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Retention, Detention, and Overland Flow for Pollutant Removal from Highway Stormwater Runoff

Volume II: Design Guidelines

Research and Development
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FOREWORD

The highway system may be a source of a wide variety of pollutants to nearby surface and groundwater. The effects of highways on water resources can have an important role in the planning, design, construction, and operation of a transportation system. The Federal Highway Administration and State highway agencies have approached the problem in a multi-phase research effort including studies to:

Phase 1 - Identify and quantify the constituents of highway runoff.
Phase 2 - Identify the sources and migration paths of these pollutants from the highways to the receiving waters.
Phase 3 - Analyze the effects of these pollutants in the receiving waters.
Phase 4 - Develop the necessary abatement/treatment methodology for objectionable constituents.

This investigation was part of the Phase 4 effort. Three management methods to remove or treat highway stormwater pollutants have been identified: vegetative controls utilizing overland flow of runoff, detention basins and wetlands, and retention basins. This study was designed to: (1) quantify, by laboratory bench-scale testing, the rate of pollutant removal from highway stormwater samples, (2) evaluate a variety of representive appropriate field installations, (3) assess the performances of these management methods, and (4) develop guidelines and specifications to assist in the implementation of the technology.

The final report of this investigation has two volumes: FHWA-RD96-095 Volume I: Research Report and FHWA-RD-96-096 Volume II: Design Guidelines.

These publications will be of interest to highway engineers and environmental practioners involved in planning and designing for the mitigation of highway runoff water quality impacts to surface and ground water. Copies of these publications are being distributed to the Federal Highway Administration regional and division offices and to each State highway agency. Additional copies may be obtained from the National Technical Information Service (NTIS), 5285 Port Royal Road, Springfield, Virginia 22161.

Charles J. Nemmers, P.E.
Director, Office of Engineering R&D

NOTICE

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16. Abstract

This volume is the second in a two-volume report entitled "Retention, Detention, and Overland Flow for Pollutant Removal From Highway Stormwater Runoff." The research developed design guidelines and specifications for measures to reduce or eliminate the impacts of highway runoff on surface waters. The titles of the volumes of this report are:

FHWA-RD-96-095	Vol. I.	Research Report
FHWA-RD-96-096	Vol. II.	Design Guidelines

This report provides design guidelines and specifications for three types of management measures for the removal of pollutants from highway stormwater runoff. The three general types of management measures, determined through previous FHWA studies to be effective in treating highway runoff, are: retention systems (basins, trenches, and wells), detention basins (wet detention basins, dry extended detentions basins, and wetlands), and overland flow (grassed channels and filter strips). These guidelines have been developed based on the experience of the project team, review of available literature, and bench-scale and field testing.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

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LIST OF SYMBOLS

A	Surface area of detention basin pool, ft^2 (m^2) Flow area, ft^2 (m^2)
A_b	Top area of retention basin, ft^2 (m^2)
A_D, A_u	Drainage area, ft^2 (m^2)
ADT	Average daily traffic
B	Bottom width of channel, ft (m) Width of overland flow system ft (m) Top width of retention facility, ft (m)
BOD_5	Biochemical oxygen demand (5-day)
C	Runoff coefficient for total runoff
C_I	Runoff coefficient for impervious areas
COD	Chemical oxygen demand
C_p	Runoff coefficient for previous areas
D	Rainfall duration, hours
D_{max}	Maximum flow depth for stability, given the channel slope, grass species and height, and erodible characteristics of the soil, ft
d_{max}	Maximum allowable depth based on: (1) depth to bedrock or seasonally high water table; or (2) infiltration rate and dewatering time, ft
d_{min}	Depth to bedrock or seasonably high water table, ft (m)
d_{reg}	Required depth to bedrock or seasonably high water table, ft (m)
d_b	Design basin depth
Δ	Mean time interval between storm midpoints, hours
E	Long-term total suspended solids removal efficiency
E_D	Dynamic removal efficiency; removal efficiency during a storm runoff event
E, E_t	TSS removal efficiency, %

LIST OF SYMBOLS
(CONTINUED)

E_Q	Quiescent removal efficiency; removal efficiency between storm runoff events
f	Soil infiltration rate, ft/hr (m/hr)
FC	Fecal coliforms
H	Hydraulic loading rate, in/hr (m/hr)
i	Rainfall intensity, in/hr (cm/hr); hydraulic gradient in Darcy's law for flow through porous media
K_2	Second order decay rate, m^3/mg-yr
L	Channel length, ft (m) Length of overload flow system, ft (m) Top length of retention facility, ft (m)
LC_{50}	Concentration lethal to 50 percent of the organisms in an acute bioassay
k	Coefficient of permeability
NO_2	Nitrite nitrogen
NO_3	Nitrate nitrogen
NS	Average number of storms per year
P	Wetted perimeter, ft (m)
P	Designed rainfall volume for retention facility, ft (m)
PCBs	Polychlorinated of biphenyls
PO_4	Phosphate phosphorus
Q	Flow rate, ft^3/s (m^3/s); Mean flow rate ft^3/hr (m^3/hr)
Q_{max}	Maximum flow rate, ft^3/s (m^3/s)
Q_R	Mean runoff flow rate, ft^3/s (m^3/s)
Q_u	Storage volume required for retention facility, ft (m)
Q/A	Detention basin overflow rate, ft/hr (m/hr)
R	Hydraulic radius; area/wetted perimeter Total phosphorus retention coefficient

LIST OF SYMBOLS
(CONTINUED)

Rv	Rainfall volume or depth, ft (m)
S, S_o	Channel slope, ft/ft (m/m)
T	Detention time, hr Average hydraulic residence time, yr
TKN	Total kjeldahl nitrogen
TC	Total coliforms
TD	Allowable dewatering time, hr
TOC	Total organic carbon
TS	Total solids
TSS	Total suspended solids
TVS	Total volatile solids
ν	Coefficient of variation; standard deviation/mean
ν_v	Coefficient of variation, rainfall volume
ν_i	Coefficient of variation, rainfall intensity
ν_D	Coefficient of variation, rainfall duration
$\nu\Delta$	Coefficient of variation, time interval between storm midpoints
V	Flow velocity, ft/sec (m/s); infiltration basin volume, ft^3 (m^3)
V_b	Assumed maximum storage capacity of retention facility, ft^3 (m^3)
V_c	Effective storage volume ft^3 (m^3)
V_d	Volume released by dewatering, ft^3 (m^3)
V_e	Effective storage volume, ft^3 (m^3)
V_r	Void ratio of trench medium
V_{ro}	Mean runoff volume, ft^3 (m^3)
V_s	Settling velocity of particulates, ft/hr (m/hr)

LIST OF SYMBOLS
(CONTINUED)

V_B	Volume of storage pool in detention basin, ft^3 (m^3)
V_R	Mean volume of runoff, ft^3
X	Impervious area/total area
Z	Side slope ratio, horizontal to vertical

1. INTRODUCTION

1. PURPOSE OF THIS MANUAL

Under the 1987 Federal Water Quality Act, stormwater discharges are to be regulated under the National Pollutant Discharge Elimination System (NPDES) permitting program. The U.S. Environmental Protection Agency (U.S. EPA) has recently released proposed regulations for the stormwater discharge permitting program, and final regulations are expected to be in place by 1990. Highway runoff is one of the many sources of stormwater pollution which are likely to be addressed by the required management programs to be developed in many urbanized areas.

In addition to the NPDES stormwater discharge permitting program, environmental assessment studies of proposed highway projects will generally have to address the stormwater runoff pollution impacts. Further, an increasing number of State and local regulatory agencies are requiring that highway runoff pollution impacts be addressed for new highway projects. With the issuance of the new U.S. EPA regulations for stormwater discharges, it is likely that the need for mitigative measures to control runoff pollution from highways will receive even more attention during the 1990's.

This report outlines a series of best management practices (BMP's) which can be used to control highway runoff pollution. Guidelines are presented for five different types of BMP's:

- Vegetative controls.
- Wet detention basins.
- Dry extended detention basins.
- Retention measures.
- Wetlands systems.

For each type of highway runoff BMP, guidelines for design, construction, and operation and maintenance are presented. Worksheets for design calculations are presented for most of the BMP's, and available data on pollutant removal efficiencies are presented. The guidelines presented herein are intended for use by highway agencies in meeting the expanding needs to mitigate the runoff pollution impacts of highway projects. These guidelines assume that the need for controlling runoff pollution from a specific highway site has been established using presented guidelines. (Dupuis, 1985)

2. HIGHWAY RUNOFF POLLUTION

Highway operation and maintenance can contribute an array of pollutants to surface and groundwater resources. Highway runoff may contain solids, heavy metals, nutrients, oil and grease, bacteria, and other pollutants. The impacts of highway runoff pollution on aquatic ecosystems are extremely site and runoff-event specific. The objective of a highway runoff pollution management program is to reduce the total highway pollutant loading that enters receiving waters. The emphasis of the management program is on total runoff, not individual events. Although all highway runoff contains pollutants, the pollutant loading does not always constitute a problem for receiving waters. This section identifies what pollutants are a concern in highway runoff, their behavior within various management measures, and what potential impacts these pollutants have on surface and groundwater resources.

A. IDENTIFICATION OF POLLUTANT SOURCES AND CONSTITUENTS

Pollutants accumulate on highway surfaces, roadside areas, and rights-of-way from highway use, maintenance, natural sources, and deposition of air pollution (see table 1). The concentrations of these pollutants are highly variable by site, and are affected by numerous factors such as traffic characteristics, climate, maintenance, adjacent land use, etc.

Physical movement of the pollutants from the highway surfaces occurs by washoff from precipitation and scour by wind or vehicle-induced air turbulence. Rainfall washoff of pollutants is dependent upon rainfall intensity and duration, street surface characteristics, and particle size. Most of the pollution potential of highway debris is contained in the very fine silt-like particles. Where streets and highways have curbs, approximately 85 to 90 percent of the pollutants accumulate within 12 in (30 cm) of the curb. (Gupta, et al., 1981) Where there is no vertical obstruction such as a curb to cause deposition, a preponderance of materials will be swept off the pavement onto median and roadside areas. Pollutants deposited on grassed roadside areas, in particular heavy metals, are immobilized by the soils and become unavailable for transport by stormwater runoff.

Highway pollutants, such as solids, heavy metals, and organics (found in fuels and motor oils) have been found to relate directly with traffic volume. Other pollutants (herbicides and nutrients) are found in highway runoff mainly as a result of highway maintenance activities and adjacent land use contributions. Management techniques for the control of traffic- or maintenance-related pollutants are, therefore, different. Maintenance-related pollutants are better controlled through the use of general measures, such as herbicide and fertilizer application management. (Rolan, 1983) Traffic-related pollutants are more applicable to site-specific control measures and are the focus of this report.

B. BEHAVIOR AND FATE

The principal transport and transformation processes that affect constituents in runoff are listed below:

Transport	Transformation
• Advection.	• Biodegradation.
• Volatilization.	• Bioassimilation/bioconcentration.
• Sorption.	• Hydrolysis.
• Settling.	• Oxidation.
• Filtering.	• Photodegradation.

Table 1. Highway pollutants and sources.

Pollutant Groups	Examples	Sources	Parameters	Concentrations (mg/l) Avg	Range
Particulates	Dust and dirt, stones, sand, gravel, grain, glass, plastics, metals, fine residues	Tire, brake, and pavement wear, car exhaust, mud and dirt accumulated on vehicles	TS	1147	145-21640
			TVS	242	26-1522
			TSS	261	4-1656
			VSS	77	1-837
Heavy Metals	Lead, zinc, iron, copper, nickel, cadmium, mercury	Use of leaded fuels, tire and brake wear, motor oil additives, rust	Pb	0.96	0.02-13.1
			Zn	0.41	0.01-3.4
			Fe	10.3	0.1-45.0
			Cu	0.103	0.01-0.88
			Ni	9.92	0.1-49.0
			Cd	0.04	0.01-0.40
			Cr	0.04	0.01-0.14
Organic Matter	Vegetation, dust and dirt humus, oils, fuels	Vegetation, litter, animal droppings, motor fuels and oils	BOD5	24	2-133
			TOC	41	5-290
			COD	14.7	5-1058
			Oil and Grease	9.47	1-104
Pesticides/ Herbicides	Weed killers	Right-of-way maintenance	Dieldrin (ug/l)	0.005	0.002-0.007
			Lindane (ug/l)	0.04	0.03-0.05
			PCBs (ug/l)	0.33	0.02-8.89
Nutrients	Nitrogen, phosphorus	Fertilizers, motor fuels and oils	TKN	2.99	0.1-14.0
			NO2 + NO3	1.14	0.01-8.4
			PO4	0.79	0.05-3.55
Pathogenic Bacteria (Indicators)	Coliforms	Soil, litter, excreta, bird and animal droppings	TC		
			FC		

Source: Gupta, et al., 1981.

The extent to which a pollutant is susceptible to these processes will be due to the chemical nature of the pollutant, its physical-chemical properties such as water solubility, vapor pressure, and tendency to absorb to organic matter or sediment. The actual processes that remove or degrade will depend not only on the above properties of the pollutant, but on the management practice being used to mitigate loading. Certain measures will not provide the time or environment to allow a particular removal process to occur. Of the transport processes, the combination of sorption and settling will be the key removal mechanisms applicable to highway runoff. Many of the constituents will be in particulate form and settle. Further, organic chemicals and heavy metals in solution will tend to adsorb to suspended sediments and then settle. Biological action, both degradation and assimilation by microbial and rooted vegetation populations, will be the most applicable transformation process. Table 2 summarizes the important fate processes for each constituent class associated with a particular management measure.

Solids

Suspended solids will be removed via settling and through filtering by vegetation and soil. Settling will be significant in wet detention basins and wetlands; where there is sufficient time allowed for smaller particles to settle. Physical removal from water can occur with all four management measures. In wet detention basins, wetlands, and particularly grass swales, rooted vegetation will reduce the velocity of the runoff and allow for some filtering and settling of particulates. Particulates will be trapped by the soil in infiltration systems.

Heavy Metals

The major fate process for heavy metals is adsorption to sediments/ particulates and settling. This will occur in each of the mitigation methods. Strictly speaking, no degradation occurs because the pollutants in these cases are elements; however, it is possible for the valence state to change which may be of toxicological importance (e.g., $Cr^{+6} \rightarrow Cr^{+3}$). Further, toxic organo-metalic complexes may be susceptible to degradation even though the resulting product may be a metal oxide.

Toxic Organics

Organic chemicals, such as aromatic hydrocarbons, chlorinated pesticides, and PCBs, will vary widely in terms of fate primarily because this is such a large class of chemicals. Any statement of fate is at best a generalization. Aromatic gasoline additives can be expected to volatilize out of the runoff water prior to any degradation. Chlorinated pesticides will be subject to sorption, volatilization, and biodegradation. PCBs will readily adsorb to sediments and the only likely transformation process will be biodegradation; the heavier compounds (greater chlorine content) can be expected to persist in the sediments. Wet detention basins and wetlands will probably be the better measures in terms of removing organics since biodegradation is the most likely transformation process; these measures should support a larger and more diverse microbial population than the soils of grass swales or infiltration systems. However, significant discharge to natural systems (as opposed to constructed management

Table 2. Principal pollutant fate processes by major management measures.

Pollutant	Management Measures			
	Vegetative Controls	Detention Basins	Infiltration Systems	Wetlands
Heavy metals	settling filtering adsorption	settling adsorption	adsorption filtration	settling adsorption
Toxic organics	settling filtering adsorption	settling adsorption biodegradation volatilization	adsorption biodegradation	settling adsorption biodegradation volatilization
Nutrients	settling filtering	bioassimilation	absorption	bioassimilation
Solids	settling filtering	settling	adsorption	settling
Oil and grease	adsorption	adsorption settling	adsorption	adsorption settling
BOD	settling biodegradation	biodegradation	biodegradation	biodegradation settling
Pathogens	settling	settling	filtration	--

Source: Dorman, et al., 1987.

measures) should be carefully monitored to determine potential impacts to habitats.

Nutrients

Nitrogen and phosphorus are the primary nutrients of concern from highway runoff. The ultimate fate of both will typically be bioassimilation by algae and rooted aquatic plants within downstream receiving waters. Nitrogen and phosphorus in particulate form will first settle, whereas dissolved portions will immediately be susceptible to bioassimilation. Removal can be expected to be more efficient in wet detention basins and wetlands, where the runoff water (and thus the nutrients) is retained allowing the indigenous flora time to assimilate the nutrients. Grass channels will aid in removal of nitrogen and phosphorus in particulate form by settling and physical filtering. Both dissolved and particulate factions may be removed within infiltration systems through filtering and adsorption processes within the underlying soil profile.

Oil and Grease

Sorption to suspended sediments and settling will be the likely transport removal mechanisms. Biodegradation will be the only transformation process likely to remove these pollutants; however, the rate of degradation is likely to be slow in all of the measures.

BOD (Organic Matter)

As the name of these pollutants implies, microbial oxidation (degradation) will be the primary transformation process. Settling and filtration will mitigate the transport of these pollutants.

Pathogens

None of the measures will effectively kill coliforms and their associated microbes. However, each of the measures should help mitigate their ultimate effect on receiving waters. The removal of particulates via filtration and settling, as mentioned above, will be the key fate mechanism for removing bacterial populations. Organic particulates will be the substrate for these organisms and thus the removal of solids will aid the removal of microbes.

C. RECEIVING WATER IMPACTS

Highway runoff pollution may affect water quality of receiving waters through shock or acute loadings and through chronic effects from long-term accumulation within the receiving water. The significance of these impacts is very site specific, and will depend heavily on the highway and receiving water characteristics. Recent research indicates few significant impacts for highways with less than 30,000 ADT. (Bertram and Kaster, 1982; Dupuis et al., 1984; Dupuis and Kobriger, 1985) Potential impacts are generally short-term, localized acute loadings from temporary water quality degradation, with few, if any, chronic effects.

There is little additional information directly relating highway runoff pollution to impacts on receiving waters. A common assumption in defining a highway runoff pollution problem is that data dealing with effects of urban runoff are applicable in some degree. Stormwater runoff pollution (including highway runoff, by assumption) can affect water quality through:

- Aesthetic impacts - General appearance of dirty or turbid water or the presence of specific unaesthetic conditions such as debris, oil films, or scum is unpleasant.

- Suspended solids - Particulates and sediment contribute to decreased flow capacity in drainageways, reduced storage volume in ponds and lakes, and smothering of benthic organisms. Suspended solids may have other pollutants adsorbed to them.

- Nutrients - The nutrients of greatest concern are nitrogen and phosphorus. High loadings of these nutrients can cause overstimulation of algae or aquatic weeds. Excessive algal concentrations in a downstream water supply can significantly increase water treatment costs, particularly with U.S. EPA's new Safe Drinking Water Act regulations. As these organisms die off and are oxidized, serious depletion of dissolved oxygen can occur (eutrophication).

- Dissolved oxygen depletion - Organic materials that consume oxygen as they decompose may use oxygen faster than natural processes can replenish it. In extreme cases, discoloration, gas formation, and odors may result. Prior to this extreme, conditions required for a balanced aquatic population of fish and other species will be adversely affected, possibly resulting in fish kills.

- Pathogenic bacteria - Excessive concentrations of pathogenic bacteria can prevent water resources from being used for water supply or recreation without purification.

- Oil and grease - Petroleum substances can destroy aquatic organisms by coating them or their habitat and blocking off the oxygen supply. Petroleum substances also exert an oxygen demand and cause aesthetic impacts.

- Toxicity - Toxicity problems can occur from either metals or pesticides and persistent organics. Impacts may be either acute or chronic depending on the concentrations, the sensitivity of each receiving species, and environmental conditions.

Highway runoff pollution impacts to receiving streams were monitored at three sites with average daily traffic (ADT) volumes of 7,400, 25,500, and 15,600. (Dupuis et al., 1984) Laboratory bioassays were also conducted with the highway runoff for ADT's up to 135,000. It was concluded:

- There were no apparent water quality impacts during storm events.

- Benthic invertebrate faunal population distribution, abundance, and composition were unaffected by the runoff.

- Periphyton communities showed no discernible impacts.

- Bioassays with undiluted highway runoff showed no acute effects on test organisms. Some sublethal chronic effects were observed; however, the use of undiluted runoff makes this a worst-case situation not likely to occur in any receiving water.

In a study of the effects of highway runoff on receiving waters, the findings of several bioassay studies of highway runoff were summarized. (Dupuis and Kobriger, 1985) Runoff from high traffic highways (one highway at 185,000 ADT and one at 50,000 ADT did have toxic effects on aquatic biota. (Winters and Gidley, 1980; Portele et al., 1982) Runoff from lower ADT rural highways did not cause discernible toxic stress to aquatic biota.

From these studies and other literature reviewed, the following conclusions can be reached regarding highway runoff pollution potential:

- Highway runoff does have the potential to adversely affect the water quality and aquatic biota of receiving waters.

- The significance of these adverse effects is variable by highway, receiving water, and runoff event.

- Runoff from urban highways with high ADT volumes may have a relatively high potential to cause adverse effects.

- Runoff from rural highways with low ADT volumes has a relatively low potential to cause adverse effects.

3. DESIGN CONTEXT

A. HIGHWAY RUNOFF CHARACTERISTICS

Mitigation measures should be designed to take advantage of the following characteristics of highway runoff:

- Nonpoint pollution discharges from frequent minor storms are more critical than discharges during infrequent major storms.

- First-flush conditions result in relatively high pollutant concentrations during the initial stages of storm runoff.

- Loadings of heavy metals and other toxicants tend to be of greater concern than loadings of nutrients and biochemical oxygen demand (BOD).

- Critical pollutants such as heavy metals tend to appear primarily in a suspended form.

Because frequent storms tend to cause runoff primarily from paved areas, they tend to produce highly concentrated discharges of highway runoff and reduced dilution by upstream runoff. As a result, most urban runoff pollution management programs rely upon controls for minor storms with relatively short recurrence intervals (e.g., less than 1 year), rather than the relatively infrequent major storms (e.g., 10-, 25-, and 100-year events) that serve as performance standards for flood management programs. Mitigation measures are typically designed to control most storms which occur each year. For example, in many sections of the U.S., mitigation measures designed to control storms producing less than 1.0 in (2.54 cm) of rainfall will control nonpoint pollution discharges from about 90 percent of the storms each year. Runoff from the more significant storm events which are not controlled tends to exhibit significant flows from nonurban areas which can dilute discharges from paved areas.

"First flush" effects refer to conditions under which a large percentage of the total storm pollution load is produced by a relatively small percentage of the runoff volume during the initial stages of runoff. As a result, the initial stages of runoff can exhibit relatively high pollutant concentrations which may induce shock-locking conditions and short-term contraventions of water quality criteria in receiving waters. Conversely, mitigation measures which can isolate first flush loadings for "treatment" may take advantage of smaller storage capacities than measures which must treat all runoff flows. Field studies have shown that the significance of first flush conditions is positively related to the amount of pavement in an urban watershed. Consequently, first flush conditions should be prevalent for most highway runoff settings. Further, first flush effects are attributed primarily to the washoff of particulates from paved areas, meaning that first flush runoff tends to exhibit relatively high loadings of suspended pollutants. Finally, heavy metals tend to exhibit a more pronounced first flush effect than other pollutants.

The BMP design guidelines presented herein are based upon control of runoff from the mean storm event. This design standard reflects the concern with controlling pavement runoff from rainfall conditions which occur frequently during the year. Since runoff from the mean storm event is expected to include most, if not all, of the first flush runoff, this design criterion should ensure adequate capture and treatment of the majority of the annual loadings in highway runoff.

Heavy metals and other toxicants in highway runoff tend to be of greater concern than other nonpoint pollutant such as nutrients. This is because paved areas tend to produce the highest per acre loadings and concentrations of heavy metals, due to contributions from vehicular traffic. Likewise, paved areas tend to exhibit less significant sources of nutrient loadings than unpaved areas. However, the control of nutrient loadings in highway runoff is likely to be of some concern if the highway project is upstream of a reservoir or estuary. This is because reservoirs and estuaries usually exhibit sufficient hydraulic residence times to cause algal blooms from excessive nutrient loadings.

Since most heavy metals and other toxicants in highway runoff tend to occur in suspended form, mitigation measures which achieve high removal efficiencies for suspended solids should also achieve significant removal efficiencies for heavy metals and other critical constituents. However, solids settling design should account for the fact that the majority of suspended loadings in highway runoff is associated with fine silt particles characterized by relatively low settling velocities.

B. CHARACTERISTICS OF MITIGATION MEASURES

Nonstructural Measures

Nonstructural mitigation measures such as curb elimination, litter control, and control of chemical applications constitute "source" controls which attempt to reduce the amount of pollution available on the highway surface. These measures address minor storms as well as major storms since overall pollution accumulations are reduced. First flush effects are reduced because these measures tend to reduce the accumulation of particulates that contribute to shock loadings.

Vegetative Controls

Vegetative controls have been shown to be effective mitigation measures for minor storms which produce relatively low velocities and long travel times. During major storms that exhibit deeper, turbulent flow conditions and shorter residence times, reduced pollutant removal rates may be expected. However, the dilution effects of these major events tends to minimize receiving water concentrations. Vegetative measures tend to be effective first flush controls since they exhibit the highest pollutant accumulations in the upstream end of the grass channel or overland flow system. The upper end of these systems is subject to the initial shock loadings, and it apparently achieves significant sedimentation and filtration of highly suspended loadings. Field studies have shown that vegetative measures for highway runoff are particularly effective at heavy metals removal within the upstream sections, probably due in large part to the significant suspended loadings. Unlike some other mitigation measures, vegetative controls are not typically used to achieve peak runoff control benefits as well as water quality control benefits.

Detention Basins

Detention basins are online or offline storage devices which can take advantage of solids settling processes as well as other mechanisms to reduce nonpoint pollution loadings in urban runoff. When used for stormwater pollution control, the storage capacity is typically sized to control minor storms (e.g., runoff from mean storm event) rather than flood-producing events. Unless a diversion structure is located upstream, the detention basin typically does not isolate first flush runoff for treatment. As a result, major storms may result in bottom sediment scouring which can cause the facility to serve as a temporary pollutant source rather than a sink. Since detention basins can be designed to achieve significant sedimentation rates, they represent a promising control measure for highway runoff due to the concern about primarily suspended loadings of heavy metals and other toxicants. Wet detention basins have a permanent pool which can also be sized to maximize biological and physical/chemical processes for removal of dissolved nutrients. Dissolved nutrients are important since this fraction tends to be more biologically available than the particulate fraction and therefore a greater threat to downstream receiving waters. Unlike wet detention basins, dry extended detention basins rely exclusively upon sedimentation processes and therefore can only achieve significant removal of suspended pollutants. Unlike most other structural measures, detention basins can be designed for cost-effective control of both peak runoff and nonpoint pollution discharges. Detention measures also represent the most effective structural control measures for spill contaminant.

Retention Measures

Retention measures can be either online or offline storage facilities. These measures take advantage of other processes besides solids settling to achieve significant pollutant removal. They can be sized for stormwater pollution management only (minor storms) or both peak runoff control and stormwater pollution management (major storms). Offline storage devices such as trenches and drainage wells isolate first flush runoff for treatment and bypass the remaining runoff from major storms. Online storage devices such as a retention basin can be sized for first flush capture, but like the detention basin, upstream diversion structures may be required to prevent overflows of runoff exceeding the first flush design volumes. Natural physical/chemical processes in the soil profile underlying these facilities are capable of achieving high pollutant removal rates for heavy metals. However, a disadvantage of retention measures is that they tend to require frequent cleanouts (e.g., every few years) in order to prevent clogging conditions which can eliminate most pollutant removal benefits.

Wetland Measures

Wetland measures are similar to wet detention basins in that they can achieve significant pollutant removal through solids settling as well as other processes. The use of both natural and constructed wetlands for highway runoff control is discussed herein. Like detention basins, wetland measures are flow-through devices which do not isolate first flush runoff for treatment. While wetlands can achieve significant removal of critical heavy metals loadings, potential deleterious impacts of metals accumulations on wetland vegetation should be considered in developing the facility design and operating procedures. Wetlands can also be very sensitive to significant shock loadings.

4. TYPES OF MANAGEMENT MEASURES

This section presents management measures currently used to control pollution from highway runoff. Descriptions of effective management techniques (general and specific) as well as measures that have been found to be ineffective are given. A guide for selecting specific management techniques is presented at the end of this section.

A. EFFECTIVE MANAGEMENT TECHNIQUES

Nonstructural Measures

Certain nonstructural measures for managing highway stormwater runoff pollution are applicable to virtually all highway situations. These measures are not directed toward site-specific problems, although they can be used in conjunction with effective site-specific measures. The practices cited are relatively low-cost and can be incorporated into existing highway design procedures and maintenance programs. They are intended to be used wherever practicable without the necessity of identifying a specific highway runoff pollution problem.

Typically, the pollutant load from highways is transported by stormwater runoff from the pavement along curbs. Most of the pollutant load in the runoff is carried as suspended solids or adsorbed to suspended solids. Therefore, management measures are usually intended to reduce the volume of particulates available for transport by runoff or to filter and settle out suspended solids. The measures, which fall into these two categories, are presented below:

- Curb elimination - Future design or reconstruction of highways should omit the use of curbs for delineation and stormwater runoff control where practicable. Curb systems act as traps for particulates and other pollutants, accumulating pollutants between storms. Omission of the curbs allows winds and vehicle-generated air turbulence to scatter the pollutants along the shoulder and rights-of-way, reducing the pollutant load available to the runoff. Where curbs are necessary for traffic control or other reasons, consider partial removal (i.e., leave gaps instead of a continuous curb) to allow air transport of pollutants off the highway. However, partial elimination of curbs should be done with caution, as discontinuous curbs may be a traffic hazard.

- Litter control - Existing litter control programs and regulations were designed primarily for aesthetic and safety objectives. However, they also achieve pollutant reduction benefits through limitation of potential pollutant sources. However, it should be noted that street sweeping cannot be considered as an effective management practice. Litter control programs should be strengthened and enforced.

- Deicing chemical use management - Proper storage and handling of deicing chemicals coupled with sound application practices will provide significant reduction for potential ground and surface water contamination. Covered storage and handling facilities designed to prevent wash off and loss of deicers coupled with good housekeeping will effectively mitigate potential pollution from these facilities. Attention to optimum application rates of chemicals along with maintenance calibration of spreading equipment will eliminate excessive deicer application. Most highway agencies have undertaken these programs which should minimize the pollution potential of deicing chemicals.

- Pesticide/herbicide use management - Use of pesticides and herbicides by State highway agencies (SHA's) are typically limited in scope and have strict controls on application, employee training, etc. The benefits of these controlled-use programs are shown by the low percent of total pollutant load attributed to pesticides/herbicides. The pesticide/herbicide controls exercised by SHA's should continue.

- Reduce direct discharges - Avoid direct discharges of highway runoff to receiving waters (including groundwater) wherever practicable. This would include collection/conveyance through closed conduits. Highway runoff should be routed through one or a combination of effective management measures prior to discharge to receiving waters.

- Reduce runoff velocity - Lowering the runoff velocity to a nonerosive level reduces the ability of the flow to transport particulates, especially bed load, and encourages sedimentation. This can be accomplished by reducing gradients, installing velocity reduction devices such as drop structures and baffles, and using grassed waterways. There will be some situations, however, where higher velocities may be required to provide for timely drainage of the highway surface and roadside areas, and where devices used to reduce gradients could be a roadside hazard.

- Establishment and maintenance of vegetation - Vegetation along highway rights-of-way is generally established and maintained for aesthetic purposes and erosion control. Vegetation, particularly dense grass cover, also provides pollutant reduction mechanisms (filtration, sedimentation, and infiltration) for highway runoff. These mechanisms can be enhanced by:

  - Establishing dense grass cover wherever practicable.

  - Minimizing the number of grass cuttings per growing season to increase the grass height and resistance to flow. Note that there is a limit to the effectiveness of this — at some height (variable by species) and flow depth, the grass will lay flat and become a less effective pollutant removal measure. The determination of the optimal number of cuttings should be based on local experience.

  - Leaving grass cuttings on the ground to act as additional filter material to encourage velocity reduction and to provide mulch.

Structural Highway Runoff BMP's

Upon identification of a potential highway stormwater runoff pollution problem, a highway runoff BMP (or combinations of BMP's) should be implemented to effectively abate the runoff impact on receiving waters through pollutant removal. There are five management measures considered cost-effective for pollutant removal from highway runoff. These are:

- Vegetative controls.
- Wet detention basins.
- Dry extended detention basins.
- Infiltration systems (also called "retention measures").
- Wetlands.

These management measures can be used alone or in combination to address site-specific highway runoff pollution problems. Table 3 summarizes the applicability of these BMP's to alternate highway configurations. Deployment of BMP's for new highway projects as well as the retrofitting of existing highways is covered in the table.

B. INEFFECTIVE MEASURES

Several stormwater runoff pollution management measures occasionally recommended as BMP's were found to be ineffective at reducing pollutant loads in highway runoff. These ineffective measures are:

- Street cleaning.
- Catchbasins.
- Porous pavements.
- Filtration devices for sediment control.

Street Cleaning

Street cleaning is accomplished either by sweeping or street flushing. While the practice has aesthetic benefits, it is not effective for highway runoff pollutant management. Recent studies which compared pollutant loads in runoff from swept and unswept streets have shown that street sweeping has virtually no effect on the pollutant loads in the runoff. (USEPA 1983) The primary effect is to improve the appearance of curbed streets by removing litter, dirt, and other debris from the gutter line. Extensive studies have been conducted of the various street cleaning methods and equipment to determine effectiveness in removing materials from streets.

Catchbasins

A catchbasin combines a storage chamber for particulates with a drainage inlet for intercepting stormwater runoff. However, the finer solids

Table 3. Applicability of structural BMP's to different highway configurations.

Management Measure	Planned Highway Construction: Interchange	Planned Highway Construction: Elevated Highway	Planned Highway Construction: At-grade Highway	Planned Highway Construction: Depressed Highway	Existing Highway Retrofit: Interchange	Existing Highway Retrofit: Elevated Highway	Existing Highway Retrofit: At-grade Highway	Existing Highway Retrofit: Depressed Highway
Vegetative controls								
Grassed channel	High	Low	High	Low	Medium	Low	High	Low
Overland flow	Medium	Low	High	Low	High	Low	High	Low
Detention basins	High	Medium	Medium	Low	Medium-High	Medium	Medium	Low
Infiltration measures								
Basin	High	Medium	Low	Low	Medium-High	Medium	Medium	Low-Medium
Trench	Low	Medium	Medium	Medium	Medium	Low-Medium	Medium	Low-Medium
Well	Medium-High	Low	Low	Low	Low-Medium	Low-Medium	Low	Low
Wetlands	Medium-High	Low	Low	Low	Low-Medium	Medium	Medium	Low

associated with most of the pollutant load are not effectively removed. From a pollutant removal effectiveness viewpoint, most catchbasins are not adequately maintained. The accumulated solids and liquids in catchbasins can be flushed by runoff inflow, and can contribute significant pollutant loadings to the receiving waters. The original purposes of catchbasins were to prevent sewer clogging by trapping materials from gravel roads, and to provide a water seal to prevent odor emanations from combined sewers. Catchbasins have been advocated as a stormwater management measure for trapping particulates and oil and grease in the runoff. When properly designed and maintained, catchbasins are effective in removing coarse solids from stormwater runoff. However, the pollutants that are associated with finer material not affected by catch basins.

Porous Pavements

Porous pavements consist of a relatively thin coat of open-graded asphalt over a base of crushed stone. The stone temporarily stores water until it percolates into the subbase material or moves laterally into a drainage channel. Potential pollutant removal occurs as the water infiltrates through the subbase. Since a key aspect of highway design is to maintain a dry subbase for structural stability, use of porous pavements is limited to parking areas and low traffic volume highways.

Filtration Systems

Filtration systems are used extensively as temporary sediment control measures during construction and vegetative cover establishment periods. Commonly used filtration systems include straw bales, sand bags, filter cloth fences, gravel and sand filters. Filtration systems are generally used to filter out larger fractions of suspended sediments and to cause some deposition upstream of the installation. Finer solids are not effectively trapped and, therefore, highway runoff pollutant removal potentials are low.

5. SELECTION OF HIGHWAY RUNOFF BMP's

A. COST-EFFECTIVENESS OF MANAGEMENT MEASURES

Management measures were rated on the basis of their pollutant removal effectiveness for specific pollutants, relative capital costs, land requirements, and operation and maintenance costs. Ratings are based on information gathered from a review of literature. Efficiencies inferred from other than specific data in the literature are identified in the table. Qualitative ratings are used because effectiveness is dependent on the design of the management measure and site-specific factors that determine runoff characteristics and pollutant loads. The ratings are shown in table 4. Ranges of monitored pollutant removal efficiencies are reported in later chapters for individual BMP's.

As may be seen in the table, the nonstructural measures can achieve relatively high pollutant removal efficiencies at little cost. For this reason, nonstructural controls should be among the first management measures considered for a particular highway.

Table 4. Effectiveness ratings of highway runoff BMP's[1].

Management Measure	Type	Pollutant Removal Effectiveness: Particulates	Pollutant Removal Effectiveness: Heavy Metals	Pollutant Removal Effectiveness: Nutrients	Pollutant Removal Effectiveness: Organics	Relative Capital Costs Per Acre[2]	Additional Land Requirements	O & M Costs: Routine	O & M Costs: Nonroutine
A. NONSTRUCTURAL CONTROLS									
Curb elimination	Post deposition	H	H	N/A	H	L	M to H	O	O
Litter control	Source	H	M	M	M	L	O	O	O
Controlled use of deicing chemicals	Source	N/A	H	N/A	H	L	O	O	O
Controlled use of pesticides/herbicides	Source	N/A	H	N/A	H	L	O	O	O
Street cleaning	Post deposition	M	L	L	L	L	O	H	O
B. STRUCTURAL CONTROLS									
Grassed channels	Post runoff	M to H	M to H	L	L to M	L	L	L	L
Overland flow	Post runoff	M to H	M to H	L	L to M	L	M to H	L	L
Dry extended detention basins	Post runoff	H	H	L to M	H	L to M	M	L	L
Wet detention basins	Post runoff	H	H	H	H	H	H	L	L
Infiltration systems	Post runoff	H	H	H	H	M to H	L to M	H	H
Wetlands	Post runoff	H	H	H	M to H	M to H	M to H	L	L
Catchbasins	Post runoff	L	L	L	L	M to H	L to M	H	H
Porous pavements	Post runoff	H	H	L to M	H	L to H	O	M	M
Filtration systems	Post runoff	M	L	L	L	L	O	M	M

1. Ratings: H = High, M = Medium, L = Low, O = None, N/A = Not Applicable.
2. Based on additional capital costs required for nonpoint pollution management, per acre.

Among the structural controls, grass channels and overland flow systems represent the most cost-effective measures for controlling loadings of toxicants in highway runoff. However, vegetative controls are among the least effective BMP's for controlling nutrient loadings. Of the other effective structural BMP's, dry extended detention basins and infiltration systems require the lowest capital costs per pound of toxicant removed, while wet detention basins and wetland systems are the most cost-effective BMP's for the control of nutrient loadings. Infiltration systems are the most maintenance-intensive BMP's for highway runoff control. If adequate funding is not available to regularly clean out (e.g., every few years) infiltration BMP's, they should not be selected for highway runoff control. Likewise, if aesthetics of the structural BMP are a critical concern, greater consideration should be given to the use of wet detention basin systems rather than dry extended detention basins. This is because the wet detention basin will require less frequent clean-outs to maintain an acceptable appearance because debris and sediment accumulate below the water surface of the permanent pool. By comparison, frequent clean-outs (e.g., every year) may be required to keep the dry detention basins from becoming eyesores.

B. GUIDELINES BASED UPON RECEIVING WATER QUALITY IMPACTS

Initial screening of structural BMP's can be based upon the potential impacts of highway runoff on receiving water quality. Since certain BMP's are more effective at controlling selected pollutants than others, the critical pollutants for the downstream receiving water are an important factor in BMP selection.

If highway runoff loadings of toxicants are the major concern, the most cost-effective BMP's are vegetative controls and dry extended detention basins. Because of their relatively low cost and widespread applicability, vegetative controls (particularly grassed channels) should be the first choice for the control of toxic loadings under most highway situations. If the level of toxic loading reductions achieved by vegetative controls does not provide sufficient protection of receiving water quality, a dry extended detention basin should be considered for deployment downstream of the vegetative control. If vegetative controls are infeasible, a detention basin BMP should be considered as the primary management measure. If adequate funding can be committed for BMP maintenance, infiltration systems might also be considered for situations where vegetative controls and detention basins are either infeasible or in need of supplementary pollution controls.

If highway runoff loadings of nutrients are the major concern, the most cost-effective BMP's are wet detention basins and wetlands systems. Although they are more expensive and require more space than dry extended detention systems, wet detention basins can achieve significant removal rates for nutrients. The receiving waters most likely to require nutrient loading controls are water bodies most vulnerable to eutrophication such as reservoirs and tidal estuaries. A grassed channel may be an appropriate pretreatment device upstream of the primary BMP's. Reductions in sediment and nutrient loadings within the grassed channel are likely to enhance the effectiveness of the downstream wet detention basin or wetlands, as well as reducing maintenance costs. Once again, an infiltration system could be substituted for the primary BMP's if adequate funding can be committed for maintenance.

If there are no particular concerns about highway runoff impacts, BMP's can probably be restricted to low-cost vegetative controls. Grassed channels should be able to achieve adequate reductions in sediment and toxicant loadings for situations where the only objective is to achieve some mitigation of runoff pollution loadings.

C. BMP SCREENING PROCEDURE

A general screening procedure which accounts for factors such as cost-effectiveness, potential water quality impacts, and design and construction considerations is summarized in figure 1. The screening procedure is based upon the general guidelines presented above. As shown, the most critical water quality concern determines whether other BMP's are required to supplement vegetative controls. If nutrient loadings are the principal concern, either a wet detention basin or a wetlands BMP should suffice, assuming adequate site area is available. If toxicant loadings are most critical, the use of a dry extended detention basin is recommended; however, if aesthetics of the BMP are a major concern, a wet detention basin BMP is preferable. Infiltration BMP's are recommended only to supplement other structural BMP's if site characteristics (e.g., soils, water table depth) are acceptable. Further, it is important that adequate maintenance funding be committed to maintain the infiltration BMP's.

A step-by-step summary of the BMP screening procedure is presented below.

1. Make maximum use of feasible nonstructural controls: Application of low-cost nonstructural approaches is a logical starting point to mitigate highway runoff pollution impacts.

2. Make maximum use of feasible vegetative controls: Because grassed channels are relatively low cost measures which are commonplace in most highway drainage systems, this BMP should receive primary consideration regardless of the potential receiving water quality impacts. Site constraints should be evaluated, including the capability to sustain an adequate vegetative cover and the length of channel required to achieve adequate pollution removal (e.g., 150 ft (45.75 m) for channel slopes greater than 2 percent and 100 ft (30.50 m) for slopes of 2 percent or less).

3. Determine potential receiving water quality impacts of highway runoff pollution: Determine the potential water quality concerns and the critical highway runoff pollutants (e.g., toxicants, nutrients).

4. Determine whether proposed vegetative controls achieve adequate water quality protection: The runoff pollution control benefits of the system identified in step 2 should be documented and compared with the water quality goals from step 3. If no serious water quality concerns are identified in step 3, presume that the vegetative control system from

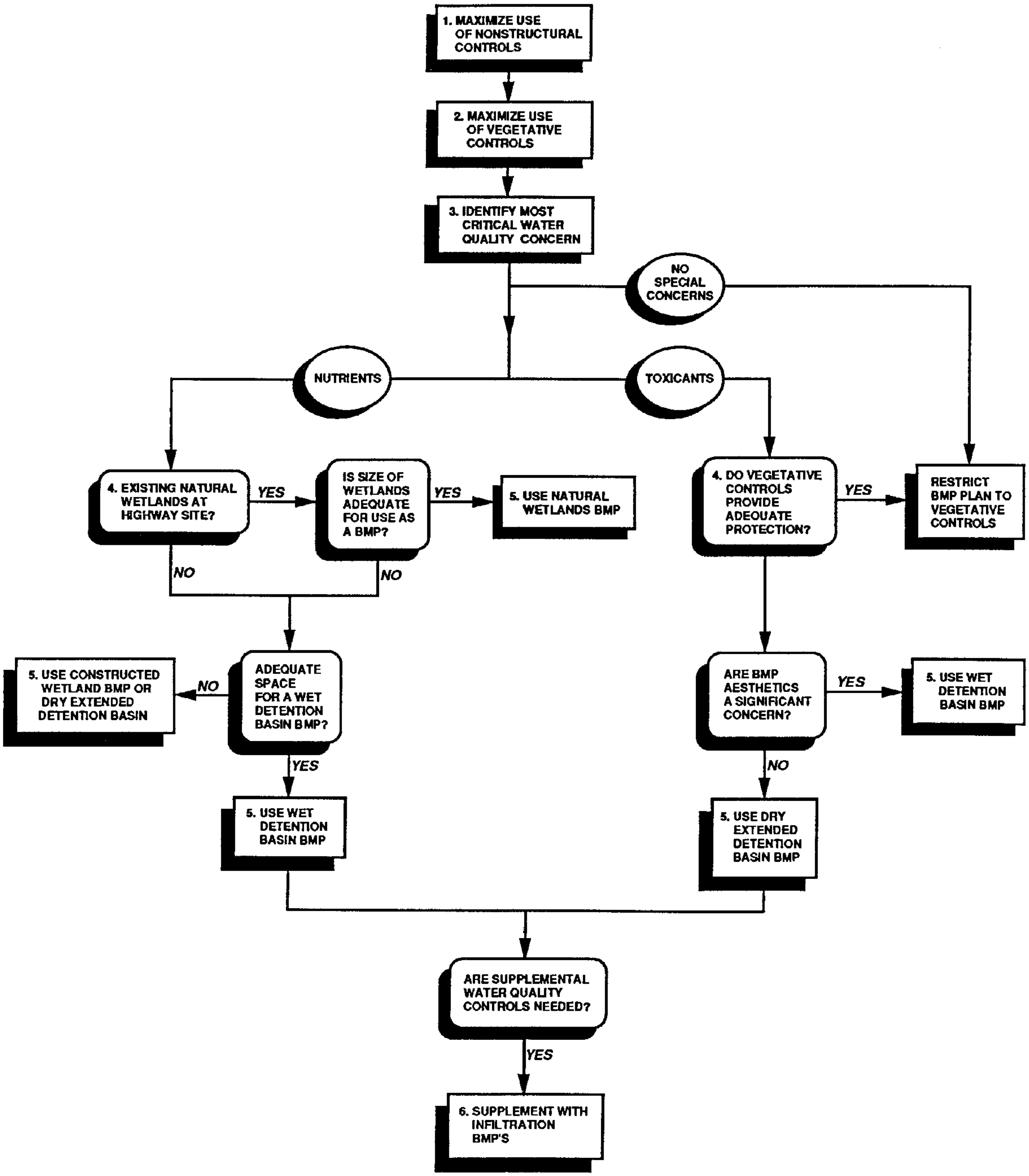

Figure 1. BMP screening procedure.

step 2 is adequate. The feasibility of using existing natural wetlands as BMP's should also be evaluated under this step.

5. If necessary, supplement vegetative controls with other structural BMP's: If toxicant loading reduction is principal concern, deploy dry extended detention basin downstream of vegetative controls. If nutrient loading reduction is principal concern, deploy wet detention basin or wetlands system downstream of vegetative controls. Screen the site constraints for the supplementary BMP's. For detention basin BMP's, site constraints include available space, configuration, locations of wetlands (i.e., may require the use of dry detention), and visibility from the highway (e.g., aesthetics). For wetlands systems, site constraints include location and size of existing wetlands, local hydrology, and configuration.

6. Infiltration systems may also be deployed to supplement vegetative controls if adequate maintenance is achievable and site characteristics are acceptable (relatively permeable soils, adequate depth to water table).

2. VEGETATIVE CONTROLS

Vegetative controls (grassed channels and overland flow areas) are the most common management measures for highway runoff pollution. They are adaptable to a variety of site conditions, are flexible in design and layout, and are a relatively inexpensive management measure. Vegetative controls can be used as sole management measures or in combination with secondary measures (e.g., detention basins, infiltration systems, and wetlands). Grass is the most common vegetation used, and is more effective at pollutant removal than shrubs, trees, or other vegetation.

1. DESIGN CONSIDERATIONS

Field studies reported in the literature and conducted to generate effectiveness data for this report suggest that sedimentation is the primary removal mechanism for stormwater pollutants in grassed channels and overland flow areas. Consequently, design considerations should focus on creating conditions that are conducive to sedimentation (i.e., shallow flow depths, sufficient detention time). Factors that affect flow depth and detention time in grassed channels include channel width, channel length, channel slope and vegetative cover. By minimizing flow depth and maximizing detention time, a substantial percentage of solids and associated pollutants can be expected to be deposited in the channel.

In addition to sedimentation, there appear to be several secondary removal mechanisms in a grass channel, including infiltration and adsorption. Unlike sedimentation, infiltration and adsorption processes result in the removal of soluble pollutants. Like sedimentation, these processes will be more effective when the flow depth is minimized and detention time is maximized. Longer detention times provide more opportunities for the stormwater pollutants to infiltrate into the soil and adsorb to surface soils within the channel.

Channel stability is an essential consideration in the design of vegetative controls, considering that an eroding channel or overland flow system can actually increase the sediment load in the stormwater, in addition to resuspending solids and pollutants removed during previous storms. Factors such as vegetative cover, soil type, slope, and design runoff flows must be considered in the design of a stable channel or overland flow system.

A. STABILITY

To ensure adequate channel stability for stormwater pollution control, the runoff event used for the design of vegetative controls should be greater than the runoff event of interest for pollution abatement. Usual practice is to design roadside channels or overland flow areas for 5- to 10-year recurrence intervals, depending on the consequences and costs of failure as well as construction and maintenance costs. One of the factors that should be considered in selecting the design runoff event is the sensitivity of receiving waters and the impact which would result from erosion and failure.

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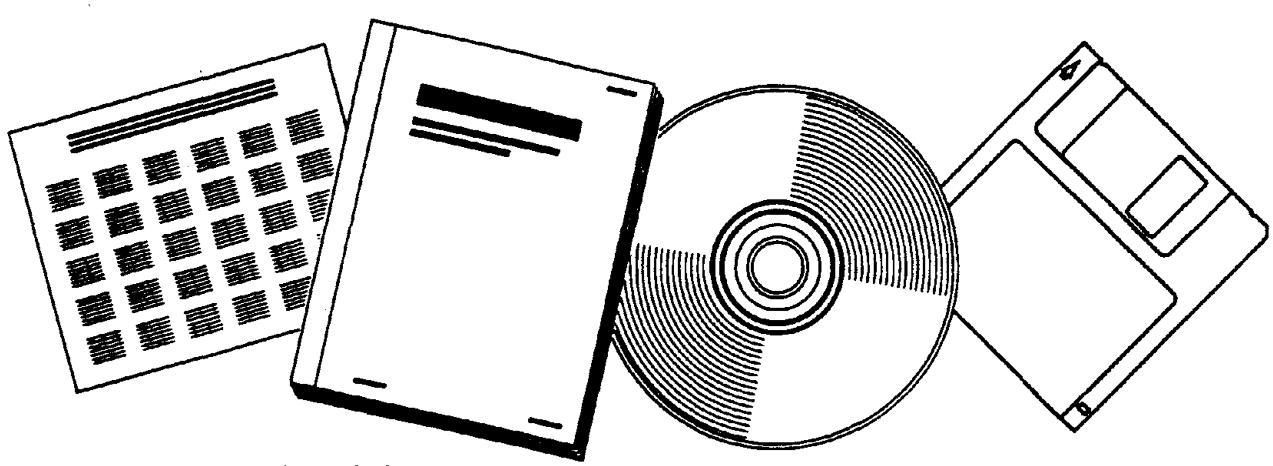
RETENTION, DETENTION AND OVERLAND FLOW FOR POLLUTANT REMOVAL FROM HIGHWAY STORMWATER RUNOFF. VOLUME 2. DESIGN GUIDELINES

VERSAR, INC., SPRINGFIELD, VA

NOV 96



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The stability of an open channel or overland flow area is dependent on the erodibility of the soils in which the channel or slope is constructed and the shear stress exerted at the soil interface by the runoff flow. For purposes here, soils with a gravel, sand, and clay mixture can be classified as erosion resistant; fine-grained sands and silts are classified as erodible; and plastic and semiplastic soils are in an intermediate range of erodibility.

A potentially unstable channel/flow area in bare soils can be made stable by lining it with grass, rock riprap, concrete, or other materials, thereby changing the susceptibility to erosion. Only a grass lining offers effective pollutant removal, however. The shear stress or tractive force can be lessened by reducing flow depth or slope, or both. Flow depth can be reduced by increasing either the width of the vegetative control measure or the velocity. Slope can be reduced by the use of grade control structures (e.g., benches and terraces, checkdams, and drop structures). Erodibility, flow depth, and slope are interdependent because a change in one will affect the magnitude of one or both of the others.

B. ESTABLISHMENT OF VEGETATION

The effectiveness of vegetative control of pollutants in runoff comes from continuous vegetation, as the vegetation enhances sedimentation, filtration, infiltration, and adsorption of pollutants. Tall dense stands of turf-forming grasses are the most effective for pollutant removal and erosion control. State highway agencies have considerable experience in establishing grass cover, and are familiar with selecting species adapted to local conditions and identifying the species establishment requirements as described below. The Soil Conservation Service and State agricultural extension services also have experience in establishing grass and can provide additional guidance.

C. FLOW DEPTH AND DETENTION TIME

The runoff event used to evaluate pollution removal should have a shorter return period than the runoff events (5- to 10-year recurrence intervals) used to analyze channel stability. Therefore, the long-term mean runoff event, is appropriate for stormwater removal evaluations. This event is defined as the mean storm rainfall intensity in inches/hr (see Chapter 3) multiplied by a runoff coefficient and drainage area.

Factors such as slope, flow width, flow length, and vegetative cover affect the flow depth and detention time in vegetative measures. Detention time can be increased by increasing the flow width, increasing the flow length, decreasing the slope, or providing a more dense vegetative cover (i.e., a greater channel roughness factor). However, decreasing the slope or providing more dense cover will also increase the flow depth, offsetting to some extent the benefit of increasing the travel time. Increasing the flow width appears to be the best alternative, because it both reduces flow depth and increases travel time. Increasing the flow length increases travel time, but does not affect flow depth.

Because stability is such an overriding concern in vegetative control design, the controls should be designed for stability first, and then adjusted if necessary to maximize pollution removal. Any changes that will be made to the control in order to increase removal efficiency will also increase the stability of the control. As discussed above, increasing the length or width of flow are the best alternatives for increasing pollution removal efficiency.

2. DESIGN PROCEDURES

The design of a grassed channel or overland flow management measure involves use of the following steps:

1. Estimate runoff flow rates for design runoff event (i.e., from 5-, 10-year storms).

2. Establish grade of proposed channel or overland flow area.

3. Select a grass cover suitable for the site.

4. Determine maximum permissible flow depth for the grass cover and slope to be used.

5. Estimate channel or overland flow area dimensions.

6. Determine flow velocity.

7. Determine if design flow is less than maximum permissible flow (stable) or greater than maximum permissible flow (unstable).

8. If channel or overland flow area is unstable, reduce flow depth by increasing bottom width or using flatter side slopes (channel), or both. Also, the maximum noneroding depth of a channel can be increased by decreasing the slope, although reductions in flow depth are preferable from a pollution removal standpoint.

9. Determine if provisions for erosion protection are necessary during establishment of grass cover.

10. Determine pollution removal efficiencies for channel or overland flow area. If removal is not sufficient, increase flow width or flow length.

A. GRASSED CHANNEL DESIGN

A synthesis of available information on the grassed channel as a highway runoff pollutant management measure, results in the following general guidelines:

- Pollutant removal effectiveness is related to average flow depth and detention time in grassed channels. Figure 2 presents the relationship between TSS removal efficiency and combinations of average flow depth and travel time, based on settling velocity distribution data for highway runoff. The curves were developed

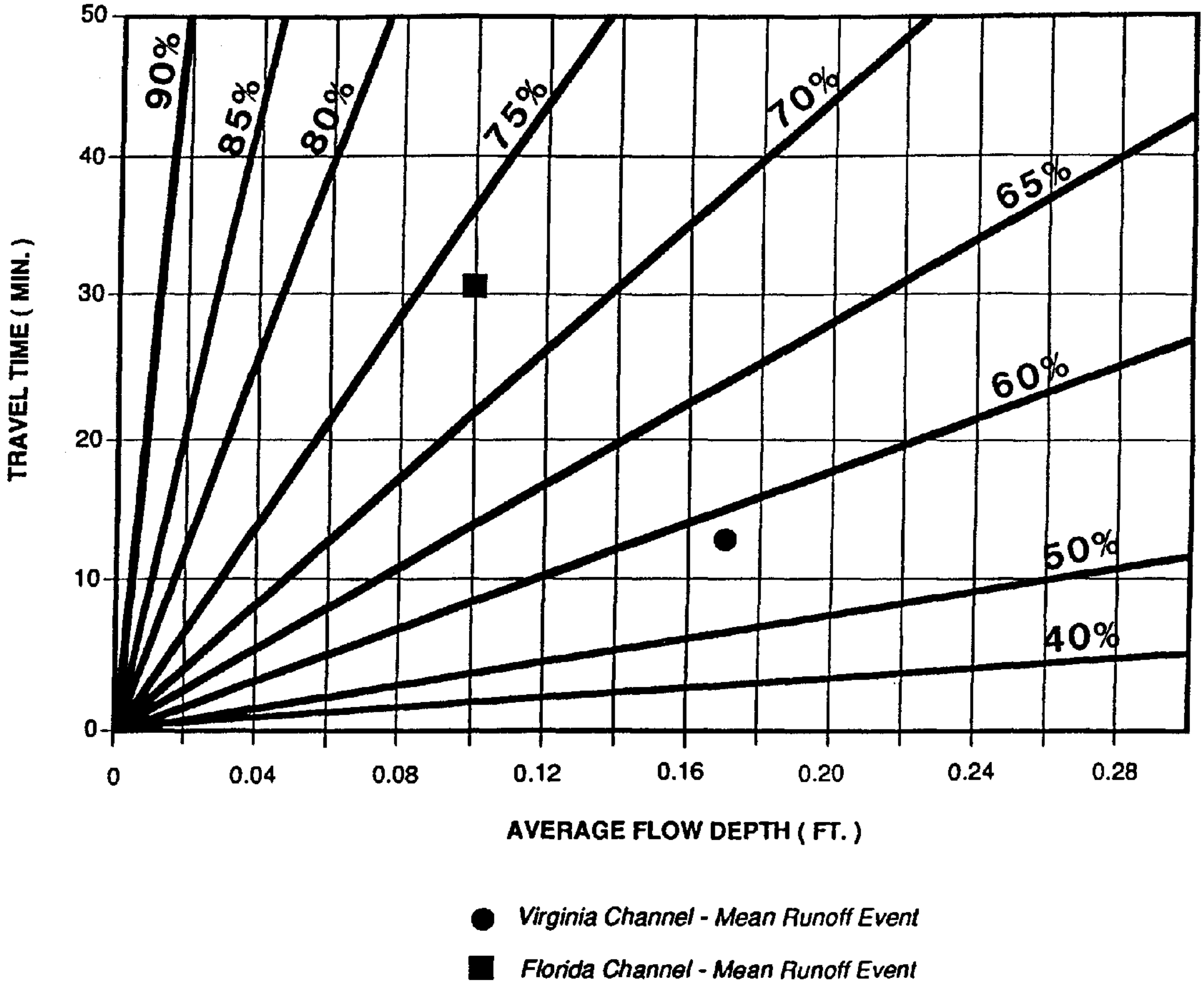


Figure 2. TSS removal efficiency for vegetative controls.

based on a sedimentation analysis procedure, which includes a parameter to account for reduced removal efficiency due to turbulence in the settling chamber. (Fair and Geyer, 1954) A high degree of turbulence was assumed for figure 2 so that the removal efficiency estimates are conservative.

Figure 2 also shows TSS removal efficiency estimates for two grassed channels that were part of a recent FHWA monitoring study of highway runoff BMP's. Based on the long-term mean runoff event for the two sites, the estimated removal efficiencies are 73 percent and 59 percent for the Florida and Virginia channels, respectively. In comparison, table 5 indicates that the monitored removal efficiencies were 90 percent for the Florida channel and 52 percent for the Virginia channel. In both cases, the estimated removal efficiency is within 20 percent of the monitored efficiency. For the Florida channel, processes other than sedimentation (e.g., infiltration and filtration) are probably responsible for the high monitored removal efficiency.

- Based upon suspended pollutant fractions measured in highway runoff field studies, average pollution removal efficiencies for lead, copper, and zinc are as follows:

  - Copper 60 percent of TSS removal efficiency
  - Lead 90 percent of TSS removal efficiency
  - Zinc 50 percent of TSS removal efficiency

  For example, 90 percent removal of TSS corresponds to a copper removal efficiency of 90 percent x 60 percent = 54 percent.

- Based upon highway runoff field studies, removal efficiencies for nutrients vary widely and do not show a strong relationship with TSS removal efficiency.

- Shallow flow depths are desirable; the erosive force in flow at a given slope increases with depth, and sedimentation is more effective at shallow depths.

- Parabolic cross sections or rounded corners in trapezoidal sections are recommended; V-shaped cross sections erode more readily.

- Channel side slopes should be at most 3:1 (H:V), and as flat as practicable for roadside safety, ease of mowing, and lesser flow depths.

- Sharp bends in the channel are potential problem areas for erosion.

- A layer of organic material spread on the soil (prior to grass establishment) should enhance metals removal efficiency through adsorption.

Table 5. Monitoring results for grassed channels.
FHWA monitoring study of highway runoff BMP's.

Monitoring Site	Channel Length (ft)	Channel Slope (ft/ft)	Estimated Travel Time (min)[2]	Average Pollutant Removal (%)[1]				
				TSS	TKN	TP	CU	ZN
Virginia	185	0.047	13.7	52	17	36	12	27
Maryland	193	0.032	16.0	(-65)-7	3-46	(-19)-33	(-2)-43	18
Florida	185	0.016	30.6	90	13	-47	42-56	69

Notes:

1. Ranges of percent removal result from inflow or outflow concentrations that are below detection limit.

2. Travel times were calculated for the mean storm event, using the rational formula to calculate a flow based on the drainage area, the runoff coefficient, and a mean rainfall intensity value selected from the rainfall characteristics table found in chapter 3.

- Alkaline soils and subsoils promote metals removal and retention through adsorption.

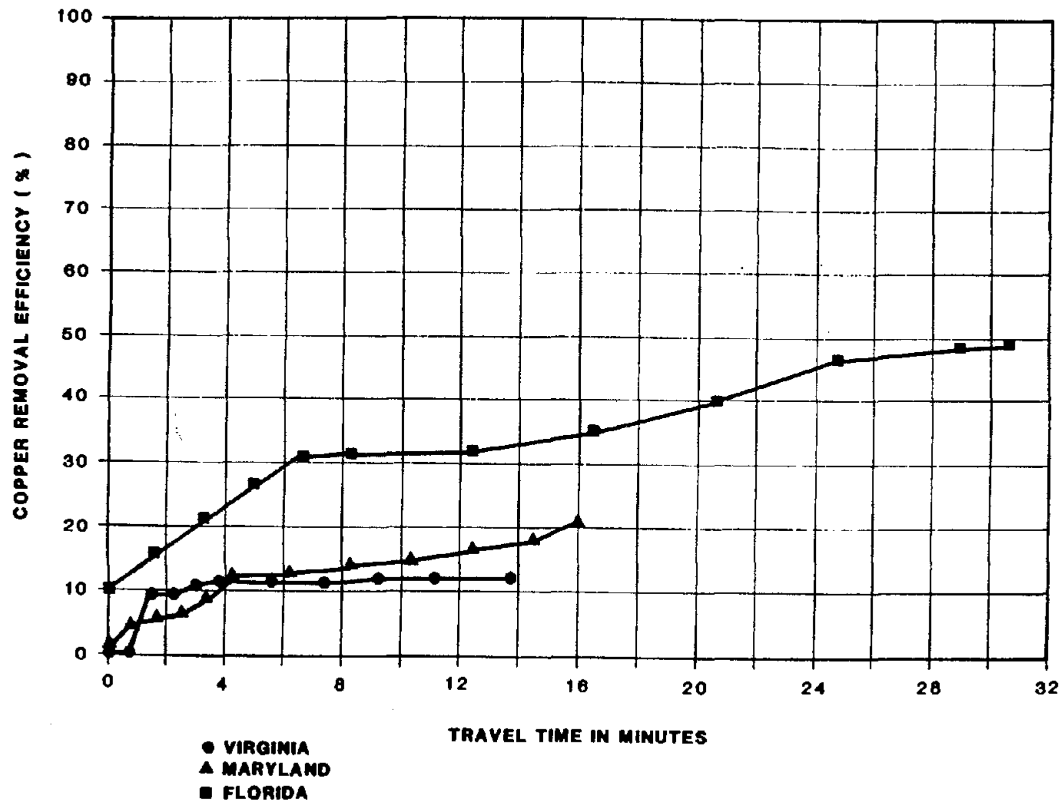
Grassed channel design for pollutant removal is based on three factors: stability, flow depth, and travel time. The channel should be stable at the design runoff event (e.g., 5-or 10-year flows) to ensure that erosion of the channel will not reduce its effectiveness in removing pollutants. Therefore, grassed channel design procedures are based on channel stability at design runoff flows. After stability criteria are met, the design is reviewed to assess long-term TSS and pollutant removal efficiency, and the design is adjusted if better performance is required.

Table 5 presents removal efficiencies for three grassed channels that were part of a recent monitoring study of highway runoff BMP's. The removal efficiencies presented in the table are based on the inflow and outflow event mean concentration (EMC) values measured for each channel. The values in the table indicate that the Florida channel is most effective in removing TSS and metals, and that low efficiencies for nutrients were observed for all three channels. Figure 3 presents average removal efficiencies for copper and zinc as a function of travel time in the three grassed channels covered by the highway runoff monitoring study. Because concentrations were only measured at the inflow and outflow points of the channel, the metals removal along the length of the channel (i.e., the shape of the removal efficiency curve) was estimated by comparing channel sediment core samples and background sediment samples. Both figures indicate that pollutant removal efficiency tends to level off after a travel time of 8 to 10 minutes. A travel time of 10 minutes corresponds to a travel length of 120 ft (36.60 m) in the Maryland channel, 135 ft (41.18 m) in the Virginia channel, and 60 ft (18.30 m) in the Florida channel. Based on these monitoring results, it recommended that a minimum length of about 150 ft (45.75 m) for grassed channels with slopes greater than 2 percent and about 100 ft (30.50 m) for channels with slopes less than 2 percent, in order to ensure travel times greater than 10 minutes.

Key design elements are channel dimensions, channel slope, vegetation type, and the flow rate in the channel. Information necessary for design of grassed channels includes:

- Runoff flow rates for design runoff event (based on the rational or other appropriate method).
- Grade and side slope of proposed channel.
- Grass cover to be used.
- Space available for construction.

Figure 4 is a worksheet that can be used for the design of grassed channels. The worksheet considers design with respect to channel stability before and after vegetation is established, as well as pollutant removal efficiency. A example of the design of grassed channels is given in chapter 2, section 5.A.



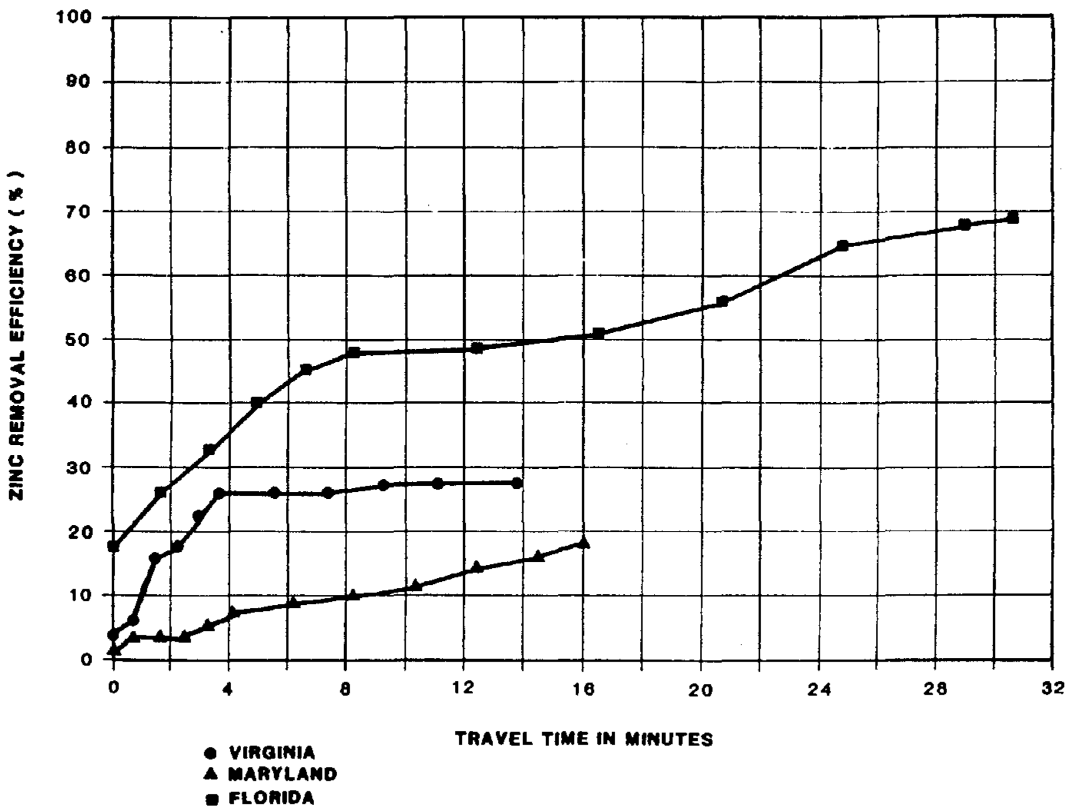


Figure 3. Copper and zinc removal in grassed channels as a function of travel time.

GRASSED CHANNEL DESIGN WORKSHEET
(for triangular or trapezodial cross sections)

*******************************DESIGN DATA********************************

Design Runoff Flow	____________________	(Q)	ft^3/s
Channel Slope	____________________	(S)	ft/ft
Channel Length	____________________	(L)	ft
Trial Bottom Width	____________________	(B)	ft
Side Slope Ratio (horizontal/ vertical)	____________________	(Z)	
Grass Species	____________________		
Vegetation Height	____________________		in
Soil Type (Circle One)	Erodible/Intermediate/Nonerodible		

********************DESIGN PROCEDURES FOR STABILITY*********************

Retardance (i.e., A, B, C) (from table 6)	____________________		
Maximum Depth of Flow (from figures 5 to 7)	____________________	(D_{max})	ft
Flow Area ($A = D_{max} \times (B + (Z \times D_{max}))$)	____________________	(A)	ft^2
Wetted Perimeter ($P = B + (2 \times D_{max}) \quad 1 + Z^2$)	____________________	(P)	ft
Hydraulic Radius ($R = A/P$)	____________________	(R)	ft
Velocity (from figures 8 to 12)	____________________	(V)	ft/s
Maximum Flow ($Q_{max} = A \times V$)	____________________	(Q_{max})	ft^3/s

If $Q_{max} > Q$ then channel will be stable. If not, increase bottom width, decrease longitudinal slope or increase side slope to increase the area of flow or to reduce velocity.

Figure 4. Grassed channel design worksheet.

GRASSED CHANNEL DESIGN WORKSHEET
(CONTINUED)

**********DETERMINATION OF LONG-TERM POLLUTANT REMOVAL EFFICIENCY**********

Long-term Mean Runoff Event Flow	______________	(Q_a)	ft^3/s
Trial Centerline Flow Depth	______________	(D)	ft
Flow Area (A = D x (B + (Z x D)))	______________	(A)	ft^2
Wetted Perimeter (P = B+ (2 x D) $\sqrt{1 + Z^2}$)	______________	(P)	ft
Hydraulic Radius (R = A/P)	______________	(R)	ft
Manning's n ($n = 1.49 \times A \times R^{2/3} \times S^{1/2}/Q_a$)	______________	(N)	
Velocity ($V = Q_a/A$)	______________	(V)	ft/s

If calculated Manning's n is in reasonable agreement with Manning's n from figure 13 as a product of V and R, then go on to next step; otherwise, revise trial depth and repeat calculations.

Average Depth $D_{avg} = A/(B+(2 \times D \times Z)))$	______________	(D_{avg})	ft
Travel Time $T = L/(60 \times V))$	______________	(T)	min
Long-term TSS Removal from figure 2)	______________	(E_{TSS})	%
Long-term Pb Removal (90% of TSS Removal)	______________	(E_{Pb})	%
Long-term Cu Removal (60% of TSS Removal)	______________	(E_{Cu})	%
Long-term Zn Removal (50% of TSS Removal)	______________	(E_{Zn})	%

Figure 4. Grassed channel design worksheet (continued).

Table 6 and figures 6 through 12 are used in the design for stability. The key design elements (flow, slope, and grass type) are necessary, together with trial channel width, side slope ratio, grass height, and soil type (erodible or nonerodible). The grass type and height are used to determine the retardance factor (a type of roughness factor) from table 6. The retardance factor, slope, and soil type are then used to determine the maximum depth of flow d_{max} using figures 5 through 7.

The hydraulic radius, slope, and retardance factor are then used to determine the velocity at d_{max} (figures 8 through 12). Using the flow area and velocity, the maximum flow (Q_{max}) is calculated and compared to the design runoff flow (Q). If the maximum flow is greater than the design flow, the channel will be stable. If not, the channel dimensions (e.g., hydraulic radius) need to be modified to reduce the channel velocity and increase the area of flow. This can be accomplished by increasing the bottom channel width or the side slope area.

After stability considerations have been addressed, pollutant removal efficiency should be investigated. Given the long-term mean runoff flow, which is defined as the mean storm rainfall intensity in inches/hour (see chapter 3) multiplied by a runoff coefficient and drainage area, and a trial flow depth, the Manning's roughness factor can be calculated. If this value is comparable to the Manning's roughness factor that is read from figure 13, then the trial flow depth is reasonable, and the average flow depth and travel time can be calculated; if not, a new trial depth must be selected. When the average flow depth and travel time are determined, figure 2 can be used to estimate long-term TSS removal. In turn, when the TSS removal value is known, removal values for copper, lead and zinc can be calculated as a percentage of TSS removal.

It should be emphasized that the metals removal estimates represent removal of metals associated with suspended solids. Therefore, if substantial infiltration or adsorption occurs in the channel, soluble metals should also be removed, and the removal efficiency should be higher than estimated as a percentage of TSS removal alone. On the other hand, suspended metals that are removed during small storm events may be resuspended during large events, meaning that the long-term removal estimates based upon figure 2 may be too high.

B. OVERLAND FLOW DESIGN

The design procedure for overland flow is somewhat different than for a grassed channel. Because flow depths in an overland flow system will be on the order of 0.1 to 0.2 in (0.01 to 0.02 ft) (0.31 to 0.61 cm), stability is not a major concern. Consequently, the focus of the discussion for overland flow design will be on travel time and associated pollutant removal.

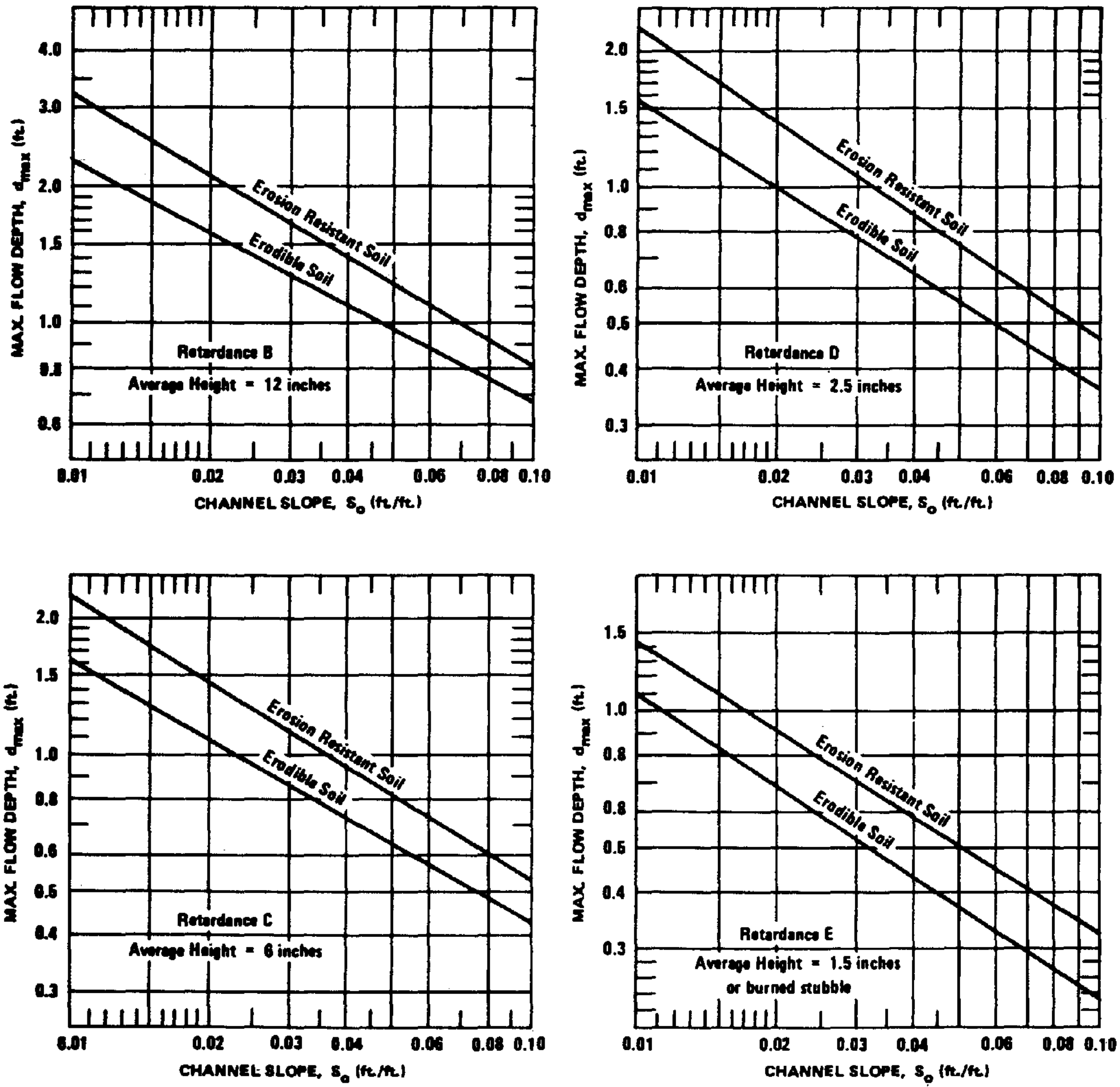
A key in the implementation of overland flow systems is to ensure that flow is being distributed uniformly to an area of established vegetation. A level spreader, defined as an excavated channel constructed at a zero gradient across a slope to convert channel flow to sheet flow, can be used

Table 6. Retardance factors for selected vegetation.

Retardance	Vegetation Type	Average Vegetation Height (in)
A	Weeping love grass (excellent stand)	30
	Yellow bluestem Ischaemum (excellent stand)	36
B	Kudzu (dense to very dense growth)	uncut
	Bermuda grass (good stand, tall)	12
	Native grass mixture - little bluestem, blue grama, and other long and short midwest grasses (good stand)	unmowed
	Weeping love grass (good stand, tall)	13-24
	Lespedeza sevicea (good stand, not woody, tall)	19
	Alfalfa (good stand)	11 (uncut)
	Blue grama (good stand)	13 (uncut)
C	Crabgrass (fair stand)	10-48 (uncut)
	Bermuda grass (good stand)	6 (mowed)
	Common lespedeza (good stand)	11 (uncut)
	Grass-legume mixture - summer - orchard grass, redtop, Italian ryegrass, and common lespedeza (good stand)	6-8 (uncut)
	Centipede grass (very dense cover)	6
	Kentucky bluegrass (good stand)	6-12 (headed)
D	Bermuda grass (good stand)	2.5
	Common lespedeza (excellent stand)	4.5
	Buffalo grass (good stand)	3-6 (uncut)
	Grass-legume mixture - fall, spring - orchard grass, redtop, Italian ryegrass, and common lespedeza (good stand)	4-5 (uncut)
	Lespedeza sericea (after cutting to 2" height - very good stand before cutting)	2
E	Bermuda grass (good stand)	1.5
	Bermuda grass (burned stubble)	

Source: Adapted from Normann 1975.

1 in = 25.4 mm

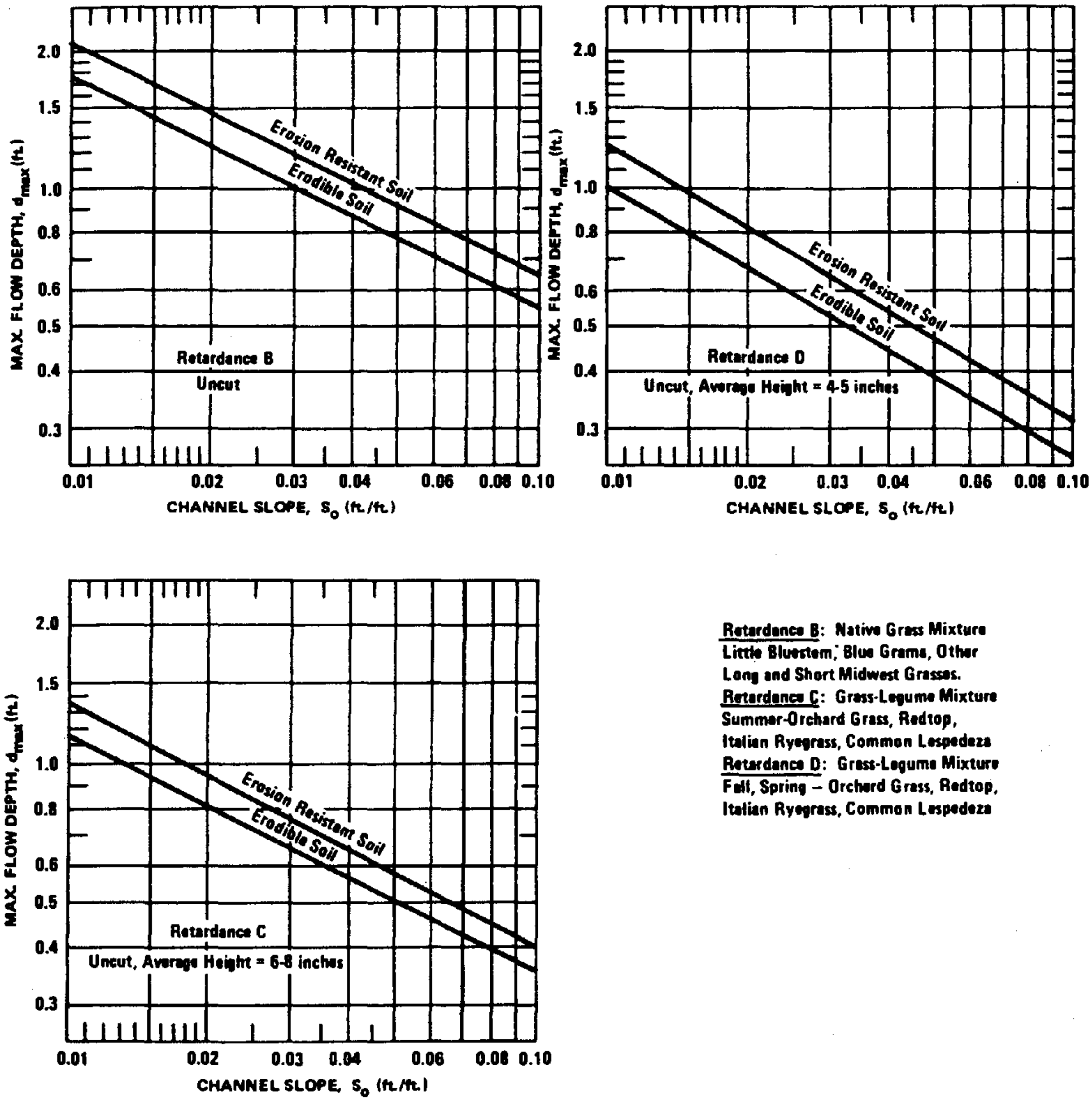


Note: Use on slopes steeper than 10 percent is not recommended.

To change ft to cm, multiply by 30.
To change in to cm, multiply by 2.5.

Source: Norman, 1975.

Figure 5. Maximum permissible depth of flow (d_{max}) for channels lined with bermuda grass, good stand, cut to various lengths.

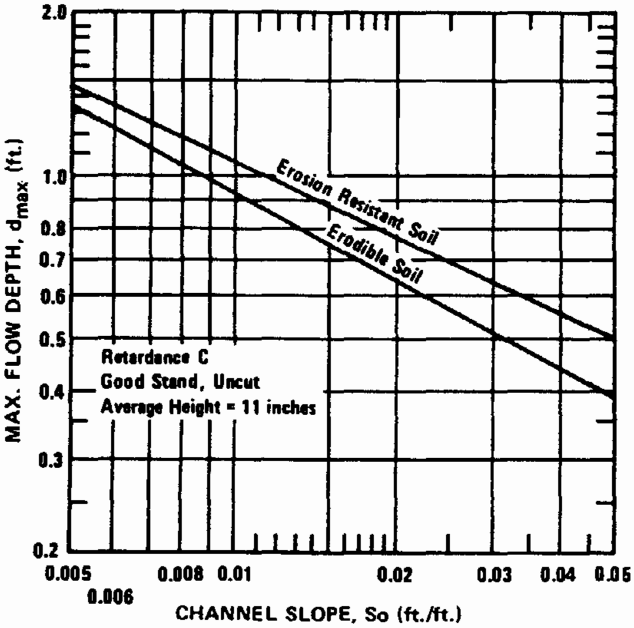


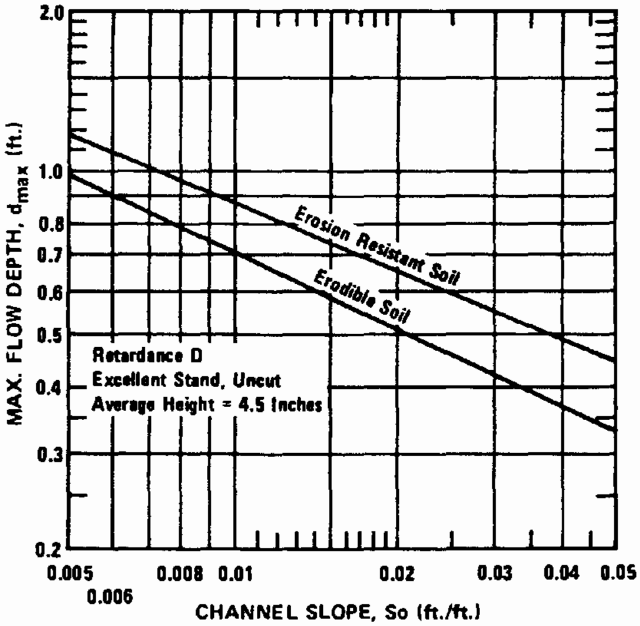
Note: Use on slopes steeper than 10 percent is not recommended.

To change ft to cm, multiply by 30.
To change in to cm, multiply by 2.5.

Source: Norman, 1975.

Figure 6. Maximum permissible depth of flow (d_{max}) for channels lined with grass mixtures, good stand, uncut.

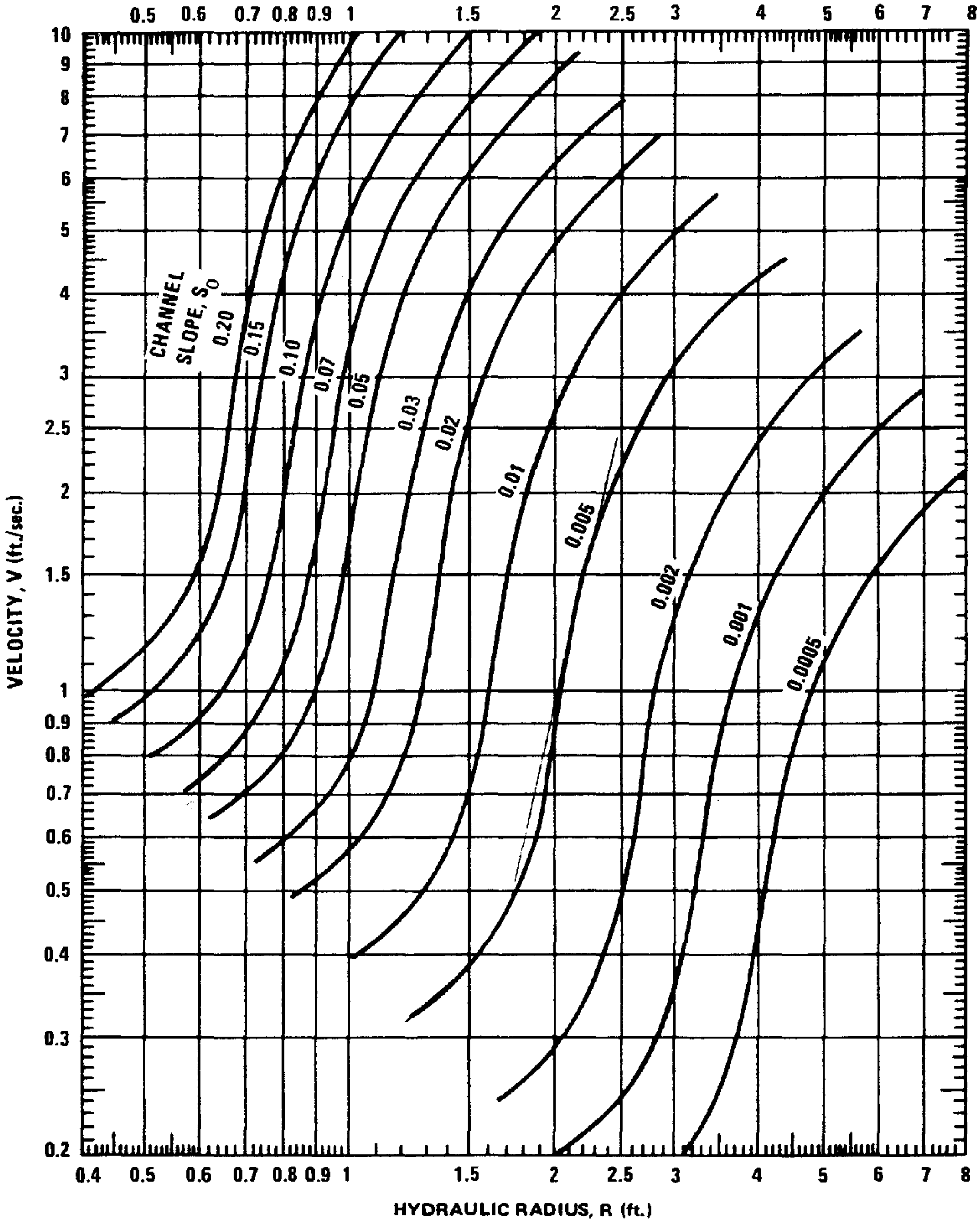




To change ft to cum, multiply by 30.
To change in to cm, multiply by 2.5.

Source: Norman, 1975.

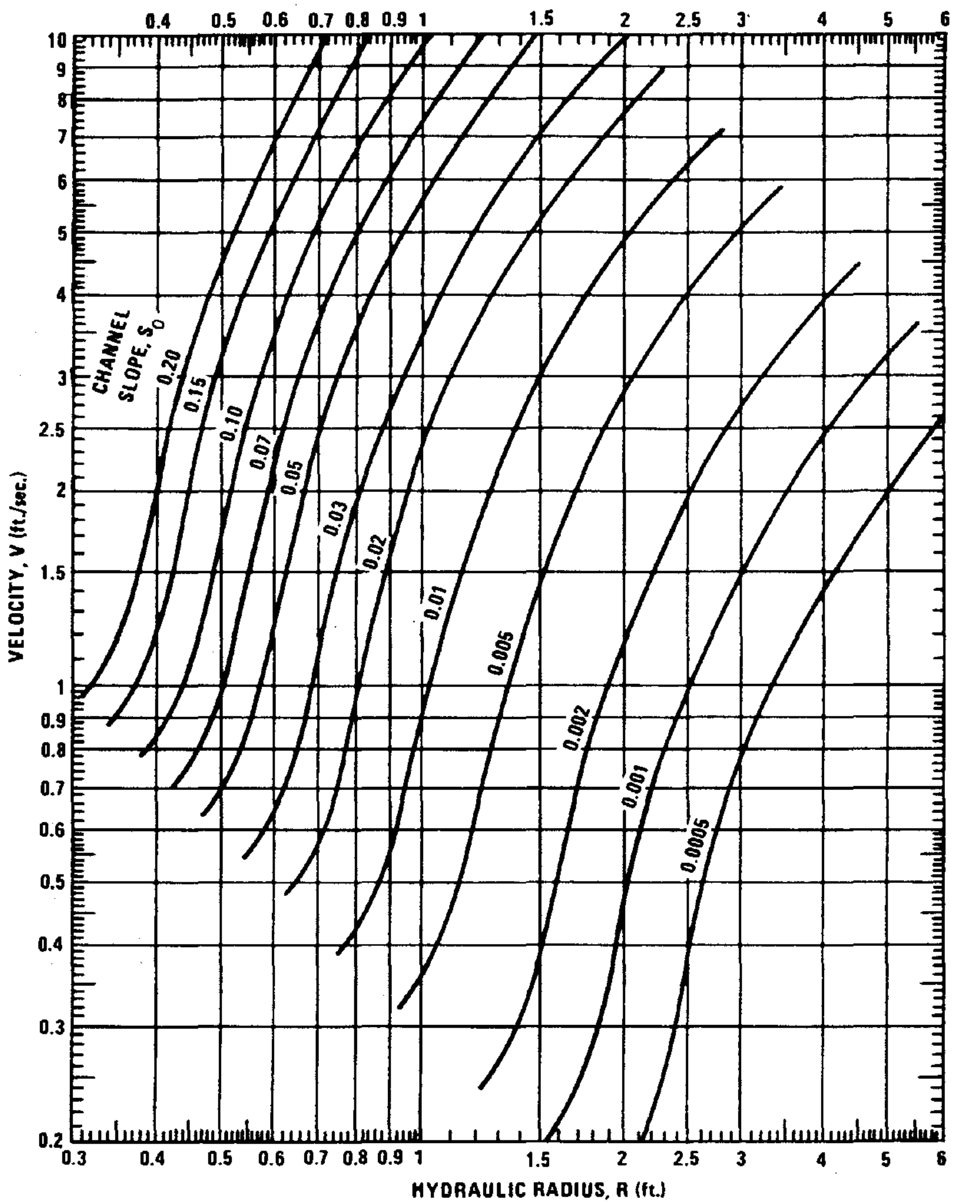
Figure 7. Maximum permissible depth of flow (d_{max}) for channels lined with common lespedeza of various lengths.



To change ft to cm, multiply by 30.
To change in to cm, multiply by 2.5.

Source: Norman, 1975.

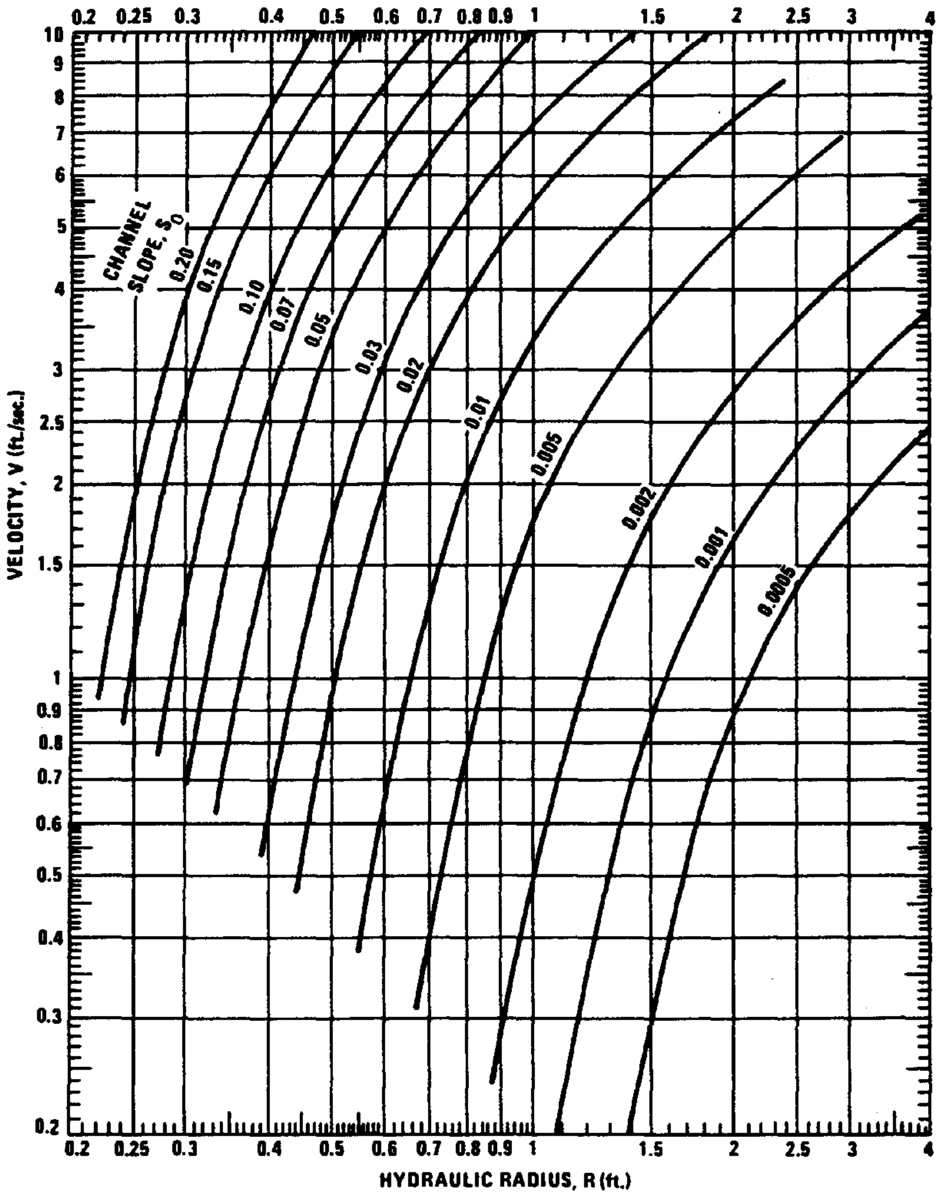
Figure 8. Flow velocity for channels lined with vegetation of retardance A.



To change ft to cm, multiply by 30.
To change in to cm, multiply by 2.5.

Source: Norman, 1975.

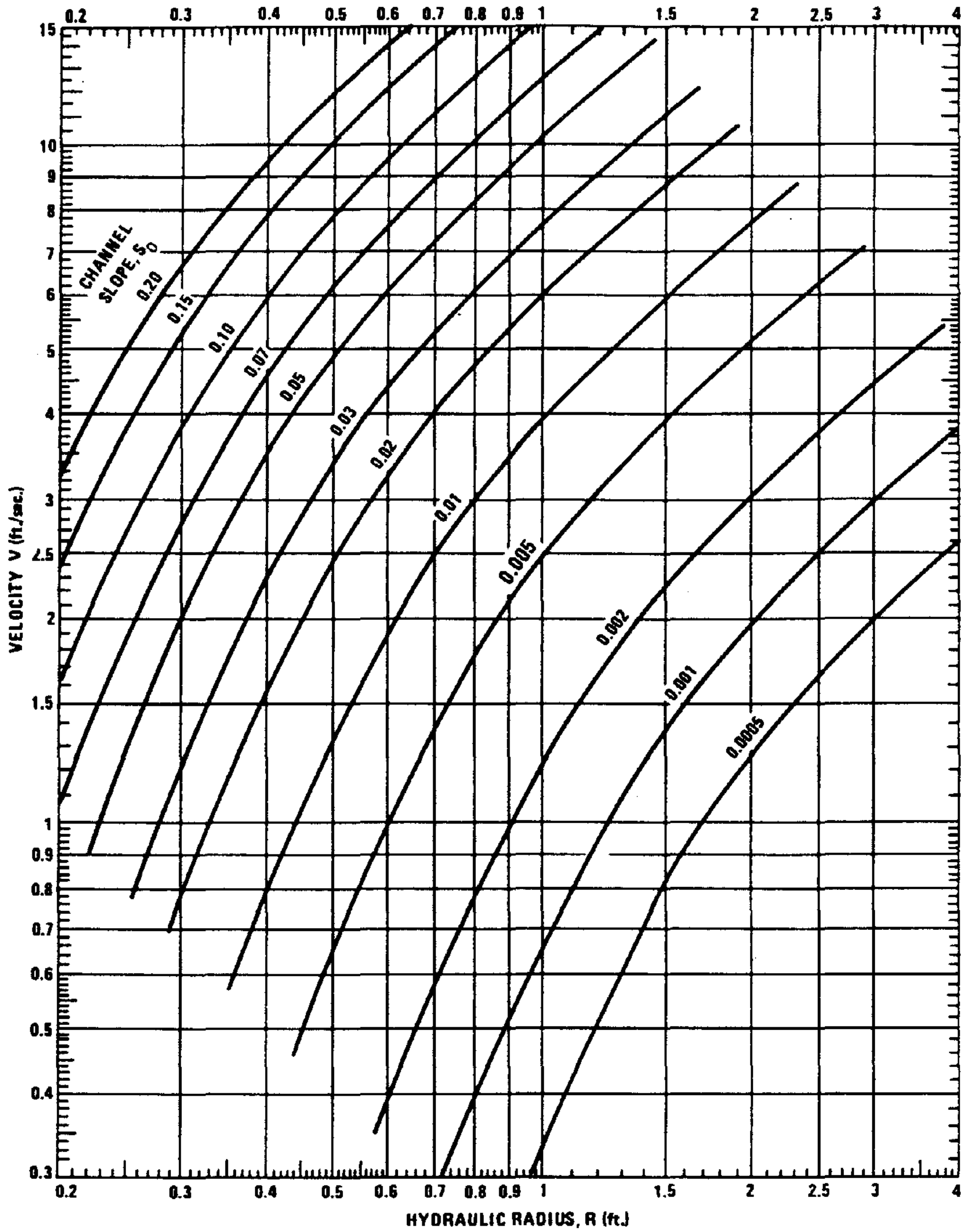
Figure 9. Flow velocity for channels lined with vegetation of retardance B.



To change ft to cm, multiply by 30.
To change in to cm, multiply by 2.5.

Source: Norman, 1975.

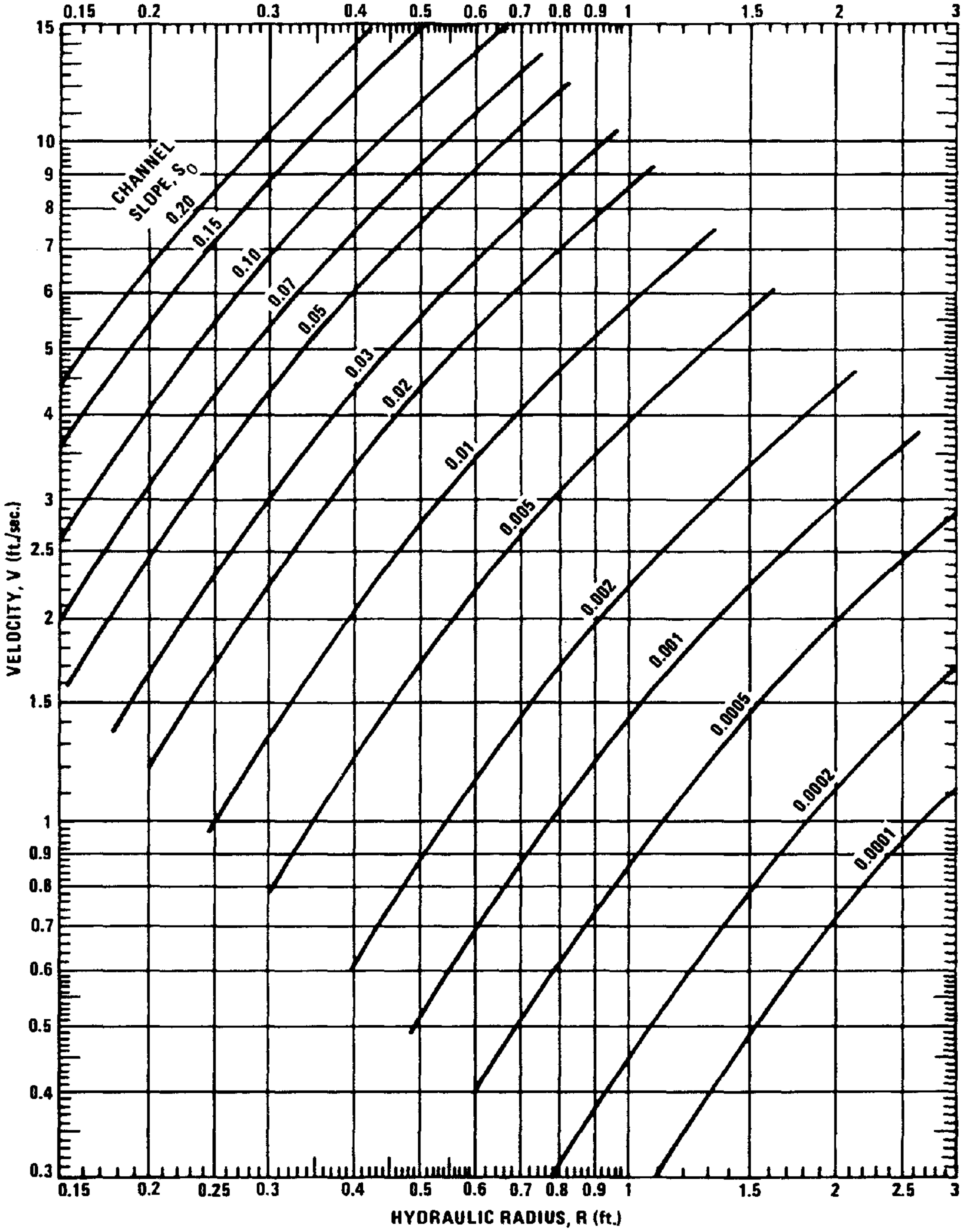
Figure 10. Flow velocity for channels lined with vegetation of retardance C.



To change ft to cm, multiply by 30.
To change in to cm, multiply by 2.5.

Source: Norman, 1975.

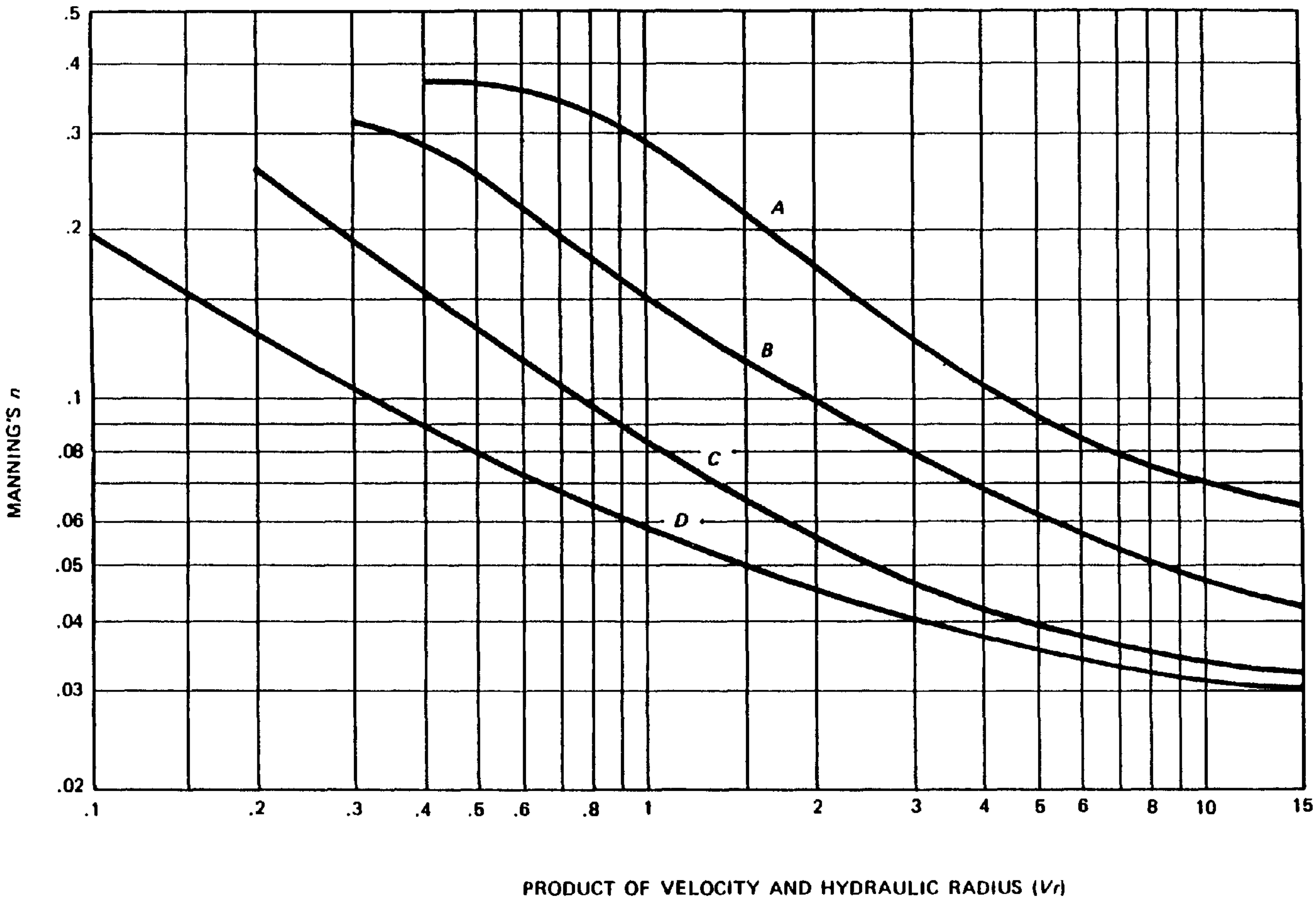
Figure 11. Flow velocity for channels lined with vegetation of retardance D.



To change ft to cm, multiply by 30.
To change in to cm, multiply by 2.5.

Source: Norman, 1975.

Figure 12. Flow velocity for channels lined with vegetation of retardance E.



Source: U.S. Soil Conservation Service, 1975.

Figure 13. Behavior of Manning's n in grassed channels for various retardance factors.

as part of the overland flow system. Flow from the overland system should be discharged to a stabilized conveyance channel (e.g., grassed channel or riprap-lined channel) in order to prevent erosion where the sheet flow again becomes concentrated.

A recent study documents the use of a level spreader to convert concentrated flow from a shopping center into sheet flow, which traveled approximately 150 ft (45.75 m) across a grassed area before discharging to an adjacent river. (Yu et al., 1987) Overall removal efficiencies of approximately 70 percent for TSS, 30 percent for TP, 25 percent for lead, and 50 percent for zinc were measured. Most of the removal occured in the first 70 ft (21.35 m) of travel.

Figure 14 is a worksheet that can be used for design of overland flow systems. The key data values in using the worksheet are the 1-year, 24-hour rainfall depth (which can be obtained from the U.S. Department of Commerce's Rainfall Frequency Atlas, also known as Technical Paper No. 40 (U.S. Department of Commerce, 1961)) and the overland flow length and slope. Using these three values, a travel time can be estimated from figure 15, which was developed using the TR-55 methodology (U.S. Soil Conservation Service, 1986) assuming a Manning's roughness coefficient of 0.20. For example, in an area with a 1-year/24-hour rainfall volume of 2.0 in (5.08 cm), a minimum overland flow length of 150 ft (45.75 m) is required to achieve a 15-minute travel time if the slope is 5 percent. After determining the travel time and estimating a flow depth (typically 0.01 to 0.02 ft) (0.31 to 0.61 cm), figure 2 can be used to estimate the average TSS removal efficiency, and average metals removal efficiencies can be estimated as a percentage of the TSS removal efficiency. An example of the design of an overland flow system is given in chapter 2, section 5.B.

As discussed previously, the removal efficiency values for metals are based on removal through sedimentation alone. Consequently, if substantial infiltration or adsorption occurs in the overland flow system, soluble metals will also be removed, and the removal efficiency will be higher than estimated as a percentage of TSS removal. On the other hand, suspended metals that are removed during small storm events may be resuspended during larger events, meaning that the long-term removal estimates based upon figure 2 may be too high.

Where an overland flow system experiences flows of long duration or where width is limited, a drain system may be required. Where flows of long duration are anticipated, the overland flow system should consist of a layer of topsoil supporting vegetation, an underlayer of sand, and a bottom layer of gravel. (Stenstrom et al., 1981) The gravel is intended as a drain and reservoir, keeping the upper layers from becoming saturated by enhancing the percolation of the runoff into the surrounding soils. In permeable soils with good porosity and drainage, the sand and gravel layers may not be necessary. However, in areas where the width of the overland flow area is limited, an underlying gravel layer may improve the pollutant removal efficiency by increasing infiltration.

OVERLAND FLOW SYSTEM DESIGN WORKSHEET

*******************************DESIGN DATA*********************************

Slope	______________________	(S)	ft/ft
Width	______________________	(B)	ft
Length	______________________	(L)	ft
Vegetation Type	______________________		
Vegetation Height	______________________		in
Soil Type (Circle One)	Erodible/Intermediate/Nonerodible		

**********DETERMINATION OF LONG-TERM POLLUTANT REMOVAL EFFICIENCY**********

1-year, 24-hour rainfall depth	______________________	(P)	in
Travel Time (from figure 15)	______________________	(T)	min
Assumed Flow Depth (typically .01-.02 ft)	______________________	(D)	ft
Long-term TSS Removal (from figure 2)	______________________	(E_{tss})	%
Long-term Pb Removal (90% of TSS removal)	______________________	(E_{pb})	%
Long-term Cu removal (60% of tss removal)	______________________	(E_{cu})	%
Long-term Zn removal (50% of TSS removal)	______________________	(E_{zn})	%

If pollutant removal efficiency is less than desired, increase length (L)

Figure 14. Overland flow design worksheet.

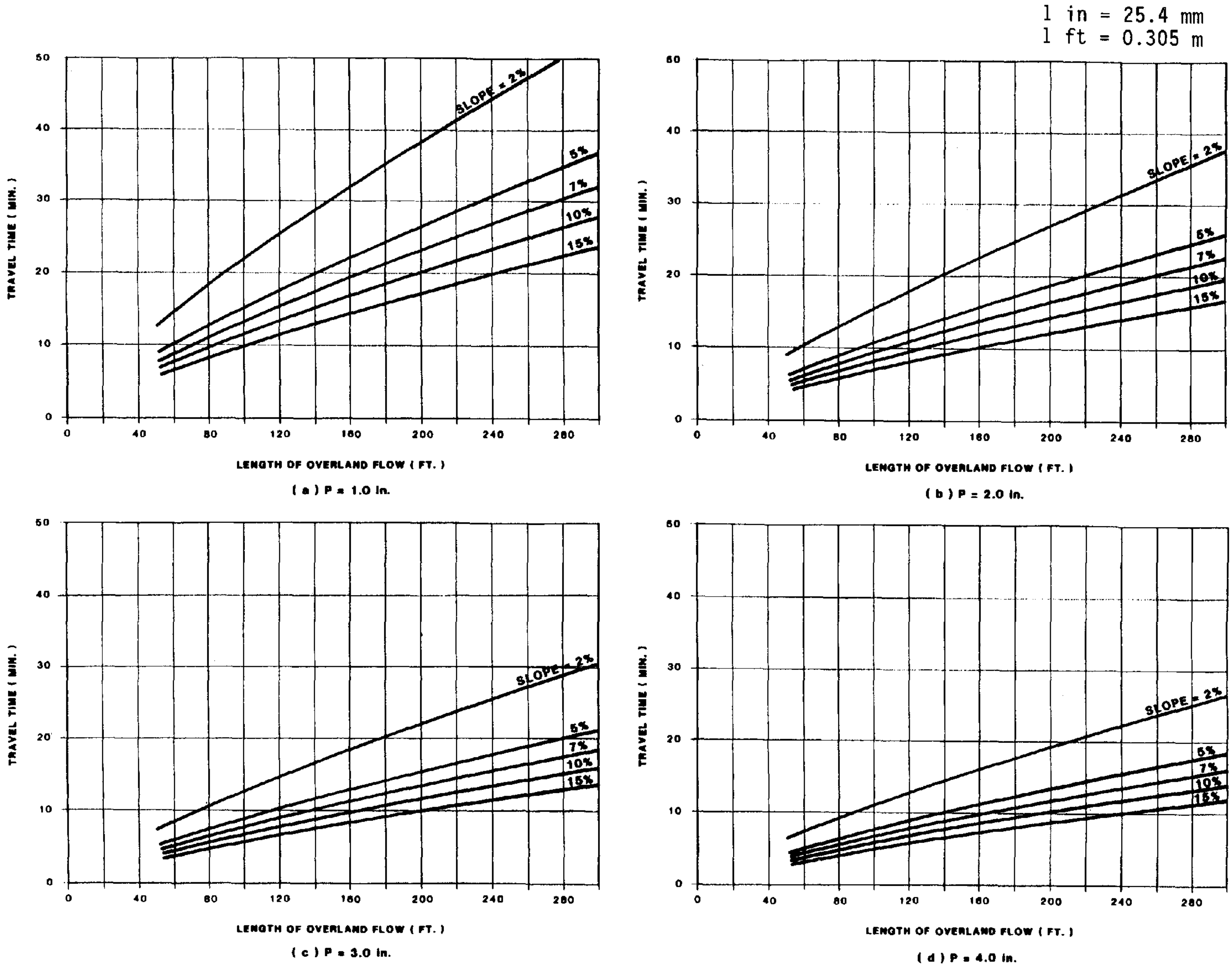


Figure 15. Travel times in overland flow systems for various 24-hour rainfall depths.

3. CONSTRUCTION CONSIDERATIONS

The most critical consideration in the construction of vegetative controls is maintaining the stability of the channel or slope. The stability is dependent on ensuring that proper grade control and contour procedures are followed and that slopes are not excessive. Chapter 2, section 1, discusses vegetation establishment and chapter 2, section 2, provides procedures for determining the need for erosion protection during establishment.

Other considerations involve the pollutant removal capabilities of the vegetative controls. For example, the incorporation of organic material into the soil prior to establishing vegetation will increase the adsorption capability of the vegetative control. In addition, tilling the soil before establishing vegetation will increase the soil's infiltration capacity, which will result in more effective pollutant removal.

4. OPERATION AND MAINTENANCE

The basic objective in the maintenance of vegetative controls is to promote the healthy growth of the established vegetation. Procedures involved in this maintenance include routine mowing, removal of grass clippings and debris, and removal of accumulated sediment.

Maintenance must also include the prompt repair of channels or overland flow systems with erosion problems. Several studies indicate that metals that are removed by grassed swales and detention basins tend to accumulate in the upper 5 to 10 cm of the sediments. (Wigington et al., 1986; Yousef, Wanielista and Harper, 1986; WASHCOG, 1983) Consequently, erosion from a grassed channel or overland flow system could carry substantial loads of metals and other pollutants in addition to the solids load. As a result, repairs by seeding or sodding should be made swiftly to maintain the vegetative cover.

Based on a recent monitoring study of three grassed channels receiving highway runoff, it is recommended that grassed channels be cleaned out once every 5 years at a minimum and that the sediment spoils be disposed of at an acceptable secure location. The monitoring data indicate that an inch or more of sediment can be expected to accumulate in the channel over a 5-year period. If this sediment is resuspended and carried downstream during a storm event, the channel will act as a source of pollution rather than a sink. The data for the Virginia monitoring site indicated metals accumulations of 5 years or less in channel bottom sediments, even though the channels had been in service for over 20 years. These monitoring results suggest that the grassed channels at these sites had been scoured out periodically due to lack of maintenance. Unless the grassed channels are cleaned out periodically, they will eventually become a source of runoff pollution loadings rather than a BMP.

5. DESIGN EXAMPLES

This section contains two design examples for vegetative controls. The first is the design of a grassed channel and the second is the design of an overland flow system.

A. GRASSED CHANNEL EXAMPLE

As shown in figure 16, the example problem considers a relatively straight section of a four-lane highway. The 10-year design runoff flow in the roadside ditches is calculated to be 8 ft^3/s (0.23 m^3/s). The channel is characterized by the following design parameters:

- Length of 200 ft (61.00 m).
- Bottom width of 4 ft (1.22 m).
- Side slopes of 4:1 horizontal-to-vertical.
- Bottom slope of 0.05 ft/ft (0.05 m/m).
- Erodible soil.
- Bermuda grass at a height of 6 in (15.24 cm).

The example shows how to determine if the channel will be stable at the design flow, and how effectively the channel will remove suspended solids and several metals. Results indicate that the channel will be stable at the design flow, and is expected to remove 73 percent of the total suspended solids, 66 percent of the lead, 44 percent of the copper, and 37 percent of the zinc entering the channel. If this is not considered sufficient, the designer would adjust the original design (e.g., increase the base width or channel length) to make the channel more effective.

B. OVERLAND FLOW EXAMPLE

As shown in figure 17, the example problem considers a relatively straight section of a four-lane highway. Runoff from the highway goes over the roadway shoulder, and travels by sheet flow to a small riprap-lined channel. The overland flow area is characterized by the following design parameters:

- Width of 300 ft (91.50 m).
- Length of 150 ft (45.80 m).
- Slope of 0.05 ft/ft (0.05 m/m).
- Erodible Soil.
- Lespedeza grass at a height of 4.5 in (11.40 cm).
- 1-year, 24-hour rainfall of 2.0 in (5.08 cm).

Figure 15 was used to select the overland flow length, to ensure that the design provided a 15-minute travel time at a 5 percent overland slope and a 1-year, 24-hour rainfall of 2.0 in (5.08 cm).

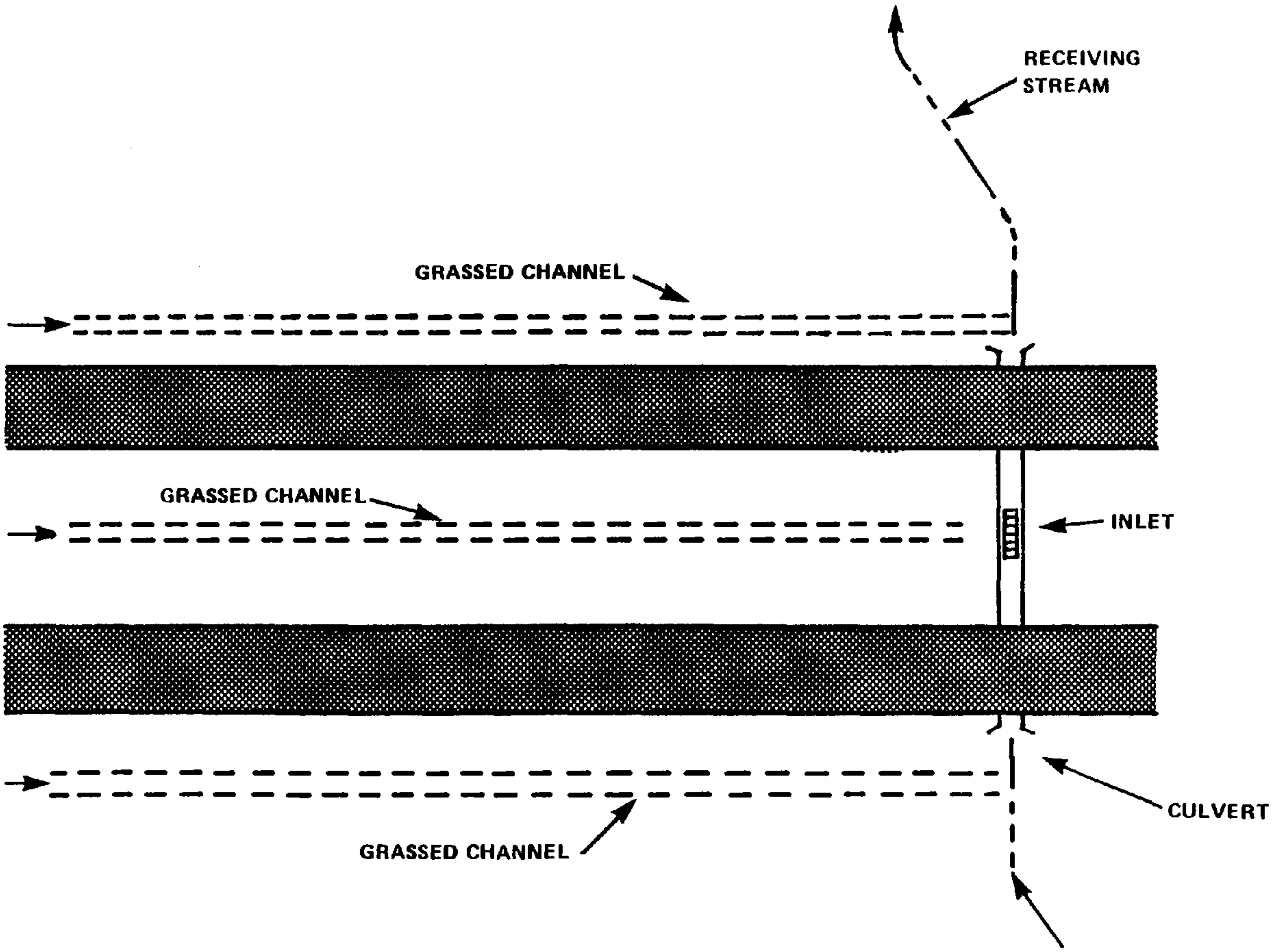


Figure 16. Grassed channel design example.

GRASSED CHANNEL DESIGN WORKSHEET
(for triangular or trapezodial cross sections)

********************************DESIGN DATA**********************************

Design Runoff Flow	8	(Q)	ft^3/s
Channel Slope	0.05	(S)	ft/ft
Channel Length	200	(L)	ft
Bottom Channel Width	4	(B)	ft
Side Slope Ratio	4	(Z)	
Vegetation Type	Bermuda grass		
Vegetation Height	6		in
Soil Type (Circle One)	Erodible/Intermediate/Nonerodible		

*********************DESIGN PROCEDURES FOR STABILITY**********************

Retardance (from table 6)	C		
Maximum Depth of Flow (from figures 5 to 7)	0.63	(D_{max})	ft
Flow Area ($A = D_{max} \times (B + (Z \times D_{max})))$	4.11	(A)	ft^2
Wetted Perimeter ($P = B + (2 \times D_{max})\ \ 1 + Z^2$)	9.20	(P)	ft
Hydraulic Radius ($R = A/P$)	0.45	(R)	ft
Velocity (from figures 8 to 12)	2.5	(V)	ft/s
Maximum Flow ($Q_{max} = A \times V$)	10	(Q_{max})	ft^3/s

If Qmax > Q then channel will be stable. If not, increase bottom width, decrease longitudinal slope or increase side slope to increase the area of flow or to reduce velocity.

Figure 16. Grassed channel design example (continued).

GRASSED CHANNEL DESIGN WORKSHEET
(CONTINUED)

**********DETERMINATION OF LONG-TERM POLLUTANT REMOVAL EFFICIENCY**********

Long-term Mean Runoff Event Flow	0.16	(Q_a)	ft^3/s
Trial Centerline Flow Depth	.15	(D)	ft
Flow Area (A = D x (B + (Z x D)))	.69	(A)	ft^2
Wetted Perimeter (P = B+ (2 x D) 1 + Z ^{2})	5.24	(P)	ft
Hydraulic Radius (R = A/P)	.13	(R)	ft
Manning's n ($n = 1.49 \times A \times R^{2/3} \times S^{1/2}/Q_a$)	.37	(n)	
Velocity ($V = Q_a/A$)	.23	(V)	ft/s

If calculated Manning's n is in reasonable agreement with Manning's n from figure 13 as a product of V and R, then go on to next step; otherwise, revise trial depth and repeat calculations.

Average Depth	.13	(D_{avg})	ft
(D_{avg} = A/(B+ (2xDxZ)))			
Travel Time T = L/(60 x V))	14.5	(T)	min
Long-term TSS Removal (from figure 2)	73	(E_{TSS})	%
Long-term Pb Removal (90% of TSS removal)	66	(E_{Pb})	%
Long-term Cu Removal (60% of TSS removal)	44	(E_{Cu})	%
Long-term Zn Removal (50% of TSS removal)	37	(E_{Zn})	%

Figure 16. Grassed channel design example (continued).

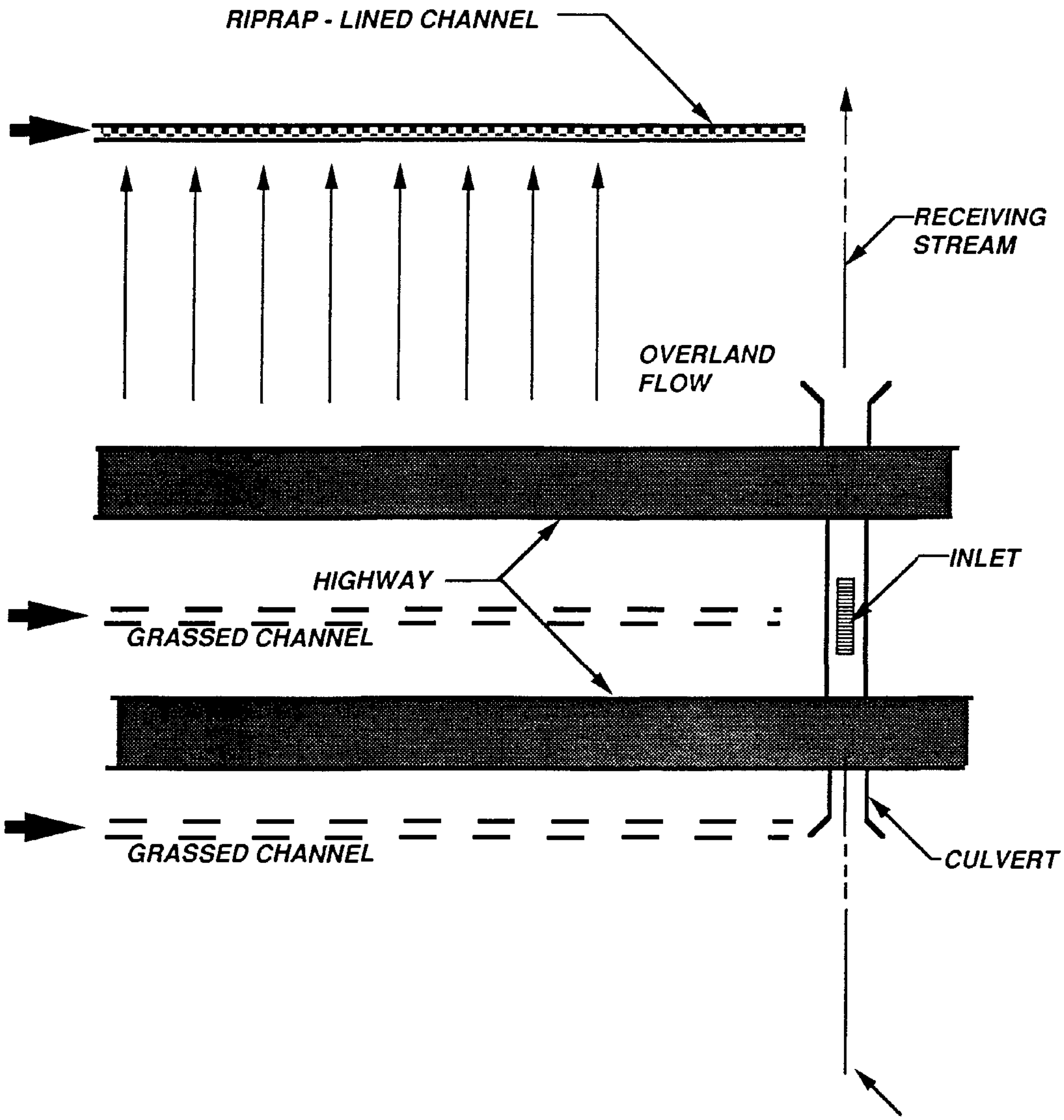


Figure 17. Overland flow design example.

OVERLAND FLOW SYSTEM DESIGN WORKSHEET
(CONTINUED)

*******************************DESIGN DATA*********************************

Slope	0.05	(S)	ft/ft
Width	300	(B)	ft
Length	150	(L)	ft
Vegetation Type	Common lespedeza		
Vegetation Height	4.5		in
Soil Type (Circle One)	Erodible/Intermediate/Nonerodible		

**********DETERMINATION OF LONG-TERM POLLUTANT REMOVAL EFFICIENCY**********

1-Year, 24-hour Rainfall Depth	2.0	(P)	in
Travel Time (from figure 15)	15	(T)	min
Assumed Flow Depth (typically 0.01-0.02 ft)	0.01	(D)	ft
Long-term TSS Removal (from figure 2)	87	(E_{TSS})	%
Long-term Pb Removal (90% of TSS Removal)	78	(E_{Pb})	%
Long-term Cu Removal (60% of TSS Removal)	52	(E_{Cu})	%
Long-term Zn Removal (50% of TSS Removal)	44	(E_{Zn})	%

If pollutant removal efficiency is less than desired, increase length (L)

Figure 17. Overland flow design example (continued).

The example shows how to determine how effectively the system will remove suspended solids and selected metals. Based on figure 1, the system is expected to remove 87 percent of the suspended solids, 78 percent of the lead, 52 percent of the copper, and 44 percent of the zinc discharged into the overland flow area. If this is not considered sufficient, the designer would adjust the original design (e.g., increase the flow length) to make the system more effective.

3. WET DETENTION BASINS

Depending upon highway runoff control needs and site conditions, wet detention basins may be the most practical and effective stormwater runoff management measure for pollution abatement. Detention is a highly effective management measure for pollutant removal if sufficient detention time is provided to permit biological uptake of nutrients and sedimentation of particulates in the stormwater runoff. Performance of basins that retain a pool of water has been found to range from poor to excellent, depending on the size of the basin relative to the size of the drainage area served, and runoff characteristics of the area. The principal mechanism for the removal of particulate forms of pollutants in wet basins is sedimentation. Further, wet detention basins can achieve substantial reductions in soluble nutrients due to biological and physical/chemical processes within the permanent pool.

1. DESIGN CONSIDERATIONS

There has been little success in characterizing the performance of detention basins for individual storm events due to the storm-to-storm variability in runoff volumes, pollutant loads, and concentrations in runoff from individual events. Detention basins typically exhibit variable performance characteristics depending on the size and characteristics of the storm, pollutants and the size distribution of pollutants transported by the runoff, and the volume of stormwater runoff processed by the basin.

Consequently, characterizations of wet detention basins are typically based upon the average pollutant removal efficiency across a wide range of storms. This characterization provides the best measure of performance, and is appropriate for evaluating receiving water quality benefits over an extended period of time, as in determining the impacts of nutrient load on lakes.

Two different approaches are recommended here to evaluate the average performance of wet detention basins and to formulate the design procedures. One approach relies upon solids settling theory and assumes that all pollutant removal within the basin is due to sedimentation. (EPA, 1983; Driscoll, 1983) The other approach views the wet detention basin as a lake achieving a controlled level of eutrophication, in an attempt to account for biological and physical/chemical processes that have been documented as the principal nutrient removal mechanisms. (Hartigan, 1988; Walker, 1987) The solids settling theory approach is most appropriate for situations where the control of heavy metals and other toxicants in highway runoff is the principal objective. This is because metals and other toxicants can be removed by sedimentation within the detention basin. The controlled level of eutrophication approach is most appropriate for situations where the principal concern is the control of nutrient loadings in highway runoff which can range from 0.1 to 14.0 mg/L for total kjeldahl nitrogen and 0.05 to 3.55 mg/L for phosphate - phosphorus (see table 1). This is because removal mechanisms for dissolved phosphorus and nitrogen are usually required to achieve effective control of nutrient loadings. The removal of the dissolved nutrient fraction typically represents the majority of total P and total N removal in wet detention basins. For example, the portion of total P removal efficiency attributable to dissolved P removal was on the order of 60 percent to 85 percent for several wet detention basins monitored during EPA's Nationwide Urban Runoff Program (NURP). This suggests that solids settling theory alone does not account for the most important nutrient removal mechanisms in wet detention basins. (Hartigan, 1988) Both approaches suggest that pollutant removal efficiency should be positively related to hydraulic residence time, although the controlled level of eutrophication approach results in greater storage capacities and longer residence times.

A. SOLIDS SETTLING METHOD

The first approach, based on settling theory, should be used where only particulate pollutant control is required and where nutrient removal is not required for protection of the receiving water. Driscoll reported a procedure based on a probabilistic analysis methodology used to compute long-term average performance from the statistical properties of detention basin inflows. (Driscoll, 1983) The analysis assumes that overall performance is due to the combined effects of removal under dynamic conditions as flows move through a basin and under quiescent conditions between storms. The methodology was tested against observed performance and monitored storm events from the NURP data base of 5 to 30 or more separate storm events at each of 13 detention basins. The procedure presented here is an adaptation of the methodology reported by Driscoll. It may be used to estimate long-term efficiency of wet detention basins or to estimate the dimensions of proposed basins to achieve desired removal rates. The procedure is not applicable to dry basins, and it cannot be used to size basins for peak flow attenuation for flood flow management.

B. CONTROLLED EUTROPHICATION METHOD

The second approach, based on a controlled level of eutrophication, is most appropriate for areas where the receiving water quality problem is caused by nutrient loadings. Since nutrients typically required extended hydraulic residence times to cause a serious receiving water quality problem, examples of situations where nutrient control is needed include watersheds of reservoirs, lakes, tidal embayments, and estuaries.

The procedure adopted here is an application of a phosphorus-limited lake eutrophication model. (Walker 1985, 1987) The lake eutrophication model requires the mean total phosphorus concentration in highway runoff discharged into the wet detention basin. Highway runoff event data show that for urban highways with average daily traffic (ADT) level of more than 30,000 vehicles per day, the median total phosphorus concentration in highway runoff is 0.40 mg/L. For rural highways with an ADT level of less than 30,000 vehicles per day, the median total phosphorus concentration is 0.16 mg/L. (Woodward-Clyde, 1987) Data from almost 1,000 separate highway runoff events monitored at 31 sites in 11 states were the basis for estimating the runoff concentrations. The highway runoff sites monitored to support the development of these design guidelines produce similar concentrations for urban and rural highways. Two grass channels had average total phosphorus inflow concentrations of 0.2 mg/L for 30,000 and 42,000 vehicles per day. The third grass channel site had an average total phosphorus inflow concentration of 0.6 mg/L for 67,000 vehicles per day.

Three detention basins exhibited average total phosphorus inflow concentrations of 0.4 mg/L, 0.4 mg/L, and 0.5 mg/L for 41,000, 51,000, and 71,000 vehicles per day, respectively.

Because the lake eutrophication model design method accounts for the biological uptake of dissolved nutrients, it produces a design which is more appropriate for nutrient control than the solids settling design method. (Hartigan, 1988) The permanent pool storage resulting from a controlled eutrophication design is on the order of two to three times larger than a design based on the solids settling model, depending upon the pollutant removal goals.

C. OTHER PERFORMANCE FACTORS

Several factors other than size and detention time influence the trap efficiency of detention basins. Among these are the stability of the banks and the distance between the location where flow enters the basin, the location of the outlet, and the outlet configuration.

Bank erosion increases the suspended sediment load entering the basin and decreases performance. Steep slopes are susceptible to erosion from wave action and from overland flow. Bank slopes should be 3:1 or flatter, and grass should be maintained on the slopes, to the extent practicable. Inlet and outlet structures should be designed to minimize erosion, which would adversely affect basin performance.

"Short-circuiting," or the failure of influent to thoroughly mix with water in the basin before discharge through the outlet structure, is another commonly recognized problem. Design aids presented in this report were developed assuming relatively poor performance with respect to short-circuiting; therefore, efficiency estimates derived by use of the design aids of this section should be conservative for most basin designs. A number of design alternatives can be used to reduce the effects of short-circuiting, including: the use of baffles, islands, or other devices to spread the inflow and increase the length of the flow path, increasing the distance between the inlet and outlet by the use of an irregular or convex shoreline; and use of an elongated, narrow basin as opposed to a nearly square basin. Designs that cause the influent to mix with water in the basin will reduce short-circuiting and improve pollutant removal efficiencies. Figure 18 illustrates some designs that increase the length of the flow path from inlet to outlet in order to reduce short-circuiting.

D. MONITORED POLLUTANT REMOVAL EFFICIENCIES FOR WET DETENTION BASINS

The EPA NURP monitored several wet detention basins draining small urban watersheds in different locations throughout the U.S. (USEPA, 1983) For wet detention basins with significant average hydraulic residence times (e.g., 2 weeks or greater), average pollutant removal rates were on the order of 50 to 60 percent for total P and 30 to 40 percent for total N. For other pollutants which are removed primarily by sedimentation processes, the average removal rates were as follows: 80 to 90 percent for TSS; 70 to 80 percent for lead; 40 to 50 percent for zinc; and 20 to 40 percent for BOD or chemical oxygen demand (COD).

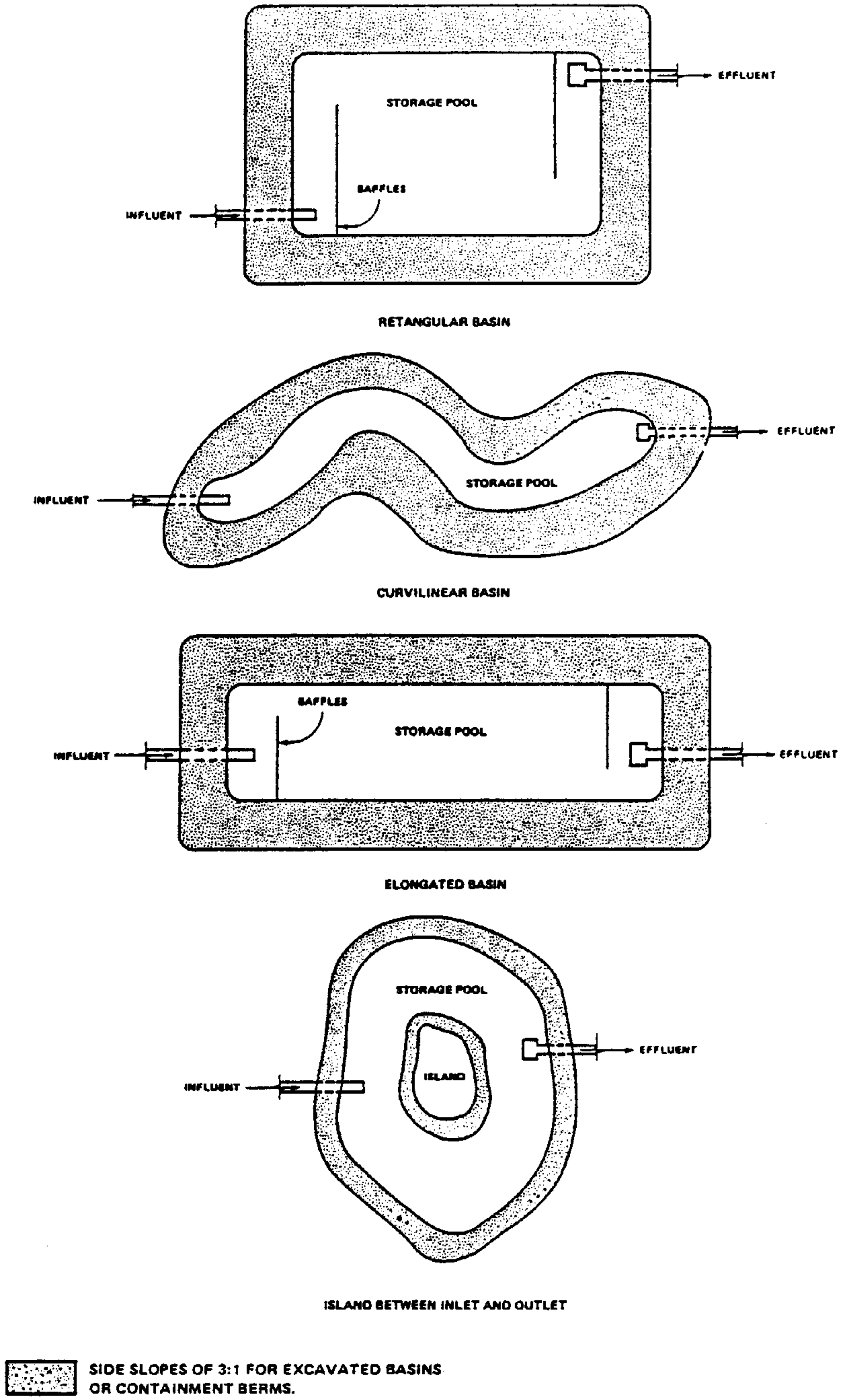


Figure 18. Examples of wet detention basins designed to increase retention time and limit short-circuiting.

To assist with preparation of this design guidelines report, wet detention basins draining highway sites in Connecticut, Minnesota, and Florida were monitored in the field. The average pollutant removal rates achieved by these wet detention basins were as follows: 10 to 60 percent for total P; 25 to 35 percent for TKN; 70 to 80 percent for lead; 20 to 65 percent for zinc; and 30 to 55 percent for copper. The wet detention basins with the greatest average hydraulic residence times typically achieved the greatest nutrient removal efficiencies. Thus, field studies of detention basins designed for highway runoff control documented pollutant removal rates similar to those found by previous field studies of detention basins which control runoff from other urban land uses. (USEPA, 1983)

2. DESIGN PROCEDURES FOR CONTROL OF TSS AND TOXICANTS (SOLIDS SETTLING METHOD)

For control of highway runoff TSS and toxicants found primarily in the suspended form, such as lead, copper, and zinc, wet detention basins rely on settling as the primary pollutant removal mechanism. Rainfall and runoff characteristics, settling velocities for suspended solids in the runoff, and the distribution of particulates and pollutants in each particle size range are needed to design wet detention basins according to the solids settling method.

The following design procedure for wet detention basins was developed for national rainfall zones delineated in figure 19 with rainfall characteristics summarized in table 7. It is recommended that the long-term mean and the coefficients of variation of rainfall event volumes, durations, intensities, and intervals between the midpoints of successive events be developed for the local area in which the highway runoff BMP is to be constructed. (See chapter 3, section 2.A, RAINFALL CHARACTERISTICS) Appendix A provides a procedure for developing design curves for areas where the rainfall statistics differ from those of table 7.

The following steps are required to use the design procedure for wet detention basins to control particulate pollution:

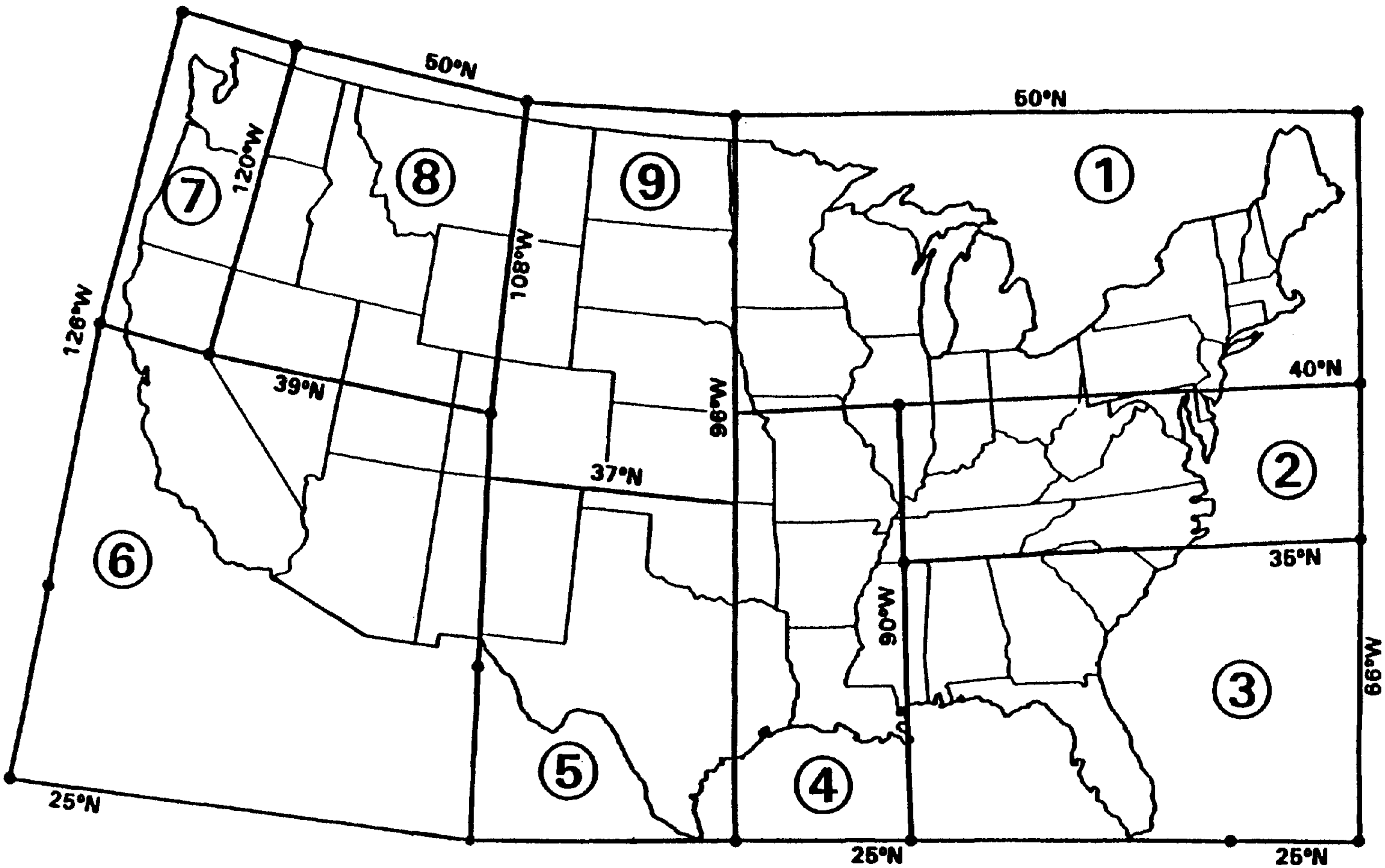
1. In the absence of local rainfall statistics, identify the rainfall zone in which the detention basin will be located from figure 19.

2. Determine the long-term mean rainfall volume (depth) for the appropriate zone by converting the value in table 7 from inches to feet.

3. Establish the runoff coefficient (see chapter 3, section 2.C, RUNOFF COEFFICIENT).

4. Compute the long-term mean event runoff volume (see chapter 3, section 2.D, RUNOFF VOLUME).

5. Establish the dimensions of the area in which a wet detention basin could be constructed.



Source: Driscoll, 1985.

Figure 19. Rainfall zones.

Table 7. Summary of rainfall characteristics.

Zone	Period	Rainfall statistics							
		Volume (in)		Intensity (in/hr)		Duration (hr)		Interval (hr)	
		Mean	υv	Mean	υi	Mean	υD	Mean	$\upsilon \Delta$
1	Annual	0.26	1.46	0.051	1.31	5.8	1.05	73	1.07
	Summer	0.32	1.38	0.082	1.29	4.4	1.14	76	1.06
2	Annual	0.36	1.45	0.066	1.32	5.9	1.05	77	1.05
	Summer	0.40	1.57	0.101	1.37	4.2	1.09	77	1.08
3	Annual	0.49	1.47	.102	1.28	6.2	1.22	89	1.05
	Summer	0.48	1.52	.133	1.34	4.9	1.33	68	1.01
4	Annual	0.58	1.46	.097	1.35	7.3	1.17	99	1.00
	Summer	0.52	1.54	.122	1.35	5.2	1.29	87	1.06
5	Annual	0.33	1.74	.080	1.37	4.0	1.07	108	1.41
	Summer	0.38	1.71	.110	1.39	3.2	1.08	112	1.49
6	Annual	0.17	1.51	.045	1.04	3.6	1.02	277	1.48
	Summer	0.17	1.61	.060	1.16	2.6	1.01	425	1.26
7	Annual	0.48	1.61	0.024	0.84	20.0	1.23	101	1.21
	Summer	0.26	1.35	0.027	1.11	11.4	1.20	188	1.15
8	Annual	0.14	1.42	.031	0.91	4.5	0.92	94	1.39
	Summer	0.14	1.51	.041	1.13	2.8	0.80	125	1.41
9	Annual	0.15	1.77	.036	1.35	4.4	1.20	94	1.24
	Summer	0.18	1.74	.059	1.44	3.1	1.14	78	1.31

Source: Driscoll 1985.

6. Compute approximate detention basin volumes for trial retained pool depths and the ratio, basin volume/runoff volume, for each trial depth. Use the volume of the permanent pool in computing the ratio, basin volume/runoff volume.

7. Enter figure 20, 21, 22, 23, or 24, as appropriate, and use the trial values computed in step 5 to select the design which will perform as desired. Alternately, enter the appropriate figure with the desired TSS removal efficiency and find the required depth of the permanent pool in the basin and the detention basin volume/runoff volume ratio to achieve that efficiency.

8. Estimate pollutant removal efficiencies for pollutants of concern from percentages given in chapter 3, section 2.G, POLLUTANT REMOVAL EFFICIENCIES. Steps 7 and 8 may be used in reverse order if the objective is to achieve a specified removal efficiency for a particular pollutant.

9. Design the basin configuration to minimize the potential for short-circuiting (chapter 3, section 1). If this is not practicable, choose a more conservative design, i.e., choose a design which will yield higher pollutant removal efficiencies to compensate for possible adverse effects from short-circuiting.

10. Bank slopes for embankment basins will follow the natural contours or should be graded for slopes at 3:1 (H:V) or flatter. For excavated basins, design all basin bank slopes at 3:1 (H:V) or flatter, specify grass cover for areas not in the retained pool of water.

A wet detention basin design worksheet for the solids settling model approach is presented in figure 25.

A. RAINFALL CHARACTERISTICS

The summary of rainfall characteristics in table 7 and the rainfall zones in figure 19 were used to develop the design curves presented in Figures 20 through 24. Statistical parameters other than the rainfall volume (depth) are accounted for in the design curves. The method reported in appendix A was used to develop the design curves. The mean event rainfall depth from table 7 should be converted from inches to feet or meters, as appropriate.

Curves in figures 20 through 24 were developed to provide conservative designs, but rainfall statistics should be developed for the area in which the management measure is to be located. Hourly precipitation data for every first-order U.S. weather station are available from the National Weather Service Data Center. A procedure is provided in appendix A for developing design curves applicable to a specific locale.

1 ft = 0.305 m

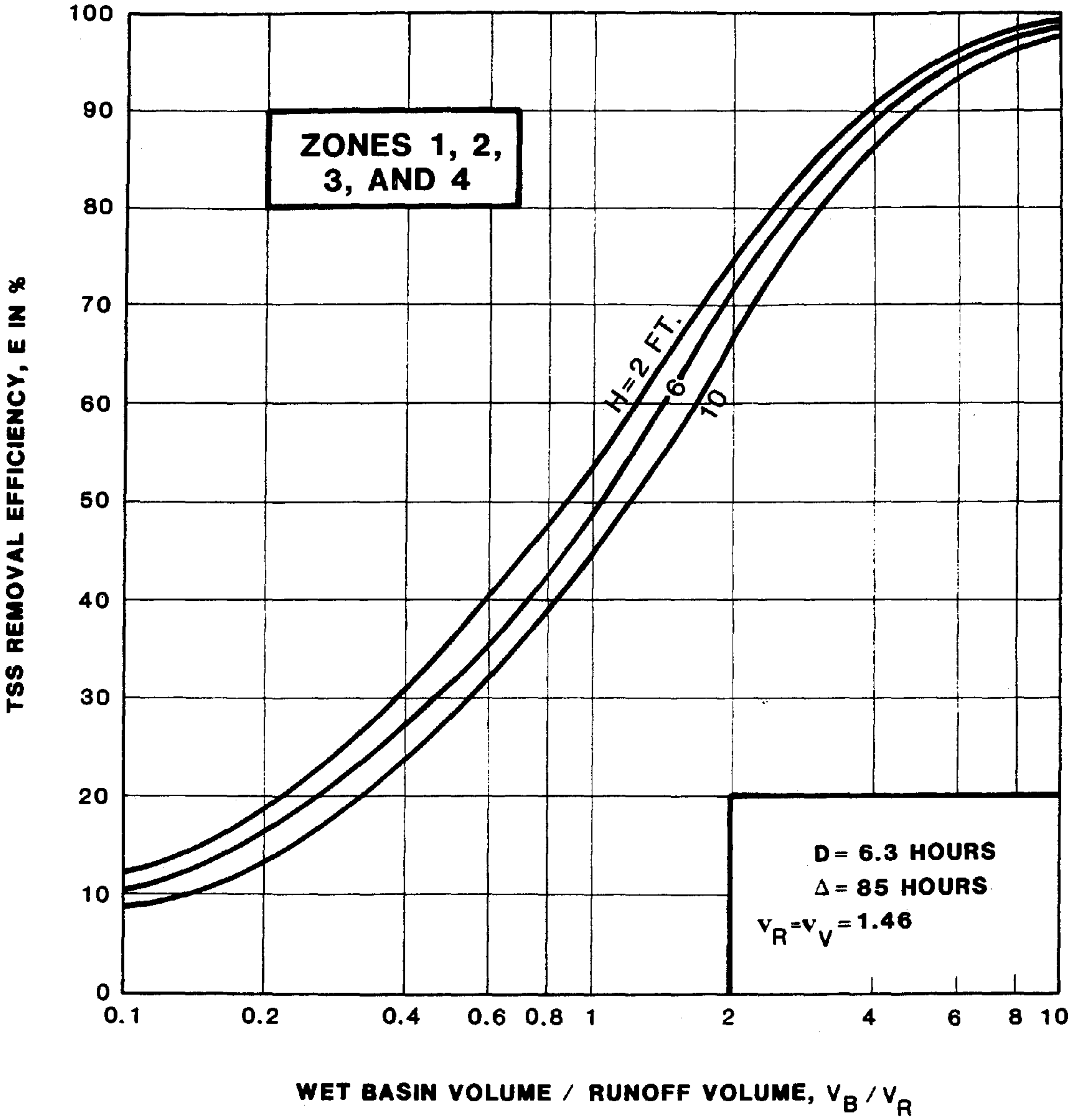


Figure 20. TSS removal versus V_B/V_R ratio for zones 1, 2, 3 and 4.

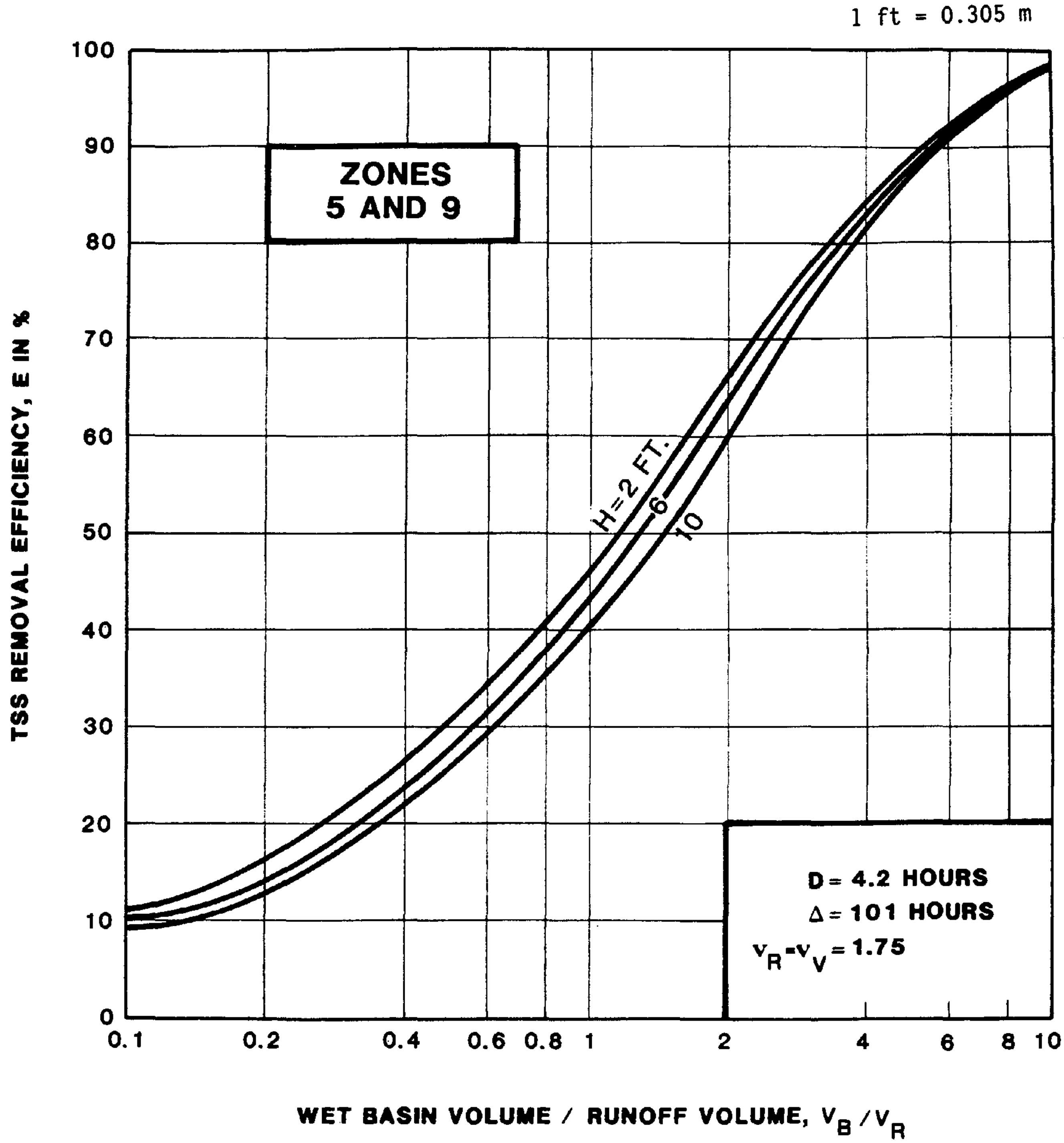


Figure 21. TSS removal versus V_B/V_R ratio for zones 5 and 9.

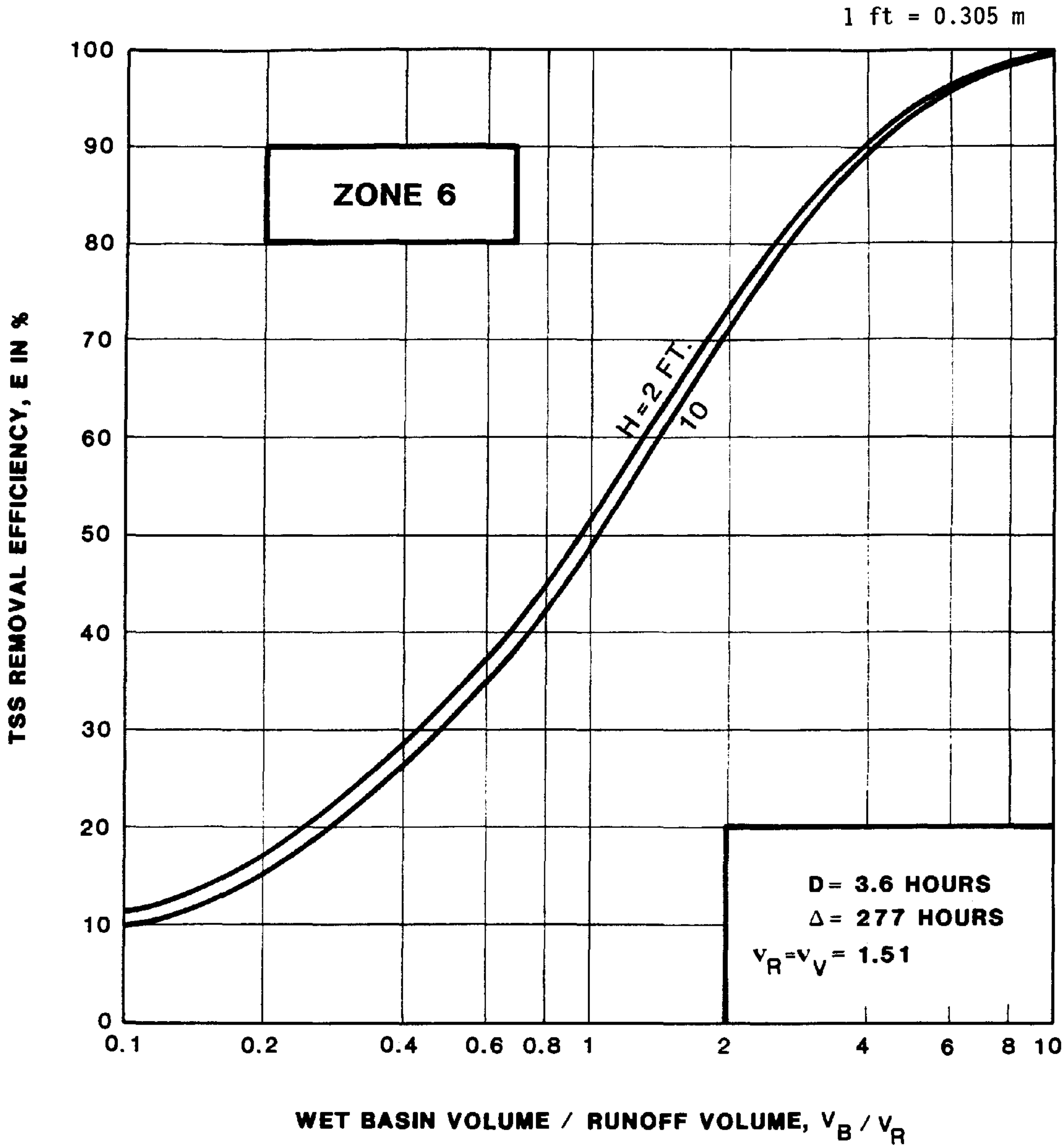


Figure 22. TSS removal versus V_B/V_R ratio for zone 6.

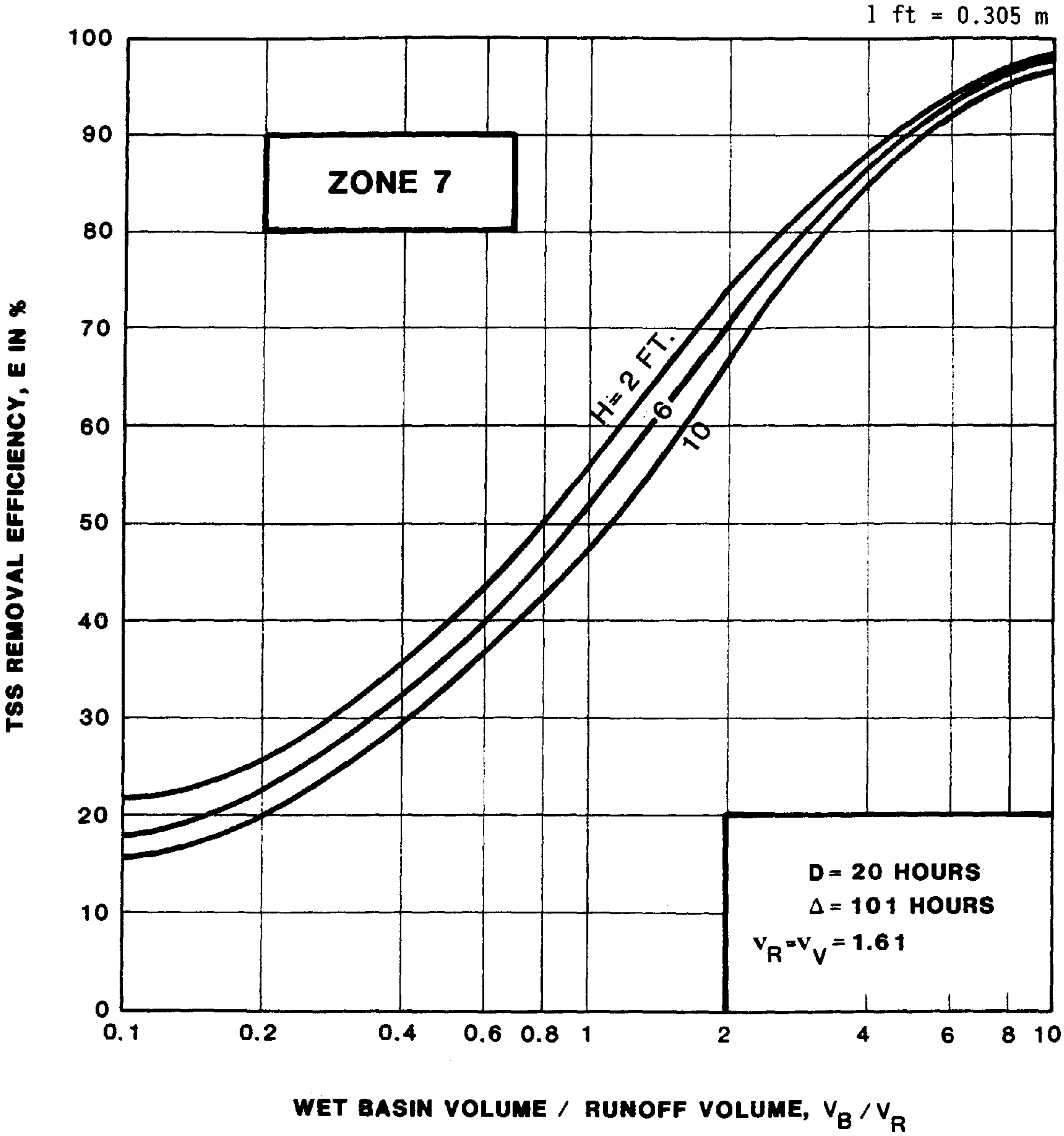


Figure 23. TSS removal versus V_B/V_R ratio for zone 7.

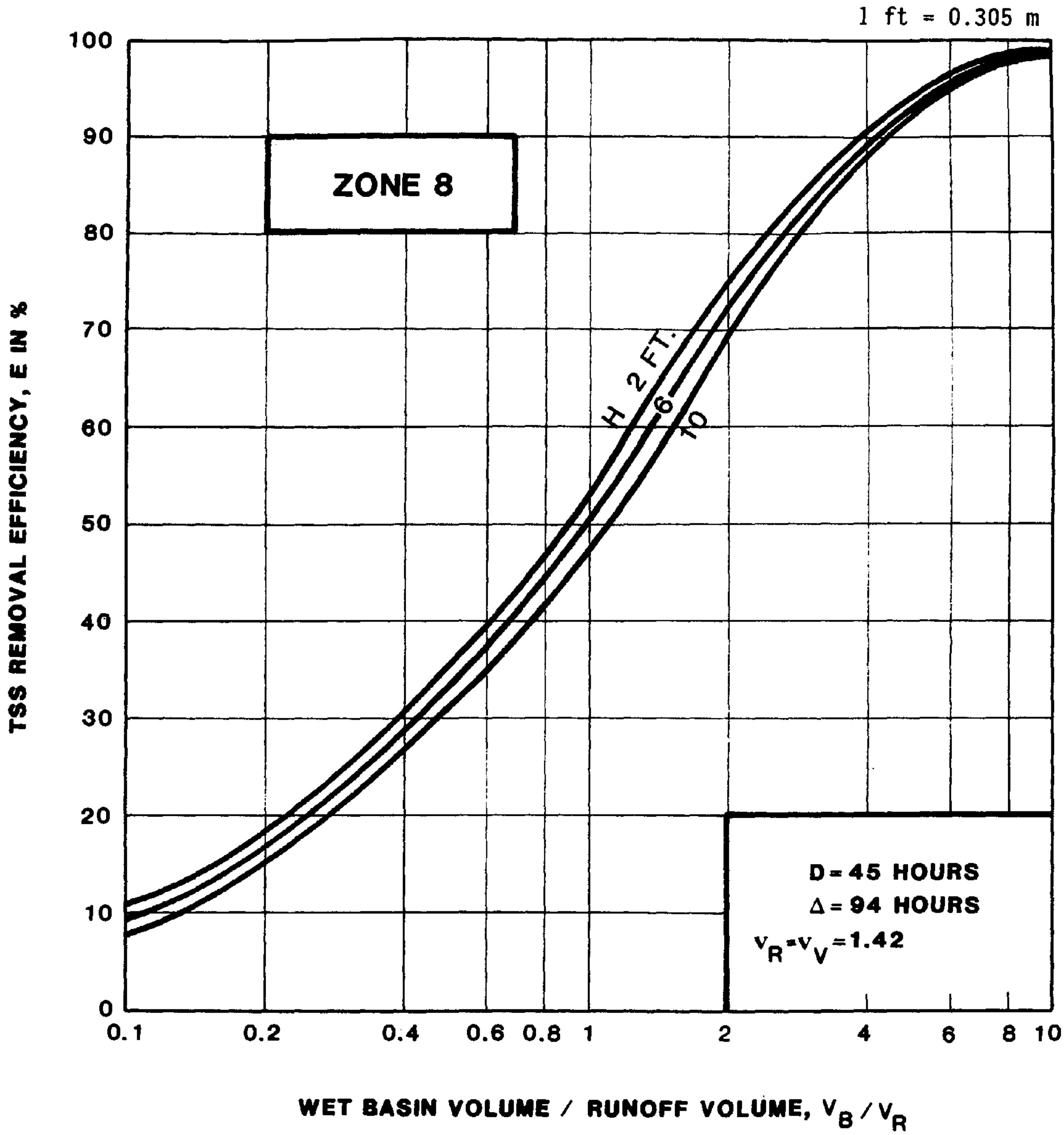


Figure 24. TSS removal versus V_B/V_R ratio for zone 8.

WET DETENTION BASIN DESIGN WORKSHEET
FOR SOLIDS SETTLING MODEL APPROACH

*********************************DESIGN DATA*******************************

Drainage Area ________________ (A_D) ac

Location ____
(City, State)

Rainfall Zone
(from figure 19) ________________

Mean Rainfall Event Volume ________________ (R_V) ft
(from table 7)

Runoff Coefficient

Pervious Area Coefficient ________________ (C_P)

Impervious Area Coefficient ________________ (C_I)

Impervious Area/Total Area ________________ (X)

Runoff Coefficient ________________ (C)
($C = C_P + (C_I - C_P)X$)

*****************************DESIGN PROCEDURE****************************

Average Runoff Volume ________________ (V_R) ac-ft
($V_R = CR_VA_D$)

Surface Area of Permanent Pool ________________ ac

Permanent Pool Volume	H = 2 ft	________________	(V_{B2}) ac-ft
	H = 4 ft	________________	(V_{B4}) ac-ft
	H = 6 ft	________________	(V_{B6}) ac-ft
	H = ___ ft	________________	($V_{B\_\_}$) ac-ft
Ratio of Permanent Pool Volume to Average Runoff Volume	H = 2 ft	________________	(V_{B2}/V_R)
	H = 4 ft	________________	(V_{B4}/V_R)
	H = 6 ft	________________	(V_{B6}/V_R)
	H = ___ ft	________________	($V_{B\_\_}/V_R$)

Figure 25. Wet detention basin design worksheet for solids settling model approach.

WET DETENTION BASIN DESIGN WORKSHEET
FOR SOLIDS SETTLING MODEL APPROACH
(CONTINUED)

************************DESIGN PROCEDURE (Continued)***********************

TSS Removal Efficiency, (from figures 20 to 24)	H = 2 ft	____________________	(E_2)	%
	H = 4 ft	____________________	(E_4)	%
	H = 6 ft	____________________	(E_6)	%
	H = ___ ft	____________________	($E_{\_}$)	%
Selected Design Depth of Permanent Pool	H = ___ ft			
Selected Design TSS Removal Efficiency (from figures 20 to 24)		____________________	(E_{TSS})	%
Long-term Average Pollutant Removal Efficiencies:				
Lead (90% of TSS)		____________________	(E_{Pb})	%
Copper (60% of TSS)		____________________	(E_{Cu})	%
Zinc (45% of TSS)		____________________	(E_{Zn})	%
TKN,BOD (10%-30% of TSS)		____________________	($E_{TKN,BOD}$)	%

Figure 25. Wet detention basin design worksheet for solids settling model approach (continued).

B. RAINFALL VOLUME

Rainfall volume in table 7 is the total depth of rainfall in a rainfall event. The rainfall volume value from the table must be multiplied by the area of the watershed over which it falls to find the total rainfall volume.

C. RUNOFF COEFFICIENT

The coefficient of runoff of interest here may differ from that used in the rational method to relate rainfall intensity to peak runoff rates. The runoff coefficient needed here represents the proportion of the total rainfall volume which runs off. A runoff coefficient of 0.2 is representative of typical urban areas. (Driscoll 1983) This can be interpreted to mean that 20 percent of the precipitation volume that falls on a typical urban watershed appears downgrade as runoff. The remaining 80 percent is infiltrated into the soil, stored in depressions, lost to evaporation, etc.

Two other studies reported coefficients for highway runoff significantly different from that reported for typical urban areas. The values reported in these studies are as follows:

Research	Site Description	Mean Runoff Coefficient
Kobriger et al. (1982)	100% paved	0.83
	51% paved	0.71
	27% paved	0.43
Mar et al. (1982)	100% paved	0.72 - 0.80
	Elevated Sections	0.70

Mar recommended use of equation 1, developed at the U.S. Army Corps of Engineers' Hydrologic Engineering Center, to estimate the runoff coefficient for partially paved drainage areas:

$$C = C_P + (C_I - C_P)X \qquad (1)$$

where:

C = Runoff coefficient or the ratio of total runoff to total rainfall

C_P = Runoff coefficient for pervious areas

C_I = Runoff coefficient for impervious areas

X = Impervious area/total area.

In the absence of local data, the equation developed at the Hydrologic Engineering Center is recommended. It is again noted that the runoff coefficient here may differ from the runoff coefficient ordinarily used in the Rational Equation for computing peak runoff rates.

D. RUNOFF VOLUME

The stormwater runoff from a watershed is dependent on rainfall volume, antecedent moisture, depression storage, slopes, soils, percent impervious, and other variables. For purposes of estimating the long-term mean event runoff volume used in the procedure presented here, these variables, except for rainfall volume, may be incorporated into a runoff coefficient. Runoff volume, V_R, can then be estimated by use of equation 2:

$$V_R = CR_VA_D \tag{2}$$

where:

V_R = Runoff volume, ft^3 (m^3) or ac-ft

C = Runoff coefficient

R_V = Rainfall depth, ft (m)

A_D = Drainage area, ft^2 (m^2) <u>or</u> ac.

E. SETTLING VELOCITIES OF SUSPENDED SOLIDS

The efficiency of wet detention basin in removing suspended solids from stormwater influent is dependent on the distribution of particle sizes and other factors such as time in residence, short-circuiting, and basin depth. The terminal settling velocity of a suspended particle in quiescent water is dependent on the size, shape, and specific gravity of the particle and the viscosity of the water. Therefore, the distribution of sizes in the silt and clay fractions is determined by settling velocities which are analogous to physical size ranges.

Driscoll reported settling velocities of particles in urban runoff as follows:

Proportion, %	Average settling velocity, ft/hr
20	0.03
20	0.3
20	1.5
20	7.0
20	65.0

SOURCE: Driscoll, 1983

To develop the guidelines presented herein, this FHWA research study relied upon settling column tests with highway runoff. The particle size distributions measured during these settling column tests are as follows:

Proportion, %	Settling velocity (ft/hr)
18	0.03
17	0.3
17	1.5
19	7.0
28	65.0

This highway runoff distribution is similar to Dricoll's urban runoff distribution given above. Therefore, an equal 20 percent proportion for each of the settling velocity categories is recommended as the particle size distribution for detention basin design.

F. SUSPENDED SOLIDS REMOVAL EFFICIENCY

Figures 20 through 24 provide for direct solution of equations developed to estimate suspended solids removal efficiency in wet detention basins.

The dimensions for average runoff volume used in the ratio, V_B/V_R, may be in ft^3 (m^3) or ac-ft, so long as units for permanent pool volume are consistent. That is, the drainage area may be measured in acres or in ft^2 (m^2) and used in equation (2) without conversion of the units. An example problem is included in this section to illustrate use of the figures.

Some observations are appropriate regarding the effects of basin design on suspended solids removal efficiencies:

> A decrease in the ratio V_B/V_R obviously reduces suspended solids removal efficiency. A reduction in efficiency can be expected as sediment accumulation reduces the permanent pool storage volume in a basin. The efficiency of an existing basin could also be decreased as a result of an increase in the imperviousness of the watershed (i.e., increase in V_R), as when additional traffic lanes are constructed.
>
> For a given permanent pool volume, a shallow pool is more efficient than a deeper basin, although it requires more space due to the larger surface area. Where rights-of-way must be acquired to construct a wet detention basin, a deeper basin may be more cost-efficient than a shallow basin.

G. POLLUTANT REMOVAL EFFICIENCIES

Specific pollutant removal efficiencies for heavy metals were defined as a percentage of the total suspended solids removal efficiencies. These removal efficiences are based upon measured suspended fractions in highway runoff samples subjected to settling column tests. The following percentages are recommended to be applied to the calculated removal efficiency of TSS for the specific pollutants:

Pollutant	Percentage
Lead (Pb)	90%
Copper (Cu)	60%
Zinc (Zn)	45%

For example, if for a given design, the TSS removal efficiency is 80 percent, then the removal efficiency for lead is approximately 72 percent based upon typical ratios of suspended lead to total lead in highway runoff. For TKN and BOD, the percentage of the TSS removal efficiency ranges from 20 to 30 percent, with the larger percentage reflecting some biological removal processes in the larger detention basins. (EPA, 1983)

H. DESIGN EXAMPLE

An example of the wet detention basin design procedure for the solids settling approach is presented below. The example illustrates the design procedure for a wet detention basin located in one quadrant of a cloverleaf interchange as shown in figure 26. The basin is assumed to be constructed in Knoxville, Tennessee. The detention basin will serve a drainage area of 40 ac (16.2 ha) and 42 percent of the area is roadway pavement. The steps used in designing the basin are detailed below and used to complete the design worksheet which follows.

Step 1. From figure 19, Knoxville, Tennessee, is in rainfall zone 2.

Step 2. From table 7, the mean storm event volume is 0.36 in (0.91 cm). $R_V = 0.36$.

Step 3. Compute the coefficient, C, for use in the equation, $V_R = CR_VA_D$. Use equation (1) to estimate the value of C.

$$C = C_p + (C_I - C_p)X$$

Assume $C_p = 0.4$

$C_I = 0.8$

$$C = 0.4 + (0.8 - 0.4)(.42)$$

$$= 0.57$$

Step 4. Compute the runoff volume.

$$V_R = CR_VA_D$$

$$= 0.57\ (0.36/12)40$$

$$= 0.68 \text{ ac-ft}$$

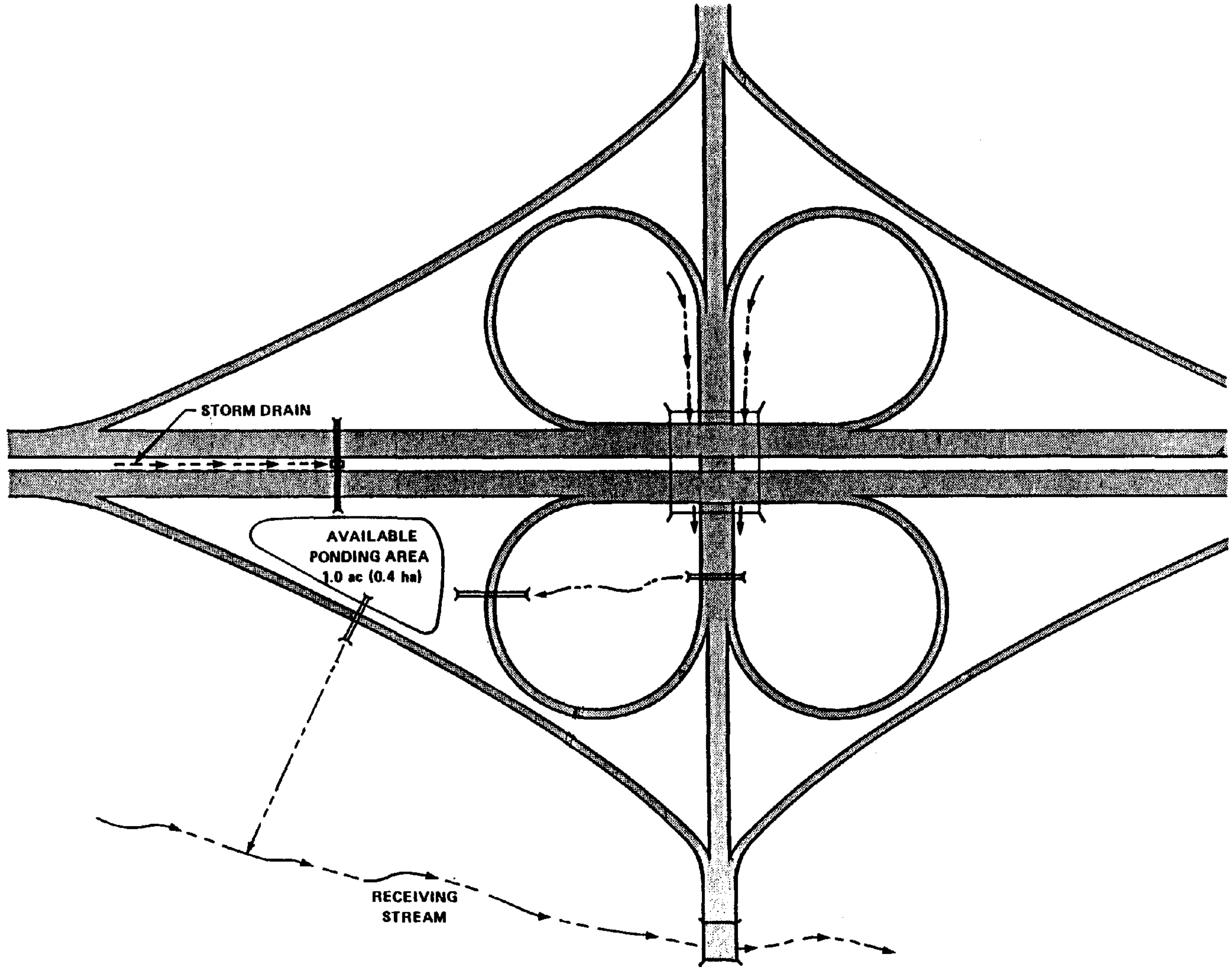


Figure 26. Example - wet detention basin design for solids settling model approach.

WET DETENTION BASIN DESIGN WORKSHEET
FOR SOLIDS SETTLING MODEL APPROACH

*********************************DESIGN DATA******************************

Drainage Area	40	(A_D) ac
Location: Knoxville, Tennessee (City, State)		
Rainfall Zone (from figure 19)	2	
Mean Rainfall Event Volume (from table 7)	0.36/12	(R_V) ft
Runoff Coefficient		
Pervious Area Coefficient	0.4	(C_P)
Impervious Area Coefficient	0.8	(C_I)
Impervious Area/Total Area	0.42	(X)
Runoff Coefficient ($C = C_P + (C_I - C_P)X$)	0.57	(C)

******************************DESIGN PROCEDURE***************************

Average Runoff Volume ($V_R = CR_VA_D$)		0.68	(V_R) ac-ft
Surface Area of Permanent Pool		1.0	ac
Permanent Pool Volume	H = 2 ft	0.62	(V_{B2}) ac-ft
	H = 3 ft	1.14	(V_{B3}) ac-ft
	H = 4 ft	1.79	(V_{B4}) ac-ft
	H = 5 ft	2.58	(V_{B5}) ac-ft
	H = 6 ft	3.51	(V_{B6}) ac-ft
Ratio of Permanent Pool Volume to Average Runoff Volume	H = 2 ft	0.9	(V_{B2}/V_R)
	H = 3 ft	1.7	(V_{B3}/V_R)
	H = 4 ft	2.6	(V_{B4}/V_R)
	H = 5 ft	3.8	(V_{B5}/V_R)
	H = 6 ft	5.2	(V_{B6}/V_R)

Figure 26. Example - wet detention basin design for solids settling model approach (continued).

WET DETENTION BASIN DESIGN WORKSHEET
FOR SOLIDS SETTLING MODEL APPROACH
(CONTINUED)

************************DESIGN PROCEDURE (Continued)***********************

TSS Removal Efficiency (from figures 20 to 24)	H = 2 ft	50	(E_2)	%
	H = 3 ft	70	(E_3)	%
	H = 4 ft	80	(E_4)	%
	H = 5 ft	88	(E_5)	%
	H = 6 ft	92	(E_6)	%

Selected Design Depth of Permanent Pool H = 5 ft

Selected Design TSS Removal Efficiency (from figures 20 to 24)	88	(E_{TSS})	%

Long-term Average Pollutant Removal Efficiencies:

Lead (0.90 x 88%)	79	(E_{Pb})	%
Copper (0.60 x 88%)	53	(E_{Cu})	%
Zinc (0.45 x 88%)	40	(E_{Zn})	%
TKN, BOD (0.20 x 88%)	18	($E_{TKN,BOD}$)	%

Figure 26. Example - wet detention basin design for solids settling model approach (continued).

Note conversion of rainfall depth from in to ft and retention of ac as the unit of measure for the drainage area. If this convention is adopted, the volume of the detention basin must be computed in ac-ft in step 6.

Step 5. A wet detention basin can be constructed within a loop ramp of the interchange in figure 26, or between the ramps in a quadrant of the interchange. An elongated basin between ramps is a more favorable shape than the near circular basin within a loop because of the reduced potential for short-circuiting.

Step 6. Basin volumes for trial permanent pool depths are typically selected from depth-area-storage relationships which are developed based on the natural contours and location of the embankment at the detention basin site. Where natural site storage is limited, excavated basins are an option to embankment basins. For this example, a depth-area-storage relationship is given below:

Depth (ft)	Area (ac)	Storage (ac-ft)
1	0.31	0.24
2	0.45	0.62
3	0.59	1.14
4	0.72	1.79
5	0.86	2.58
6	1.00	3.51

Figure 26 presents the permanent pool volumes and ratio of permanent pool volume to average runoff volume for depths of 2, 3, 4, 5, and 6 ft (0.92, 1.22, 1.53, and 1.83 m).

Step 7. For each of the selected depths and computed ratios of permanent pool volume to average runoff volume, the TSS removal efficiencies are determined from figure 20 and are as follows:

Depth (ft)	Ratio of Pool Volume to Runoff Volume	TSS Removal Efficiency (%)
2	0.9	50
3	1.7	70
4	2.6	80
5	3.8	88
6	5.2	92

The above depths and efficiencies and the curves of figure 20 show that diminishing returns for TSS removal efficiency are realized for B_V/V_R ratios in excess of about 3.7. Thus, for this example site, a depth of 5 ft is selected which produces a TSS removal efficiency of 88 percent.

Step 8. For TSS removal efficiency E = 88 percent, pollutant removal efficiencies are:

Pb removal efficiency (.90 x 88) = 79 percent
Cu removal efficiency (.60 x 88) = 53 percent
Zn removal efficiency (.45 x 88) = 40 percent

Step 9. The selected wet detention basin is conservatively designed. Precautions could be taken against short-circuiting by adopting an irregular shore-line to increase the distance between the inlet and outlet (chapter 3, section 2.A). If use of an irregular basin perimeter results in a reduction in permanent pool volume and significant reductions in the estimated pollutant removal efficiencies, the reduced volume can be compensated for by a small increase in the depth of the permanent pool.

Step 10. Bank slope adopted for this design should be at 3:1 (H:V) or flatter. Any bank slopes above the invert elevation of the outlet culvert should be grassed.

3. DESIGN PROCEDURES FOR PARTICULATE AND SOLUBLE POLLUTANT CONTROL (CONTROLLED LAKE EUTROPHICATION METHOD)

This approach assumes that a wet detention basin is a small eutrophic lake which can be represented by empirical models used to evaluate lake eutrophication impacts. The intent of this approach is to use lake eutrophication models to account for the significant removal of dissolved nutrients observed in the field and attributable to biological processes such as uptake by algae and rooted aquatic vegetation. Using this design method, a wet detention basin can be sized to achieve a controlled rate of eutrophication and an associated removal rate for nutrients.

The following design procedure is based on the phosphorus retention coefficient model. (Walker (1987) Like most input/output lake eutrophication models, the Walker model is an empirical approach which treats the permanent pool as a completely mixed system and assumes that it is not necessary to consider the temporal variability associated with individual storm events. The Walker model is based upon annual runoff flows and stormwater pollution loadings.

The following steps are required to use the design procedure for wet detention basins to control nutrient loadings (particulate and soluble):

1. Identify the rainfall zone in which the basin will be located from figure 19.

2. Determine the mean storm event rainfall volume (depth) for the appropriate zone by converting the value in table 7 from inches to feet.

3. Establish the runoff coefficient (see chapter 3, section 2.C, RUNOFF COEFFICIENT).

4. Compute the mean storm event runoff volume (see chapter 3, section 2.D, RUNOFF VOLUME).

5. Establish the dimensions of the area in which a wet detention basin could be constructed.

6. Select an average hydraulic residence time and compute the permanent pool volume (see chapter 3, section 3.A, PERMANENT POOL VOLUME). For most locations in the U.S., the optimum average hydraulic residence time for most designs will be 2 to 3 weeks. (Hartigan, 1988)

7. Establish mean depth (see chapter 3, section 3.B, DEPTH OF PERMANENT POOL).

8. Establish total phosphorus loadings and ratio of ortho-phosphorus to total phosphorus loadings in highway runoff discharged into wet detention basin (see chapter 3, section 3.D, PHOSPHORUS LOADS IN HIGHWAY RUNOFF).

9. Compute phosphorus removal efficiency (see chapter 3, section 3.E, LAKE EUTROPHICATION MODEL).

10. Determine removal efficiency for particulate pollutants.

11. Design the basin configuration to minimize the potential for short-circuiting (chapter 3, section 1). If this is not practicable, choose a more conservative design, i.e., choose a design which will yield higher removal efficiencies to compensate for possible adverse effects from short-circuiting.

12. Design side slopes along shoreline of permanent pool (see chapter 3, section 3.C, SIDE SCOPES ALONG SHORELINE).

A wet detention basin design worksheet for the lake eutrophication model approach is presented in figure 27.

A. PERMANENT POOL VOLUME

The permanent pool storage volume required for a wet detention basin to effectively control nutrient loadings is dependent on the average hydraulic residence time (T) which is a key parameter in the eutrophication modeling approach. Based upon typical urban runoff pollution loadings, average hydraulic residence times have been related to average total P removal. (Hartigan 1988) Figure 28 is an example of such a relationship for wet detention basins in northern Virginia. The design curve examples in figure 28 show that an average hydraulic residence time of 2 to 3 weeks achieves an optimum level of control.

Wet detention basins with hydraulic residence times much greater than 3 weeks would have a greater risk of thermal stratification and anaerobic bottom waters. As a result, there would be increased risk of short-circuiting and significant export of nutrients from bottom sediments subject to anaerobic conditions. Consequently, it is advisable to maintain

WET DETENTION BASIN DESIGN WORKSHEET
FOR LAKE EUTROPHICATION MODEL APPROACH

********************************DESIGN DATA********************************

Drainage Area ______________ (A_D) ac

Location ______________________________
(City, State)

Rainfall Zone
(from figure 19) ______________

Mean Rainfall Event Volume ______________ (R_V) ft
(from table 7)

Runoff Coefficient

Pervious Area Coefficient ______________ (C_P)

Impervious Area Coefficient ______________ (C_I)

Impervious Area/Total Area ______________ (X)

Runoff Coefficient ______________ (C)
($C = C_P + (C_I - C_P)X$)

Phosphorus Values

Total Phosphorus Inflow ______________ (P)mg/L
(0.4 mg/L for Urban Highways and or
0.20 mg/L for Rural Highways) ______________ (P)ug/L

Orthophosphorus/Total Phosphorus ______________ (F)
(use 0.25)

****************************DESIGN PROCEDURE****************************

Runoff Volume ______________ (V_R) ac-ft
($V_R = CR_VA_D$)

Surface Area of Permanent Pool ______________ ac

Average Hydraulic Residence Time ______________ (T) yr
(____ wks/52)

Number of Storms ______________ (NS)
(365 x 24/____ interval from table 7)

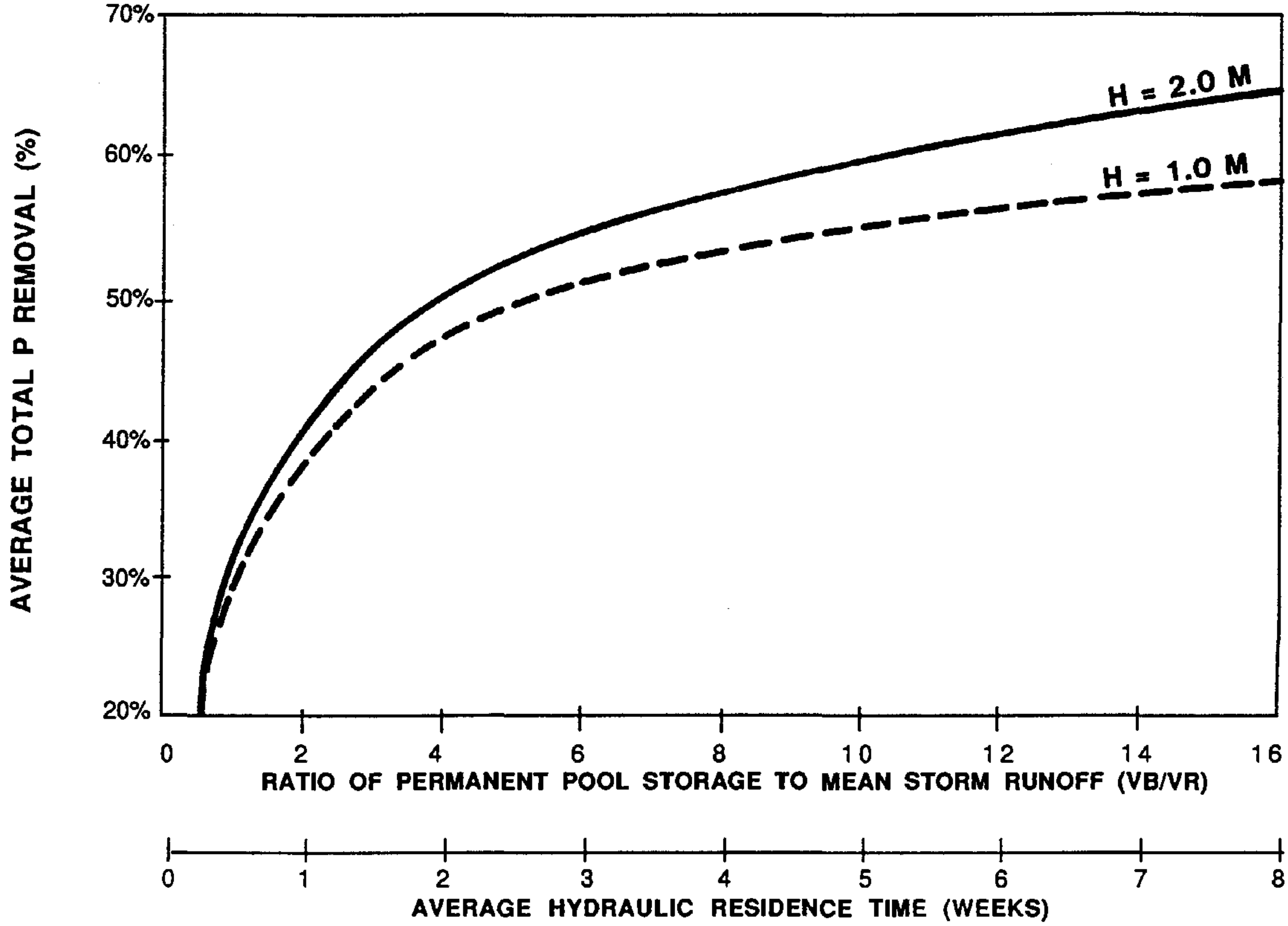
Figure 27. Wet detention basin design worksheet for lake eutrophication model approach.

WET DETENTION BASIN DESIGN WORKSHEET
FOR LAKE EUTROPHICATION MODEL APPROACH
(CONTINUED)

**********************DESIGN PROCEDURE (Continued)************************

Permanent Pool Volume (V_B = T x V_R x NS)	______________	(V_B) ac-ft
Mean Depth of Permanent Pool (V_B/A_S)	______________	(Z) ft
		or
X 0.3048 =	______________	(Z) m
Mean Overflow Rate (Q_s = Z/T)	______________	(Q_s) m/yr
Decay Rate ($K_2 = (0.056)(Q_s)(F)^{-1}/(Q_s+13.3)$)	______________	(K_2)
N Factor (N = K_2 x P x T)	______________	(N)
Total Phosphorus Removal Efficiency ($R = 1 + (1 - (1+4N)^{0.5})/(2N)$)	______________	(R) or
	______________	%
Additional Pollutant Removal Efficiencies:		
Basin Volume/Runoff Volume	______________	(V_B/V_R)
TSS Removal Efficiency (from figures 20 to 24)	______________	(E_{TSS}) %
Lead Removal Efficiency (90% of TSS)	______________	(E_{Pb}) %
Copper Removal Efficiency (60% of TSS)	______________	(E_{Cu}) %
Zinc Removal Efficiency (45% of TSS)	______________	(E_{Zn}) %
TKN, BOD (10%-30% of TSS)	______________	($E_{TKN,BOD}$) %

Figure 27. Wet detention basin design worksheet for lake eutrophication model approach (continued).



Source: Hartigan, 1988.

Figure 28. Eutrophication design method for wet detention basins: northern Virginia example.

the average residence time at the lowest level which can ensure adequate nutrient uptake. Therefore, an average hydraulic residence time of 2 to 3 weeks is recommended as a design criterion for highway runoff controls.

The average hydraulic residence time, T (in units of "years"), is equal to the ratio of permanent pool storage volume (V_B) to the product of mean storm runoff (V_R) times the average number of storms per year. For example, for eastern U.S. locations in zone 2 (figure 19) which average about 114 storm events per year based on a mean interval between storms of 77 hrs (table 7), the average hydraulic residence time (in years) is equal to V_B/V_R divided by 114.

The required permanent pool volume can be calculated from the two-week hydraulic residence time, the mean storm runoff and the number of storms per year by applying the following equation:

$$V_B = (T)(V_R)(NS) \tag{3}$$

where:

V_B = Permanent pool volume, ac-ft

T = Average hydraulic residence time, years

V_R = Mean storm runoff volume, ac-ft

NS = Average number of storms per year

$$NS = \frac{(365\ \text{day})\ \text{x}\ (24\ \text{hrs})}{(\text{mean storm interval, hrs})}$$

For mean storm interval in appropriate zone, see table 7.

B. DEPTH OF PERMANENT POOL

Mean depth of the detention basin is calculated by dividing the storage by the surface area. To achieve adequate control of nutrient loadings, the mean depth should be low enough to minimize the risk of thermal stratification, but high enough to ensure that algal blooms are not excessive and to minimize resuspension of settled pollutant during major storm events. The prevention of significant thermal stratification will help minimize short-circuiting and maintain the aerobic bottom waters that should maximize sediment uptake and minimize the release of nutrients from bottom sediments into the water column. A mean depth of about 1 to 3 m should be capable of maintaining an acceptable environment within the permanent pool for the average hydraulic residence times recommended herein. (Hartigan, 1988) The mean depths of the more effective wet detention basins monitored by the NURP study typically fall within this range as do the recommendations of recent Florida monitoring studies of retention basins. (Yousef, 1985)

The maximum depth of the permanent pool should be set at a level which minimizes the risk of thermal stratification. Based upon typical thermal profiles for different impoundment sizes and geographical regions, a maximum depth of no greater than 4 to 6 m should be acceptable for most regions assuming an average hydraulic residence time of 2 weeks. (Mills, 1982)

C. SIDE SLOPES ALONG SHORELINE

The slope of the littoral zone around the perimeter of the permanent pool should be gradual enough to minimize safety hazards, to promote the growth of wetland vegetation along the shoreline, and to facilitate maintenance (e.g., grass mowing). Side slopes in the range 5:1 (H:V) to 10:1 (H:V) are recommended. The side slopes should also be topsoiled, nurtured, or planted from 2 ft (0.61 m) below to 1 ft (0.31 m) above the permanent pool control elevation to promote vegetative growth. Wetland vegetation will not only improve the aesthetic qualities of the detention facility, but they will also help minimize the proliferation of free-floating algae. The nutrient uptake achieved by wetland vegetation will help keep the algae concentrations in check by limiting the amount of nutrients available for phytoplankton. Additional guidelines for using wetland vegetation within shallow sections of the permanent pool have been published by the State of Maryland. (Maryland DNR, 1987)

D. PHOSPHORUS LOADS IN HIGHWAY RUNOFF

The lake eutrophication model requires the mean total phosphorus concentration in highway runoff discharged into the wet detention basin. Based on highway runoff data, the following concentration of total phosphorus from highway runoff are recommended for use in this wet detention basin design procedure: (Woodward-Clyde, 1987)

- Urban highways (ATD > 30,000): use 0.4 mg/L (400 ug/L) TP
- Rural highways (ATD < 30,000): use 0.2 ug/L (200 ug/L) TP

The lake eutrophication model also requires the ratio of orthophosphorus to total phosphorus loadings in highway runoff. An ortho P/total P ratio, F, of 0.25 is recommended based upon previous runoff pollution monitoring studies of urban areas with significant levels of imperviousness. (Northern Virginia Planning District Commission, 1979)

E. LAKE EUTROPHICATION MODEL

The phosphorus retention coefficient model is applied in two parts: (Walker, 1987)

$$K_2 = (0.056)(Q_s)(F)^{-1}/(Q_s + 13.3) \qquad (4)$$

where

K_2 = second order decay rate (m^3/mg-yr)

Q_s = mean overflow rate (m/yr) = Z/T

F = inflow ortho P/total P ratio

Z = mean depth of permanent pool (m)

T = average hydraulic residence time (yr)

and

$$R = 1 + (1 - (1 + 4N)^{0.5})/(2N) \qquad (5)$$

where

R = total P retention coefficient = removal efficiency

$N = (K_2)(P)(T)$

P = inflow total P (ug/L)

As may be seen, the model relies upon a second order reaction rate which means that the total P removal per unit volume is proportional to the concentration squared. The second order decay rate (K_2) is calculated from the mean overflow rate and the ortho P fraction of total P. The average total P removal rate (R) is then calculated from the decay rate, the inflow total P concentration, and the average hydraulic residence time. The model was developed from a database for 60 Corps of Engineers' reservoirs and verified for 20 other reservoirs.

F. DESIGN EXAMPLE

This example of the wet detention basin design for control of nutrient loadings is based on the same site previously used for the wet detention basin design in chapter 3, section 2.H. Figure 29 presents a schematic of the site location in one quadrant of a cloverleaf interchange in Knoxville, Tennessee with accompanying worksheet. The detention basin will serve a drainage area of 40 ac (16.2 ha) and 42 percent of the area is roadway pavement. The steps used in designing the basin are detailed below and are used to complete the design worksheet.

Step 1. From figure 19, Knoxville, Tennessee, is in rainfall zone 2.

Step 2. From table 7, the mean annual rainfall event volume is 0.36 inches. $R_V = 0.36$.

Step 3. Compute the coefficient, C, for use in the equation, $V_R = CR_VA_D$. Use equation 1 to estimate the value of C.

$$C = C_p + (C_I - C_p)X$$

Assume $C_p = 0.4$

$C_I = 0.8$

$$C = 0.4 + (0.8 - 0.4).42$$

$$= 0.57$$

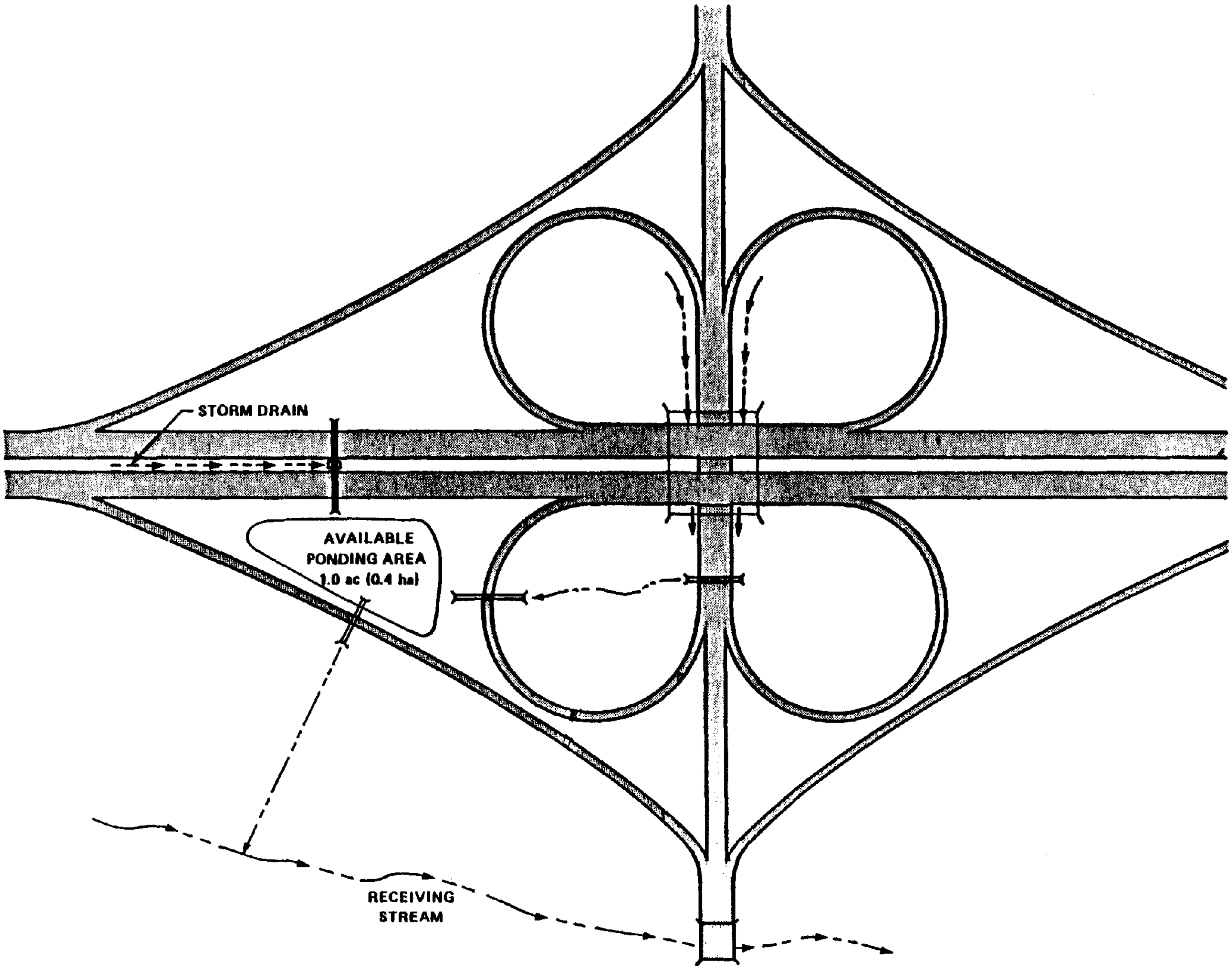


Figure 29. Example - wet detention basin design for lake eutrophication model approach.

WET DETENTION BASIN DESIGN WORKSHEET
FOR LAKE EUTROPHICATION MODEL APPROACH

**********************************DESIGN DATA********************************

Drainage Area	40	(A_D) ac
Location	Knoxville, Tennessee (City, State)	
Rainfall Zone (from figure 19)	4	
Mean Rainfall Event Volume (from table 7)	0.36	(R_V) ft
Runoff Coefficient		
Pervious Area Coefficient	0.4	(C_P)
Impervious Area Coefficient	0.8	(C_I)
Impervious Area/Total Area	0.42	(X)
Runoff Coefficient ($C = C_P + (C_I - C_P)X$)	0.57	(C)
Phosphorus Values		
Total Phosphorus Inflow (0.4 mg/L for Urban Highways and 0.20 mg/L for Rural Highways)	0.4	(P)mg/L
		or
	400	(P)ug/L
Orthophosphorus/Total Phosphorus (use 0.25)	0.25	(F)

******************************DESIGN PROCEDURE*****************************

Runoff Volume ($V_R = CR_VA_D$)	0.68	(V_R) ac-ft
Surface Area of Permanent Pool	1.0	ac
Average Hydraulic Residence Time (2.5 wks/52)	0.0481	(T) yr
Number of Storms (365 x 24/77 interval from table 7)	114	(NS)

Figure 29. Example - wet detention basin design worksheet for lake eutrophication model approach (continued).

WET DETENTION BASIN DESIGN WORKSHEET
FOR LAKE EUTROPHICATION MODEL APPROACH
(CONTINUED)

***********************DESIGN PROCEDURE (Continued)************************

Permanent Pool Volume ($V_B = T \times V_R \times NS$)	3.7	(V_B) ac-ft
Mean Depth of Permanent Pool (V_B/A_S)	3.7	(Z) ft or
x 0.3048 =	1.1	(Z) m
Mean Overflow Rate ($Q_S = Z/T$)	22.9	(Q_S) m/yr
Decay Rate ($K_2 = (0.056)(Q_S)(F)^{-1}/(Q_S+13.3)$)	0.142	(K_2)
N Factor ($N = K_2 \times P \times T$)	2.73	(N)
Total Phosphorus Removal Efficiency ($R = 1 + (1 - (1+4N)^{0.5})/(2N)$)	0.55	(R) or
	55	%

Additional Pollutant Removal Efficiencies:

Basin Volume/Runoff Volume	5.4	(V_B/V_R)
TSS Removal Efficiency (from figures 20 to 24)	93	(E_{TSS}) %
Lead Removal Efficiency (90% of TSS)	84	(E_{Pb}) %
Copper Removal Efficiency (60% of TSS)	56	(E_{Cu}) %
Zinc Removal Efficiency (45% of TSS)	42	(E_{Zn}) %
TKN, BOD (10%-30% of TSS)	19	($E_{TKN,BOD}$) %

Figure 29. Example - wet detention basin design worksheet for lake eutrophication model approach (continued).

Step 4. Compute the runoff volume.

$V_R = CR_VA_D$

$= 0.57\ (0.36/12)40$

$= 0.68$ ac-ft

Note conversion of rainfall depth from in to ft and retention of ac as the unit of measure for the drainage area. If this convention is adopted, the volume of the detention basin must be computed in ac-ft in step 6.

Step 5. A wet detention basin can be constructed within a loop ramp of the interchange in figure 29, or between the ramps in a quadrant of the interchange. An elongated basin between ramps is a more favorable shape than the near circular basin within a loop because of the reduced potential for short-circuiting.

Step 6. Compute basin volume required assuming a 2.5-week average hydraulic residence time (convert to years: 2.5/52 = 0.0481 years) using the equation $V_B = (T)(V_R)(NS)$. First compute the number of storms (NS) by:

$$NS = \frac{(365 \text{ days})(24 \text{ hrs})}{(77 \text{ hrs})} = 114 \text{ storms}$$

where 77 hours is the mean rainfall interval for zone 2 as given in table 7.

$V_B = (T)(V_R)(NS)$

$= (0.0481)(0.68)(114)$

$= 3.7$ ac-ft

Step 7. Compute mean depth (Z) of permanent pool for lake eutrophication model

Z = Basin Volume/Surface Area (V_B/A_S)

$= 3.7/1.0$

$= 3.7$ ft <u>or</u> 1.1 m

(lake eutrophication model requires mean depth in units of meters)

A mean depth of 1.1 meters is within the recommended range of 1 to 3 meters.

Step 8. Assume that the cloverleaf interchange is an urban highway and the average daily traffic has more than 30,000 vehicles per day. Therefore, the mean total phosphorus in highway runoff discharges is chosen to be 0.4 mg/L or 400 ug/L which are the units required for the model. The orthophosphorus to total phosphorus ratio is set at 0.25.

Step 9. Compute the phosphorus removal efficiency from the lake eutrophication model equations. First apply the second order decay rate equation:

$K_2 = 0.056(Q_s)(F)^{-1}/(Q_s + 13.3)$

where $F = 0.25$ and $Q_s = Z/T = 1.1/0.0481 = 22.9$ m/yr

$K_2 = 0.056(22.9)(0.25)^{-1}/(22.9 + 13.3)$

$= 0.142$

Then, compute removal efficiency with the equation:

$R = 1 + [1 - (1 + 4N)^{0.5}]/(2N)$

where $N = (K_2)(P)(T)$

$= (0.142)(400)(0.0481)$

$= 2.73$

$R = 1 + [1- (1 + (4 \times 2.73))^{0.5}]/(2 \times 2.73)$

$= 1 + [- 2.45/5.46]$

$= 0.55$ or

Total Phosphorus Removal Efficiency = 55 percent

Step 10. Particulate removal efficiency for TSS can be estimated from figure 20.

$V_B/V_R = 3.7/0.68 = 5.4$

$H = 6$ ft

(For a volume of 3.7 ac-ft, the depth-volume relationship gives a depth of approximately 6 ft

E = 93 percent (figure 20)

For TSS removal efficiency E = 93 percent, additional pollutant removal efficiencies are:

Lead removal efficiency (.90 x 93) = 84 percent

Copper removal efficiency (.60 x 93) = 47 percent

Zinc removal efficiency (.45 x 93) = 42 percent

TKN, BOD removal efficiency (.20 x 90) = 19 percent

Step 11. The selected wet detention basin is conservatively designed. Precautions could be taken against short-circuiting by adopting an irregular shoreline to increase the distance between the inlet and outlet. If use on an irregular basin perimeter results in a reduction in basin volume and significant reductions in the estimated pollutant removal efficiencies, the reduced volume can be compensated for by a small increase in basin depth.

Step 12. Final design for bank slopes will be determined based on the natural contours and grading as required to maintain a slope ratio of 3:1 (H:V). The side slopes along the shoreline should be 5:1 (H:V) to minimize safety hazards and promote the growth of wetland vegetation.

4. CONSTRUCTION CONSIDERATIONS

The total cost of wet detention basins is highly dependent on land acquisition costs if the basin cannot be built within the rights-of-way acquired for construction of the highway. Costs per unit area served typically decrease with increasing drainage area size. This makes it advisable, where practicable, to construct basins to serve larger combined areas, as opposed to constructing numerous smaller detention basins serving individual drainage areas.

Detention basins may be the most practical stormwater management measure for highways where BMP retrofitting is necessary. However, the costs of retrofit detention basins may be significantly greater than the costs of basins included in plans for highway construction, particularly where traffic conflicts with construction equipment will be difficult to handle, where additional rights-of-way will be required, and where drainage system modifications will be necessary. If practicable, retrofitting should be accomplished in conjunction with a larger project to upgrade the level of traffic service so that necessary traffic disruption will be more acceptable to the public.

5. OPERATION AND MAINTENANCE

A. INSPECTIONS

Inspections should be performed at regular intervals to assure that the detention basin is operating as designed. Annual inspections should be considered at a minimum with additional inspections following storm events. Some inspections can be arranged to coincide with scheduled maintenance visits in order to minimize site visits and to ascertain that maintenance activities are performed satisfactorily. The embankment, emergency spillway, and side slopes of the basin should be checked to ensure that they do not show signs of erosion, settlement, slope failure, tree growth, wildlife damage, or vehicular damage.

B. ROUTINE MAINTENANCE

Routine or preventive maintenance refers to procedures which are performed on a regular basis in order to keep the basin sightly and in proper working order. Routine maintenance should include grass mowing, debris removal, and nuisance control of insects, weeds, odors, and algae as required.

C. NONROUTINE MAINTENANCE

Nonroutine or corrective maintenance refers to a rehabilitative activity that is not performed on a regular basis.

Erosion and Structural Repair

Areas of erosion and slope failure should be filled and compacted, if necessary, and reseeded as soon as possible. Eroded areas near the inlet or outlet should be revegetated and, if necessary, be filled, compacted, and reseeded or lined with riprap. Damaged side slopes and embankments should be repaired using fill dirt of adequate permeability. Major damage to inlet/outlet and riser structures should be repaired as soon as possible.

Access to wet detention basins is necessary for excavating equipment, trucks, mowers, and personnel for routine maintenance and erosion repair and for the removal of sediment accumulation. Where access is particularly difficult or impractical, basins should be overdesigned to allow for sediment accumulation.

Sediment Removal and Disposal

Sediment removal is a very important maintenance activity because wet detention basins are designed to remove pollutants by sedimentation. Sediments collect at the bottom of the basin, reducing storage volume, and accumulated sediment can reduce the pollutant removal efficiency of the basin.

Under existing EPA regulations (40 CFR 261), any material cleaned from a wet detention basin should be screened to determine whether it is a solid waste and whether it is a hazardous waste. Sediment accumulated in a wet detention basin qualifies as a solid waste and is subject to the Extraction Procedure (EP) toxicity test. This test should be carried out for accumulated sediment. If the sediment fails the test, it is subject to the Resource Conservation and Recovery Act (RCRA) regulations, and must be disposed of in an approved manner at a RCRA approved facility. If the EP toxicity test is negative, then the States are free to impose their own solid waste regulations.

For sediment which is not classified as a hazardous waste, two major options of disposal are available: onsite and landfill disposal. The area required for onsite disposal must be determined to assure adequate space for sediment disposal. For wet detention basins, sediment removal may be required approximately every 10 years. The disposal area should be large enough to stockpile two sediment clean-outs assuming the area can accept a 12-in (30.48 cm) depth of wet sediment for each clean-out. (MWCOG, 1987) Any onsite disposal areas must be protected with sediment control measures to prevent material from reentering the watercourses. To be consistent with guidelines for landfilling and land application of sludge, the disposal area should not be in the 100-year floodplain nor in wetlands. (EPA, 1988) The minimum depth to groundwater should be 5 ft (1.53 m), and the location should be a minimum distance of 100 ft (30.50 m) from surface waters.

If onsite disposal areas are not available or are inadequate in size, which may be the case for larger detention basins, then steps must be taken to transport the material to local landfills. Detention basin sediment is typically accepted at landfills by local government departments of solid waste if the material has been sufficiently dried to be a "workable material" and can pass an EP toxicity test.

Detention Basin Sediment and Municipal Sludge Comparison

Core sample data from wet detention basins were analyzed to determine the concentrations of metals in the sediment. These concentrations were compared to the concentrations found in sludges produced by municipal wastewater treatment plants in order to demonstrate the relative quantity of pollutants contained in detention basin sediment compared to other solid wastes. Table 8 presents a summary of the bottom sediment data for detention basin BMP's which control highway runoff in Connecticut, Minnesota, and Florida. Average, maximum, and minimum sediment concentrations for chromium (Cr), copper (Cu), lead (Pb), and zinc (Zn) are listed in the table. Table 8 also gives background information on the sites, including years in service, ADT, and surface area.

By comparison, the EPA mean concentrations found in sludges produced by publicly owned wastewater treatment works for the four metals are presented at the bottom of table 8. (EPA, 1982)

As shown in table 8, the maximum and mean sediment concentrations in the wet detention basin bottom sediments are typically an order of magnitude less than concentrations found in municipal wastewater sludge. Lead concentrations in the BMP bottom sediments typically come closest to the municipal wastewater sludge levels, although the levels in BMP sediments are still much lower. In addition to the general guidelines for sediment removal from wet detention basins (approximately every 10 years), core samples from detention basins every few years could be used to monitor the build-up of pollutants. If bottom sediment concentrations approach levels which would restrict disposal onsite or in local landfills, then clean-out may be required more frequently than every 10 years.

Table 8. Concentrations of selected heavy metals in detention basin bottom sediments and wastewater sludge.

Location	Background Data		Sediment Concentration (mg/kg)			
			Chromium	Copper	Lead	Zinc
Connecticut	Years in Service = 3	Avg.	13.5	18.6	39.4	51.5
	ADT = 71,000	Min.	4.1	6.6	13.0	19.8
	Surface Area = 0.11 ac	Max.	25.7	34.9	73.5	93.8
Minnesota	Years in Service = 5	Avg.	21.5	23.8	59.9	110.0
	ADT = 53,000	Min.	11.0	12.0	21.0	53.0
	Surface Area = 0.65 ac	Max.	31.0	38.0	97.0	198.0
Florida	Years in Service = 5	Avg.	30.7	13.0	124.8	105.8
	ADT = 42,000	Min.	7.8	5.2	28.8	35.4
	Surface Area = 1.6 ac	Max.	54.6	37.1	294.8	349.3
Publicly owned treatment works sludge[1]		Avg.	428.0	564.0	378.0	1,410.0

1. Source: EPA, 1982.

4. DRY EXTENDED DETENTION BASINS

Dry extended detention basins can be used in place of wet detention basins where the major concern is for the removal of particulate forms of pollutants and not the additional removal of soluble pollutants. Dry extended detention basins capture stormwater runoff and release it over an extended period of time. Sedimentation of suspended solids during the extended dewatering period is the primary removal mechanism for pollutants.

Dry extended detention basins can be designed not only for extended detention but also for flooding and erosion control. Figure 30 is an example of a two-stage design which has a perforated riser for drawdown of the extended detention pool with additional storage for flooding and erosion control. Detention basin designs will also include an emergency spillway to prevent overtopping of the dam during extreme storm events.

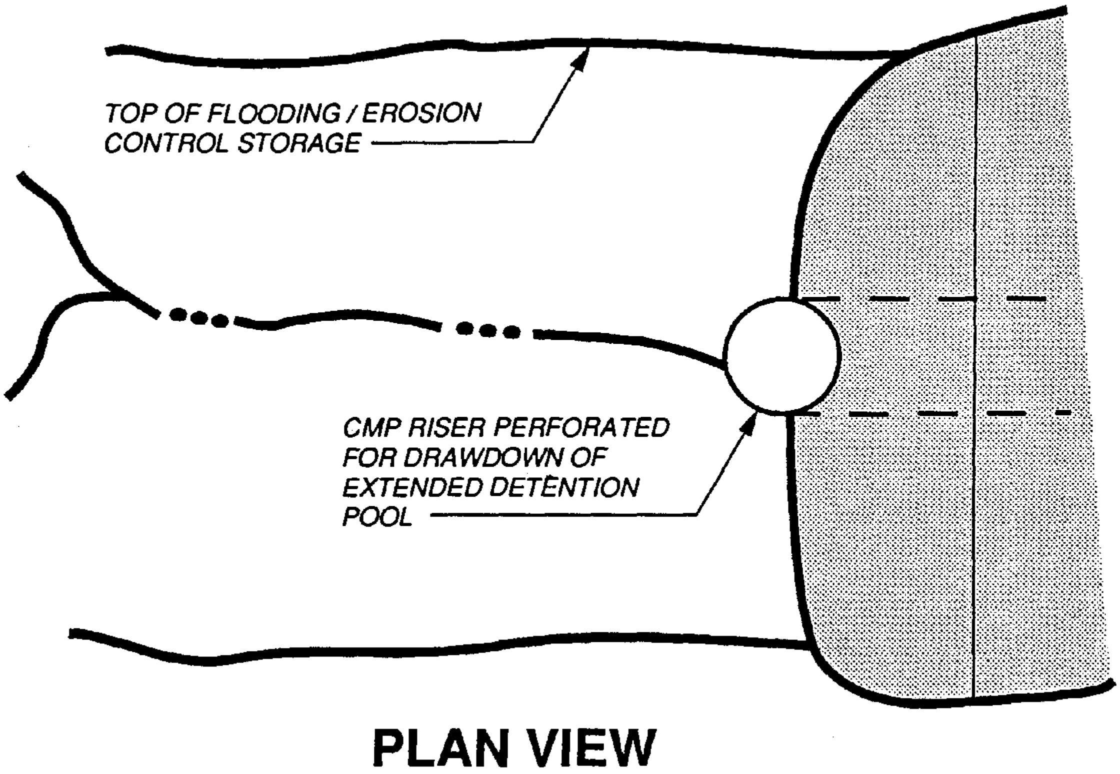
For pollutants like heavy metals and sediment which can be removed through sedimentation processes, dry extended detention basins can be designed to achieve pollutant removal efficiencies similar to wet detention basins. Since a significant percentage of the nutrient loadings in stormwater runoff is in dissolved form, dry extended detention basins are less efficient than wet detention basins for the removal of phosphorus and nitrogen. Consequently, dry extended detention basins are most appropriate for situations where the receiving water quality concern is <u>not</u> related to nutrient loadings.

Dry extended detention basins typically require much smaller storage volumes to achieve the same level of pollutant removal efficiency for metals, sediment, and pollutants found primarily in suspended form. The dry basins have a cost advantage over the wet basins in that the former require much less storage volume and space to achieve the same pollutant removal efficiency for suspended pollutants. Therefore, dry extended detention basins are likely to be more cost-effective than wet detention basins if the control of nutrient loadings is not critical. Further, dry detention basins may be preferable for some sites where major wetlands destruction would result from the construction of a wet detention basin.

1. <u>DESIGN CONSIDERATIONS</u>

Dry extended detention basin performance can be characterized by the amount of runoff detained, the duration of the dewatering period, and the removal efficiency associated with the extended period of time that the runoff volume is detained.

The storage volume subjected to extended detention for pollutant removal from highway stormwater runoff should be large enough to capture and "treat" a significant percentage of annual nonpoint pollution loadings. Criteria for the extended detention storage volume typically are based upon "first-flush" runoff. (NVPDC, 1979; Hartigan et al., 1980) The term "first-flush" runoff refers to the washoff of a large percentage of the total storm pollution load by a relatively small percentage of the storm runoff volume. Thus, capture and treatment (i.e., sedimentation) of a relatively small percentage of total annual runoff volume can achieve



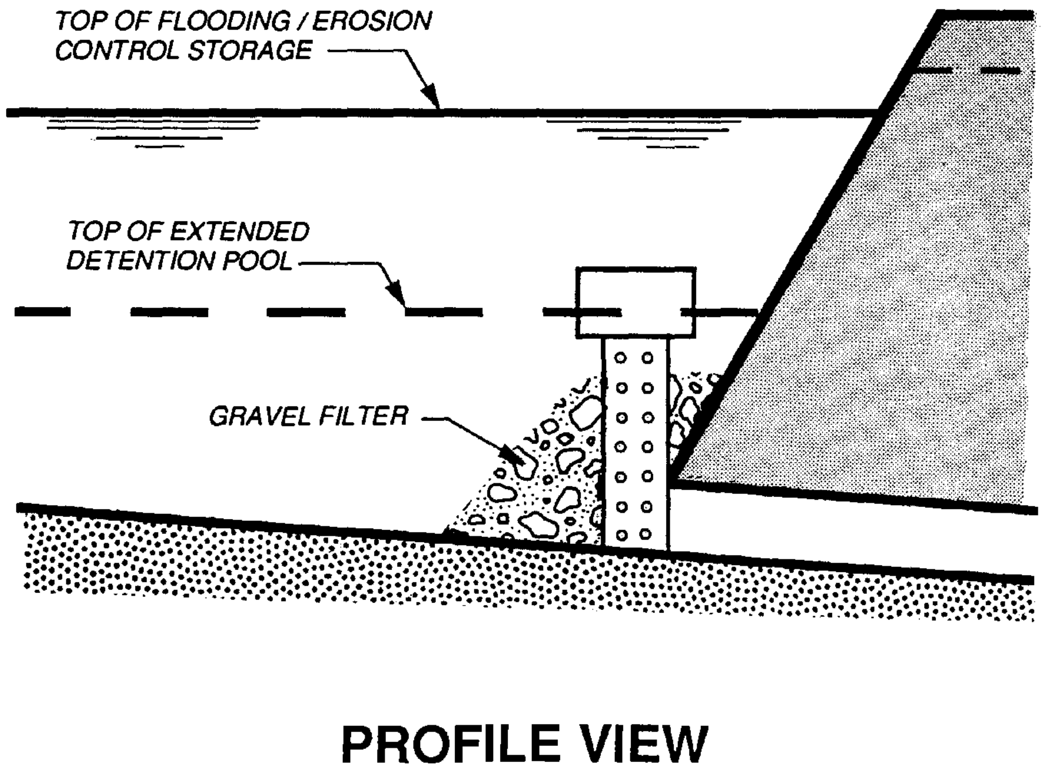


Figure 30. Schematic of dry extended detention basin.

significant removal of suspended pollution loadings. The design procedure recommended here for water quality control is based on the extended detention of runoff produced by the mean storm. Rainfall volumes and runoff coefficients used for dry extended detention basins are consistent with those presented for the analysis of wet detention basins.

Pollutant removal efficiency is based on settling behavior of the particulate pollutants. Experimental settling column data and field monitoring data for dry extended detention basins have been used to evaluate pollutant removal performance. Settling column experiments were performed and removal efficiencies of various pollutants for different settling times was determined. (Driscoll, 1986; Grizzard et al., 1986; and Occoquan Watershed Monitoring Laboratory, 1983) Settling column studies were also performed as part of this study for 13 storms monitored at 6 highway sites in the Virginia suburbs of Washington, D.C. Two field studies at extended detention basins in the Washington, D.C. area were also performed: one during the USEPA NURP study in Montgomery County, Maryland; and one in northern Virginia. (MWCOG, 1983; Occoquan Watershed Monitoring Laboratory, 1987)

Based on the available settling column data for urban and highway runoff, it can be concluded that the majority of pollutant removal by settling occurs within the first 6 to 12 hours after the runoff is discharged into the detention basin. This indicates that dry basins which achieve extended detention times on the order of 6 to 12 hours should maximize the removal of suspended pollutants. A dry extended detention basin should be designed to achieve a total dewatering period of 24 hours which will result in an average detention time on the order of 12 hours.

The method adopted here for the determining the performance of a dry extended detention basin is based on settling column studies and field measurements. The pollutant removal efficiencies presented in the Design Procedures section are based on 12 hour detention times.

Several other factors other than detention volume and detention time influence the performance of dry extended detention basins. The extended detention control device must be designed to minimize the resuspension of pollutants with the advent of each new storm. As with wet detention basins, bank erosion increases the suspended sediment load entering the basin and decreases performance. Bank slopes should be 3:1 or flatter and grass should be maintained on the slopes. Inlet and outlet structures should be designed to minimize erosion. Relatively high length: width ratios can enhance sedimentation in extended dry basins. In addition, the location of the outlet structure within the detention basin should maximize travel time from the inlet to the outlet.

In addition to the aforementioned general design considerations, diversion structures can be used to achieve "offline" operations of dry extended detention basins. A diversion structure should be designed to channel the first-flush runoff into the detention basin. This can be achieved by placing a weir across a natural or man-made channel to divert the flow to the basin and allow any flow which exceeds the first-flush flow to continue down the channel.

A "diversion box" can be used for areas with storm sewers discharging to a dry extended detention basin. A diversion box is a concrete chamber that is bisected by a side weir to divert flows exceeding the first-flush criterion into a storm sewer which bypasses the detention basin. A schematic of a diversion box operation for highway runoff control is shown in figure 31.

Diversion structures can also be used to capture the first-flush from an adjacent highway for treatment in an offline detention basin. Subsequent larger flows from upstream offsite areas would overflow the diversion weir and continue downstream.

2. DESIGN PROCEDURES

The following design procedure for dry extended detention basins relies upon many of the same rainfall and runoff characteristics used in the wet detention basin design procedures (chapter 3). Important design criteria are as follows:

- Detention storage volume: runoff from long-term mean storm rainfall.
- Dewatering period: 24 hours (average detention time of 12 hours).
- Surface area.
- Mean depth: 2 to 6 ft (0.61 to 1.83 m) should be appropriate for most highway runoff designs.

The design procedure for extended dry detention basins consists of the following steps:

1. In the absence of local rainfall statistics, identify the rainfall zone in which the detention basin will be located from figure 19.
2. Determine the long-term mean rainfall volume (depth) for the appropriate zone by converting the value in table 7 from inches to feet.
3. Establish the runoff coefficient (use procedure presented in chapter 3, section 2.C, RUNOFF COEFFICIENT).
4. Compute the long-term mean event runoff volume to be stored within the extended detention pool (use procedure presented in chapter 3, section 2.D, RUNOFF VOLUME).
5. Establish the dimensions of the area in which a dry extended detention basin could be constructed.
6. Select the extended detention depth and determine basin surface area required based on side slopes, storage volume, and depth.

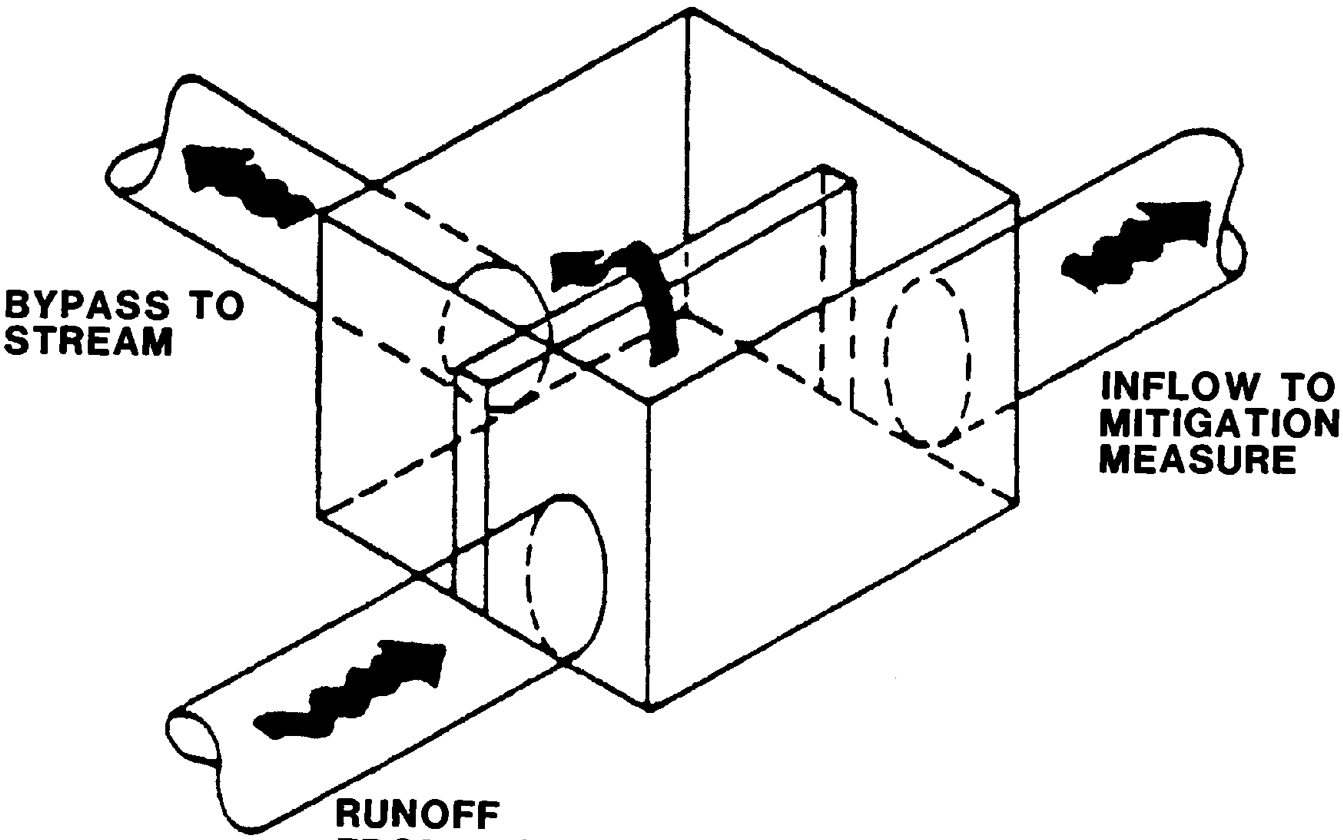


Figure 31. Schematic of diversion box operation for highway runoff control.

7. Design basin configuration for maximum length to width ratio specifying bank slopes at 3:1 (H:V) or flatter, design trickle channel to provide drainage to outlet works, grass cover for basin area, and 24-hour draw down period to provide an average 12-hour detention period.

8. Size the outlet structure to achieve the required dewatering time of 24 hours (see chapter 4, section 2.A, OUTLET STRUCTURE).

9. Determine pollutant removal efficiencies (see chapter 4, section 2.B, POLLUTANT REMOVAL EFFICIENCIES).

A design worksheet is presented in figure 32 to facilitate the design procedure and to record pertinent design parameters.

A. OUTLET STRUCTURE

Outlet structures for dry extended detention basins are designed to release the ponded water over an extended period of time (e.g., 24-hour dewatering time to provide an average detention time of 12 hours). The most common design is a corrugated metal pipe riser which is perforated with a series of small holes about one-half to 1 in (2.54 cm) in diameter. The total area of area of these holes controls the slow release of the water detained in the extended detention pool. If a concrete box riser is used as the outlet structure, then a metal plate with a series of holes can be placed over an opening in the riser. Gravel is placed around the riser holes to prevent the holes from clogging with sediment. The number of holes designed for the riser is based on the flow rate required to dewater the stored volume over a given period of time. The flow rate is calculated by the following equation:

$$Q = V_R\ (43560)/T_D\ (3600) \tag{6}$$

where:

Q = Release rate, ft^3/sec

V_R = Runoff volume to be stored, ac-ft

T_D = Dewatering time, hrs

The total area of the holes is calculated from the release rate and driving head by using the orifice equation:

$$Q = CA\ (2\ gh)^{0.5} \tag{7}$$

where:

Q = Release rate, ft^3/sec

C = Orifice flow coefficient

DRY EXTENDED DETENTION BASIN DESIGN WORKSHEET

*****************************DESIGN DATA************************************

Drainage Area ______________ (A_D) ac

Location ____
(City, State)

Rainfall Zone ______________
(from figure 19)

Mean Storm Event Volume ______________ (R_V) ft
(from table 7)

Runoff Coefficient

Pervious Area Coefficient ______________ (C_P)

Impervious Area Coefficient ______________ (C_I)

Impervious Area/Total Area ______________ (X)

Runoff Coefficient ______________ (C)
($C = C_P + (C_I - C_P)X$)

******************************DESIGN PROCEDURE*****************************

Runoff Volume from Mean Storm Event ______________ (V_R) ac-ft
($V_R = CR_VA_D$)

Area of Space Available for Basin ______________ ac

Selected Mean Depth of Extended Detention Pool ______________ ft

Surface Area of Extended Detention Pool ______________ ac

Outlet Structure

Release Rate ______________ ft^3/sec

Type and Size of Opening ______________

____

Figure 32. Dry extended detention basin design worksheet.

A = Total area of holes, ft^2

g = Gravity constant, 32.2 ft/sec/sec

h = Driving head, ft

B. POLLUTANT REMOVAL EFFICIENCIES

Table 9 summarizes average pollutant removal efficiencies for dry extended detention basins based on settling column data and field monitoring data. Settling column data from NURP studies, and the settling column data produced as part of this study were evaluated to establish the removal efficiencies for TSS, lead, copper, and zinc. (EPA, 1983; OWML, 1983) Removal efficiences for phosphorus, TKN, and BOD were determined by evaluating the results of two field monitoring studies of dry extended detention basins in the metropolitan Washington, D.C. region. (MWCOG, 1983; Occoquan Watershed Monitoring Laboratory, 1987)

3. CONSTRUCTION CONSIDERATIONS

As with wet detention basins, the total cost of dry extended detention basins is highly dependent on land acquisition costs if the basin cannot be built within the rights-of-way acquired for construction of the highway. Cost per unit area served decreases with increasing drainage area size. This makes it advisable, where practicable, to construct basins to serve larger combined areas, as opposed to constructing numerous smaller detention basins serving individual drainage areas.

Detention basins may be the most practical stormwater management measure where retrofitting is necessary. However, the costs of retrofit detention basins may be significantly greater than the costs of basins included in plans for highway construction, particularly where traffic conflicts with construction equipment will be difficult to handle, where additional rights-of-way will be required, and where drainage system modifications will be necessary. If practicable, retrofitting should accomplished in conjunction with a larger project to upgrade the level of traffic service so that necessary traffic disruption will be more acceptable to the public.

4. OPERATION AND MAINTENANCE

A. INSPECTIONS

Inspections should be performed at regular intervals to assure that the detention basin is operating as designed. Annual inspection should be considered at a minimum with additional inspections following storm events. For the inspection following a major storm, the inspector should visit the site at the end of the specified dewatering period to ensure that the extended detention device is draining properly, checking for clogging or poor design which would release the water too rapidly. Some inspections can be arranged to coincide with scheduled maintenance visits in order to minimize site visits and to ascertain that maintenance activities are performed satisfactorily. At the time of all site visits, the inspector should check accumulations of debris and sediment. The embankment,

Table 9. Average pollutant removal efficiencies for dry extended detention basins.

Pollutant	Removal Efficiency (%)
TSS	80-90
Total Lead	70-80
Copper	50-60
Zinc	40-50
Total Phosphorus	20-30
Total Kjeldahl Nitrogen	20-30
BOD	20-30

SOURCE: Hartigan, 1988.

emergency spillway and side slopes of the basin should be checked to ensure that they do not show signs of erosion, settlement, slope failure, tree growth, wildlife damage, or vehicular damage.

B. ROUTINE MAINTENANCE

Routine or preventive maintenance refers to procedures which are performed on a regular basis in order to keep the basin sightly and in proper working order. Routine maintenance should include grass mowing, debris removal, and clearing around the extended detention control devise to prevent clogging.

C. NONROUTINE MAINTENANCE

Nonroutine or corrective maintenance refers to a rehabilitative activity that is not performed on a regular basis.

Erosion and Structural Repair

Areas of erosion and slope failure should be filled and compacted, if necessary, and reseeded as soon as possible. Eroded areas near the inlet or outlet should be revegetated and, if necessary, be filled, compacted, and reseeded or lined with riprap. Damaged side slopes and embankments should be repaired using fill dirt of adequate permeability. Major damage to inlet/outlet and riser structures should be repaired as soon as possible.

Access to dry extended detention basins is necessary for excavating equipment, trucks, mowers, and personnel for routine maintenance and erosion repair and for the removal of sediment accumulation. Where access is particularly difficult or impractical, basins should be overdesigned to allow for sediment accumulation.

Sediment Removal and Disposal

Sediment removal is a very important maintenance activity for dry extended detention basins because these facilities are designed to remove pollutants by sedimentation. Sediments collect at the bottom of the basin reducing storage volume and increasing the likelihood of clogging the orifices of the extended detention outlet structure. As discussed above, dry extended detention basins may have to be cleaned out more frequently than wet detention basins for aesthetic reasons.

The guidelines presented in chapter 3 for the disposal of wet detention basin bottom sediments also apply here.

5. DESIGN EXAMPLE

The site used to illustrate the design procedure for the dry extended detention basin is the same site used for the wet detention basin example and is shown in figure 33. The basin location is in one quadrant of a cloverleaf interchange. The basin will serve a drainage area of 40 acres (16.2 ha) and 42 percent of the area is roadway pavement. The basin is

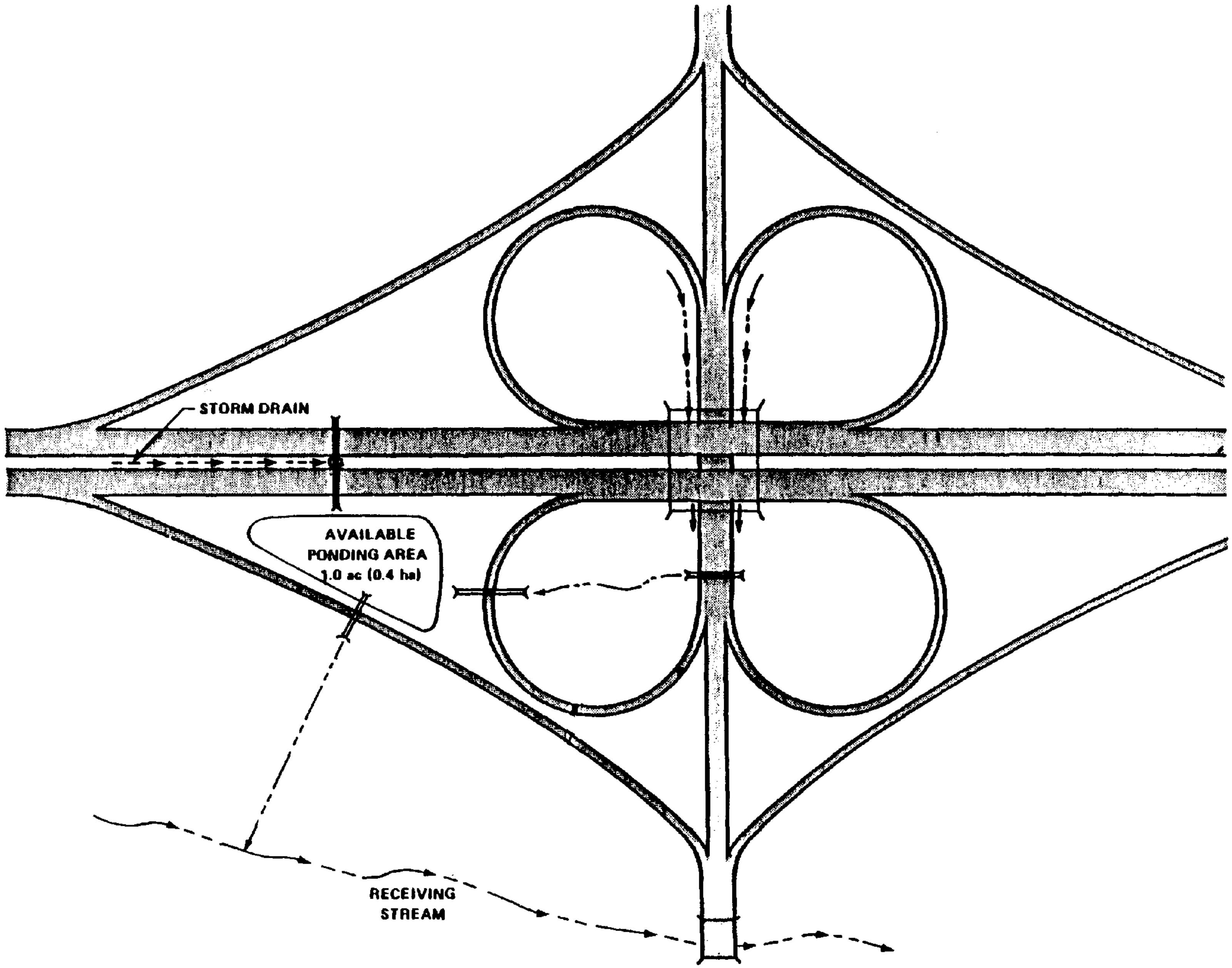


Figure 33. Example - dry extended detention basin design.

DRY EXTENDED DETENTION BASIN DESIGN WORKSHEET
(CONTINUED)

****************************DESIGN DATA************************************

Drainage Area	40	(A_D) ac
Location	Knoxville, Tennessee (City, State)	
Rainfall Zone (from figure 19)	2	
Mean Storm Event Volume (from table 7)	0.36/12	(R_V) ft
Runoff Coefficient		
Pervious Area Coefficient	0.4	(C_P)
Impervious Area Coefficient	0.8	(C_I)
Impervious Area/Total Area	0.42	(X)
Runoff Coefficient ($C = C_P + (C_I - C_P)X$)	0.57	(C)

****************************DESIGN RESULTS********************************

Runoff Volume = Basin Volume ($V_R = CR_VA_D$)	0.68	(V_R) ac-ft
Area of Space Available for Basin	1.0	ac
Selected Mean Depth of Extended Detention Pool	2.0	ft
Surface Area of Extended Detention Pool	0.45	ac
Outlet Structure		
Release Rate	0.34	ft^3/sec
Type and Size of Opening	perforated riser, 12 1-inch holes	

Figure 33. Dry extended detention basin design example (continued).

assumed to be constructed in Knoxville, Tennessee. The steps used in designing the basin are detailed below and used to complete the design worksheet (figure 33).

Step 1. From figure 19, Knoxville, Tennessee, is in rainfall zone 2.

Step 2. From table 7, the mean storm event rainfall volume is 0.36 inches.

Use $R_V = 0.36$.

Step 3. Compute the coefficient, C, for use in the equation, $V_R = CR_VA_D$. Use equation (1) to estimate the value of C.

$$C = C_P + (C_I - C_P)X$$

Assume $C_P = 0.4$

$C_I = 0.8$

$$C = 0.4 + (0.8 - 0.4)(.42)$$

$$= 0.57$$

Step 4. Compute the runoff volume used to size the extended detention pool.

$$V_R = CR_VA_D$$

$$= 0.57\ (0.36/12)40$$

$$= 0.68 \text{ ac-ft}$$

Note conversion of rainfall depth from in to ft and retention of ac as the unit of measure for the drainage area. If this convention is adopted, the volume of the detention basin must be computed in ac-ft in step 6.

Step 5. A dry extended detention basin can be constructed within a loop ramp of the interchange in figure 33, or between the ramps in a quadrant of the interchange. An elongated basin between ramps is a more favorable shape than the near circular basin within a loop for maximum length to width ratio.

Step 6. Basin depths and volumes are typically selected from depth-area-storage relationships which are developed based on the natural contours and location of the embankment at the detention basin site. Where natural site storage is limited, excavated basins are an option to embankment basins. For this example, a depth-area-storage relationship is given below:

Depth (ft)	(m)	Area (ac)	(ha)	Storage (ac-ft)	(ha m)
1	.305	0.31	.126	0.24	.038
2	.61	0.45	.182	0.62	.111
3	.915	0.59	.239	1.14	.219
4	1.22	0.72	.292	1.79	.356
5	1.525	0.86	.348	2.58	.531
6	1.83	1.00	.405	3.51	.851

The storage volume required for the extended detention pool is equal to the runoff volume of 0.68 ac-ft (839 m^3) computed in Step 4. From the depth-area-storage relationship, a mean depth of 2.1 ft (0.64 m) would be required. Thus, the detention basin should be designed for the mean depth of 2 ft (0.61 m), which would give a surface area of 0.45 acres (1821 m^2).

Step 7. Bank slope adopted for this design should be 3:1 (H:V) or flatter and the basin area should be grassed. The extended detention control device should be designed (e.g., perforated riser) to release the captured stormwater over a drawdown period of 24 hours which would provide for an average detention period of 12 hours.

Step 8. To size the outlet structure, first determine the release rate for a dry extended detention volume of 0.68 ac-ft and a drawdown time of 24 hours by equation 6:

$$Q = (0.68)(43560)/(24)(3600)$$

$$Q = 0.34\ ft^3/sec$$

Assume structure to be perforated riser with 1-in (2.54 cm) diameter holes. The maximum extended detention pool level is 2 ft (0.61 m). Based on an average depth of 1 ft (0.31 m) during drawdown and an orifice flow coefficient of 0.6, the total opening area is calculated from equation 7:

$$Q = CA(2gh)^{0.5}$$

where area is calculated by:

$$A = Q/(C)(2gh)^{0.5}$$

$$A = 0.34/(0.6)(2*32.2*1)^{0.5}$$

$$A = 0.07\ ft^2 \text{ or } 10\ in^2$$

For a total area of 10 in^2 (62.50 cm^2), about 12 1-in (2.54 cm) diameter holes would be required.

Step 9. Use table 9 to determine pollutant removal efficiency for pollutants of interest. Removal efficiencies are:

TSS	80 to 90 percent
Pb	70 to 80 percent
Cu	50 to 60 percent
Zn	40 to 50 percent
P	20 to 30 percent
TKN	20 to 30 percent
BOD	20 to 30 percent

5. RETENTION MEASURES

1. INTRODUCTION

Retention facilities differ from detention facilities in that they do not discharge "treated" waters into the surface runoff conveyance system. Instead, these measures release captured stormwater into the soil profile beneath the retention storage basin, thereby achieving significant pollutant removal through natural processes within the soil profile underlying the facility. The use of retention practices as highway runoff BMP's can result in several advantages in comparison with typical above-ground detention measures:

- Retention measures provide potential for significant reduction of both dissolved and suspended nonpoint pollutant loadings due to physical, chemical, and biological processes within the soil profile.

- Retention measures provide a natural means of groundwater recharge, thereby augmenting post-development baseflows and dry period low flows.

- Retention measures can significantly reduce total annual surface runoff volumes, and peak runoff from many storms each year, thereby reducing the adverse impacts highway runoff can have upon stream habitats.

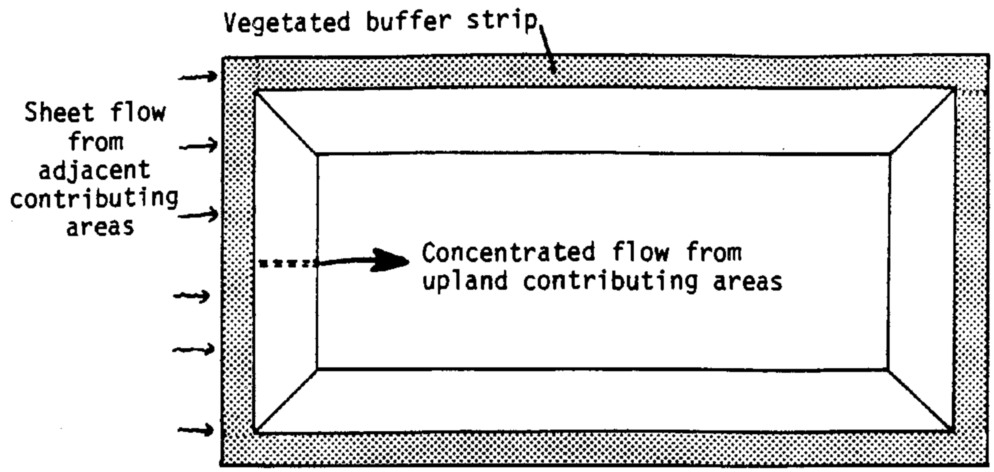
- Retention measures can result in improved control of post-development flooding and streambank erosion through the maintenance of outlet hydrographs which more closely resemble pre-development conditions.

In recognition of these advantages, State regulations specify that retention measures are the runoff control method of choice in Maryland and Florida. The use of these measures is also encouraged in other States.

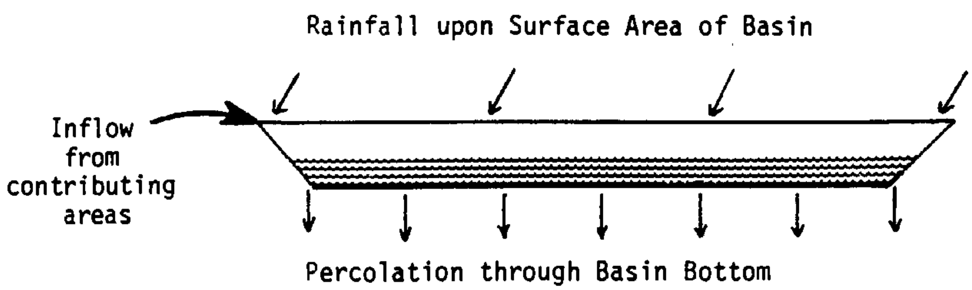
Highway runoff tends to exhibit a "first flush" phenomenon, whereby a large portion of the total pollutant load is concentrated in a relatively small portion of the total runoff volume associated with the rising limb of the runoff hydrograph. This first flush tendency enables retention measures to achieve considerable water quality benefits while storing fairly small volumes of highway runoff.

Retention measures which may be suitable for highway applications include:

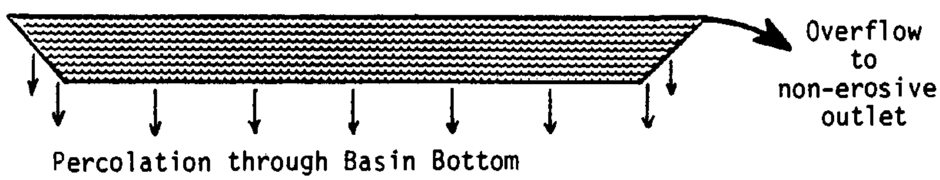
- Retention basin: An open pit or impoundment with vegetated sides which releases stored runoff by infiltration through the bottom and sides of the basin and is generally suitable for drainage areas of 5 to 50 acres (2 to 20 ha) (see figure 34).



(a): Top View of Infiltration Basin During Storm



(b): Cross Section of Infiltration Basin During Storm



(c): Cross Section of Overflowing Infiltration Basin

Figure 34. Schematic of retention basin during storm.

- Retention trench: An excavated trench backfilled with stone suitable for use on small watersheds, generally less than 5 acres (2 ha) (see figure 35).

- Retention well: A vertical shaft extending to pervious strata which either may be backfilled with aggregate or may be lined with precast concrete or metal pipe suitable for use on small watersheds, generally less than 2 acres (0.8 ha) (see figure 36).

Retention basin are the preferable mitigation measure for highway runoff since they are easier to maintain than retention trenches and wells. Trenches and wells are susceptible to similar clogging problems due to sediment and, to a lesser extent, oil and grease in highway runoff, although the trenches can be expected to clog primarily at the surface of the facility (i.e., upper layers of stone) while wells can exhibit clogging of the soil around the bottom and sides of the well. Due to the higher construction costs (e.g., excavation, casings) per cubic foot of runoff storage and the potential for clogging of deeper layers and less filtering prior to recharging groundwater, well systems may be less desirable measures than retention trenches for certain highway site conditions.

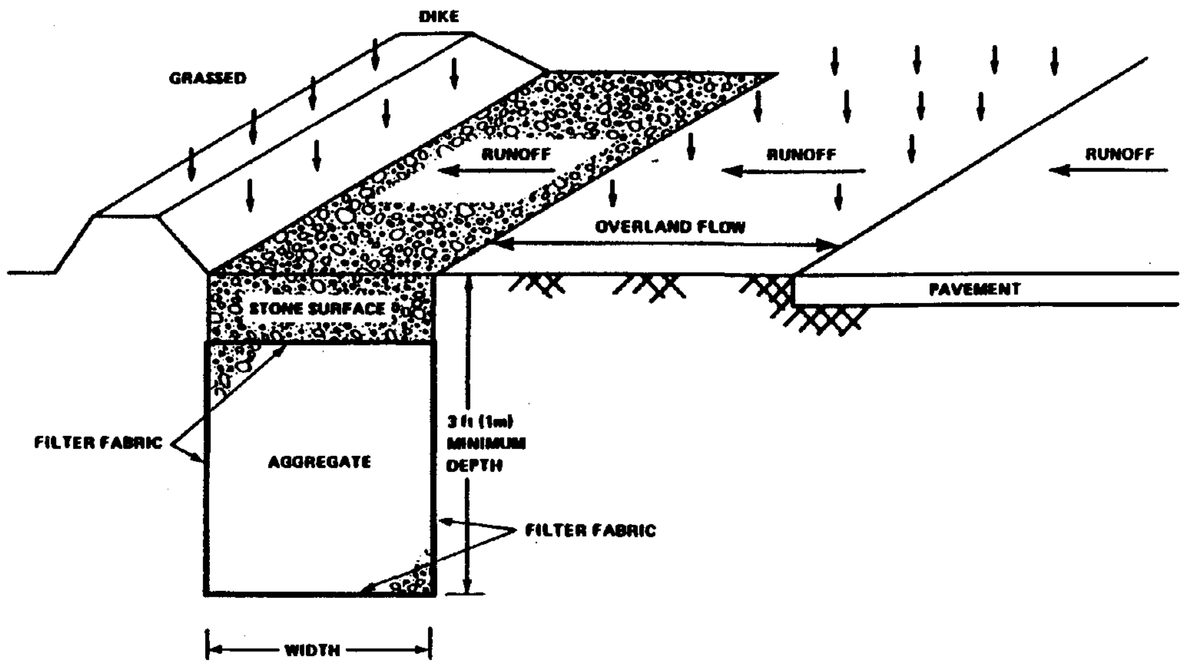
2. DESIGN CONSIDERATIONS

A. GENERAL CRITERIA

Unlike detention measures and vegetative controls, the feasibility of retention measures is very dependent upon site conditions. Typically, the site must satisfy the following feasibility tests in order to be suitable for a retention measure:

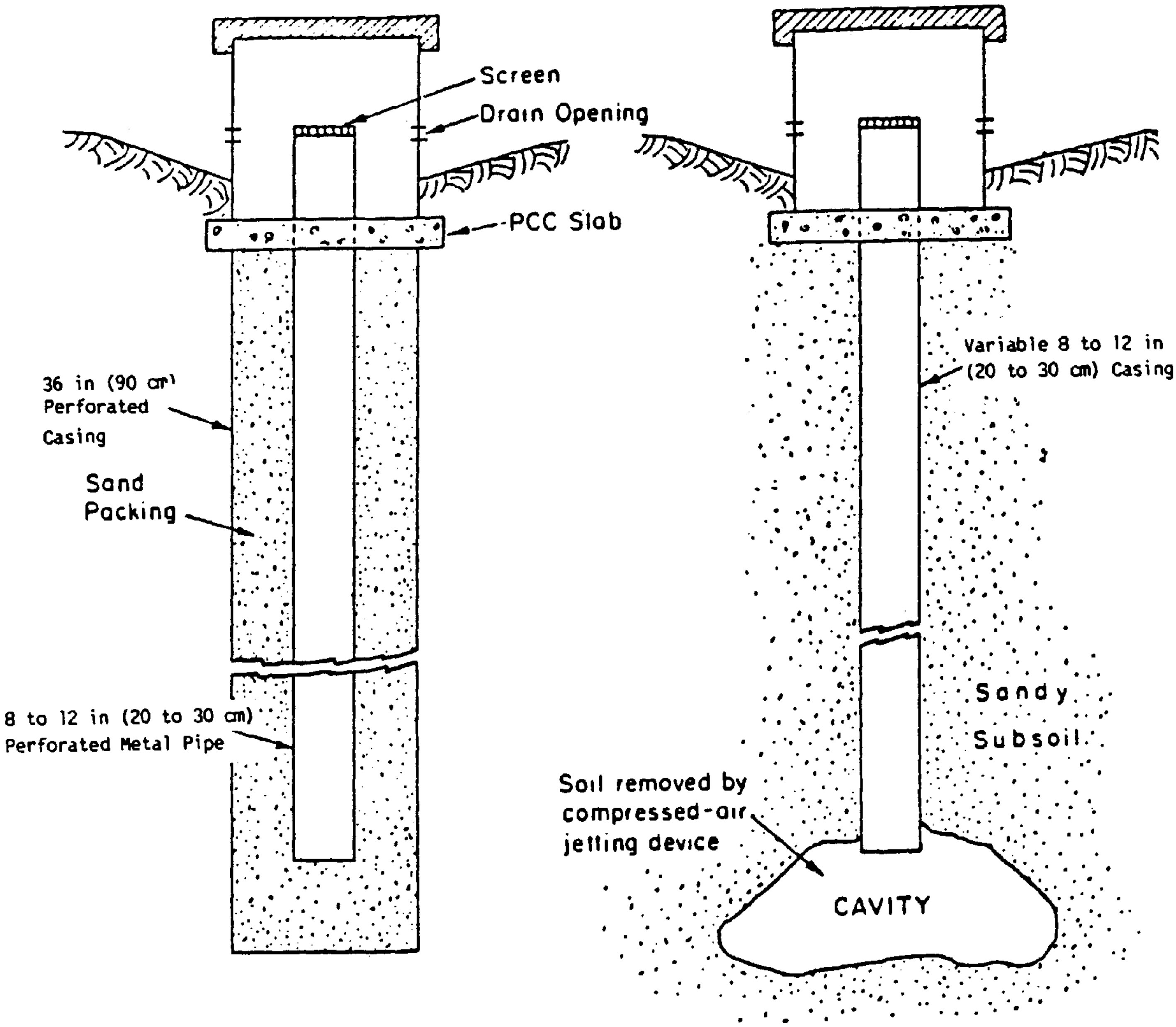
1. Saturated soil infiltration rate that permits adequate percolation of stored runoff: Typical values based on soil texture classifications are shown in table 10. Ideally, this value should be measured in the field by an appropriate method such as the double concentric ring infiltrometer test (ASTM Test Method D3385). It is recommended that retention measures be restricted to sites with minimum infiltration rates of about 0.3 in/hr (0.76 cm/hr) (i.e., silt loam soils in table 10) within the underlying and surrounding soil profile.

2. Maximum allowable dewatering time should minimize the risk of carryover runoff storage between rainstorms: If the retention facility requires an excessive amount of time to dewater, storage will not be available for runoff from subsequent rainstorms. Ideally, the dewatering time should be related to statistics on the interval between rainstorms in a particular rainfall zone (see figure 19). The States of Maryland and Florida require the use of a maximum 72-hour dewatering period for retention measure design, which is in the same ballpark as the mean interstorm intervals for rainfall zones 2 and 3 as shown in table 11. It is recommended that similar criteria be used for highway applications.



Source: Maryland WRA, 1984.

Figure 35. Retention trench controlling pavement runoff.



Source: Jackura, 1980.

Figure 36. Alternate drainage well installations.

Table 10. Saturated soil infiltration rates for soils suitable for retention measures.

Soil Texture	Saturated Soil Infiltration Rate (inches/hr)
Sand	8.3
Loamy Sand	2.4
Sandy Loam	1.0
Loam	0.5
Silt Loam	0.3

SOURCE: Adapted from Maryland WRA, 1984.

Table 11. Recommended dewatering times for different rainfall zones in the United States.

Rainfall Zone	Dewatering Time (hours)
1	72
2	72
3	72
4	72
5	96
6	264
7	96
8	96
9	72

NOTE: Dewatering times based upon the lower of the "annual" and "summer" values reported in table 7.

Table 11 summarizes the dewatering times recommended by this study for each rainfall zone in the United States, based upon the lower of the "annual" and "summer" values reported in table 7.

3. Minimum distance between the bottom of the facility and the seasonally high water table, bedrock, limestone, or other water-conducting strata: Adequate travel time through unsaturated soil is required to ensure sufficient pollutant removal. In the eastern United States, a minimum separation distance of 2 to 4 ft (0.61 to 1.22 m) is typically used in areas (e.g., Maryland and Florida) where water table depths are relatively shallow, while 10 ft (3.05 m) is typically used in some western States. If there are no standards required for a particular area, a minimum distance in the range 3 to 10 ft (0.92 to 3.05 m) should suffice, with the upper end of this range most suitable for areas where there is considerable concern about groundwater contamination potential. Where feasible, 8 to 10 ft (2.44 to 3.05 m) deep test pits should be excavated at prospective retention measure sites to map the stratigraphic profile and collect soil samples for gradation testing and analysis of stain markings which can indicate high water table elevations.

4. Acceptable topographic features: Certain retention measures may not be suitable for areas with relatively steep slopes (e.g., greater than 7 percent). Likewise, the use of retention measures on fill material is not recommended due to the possibility of creating an unstable subgrade. Finally, a retention facility should exhibit a minimum horizontal separation of 100 ft (30.50 m) from any water supply well adjoining the highway.

All three retention measures also require some type of upstream "pre-treatment" facility to minimize the loadings of solids and debris that can cause clogging problems. The most appropriate upstream pretreatment devices are vegetative controls such as a grassed channel and grassed over-land flow areas. Because the vegetative controls are intended primarily to remove the larger sediment particles which can cause clogging problems in the retention measure, shorter travel lengths than those recommended in chapter 3, section 1 should suffice for pretreatment purposes.

B. DIVERSION STRUCTURES

Diversion structures are required for retention basins which are designed as off-line retention measures. Off-line locations are especially applicable in areas where infiltration cannot be achieved by constructing a basin in the main channel. A diversion structure should be designed to channel the first-flush component of highway runoff into the retention basin. This can be achieved by placing a weir across a natural or manmade channel to divert the flow to the basin and allow any flow which exceeds the first-flush flow to continue down the channel.

For areas that have storm sewers that would discharge to a retention basin, a "diversion box" can be used to isolate first-flush runoff for treatment. A diversion box is a concrete chamber that is bisected by a side weir to divert flows exceeding the first-flush runoff criterion into a storm sewer which bypasses the retention basin. A schematic of a diversion box operation for highway runoff control is presented in figure 31.

Diversion structures can also be used for off-line retention basins to capture the first-flush from adjacent highway runoff, and to bypass the main channel flow which would primarily carry off-site stormwater from larger upstream areas.

In addition to the aforementioned general design criteria and the required runoff storage volume, each retention measure has unique design criteria which are outlined below.

C. ADDITIONAL RETENTION BASIN CRITERIA

Depth

The maximum allowable depth of the basin is calculated by multiplying the saturated soil infiltration rate by the maximum allowable dewatering time shown in table 11.

Side Slopes

Basin side slopes should be 3:1(H:V) or flatter to prevent erosion, improve appearance, and facilitate maintenance. Also, the basin's side slopes should be vegetated with grass. The grass cover will provide protection from erosion and sloughing, and also provide a means of maintaining relatively high infiltration rates. Grasses used should be adaptable to dry sandy soils, drought resistant, hardy, and able to withstand periodic inundation. SHA's should select grass or grass mixes based on their own in-house experience for their regions.

Embankment

These guidelines apply to basins with earth embankment dams. Assuming that the total height of the embankment is less than 15 ft (4.58 m), a minimum top width of 8 ft (2.44 m) should be adequate. The combined upstream and downstream side slopes of the settled embankment should not be less than 5:1(H:V), with neither slope steeper than 2:1(H:V). For example, an embankment could be graded with upstream side slope of 3:1(H:V) and a downstream side slope of 2:1(H:V). Slopes should be designed to be stable in all cases, even if flatter side slopes are required.

The minimum freeboard requirement for embankment dams is a 2.0 ft (0.61 m) difference in elevation between the crest of the emergency spillway and the settled top of dam.

Outlet

For basins created by an embankment, particularly those with relatively large drainage areas, consideration should be given to the provision of a riser-pipe emergency spillway designed to pass an appropriate design storm (e.g., 100-year event). At a minimum, a nonerosive overflow or outlet channel leading to a stabilized watercourse should be provided for all basins.

D. ADDITIONAL RETENTION TRENCH CRITERIA

Depth

The depth of the retention trench will typically range from 3 to 10 ft (0.92 to 3.05 m). In general, the trench should be as deep as possible to minimize the surface area. The maximum allowable trench depth will be based upon whichever of the following is smaller: (a) the product of the maximum dewatering time (table 11) and the saturated soil infiltration rate divided by the void ratio; or (b) the maximum depth based upon the required separation distance between the bottom of the trench and bedrock or the seasonally high water table.

Backfill Material

The aggregate material for the trench should consist of clean stone with diameters in the range 1.5 to 3.0 in (3.81 to 7.62 cm). The void ratio for this aggregate material should be in the range 0.3 to 0.4.

Filter Fabric

As shown in figure 35, below the top 1 ft (0.31 m) of depth, the top, bottom, and sides of aggregate material should be completely surrounded with an appropriate geotextile filter fabric.

Outlet

Because of the small drainage areas controlled by a retention trench, an emergency spillway is not necessary. However, a nonerosive overflow channel leading to a stabilized watercourse should be provided.

E. ADDITIONAL RETENTION WELL CRITERIA

Depth

In the eastern United States, the depth of shallow dry wells backfilled with aggregate is typically about 3 to 12 ft (0.92 to 3.66 m). In western States and other sections of the United States, depths of drainage wells may be 30 ft (9.15 m) or greater. For shallow dry wells, the procedure for determining the maximum allowable depth is the same as the procedure described for a retention trench. For deep wells, the procedure for determining maximum allowable depth is related to the procedures used for a shallow dry well although the analysis is more rigorous. A rational deep well design procedure relies upon infiltration theory for the case of a constant head suddenly applied over a semi-infinite porous medium. (Weaver, 1971)

Backfill Material

The aggregate material for shallow dry wells should consist of clean stove with diameters in the range of 1.5 to 3.0 in (3.81 to 7.62 cm). The void ratio for this aggregate material should be in the range 0.3 to 0.4.

Filter Media

Similar to a retention trench, the top, bottom, and sides of aggregate material in a shallow dry well should be completely surrounded with an appropriate geotextile filter fabric. In addition to inlet designs that prevent debris from being washed directly into well chambers, filter cloth may also be placed at the top of deep wells to help minimize the frequency of clogging.

Upstream Detention Basins

Due to the limited storage available in well chambers which are typically no more than 12 in (30.48 cm) in diameter, upstream detention basins may be required for flow equalization purposes.

Outlet

Like retention trenches, a nonerosive overflow channel leading to a stabilized water course should be provided.

F. STORAGE REQUIREMENTS FOR NONPOINT POLLUTION MANAGEMENT

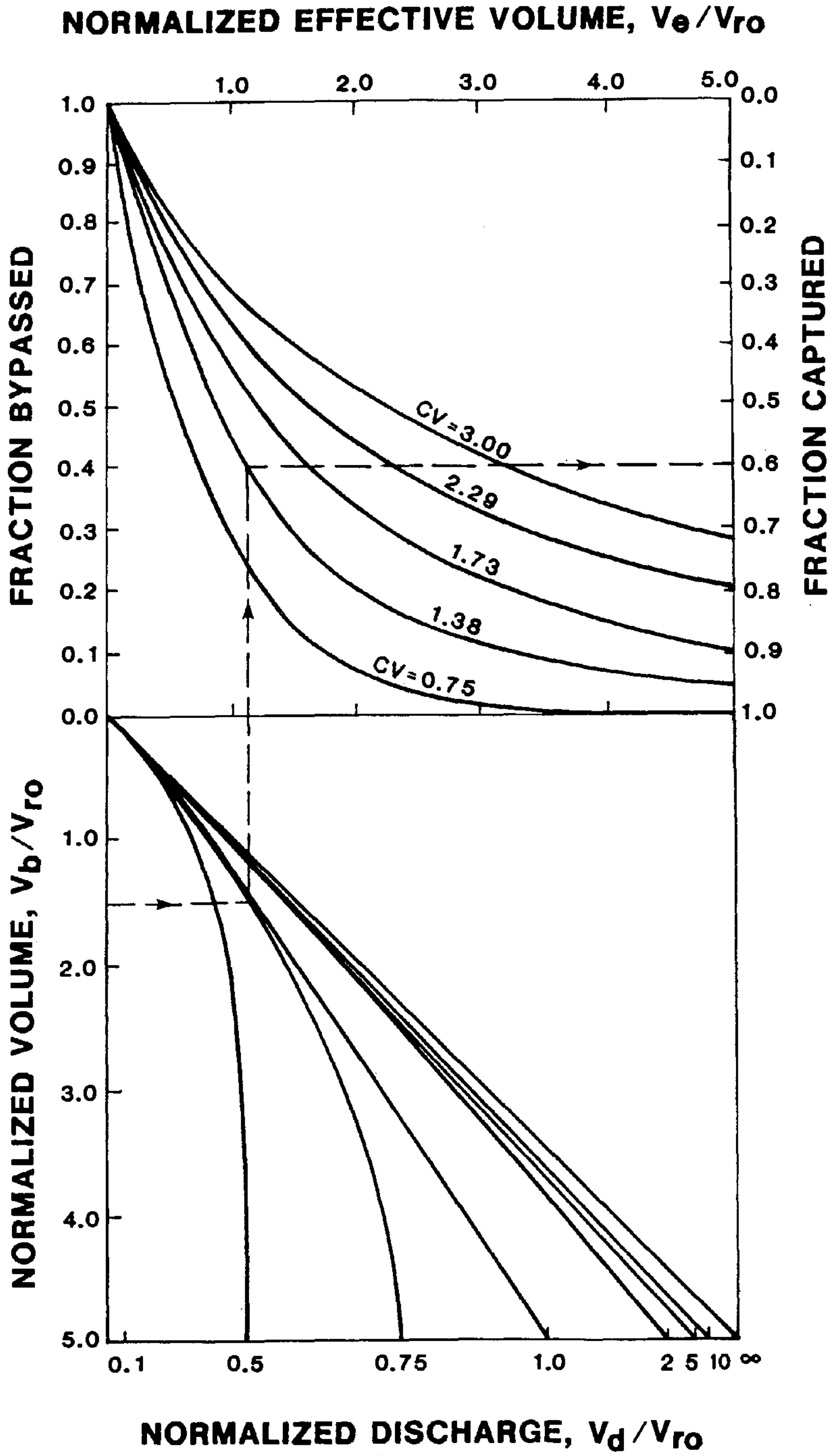
Retention measures are typically designed to capture and "treat" the first-flush flows in urban runoff. For example, Florida and Maryland have State regulations which require retention measures to provide storage for first-flush runoff. Florida requires a minimum storage capacity equivalent to the first 0.5 in (1.27 cm) of runoff from the entire drainage area, while Maryland requires a minimum storage equal to the first 0.5 in (1.27 cm) of runoff from impervious areas only.

Two of the retention measures (trenches and wells) operate as offline storage devices which capture the initial stages of runoff and automatically bypass subsequent flows when the available storage capacity is filled. Retention basins can operate as either online or offline storage devices which will exhibit overflows of captured first-flush runoff when storage capacities are exceeded. Therefore, an important design criterion is the volume of first-flush runoff which will be stored in retention facilities before bypasses or overflows occur. The larger the storage requirement, the higher the runoff capture efficiency and vice versa. It is typically assumed that natural mechanisms in the soil profile underlying retention facilities will achieve relatively high pollution removal rates for captured runoff waters, with removal rates of 90 percent or greater projected for heavy metals, BOD, and sediment and about 50 to 65 percent for nutrients. (Northern Virginia Planning District Commission, 1979)

Storage criteria for retention measures are typically based on analyses of the runoff capture statistics associated with different first-flush storage volumes. Two methods are available for developing "runoff capture-storage" relationships which account for the variability of runoff characteristics from storm to storm: (a) applications of continuous simulation models such as STORM (Hydrologic Engineering Center, 1977); SWMM (Huber et al., 1988); and NPS (Donigian et al., 1976) to route a long-term runoff record (e.g., 25 to 30 years) through the assumed offline storage volume; and (b) statistical methods which approximate capture-storage relationships based upon statistical properties of rainfall and runoff. (Goforth et al., 1983) The statistical method, which is similar to the detention basin design method (solids settling approach) outlined in an earlier section, is much simpler and easier to apply than the continuous simulation method. Comparisons of the two methods indicate that the statistical method can provide an adequate approximation of continuous simulation results with the statistical method typically producing a more conservative estimate (i.e., lower values) of runoff capture. (DiToro and Small, 1979; Goforth, et al., 1983) Because of the ease of application and the conservative results, the statistical method was selected for the retention storage analyses presented herein.

The selected method was originally developed for analyses of combined sewer overflow problems. (DiToro and Small, 1979) The method treats the duration, volume, and flow of runoff as independent and exponentially distributed random variables. The long-term average capture of runoff flows (expressed as a fraction of runoff flows) is related to the ratio of "effective storage volume" (V_e) and mean runoff volume (V_{ro}) and to the coefficient of variation for runoff volumes. V_e is defined as the storage capacity that is available on the average, after accounting for carryover storage between storms. For the same facility dewatering rate and runoff statistics, the smaller the selected first-flush storage volume, the greater the impact of carryover storage between storms and the lower the runoff capture efficiency. The end product of the statistical method is the set of graphs shown in figure 37. (Goforth, et al., 1983) These graphs enable the user to solve for V_e and "Fraction of flow captured," as a function of the storage released by dewatering (V_d), V_{ro}, and the assumed maximum storage capacity of the retention facility (V_b). The graphs in figure 36 are used as follows: (1) enter the lower graph at the V_b/V_{ro} ratio associated with the assumed storage capacity (V_b); (2) move horizontally until intersecting the appropriate normalized discharge curve (V_d/V_{ro}); (3) move vertically to the upper graph at the corresponding effective volume ratio (V_e/V_{ro}), the common axis between the graphs; (4) continue moving vertically until intersecting the appropriate runoff volume coefficient of variation curve (CV); and (5) move horizontally and exit at the "fraction captured" axis. For example, as shown in figure 37, a V_b/V_{ro} ratio of approximately 1.5 and a V_d/V_{ro} ratio of 1.0 yield a V_e/V_{ro} ratio of about 1.15 (i.e., the average effective storage is 15 percent greater than the mean runoff volume), which in turn yields a "fraction captured" of about 0.6 for a runoff volume coefficient of variation equal to 1.38.

To develop generalized estimates of runoff capture efficiency for different retention storage capacities, the graphs shown in figure 37 were applied to rainfall statistics (see table 7) for the nine U.S. rainfall zones (see figure 19). The rainfall statistics were developed from applications of the SYNOP program to long-term hourly rainfall records. (Hydroscience, 1976) The results are summarized in table 12 for 0.5 in (1.27 cm) and 1.0 in (2.54 cm) retention storage capacities. This sample analysis



Source: Goforth et al., 1983.

Figure 37. Statistical method for determining long-term runoff capture efficiency.

Table 12. Summary of long-term runoff capture efficiencies for different retention storage capacities: highway pavement drainage area.

Rainfall Zone	Runoff Capture Efficiency (%)	
	0.5 in. Storage	1.0 in. Storage
1	76%	91%
2	67%	87%
3	56%	80%
4	51%	73%
5	59%	80%
6	85%	95%
7	52%	77%
8	92%	99%
9	82%	93%

NOTES: (1) Drainage area was assumed to be 100% impervious (C = 0.9).

(2) Minimum dewatering rate was assumed to be 0.3 in/hr.

presented in table 12 assumed that the retention facility drainage area consisted of highway pavement which was 100 percent impervious and exhibited a runoff coefficient(C) of 0.9. If pervious areas are to be considered in the actual designs, the runoff coefficients for the pervious and impervious areas should be area weighted to derive a composite runoff coefficient which is used to compute the mean runoff volume (V_{ro}). In order to be conservative, the dewatering rate used to calculate V_d assumed a minimum permissible value of 0.3 in/hr (0.76 cm/hr). V_d was calulated by multiplying 0.3 in/hr (0.76 cm/hr) by the recommended dewatering times in table 12. The minimum soil infiltration rate at the site should be used for actual designs.

As may be seen in table 12, a 0.5 in (1.27 cm) storage capacity would achieve efficiencies on the order of 50 to 60 percent for zones 3, 4, 5, and 7; on the order of 70 to 75 percent for zones 1 and 2; and on the order of 80 to 90 percent for zones 6, 8, and 9. Similar trends are evident for a 1.0 in storage capacity: about 70 to 80 percent efficiency for zones 3, 4, 5, and 7; about 90 percent for zones 1 and 2; and about 95 percent or greater for zones 6, 8, and 9. In general, the zones with the highest mean runoff volume (V_{ro}) exhibited the lowest efficiencies and vice versa. This suggests that higher storage requirements may be justified in the zones exhibiting the highest mean runoff volumes per storm. Likewise, a higher coefficient of variation for runoff volume resulted in a lower efficiency and vice versa.

The results in table 12 also highlight the diminishing benefits associated with significantly increasing storage capacity beyond 0.5 in (1.27 cm). As shown, a 100 percent increase in retention storage capacity (i.e., 1.0 in vs. 0.5 in) (2.54 vs. 1.27 cm) results in a much smaller increase in runoff capture efficiency.

As indicated above, the efficiencies reported in table 12 are typically lower than the efficiencies based upon the continuous simulation method, with the most significant underestimate for the lower efficiencies and vice versa. For example, in an application to a section of rainfall zone 3, underestimates were reported on the order of 15 to 20 percent for efficiencies in the 50 to 60 percent range, on the order of 10 percent for efficiencies in the 65 to 75 percent range, and about 5 percent for efficiencies on the order of 80 percent. (Goforth, et al. 1983) Thus, the efficiencies shown in table 12 are best viewed as conservative efficiency estimates suitable for general comparisons among rainfall zones.

The efficiencies summarized in table 12 only account for the capture of runoff volumes. If it is assumed that nonpoint pollution concentrations are relatively constant during the runoff event, then the runoff capture efficiencies represent an adequate approximation of long-term capture of pollutant mass. As indicated above, highway runoff typically exhibits first-flush conditions, meaning that the pollutant concentrations in the initial runoff captured by the retention facility will typically be higher than the concentrations of the later stages of runoff which are bypassed. Therefore, for retention measures designed to control highway runoff, long-term efficiencies for pollutant mass capture will typically be greater than the runoff capture efficiencies based on figure 37.

As an expansion of the statistical method applied herein, estimates were derived for the increased efficiency for pollutant mass capture under a range of first-flush conditions. (DiToro and Small, 1979) Their analysis assumes that the temporal concentration profile for highway runoff has an exponential shape. For a "moderate" first-flush effect defined by ratio of peak to final concentrations in the range 1.5 to 4.0, the increased efficiency due to first-flush conditions is about 10 percent for runoff capture efficiencies of 50 to 70 percent and about 5 to 7 percent for runoff capture efficiencies of 75 to 90 percent. For example, these first-flush adjustment factors suggest that the pollutant mass capture efficiency for highway runoff discharges into a 0.5 in (1.27 cm) capacity retention measure in rainfall zone 2 should be about 10 percent greater than the runoff capture efficiency, or 77 percent.

In summary, a statistical method has been used to analyze the expected long-term efficiencies of highway runoff capture and pollutant mass capture for typical retention storage requirements (0.5 to 1.0 in) (1.27 to 2.54 cm) in effect around the United States. The results suggest that a 0.5-in (1.27-cm) storage requirement can achieve a minimum runoff capture efficiency of about 60 to 70 percent for most rainfall zones and a minimum pollutant mass capture efficiency of about 70 to 80 percent after accounting for first-flush effects. Capture efficiencies for a 1.0-in (2.54-cm) storage requirement are about 10 to 20 percent higher than the values associated with a 0.5-in (1.27-cm) storage level. Given the estimated pollutant removal rates for the soil profile beneath retention facilities, these relatively high capture efficiencies indicate that retention measures for highway runoff control should be very competitive with other control measures in terms of expected pollutant removal rates.

3. DESIGN PROCEDURES

A. STORAGE REQUIREMENT

The previous section has demonstrated that retention storages of 0.5 to 1.0 in (1.27 to 2.54 cm) are desirable in order to produce capture efficient of 60 to 80 percent. Table 12, showing runoff capture efficiencies of 0.5- and 1.0-in (1.27- and 2.54-cm) storage within each rainfall zone, can be used as a guide to determine retention storage requirements. However, for a more detailed analysis at a given site, the storage requirements can be calculated with site-specific data. The design standard used in the procedures outlined below involves setting the retention storage capacity based upon the total runoff from the long-term mean rainfall volume.

The following steps and the design worksheet given in figure 38 can be used to determine the storage required for retention basins and retention trenches:

1. Identify the rainfall zone in which the basin will be located from figure 19.

2. Determine the long-term mean rainfall volume (R_v in inches) for the appropriate zone shown in table 7.

STORAGE REQUIREMENT FOR RETENTION MEASURES WORKSHEET

**********************************DESIGN DATA**********************************

Location ____
(City, State)

Rainfall Zone ______________________
(from figure 19)

Mean Rainfall Event Volume ______________________ (R_v) in
(from table 7)

Runoff Coefficient

Pervious Area Coefficient ______________________ (C_p)

Impervious Area Coefficient ______________________ (C_I)

Impervious Area/Total Area ______________________ (X)

Runoff Coefficient ______________________ (C)
($C = C_p + (C_I + C_p)X$)

Saturated Soil Infiltration Rate ______________________ (I_R) in/hr
(from table 10)

Dewatering time ______________________ (T_d) hrs
(from table 11)

*******************************DESIGN PROCEDURE*******************************

Runoff Volume from mean rainfall event volume
$V_{ro} = CR_v$ ______________________ (V_{ro}) in

Volume Released by Dewatering ______________________ (V_d) in
$V_d = I_R T_d$

Ratio of V_d/V_{ro} ______________________ V_d/V_{ro}

Fraction Captured
(from figure 37)

For Basin Volume to Runoff Volume Ratios (Trial Values)

V_b/V_{ro} = ________ E = ______ %

V_b/V_{ro} = ________ E = ______ %

V_b/V_{ro} = ________ E = ______ %

Selected Ratio ________

Design Storage ________ in
$(V_b/V_{ro})(V_{ro})$

Figure 38. Storage requirement for retention measures worksheet.

3. Establish the runoff coefficient

$$C = C_p + (C_I - C_p)X \tag{8}$$

where c = Runoff coefficient
C_p = Runoff coefficient for pervious areas
C_I = Runoff coefficient for impervious areas
X = Impervious area/total area.

4. Compute the long-term mean event runoff volume

$$V_{ro} = CR_V \tag{9}$$

where V_{ro} = Runoff volume, inches
C = Runoff coefficient
R_V = Rainfall depth, inches

5. Compute storage release by dewatering

$$V_d = T_R T_d \tag{10}$$

where V_d = Storage released by dewatering, inches
I_R = Saturated soil infiltration rate (table 10), inches/hour
T_d = Dewatering time (table 11), hours

6. Compute V_d/V_{ro} ratio

7. Select ratio of retention basin volume to runoff volume (V_b/V_{ro}) and choose coefficient of variation, CV, for rainfall volume, from table 7 for appropriate rainfall zone. Select percent fraction captured (E) between 60 and 80 percent using figure 37. Select design basin volume.

8. Continue retention basin and retention trench design as described in following sections.

B. RETENTION BASIN

Two different worksheets are presented to assist with retention basin design: a Part A worksheet for important design criteria (figure 39), and a Part B worksheet for performing iterative design calculations to size an adequate device (figure 40).

The Part A worksheet shown in figure 39 summarizes criteria such as required storage volume, design rainfall (deposited on the basin surface),

RETENTION BASIN DESIGN WORKSHEET: PART A

Site Name and Location ______________________________

Subarea Number _______

------------------------------Design Data------------------------------

1.	Storage required for nonpoint pollution management, Q_u	________	ft
2.	Design rainfall volume for nonpoint pollution management, P	________	ft
3.	Saturated soil infiltration rate, f	________	ft/hr
4.	Allowable dewatering time, T_d	________	hr
5.	Depth to bedrock or seasonally high water table, d_{min}	________	ft
6.	Required distance between facility bottom and bedrock or seasonally high water table, d_{req}	________	ft
7.	Maximum allowable depth based on depth to bedrock or seasonally high water table [5 - 6], d_{max}	________	ft
8.	Maximum allowable depth based on infiltration rate and dewatering time [3 x 4], d_{max}	________	ft
9.	Final maximum basin depth; i.e., lesser value of [7] and [8] ,d_b	________	ft

Figure 39. Retention basin design worksheet: Part A.

RETENTION BASIN DESIGN WORKSHEET: PART B

Site Name and Location ______________________________

Subarea Number ________

BASIN DESIGN

	BASIC PARAMETERS		SELECTED PARAMETERS		COMPUTATION	STORAGE COMPARISON			
Case	Area Controlled A_u (ft^2)	Depth d_b (ft)	Top Width W (ft)	Side Slopes Z (H:V)	Top Length L (ft) [eq. 11]	Storage Required (ft^3) [eq. 12]	Actual Storage (ft^3) [eq. 13]	Meets Min. Reqt.?	Total* Storage (ft^3) [eq. 14]

*Total storage includes the storage of upland runoff and of rainfall incident to the basin surface.

Figure 40. Retention basin design worksheet: Part B.

saturated soil infiltration rate, and the data required to calculate maximum allowable basin depth.

The Part B worksheet shown in figure 40 summarizes the key assumptions and results of iterative design calculations. The object is to size a basin to store the nonpoint pollution management storage, Q_u. The basin must also store the volume of rainfall (P) which occurs over its surface area, which can be assumed to equal Q_u (i.e., impervious area runoff for design condition). For this design procedure, it is conservatively assumed that the exfiltration volume through the bottom of the basin is relatively small in comparison with the runoff discharges into the basin during the period of time (e.g., less than 2 hrs) when the basin is being filled with highway runoff, and therefore that this portion of exfiltration may be ignored with little loss in accuracy. The key parameters in the detention basin design are illustrated in figure 41. In the basin design section under the heading "BASIC PARAMETERS," the area controlled is the area of the watershed excluding the infiltration basin surface area assumed for the first trial calculations. For example, if a 5-acre (2 ha) watershed is to be served by an infiltration basin and it is estimated that the basin surface area will be 9,000 ft^2 ($836 m^2$), the drainage area controlled, A_u, is:

$$(5 \text{ acres})(43{,}560 \text{ ft}^2/\text{acre}) - 9{,}000 \text{ ft}^2 = 208{,}800 \text{ ft}^2$$

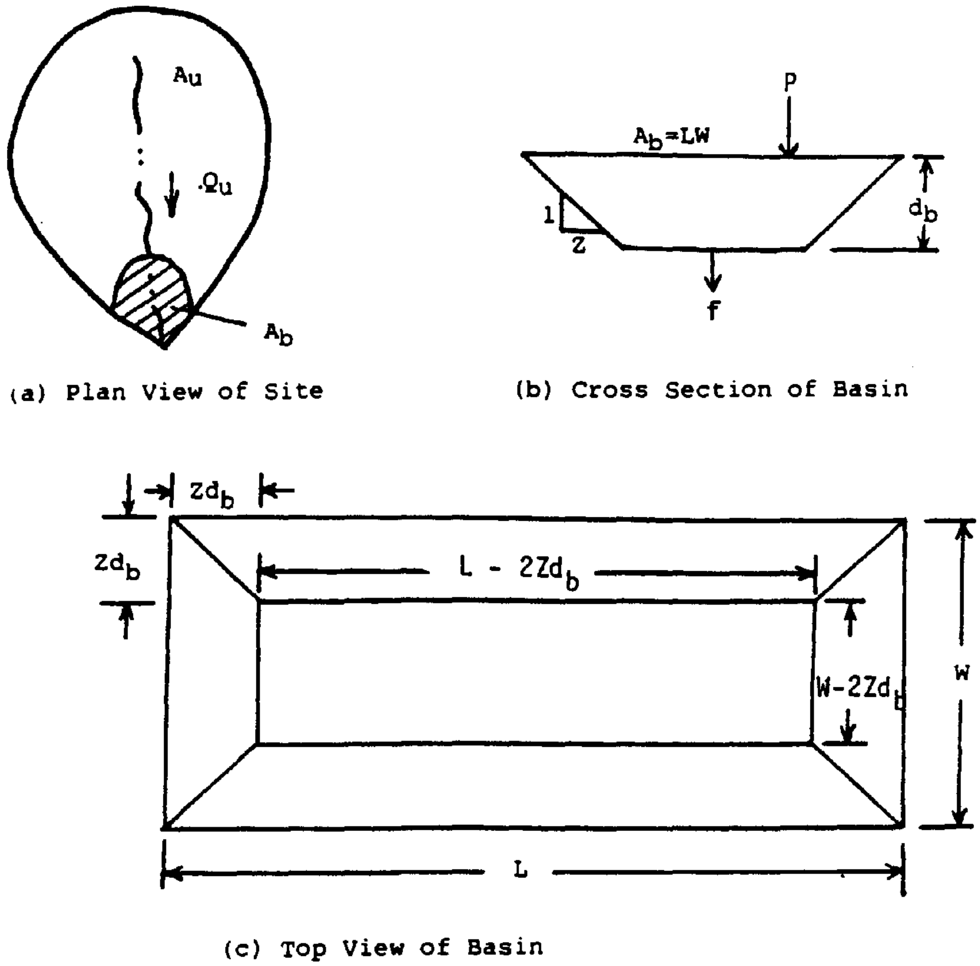
The area controlled should be recalculated each time the basin surface area changes.

The second "BASIC PARAMETER" is the basin depth, d_b, which is calculated in the worksheet shown in figure 39.

At this point, a basin width, W (ft), and side slopes, Z (H:V) are selected. The width selected must be greater than $2Zd_b$. The required basin length is then computed by:

$$L = \frac{(Q_u A_u) + (Zd_b^2)(W - 2Zd_b)}{W(d_b - P) - Zd_b^2} \tag{11}$$

where L is the required basin length in ft, Q_u is the required storage in ft (i.e., inches of runoff divided by 12), A_u is the upland drainage area (excluding the infiltration basin surface area) in ft^2, Z is the side slope ratio (horizontal to vertical), d_b is the basin depth in ft, W is the basin width in ft, and P is the rainfall volume in ft (i.e., assumed to equal Q_u). (Maryland WRA, 1984) The calculated L must be greater than $2Zd_b$. If the resultant surface area of the retention basin (LW) is substantially different from the one assumed in order to derive A_u, another iteration should be performed using the calculated surface area of the infiltration basin and recalculating A_u. When the basin is sized, the storage comparison section of the Part B worksheet is applied.



(a) Plan View of Site

(b) Cross Section of Basin

(c) Top View of Basin

<u>Notation</u>

A_u = upland contributing area (ft^2)
Q_u = peak flow control storage (ft)
A_b = top surface area of basin (ft^2)
L = top length of basin (ft)
W = top width of basin (ft)
d_b = basin depth (ft)
Z = basin side slope ratio (H:V)
P = rainfall volume (ft)
f = saturated soil infiltration rate (ft/hr)

Source: Adapted from MD WRA, 1984.

Figure 41. Schematic of retention basin.

The final storage requirement is given by:

$$\text{Required Storage} = Q_u A_u \tag{12}$$

where Q_u is the storage in ft required for nonpoint pollution management and A_u is the upland contributing area in ft^2.

The actual storage provided by the basin can be calculated by:

$$\text{Actual Storage} = \frac{LW + (L-2Zd_b)(W-2Zd_b)}{2}(d_b)-(2/3)Z^2d_b^3 - PLW \tag{13}$$

where L,W, and d_b are the length, width, and depth of the basin in ft, respectively, Z is the side slope ratio (H:V), and P is the design rainfall volume in ft.

The required and actual storages are then compared. If the design is complete, the total storage can then be computed by:

$$\text{Total Storage} = \frac{LW + (L-2Zd_b)(W-2Zd_b)}{2}(d_b)-(2/3)Z^2d_b^3 \tag{14}$$

where all parameters are the same as in equation 13.

C. RETENTION TRENCH

Figures 42 and 43 are worksheets for trench design similar to the basin worksheets shown above. A schematic illustrating the key parameters in the trench design is shown in figure 44.

The upland area, A_u, is the total area controlled by the trench less the trench surface area (A_t). For the first design iteration, the surface area of the trench must be estimated. If it is found that the calculated trench area differs significantly from the assumed area, another iteration should be performed using the calculated trench area.

Following the completion of the Part A worksheet, the actual design of the trench can be carried out. The trench should be designed to control the nonpoint pollution management storage, Q_u. In the "BASIC PARAMETERS" section of the Part B worksheet, the upland area A_u is determined as described above. The trench depth is determined in the worksheet.

For the nonpoint pollution management storage (Q_u) in rock-covered trenches, the required surface area of the trench can be derived directly by:

$$A_t = \frac{Q_u A_u}{(V_r d_t)} \tag{15}$$

where A_t is the required trench area in square feet, Q_u is the required runoff storage in feet (i.e., inches of runoff divided by 12), A_u is the total upland drainage area in square feet, V_r is the void ratio of the trench aggregate, and d_t is the trench depth in feet.

TRENCH AND SHALLOW DRY WELL DESIGN WORKSHEET: PART A

Site Name and Location ______________________________

Subarea Number _______

--------------------------------Design Data----------------------------------

1. Storage required for nonpoint pollution management, Q_u	________	ft
2. Saturated soil infiltration rate, f	________	ft/hr
3. Allowable dewatering time, T_d	________	hr
4. Void ratio of trench aggregate, V_r	________	
5. Depth to bedrock or seasonally high water table, d_{min}	________	ft
6. Required distance between facility bottom and bedrock or seasonally high water table, d_{req}	________	ft
7. Maximum allowable depth based on depth to bedrock or seasonally high water table [5 - 6], d_{max}	________	ft
8. Maximum allowable depth based on infiltration rate and dewatering time [2 x 3/4], d_{max}	________	ft
9. Final maximum trench depth; i.e., lesser value of [7] and [8], d_t	________	ft

Figure 42. Retention trench and shallow dry well design worksheet: Part A.

RETENTION TRENCH AND SHALLOW DRY WELL DESIGN WORKSHEET: PART B

Site Name and Location ____

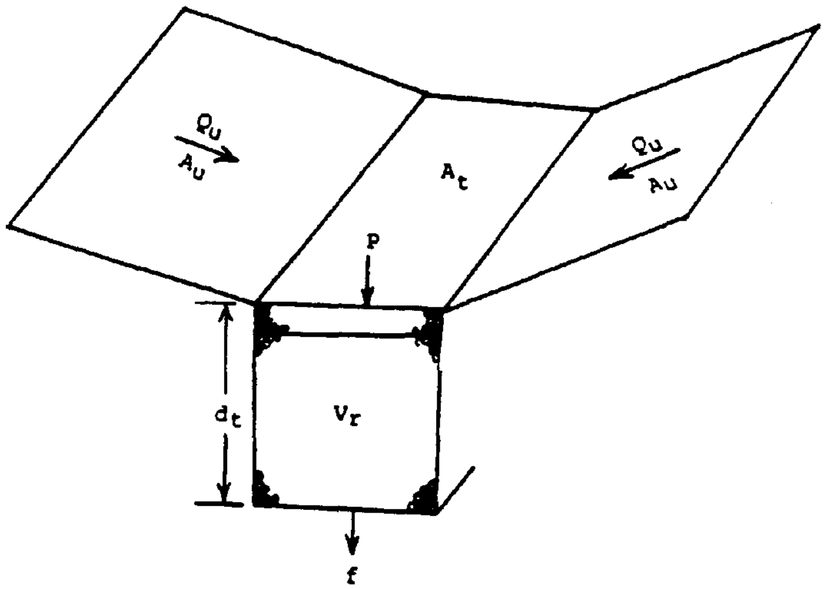
Subarea Number _________

TRENCH DESIGN

	BASIC PARAMETERS		SELECTED PARAMETERS		COMPUTATIONS		STORAGE COMPARISON		
Case	Area Controlled A_u (ft^2)	Depth d_t (ft)	Width W_t (ft)	Side Slopes Z (H:V)	Area A_t (ft^2) [eq. 15]	Length L_t (ft) [eq. 16 or 17]	Storage Required (ft^3) [eq. 18]	Actual Storage (ft^3) [eq. 19]	Meets Min. Reqt. ?

*Total storage includes the storage of upland runoff and of rainfall incident to the trench surface.

Figure 43. Retention trench and shallow dry well design worksheet: Part B.



Notation

P = rainfall volume (ft)

A_u = upland contributing area (ft^2)

Q_u = peak flow control storage (ft)

A_t = trench surface area (ft^2)

d_t = trench depth (ft)

f = saturated soil infiltration rate (ft/hr)

V_r = void ratio of trench medium

Figure 44. Schematic of retention trench.

For a trench with vertical sidewalls, the width can be set and the length derived by dividing the area by the width:

$$L_t = A_t/W_t \tag{16}$$

where the length of trench L_t and the trench width W_t are in feet and the trench area A_t is in square feet. If side slopes Z (H:V) are included as part of the design, a width can be selected ($W_t \geq 2Zd_t$) and the length computed by:

$$L_t = Zd_t + \frac{A_t}{(W_t - Zd_t)} \tag{17}$$

where L_t is the trench length in feet, Z is the side slope ratio (H:V), d_t is the trench depth in feet, A_t is the trench area in square feet computed by equation 6, and W_t is the trench width in feet.

After the trench has been sized, the storage comparison section of the Part B worksheet is applied. The required storage is calculated by:

$$\text{Required Storage} = Q_u A_u \tag{18}$$

where Q_u is the storage in ft required for peak flow control and A_u is the upland contributing drainage area in ft^2. The actual storage is calculated by:

$$\text{Actual Storage} = A_t d_t V_r \tag{19}$$

where A_t is the trench surface area in ft^2, d_t is the trench depth in ft, and V_r is the void ratio of the trench aggregate.

D. WELLS

The worksheets shown in figures 42 and 43 may also be used for shallow dry well designs. The major differences between the dry well and trench designs are that the surface area (A_t) for the former will generally be much smaller and circular in shape (in comparison with a rectangular shape for the trench).

Due to the limited storage volumes in individual wells, deep wells will typically be used in groups. The design procedure involves determining how many wells are required to control the specified highway runoff volume and evaluating the need for a detention basin upstream of the well system to serve a flow equalization function. The capacity of individual deep wells can be determined using infiltration equations for the case of a constant head suddenly applied over a semi-infinite porous medium. For deep wells packed in sand or gravel, calculations of storage capacity should account for storage in the void spaces of the packing area (e.g., void ratio of 0.25 or less for sand).

4. CONSTRUCTION CONSIDERATIONS

A. RETENTION BASIN

Protection of Device during Construction

In order to preserve natural infiltration rates at the retention basin site, particularly for basins with embankment dams, the site should be roped off to prevent the entry of heavy construction traffic which would cause excessive compaction of the soil.

Initial Excavation

Initial excavation should be carried out with light-weight equipment to minimize compaction of the soil profile. Where possible, excavation should take place from the sides of the device rather than from the device floor. Excavated materials should be placed at sufficient distance from the sides of the device to help prevent side failures and also to prevent migration of soil particles back into the device. Excavation in clay soils (clay content > 25 percent by weight) should only proceed when the soil is sufficiently dry to resist forming a "wire" when rolled between the palms of the hands. This will prevent excessive compaction and/or sealing of the soil surface.

During excavation, parts of the soil surface may become smeared (i.e., the pores may be sealed). Upon the completion of excavation, smeared surfaces should be scarified, protruding tree roots should be trimmed, and any other objectionable materials (e.g., large rocks) should be removed. Voids left in the sides or floor of a device by the removal of objectionable materials should be repacked with highly permeable soils. If the infiltration basin is to be used as a sedimentation basin during construction, initial excavation should be carried out to approximately 1 ft (0.31 m) above final grade.

Upon completion and inspection of the initial excavation, the side slopes of the basin as well as any embankments and the downstream outlet and/or emergency spillway, should be stabilized.

Embankment

When an embankment is used in conjunction with an infiltration basin, the fill should be sufficiently compacted to prevent seepage. When initial excavation and/or embankment construction are completed, the basin should be visually inspected.

Final Excavation

When all areas contributing runoff to the sediment basin have been stabilized, the excavation of the basin to finished grade should proceed following the removal of all accumulated sediments. Final excavation/grading should be performed with light-weight equipment in order to avoid excessive compaction of the basin floor.

Inlet/Outlet

The inlet to the infiltration basin should be designed to help prevent erosion in areas adjacent to the inlet. Energy dissipators such as riprap may be required to help control erosion near the inlet.

Outlets should be designed to protect against erosion and scour due to high velocities. Riprap should be provided as needed along the outflow channel.

Tilling

Tilling is recommended after basin construction has been completed to restore natural recharge (infiltration) rates to compacted soils. Tilling should be accomplished using light-weight equipment (e.g., small tractor) with rotary tillers or disc harrows. If heavy equipment has traversed the infiltration area, tilling should be preceded by deep plowing. A leveling drag should be towed behind the equipment on the last pass to ensure a level and smooth infiltration surface, which will facilitate future cleanout operations.

Lining

Infiltration basins may be lined with a 6- to 12-in (15.24- to 30.48-cm) layer of filter material such as coarse sand in order to prevent the buildup of impervious deposits on the natural soil surface. To increase the permeability of clayey soils, a 6-in (15.24-cm) layer of coarse organic material is sometimes specified for discing or spading into the soil.

B. RETENTION TRENCH

Protection of Device during Construction

It is important to prevent any stormwater from being discharged into retention trenches until all areas contributing stormwater runoff to a device have been stabilized. This will prevent premature clogging of upper layer void spaces due to the high sediment concentrations in construction site runoff.

Excavation

Trenches should be excavated using a backhoe or a wheel or ladder type trencher. Front-end loaders or bulldozers should not be used because the blades can smear the infiltration surface. In addition, these machines may cause undue compaction of the trench floor. Excavated materials should be placed a sufficient distance from the sides of the device to minimize the risk of sidewall cave-ins and also to prevent migration of soil particles back into the trench after the aggregate has been placed. Work should be scheduled so that the amount of trench excavated can be covered in one day in order to prevent windblown or waterborne sediment from entering the trench. Upon the completion of trench excavation, an inspection should be conducted. At this time, the inspector should also evaluate the quality and size of the trench aggregate, which should be clean and should conform to design specifications. Materials used in the trench (e.g., filter fabric) should be clean and free from defects.

Filter Fabric Laydown

When excavation and the subsequent inspection of the trench are complete, a filter fabric layer should be placed along the bottom and sides of the trench, with sufficient length left on top to overlap 6 in (15.24 cm) or more after the aggregate has been placed. The filter fabric should be free of large holes. Overlaps between rolls should be a mimimum of 2 ft (0.61 m) with the upstream roll atop the downstream roll.

Filter fabrics are very sensitive to long-term exposure to ultraviolet light. Therefore, they should not be left in the sun for any significant period of time. Some filter fabrics are affected by alkalies, acidic materials, asphalt components, and fuel oils. The selected filter fabric should also have a water permeability rate more rapid than that of the natural soil.

Aggregate Placement

Once the filter fabric lining has been placed along the bottom and sides of the trench, the aggregate should be laid in the trench to a depth of 1 ft (0.31 m) below the top by a backhoe or front-end loader rather than dumped in by a truck. This can be accomplished from the sides of the trench. It is recommended that the aggregate be placed in loose lifts 12 in (30.48 cm) thick (maximum) and compacted using plate compactors. When the depth of aggregate is within 1 ft (0.31 m) of the top of the trench, the filter fabric should be laid down and overlapped on the top of the trench. Following the final fabric laydown, the final layer of aggregate should be placed in the trench on top of the fabric until flush with finished grade.

C. WELLS

The construction practices for shallow dry wells are very similar to those cited above for the retention trench. Deep wells are typically constructed by boring with a large auger.

5. OPERATION AND MAINTENANCE

A. INTRODUCTION

For retention measures, an effective O&M program depends upon the use of proper design and construction practices. Pretreatment facilities such as grass channels and buffer strips are necessary to minimize the sediment and debris discharges into the retention facility. In the absence of effective pretreatment measures, the costs for frequent major cleanout operations to relieve clogging conditions may be prohibitive, particularly for trenches and wells. Likewise, the use of filter fabric lining is essential for trench and well systems. Finally, it is important that the retention facility not be activated until the entire drainage area contributing stormwater runoff has been stabilized.

Presented below are guidelines for facility inspections, routine maintenance activities which are performed on a regular basis and are preventive in nature, and nonroutine maintenance activities which are rehabilitative in nature.

B. FACILITY INSPECTIONS

Retention facilities should be inspected following at least one storm per year and at the time any maintenance activities are performed. For the inspection following a major storm, the inspector should visit the site at the end of the specified dewatering period (see table 11) to ensure that the facility is draining properly. At the time of all site visits, the inspector should check accumulations of debris, sediment, and oil and grease (aggregate filled measures only) within the retention facility, at inlets and outlets, and within major pretreatment areas.

For retention basins, the embankment (if applicable) and side slopes of the basin should be checked to ensure that they exhibit no visible signs of erosion, settlement, slope failure, wildlife damage, or vehicle damage.

For retention trenches and shallow dry wells, inspections to check for surface clogging should be made once or twice per year during nonfreezing conditions. Approximately every few years, a trench with an aggregate surface can be expected to exhibit clogging of the surface layers and the top roll of filter fabric. In the absence of periodic maintenance, the surface layers of the trench will eventually reach a fully clogged condition that approximates an impervious surface.

In addition to visual inspection, the existence of surface clogging at a trench or shallow dry well should be checked by pouring about one gallon of water onto a 1-ft by 1-ft (2.54-cm by 2.54-cm) section (i.e., 1.0 ft^2 (.09 m^2)). Assuming that the water is applied fairly evenly to the 1.0 ft^2 (.09 m^2) section over about a 15-second period, the water should percolate into the lower layers fairly rapidly so that there is no significant ponding and/or runoff. Several sections should be checked in this manner to ascertain if the clogging problem is widespread or localized. The top aggregate layer (approximately 1 ft (.31 m) deep) should be removed in a small area (by hand or with the aid of a trowel) and the condition of the filter fabric should be checked to confirm the existence of clogging conditions.

C. ROUTINE MAINTENANCE

Grass can be mowed occasionally if desired. Grasses of the fescue family can be mowed twice per year, in June and September. In addition to grass maintenance, any other vegetation in the retention basin area or access area which has reached nuisance levels (e.g., bushes and weeds) should be trimmed or removed. Fertilization activities may not be necessary due to the nutrient concentrations in highway runoff.

For the retention basin, if the inspector determines that the dewatering rate is too slow, the basin should be tilled. It is anticipated that tilling operations will be required about once a year. Before the basin can be tilled, however, all accumulated sediment must be removed. Sediment should be removed using light equipment only after the layer has dried, cracked, and separated from the natural floor of the basin. After the sediment accumulations have been carefully removed, tilling should be performed using the methods outlined above for construction practices.

For trenches and well systems, the elimination of clogging problems falls under the category of nonroutine maintenance activities.

Debris should be removed from the surface of the retention facility, the inlet/outlet, and major pretreatment areas whenever the site is inspected, if feasible. Most debris can be removed by hand or with hand tools (e.g., shovel). Some larger objects such as fallen tree limbs may have to be cut up before removal by hand is possible.

D. NONROUTINE MAINTENANCE

Eroded areas should be filled and compacted, if necessary, and reseeded as soon as possible. Eroded areas near the inlet or outlet should be revegetated and, if necessary, be filled, compacted and reseeded or lined with riprap. Damaged side slopes in retention basins should be repaired using fill dirt of adequate permeability. Major damage to inlet/outlet structures and the embankment (retention basin) should be repaired as soon as possible.

For retention basins, significant sediment accumulations in the basin are likely to require removal (followed by tilling) at a frequency of about once every 5 years or less.

In order to eliminate clogging problems in a retention trench or a dry well backfilled with aggregate, the surface layer of aggregate and the filter fabric covering the top of the trench should be replaced. First, the old aggregate should be carefully removed. Then, the filter cloth overlaying the top of the trench or well should be cut on either side of the trench and replaced with a new strip, with a minimum overlap between old and new cloth of 1.0 ft (0.31 m). Clean aggregate should then be laid on top of the new filter fabric layer until flush with the finished grade. Based upon typical sediment discharge rates, it is estimated that surface cleanout operations and replacement of the filter fabric cover could be required on the order of once every few years. The frequency of cleanout operations will depend, to a large extent, upon whether satisfactory pretreatment areas are included in the retention system design.

When the inspector determines that the trench or dry well is completely clogged, the entire trench should be rehabilitated, starting with excavation of all aggregate, removal of all filter cloth, and rescarification of the bottom and sides of the trench. New filter fabric and clean aggregate should be laid in the trench. It is estimated that these major rehabilitation projects could be required on the order of once every 10 to 15 years for trenches and dry wells backfilled with aggregate.

6. HIGHWAY APPLICATIONS

Retention systems can be readily adapted to fit the requirements of highway systems, assuming acceptable topographic, soils, and water table conditions.

Retention basins can be shaped into deep-sided linear basins constructed in borrow pits, or linked in a series of small basins which can be located within median strips or other open areas within the right-of-way. Small basins can be used in cloverleaf interchanges and other areas where surface area and drainage area (i.e., runoff volume) are both limited.

Retention trenches are probably best suited for linear drainage systems such as median strips or right-of-way areas adjoining the highway shoulder, with inflow via overland flow paths. Trenches can also be located in the bottom of grass channels, preferably upstream of small check dams (6 to 18 in (15.24 to 45.72 cm) high) which will retard and pond runoff sufficiently to permit infiltration through the trench surface. Trenches can also be located in interchange loops.

Wells are suitable for linear drainage systems and in interchange loops, and may be a particularly appropriate control measure for depressed highways where the lack of pervious open areas prevents the use of grass channels, overland flow systems, and detention basins.

Because retention measures can achieve peak runoff reductions, groundwater recharge, and thereby augment dry period low flows, they may be particularly appropriate for locations where highway runoff impacts on stream channel habitats are a significant environmental concern.

Retention systems are least suitable for major aquifer recharge areas, particularly in northern states where roadway deicing chemicals are required. Likewise, relatively deep retention systems, such as wells and perhaps trenches, may be unsuitable for most areas where the potential for groundwater contamination is a significant concern.

7. DESIGN EXAMPLES

A. RETENTION BASINS

Figure 45 presents an example of an excavated retention basin to be designed for nonpoint pollution management in Knoxville, Tennessee. The basin is to serve a highway drainage area of 15 acres (6 ha) (of which 75 percent is roadway pavement). The saturated soil infiltration rate is 1.0 in/hr (2.54 cm/hr). The minimum depth to bedrock or the seasonally high water table is 30 ft (9.15 m). Find the dimensions of the retention basin that will hold the required nonpoint pollution management storage.

The following steps are followed to complete the retention basin design worksheet for the STORAGE REQUIREMENT shown in figure 45.

Step 1. From figure 19, Knoxville, Tennessee is in rainfall zone 2.

Step 2. From table 7, the mean annual rainfall volume is 0.36 in (0.91 cm).

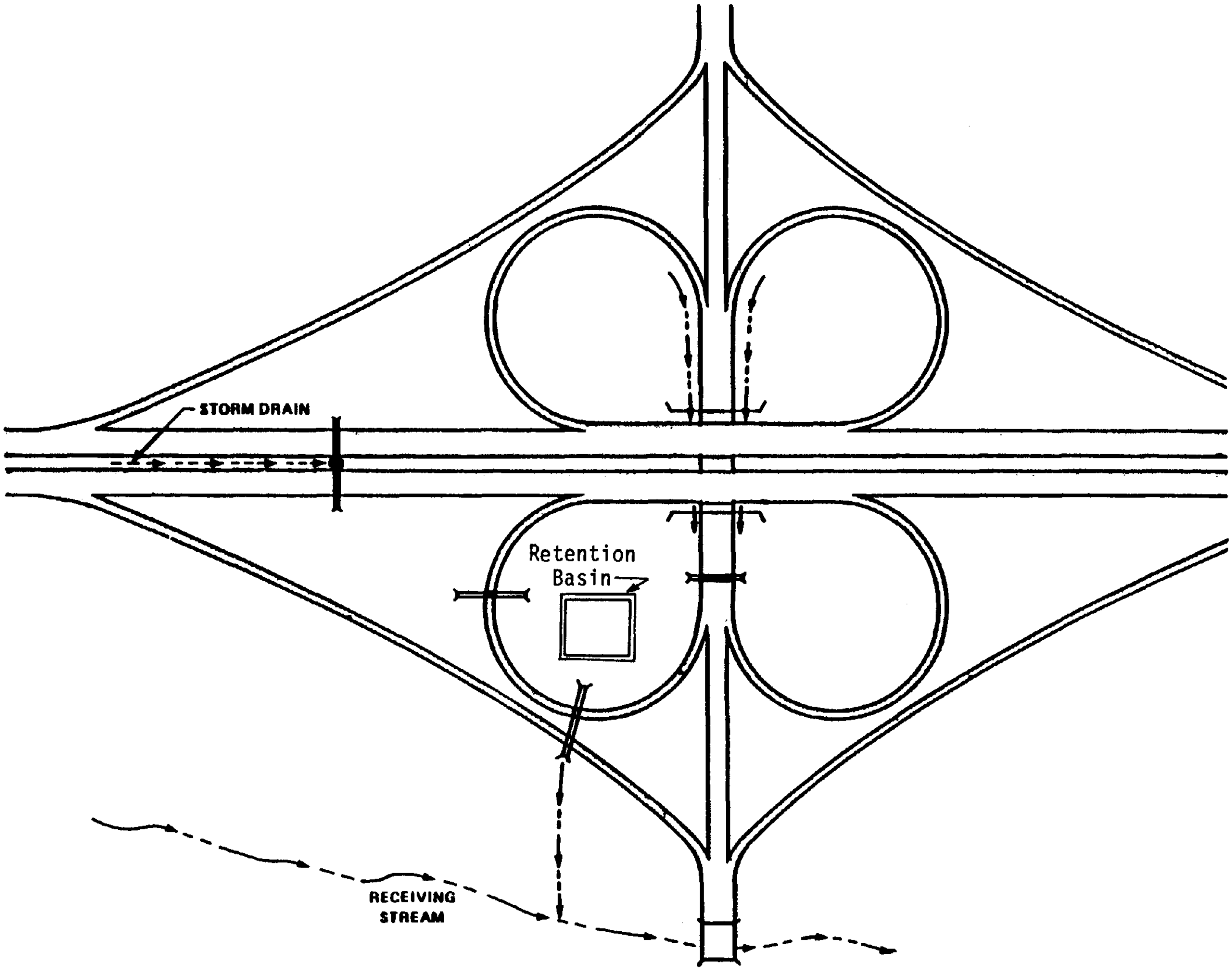


Figure 45. Example - retention basin in cloverleaf interchange.

STORAGE REQUIREMENT FOR RETENTION MEASURES WORKSHEET

**********************************DESIGN DATA**********************************

Location	Knoxville, Tennessee (City, State)	
Rainfall Zone (from figure 19)	2	
Mean Rainfall Event Volume (from table 7)	0.36	(R_V) in
Runoff Coefficient		
Pervious Area Coefficient	0.4	(C_p)
Impervious Area Coefficient	0.8	(C_I)
Impervious Area/Total Area	0.75	(X)
Runoff Coefficient ($C = C_p + (C_I + C_p)X$)	0.7	(C)
Saturated Soil Infiltration Rate (from table 10)	1.0	(I_R) in/hr
Dewatering time (from table 11)	72	(T_d) hrs

*******************************DESIGN PROCEDURE*******************************

Runoff Volume from mean rainfall event volume $V_{ro} = CR_V$	0.22	(V_{ro}) in
Volume Released by Dewatering $V_d = I_R T_d$	72	(V_d) in
Ratio of V_d/V_{ro}	∞	V_d/V_{ro}

Fraction captured (from figure 37)

For Basin Volume to Runoff Volume Ratios (Trial Values)

V_b/V_{ro} = 1.0	E = 53 %
V_b/V_{ro} = 1.5	E = 70 %
V_b/V_{ro} = 2.0	E = 79 %

Selected Ratio 2.0

Design Storage (V_b/V_{ro})(V_{ro}) 0.5 in

Figure 45. Example - retention basin in cloverleaf interchange: storage requirement (continued).

RETENTION BASIN DESIGN WORKSHEET: PART A

Site Name and Location Cloverleaf Interchange Example Problem

Subarea Number 1

------------------------------Design Data------------------------------

1.	Storage required for nonpoint pollution management, Q_u	0.042	ft
2.	Design rainfall volume for nonpoint pollution management, P	0.042	ft
3.	Saturated soil infiltration rate, f	0.083	ft/hr
4.	Allowable dewatering time, T_d	72	hr
5.	Depth to bedrock or seasonally high water table, d_{min}	30	ft
6.	Required distance between facility bottom and bedrock or seasonally high water table, d_{req}	10	ft
7.	Maximum allowable depth based on depth to bedrock or seasonally high water table [5 - 6], d_{max}	20	ft
8.	Maximum allowable depth based on infiltration rate and ponding time [3 x 4], d_{max}	6	ft
9.	Final maximum basin depth; i.e., lesser value of [7] and [8] ,d_b	6	ft

Figure 45. Example - Retention basin in a cloverleaf interchange: Part A (continued).

RETENTION BASIN DESIGN WORKSHEET: PART B

Site Name and Location Cloverleaf Interchange Example Problem

Subarea Number 1

BASIN DESIGN

	BASIC PARAMETERS		SELECTED PARAMETERS		COMPUTATION	STORAGE COMPARISON			
Case	Area Controlled A_u (ft^2)	Depth d_b (ft)	Top Width W (ft)	Side Slopes Z (H:V)	Top Length L (ft) [eq. 11]	Storage Required (ft^3) [eq. 12]	Actual Storage (ft^3) [eq. 13]	Meets Min. Reqt.?	Total* Storage (ft^3) [eq. 14]
1	647,000	6	80	3:1	90	27,174	27,130	No	—
2	646,040	6	80	3:1	92	27,134	27,867	Yes	28,176

*Total storage includes the storage of upland runoff and of rainfall incident to the basin surface.

Figure 45. Example - Retention basin in a cloverleaf interchange: Part B (continued).

Step 3. Compute the runoff coefficient

$C = C_p + (C_I - C_p)X$

Assume $C_p = 0.4$
$C_I = 0.8$

$C = 0.4 + (0.8 - 0.4)0.75$

$= 0.7$

Step 4. Compute runoff volume from mean event rainfall

$V_{ro} = CR_v$

$= (0.7)(0.36)$

$= 0.25$ in

Step 5. Compute storage released by dewatering

$V_d = I_R T_d$

$= (1.0)(72)$

$= 72$ in

Step 6. Compute V_d/V_{ro} ratio

$72/0.22 = 327$

Therefore, for figure 37 use infinity curve (∞).

Step 7. Select ratio of retention basin volume to runoff volume and use figure 37 to determine fraction captured. Use CV = 1.45 which is coefficient of variation for annual rainfall volume in zone 2 given in table 7. The coefficient of variation for volume in table 7 is variable "νv".

For V_b/V_{ro} of 1.0, fraction captured = 53 percent

V_b/V_{ro} of 1.5, fraction captured = 70 percent

V_b/V_{ro} of 2.0, fraction captured = 79 percent

Select a ratio of 2.0 for a high level of efficiency (79 percent capture). This gives a design storage of 0.5 in (1.27 cm) (i.e., 2 times 0.25 in).

The following steps are used to complete PART A and PART B worksheets (figure 45) for this example problem:

Step 1. The storage requirement is given in inches. Convert to units of feet.

Q_u = 0.5 in x 1 ft/12 in = 0.042 ft

Step 2. The design rainfall volume is assumed to be equal to Q_u.

P = Q_u = 0.042 ft

Step 3. The saturated soil infiltration rate is given.

f = 1.0 in/hr x 1 ft/12 in = 0.083 ft/hr

Step 4. Based upon table 11, an allowable dewatering time of 72 hr will be assumed for rainfall zone 2.

Step 5. The depth to bedrock or seasonally high water table is given.

d_{min} = 30 ft

Step 6. The required depth betwen the practice bottom and bedrock will be set at:

d_{req} = 10 ft

Step 7. The maximum basin depth based on line 7 of the Part A worksheet is:

d_{max} = 30 ft - 10 ft = 20 ft

Step 8. The maximum basin depth based on line 8 of the Part A worksheet is:

d_{max} = (0.083 ft/hr)(72 hr) = 6.0 ft

Step 9. Use lower of d_{max} from Steps 7 and 8 for basin depth.

d_b = 6 ft

Step 10. The design data worksheet (Part A) has been completed. Now enter the Part B worksheet.

Step 11. Assume the basin dimensions are 80 ft by 80 ft. The resultant upland area is:

A_u = 15 ac (43,560 ft^2/ac) - (80 ft)(80 ft) = 647,000 ft^2

Step 12. The basin depth is determined in the design data section.

d_b = 6 ft

Step 13. The selected parameters must now be chosen.

Select a top width W = 80 ft

W must be $\geq 2Zd_b$

Select side slopes Z = 3:1 (H:V), so $2Zd_b$ = 36 ft (OK)

Step 14. Compute the required length by equation 11:

$$L = \frac{(0.042 \text{ ft})(647{,}000 \text{ ft}^2) + 3(36 \text{ ft}^2)[80 \text{ ft}-(2)(3)(6 \text{ ft})]}{(80 \text{ ft } (6 \text{ ft} - 0.042 \text{ ft})) - 3(36 \text{ ft}^2)}$$

L = 86.6 ft; use L = 90 ft

Step 15. The required storage is calculated by equation 12:

Required Storage = $Q_u A_u$ = (0.042)(647,000) = 27,174 ft^3

Step 16. The actual storage provided is calculated by equation 13:

Actual Storage Provided =

$$\frac{(90)(80)+(90-2(3)(6))(80-2(3)(6))(6)}{2} -(2/3)(3)^2(6)^3-(0.042)(90)(80)$$

Actual Storage Provided = 27,130 ft^3

Step 17. Since the actual storage provided is less than the required storage, the size of the basin must be increased slightly. Therefore, increase the length to 92 ft.

Step 18. The new basin area is:

(92 ft)(80 ft) = 7,360 ft^2

The associated upland area A_u is now:

A_u = (15 ac)(43,560 ft^2/ac) - 7,360 ft^2 = 646,040 ft^2

This value is entered in the design table.

Step 19. The required storage is now (equation 12):

Required Storage = (0.042)(646,040) = 27,134 ft^3

Step 20. The actual storage provided is now (equation 13):

Actual Storage Provided =

$$\frac{((92)(80)+(92-2(3)(6))(80-2(3)(6)))(6)}{2} -(2/3)(3)^2(6)^3-(0.042)(92)(80)$$

Actual Storage Provided = 27,867 ft^3

Step 21. Since the actual storage is greater than the required storage, the design satisfies the minimum requirement. The total storage can be calculated by equation 14, if desired:

Total Storage = (Actual Storage) + (PLW) = 28,176 ft^3

Step 22. The final dimensions of the retention basin are as follows:

Length = 92 ft
Width = 80 ft
Depth = 6 ft
Side Slopes = 3:1 (H:V)

B. RETENTION TRENCHES

This example (figure 46) covers a system of retention trenches, to be located in each quadrant of a diamond interchange in rainfall zone 2, with each trench to serve an area of 3 acres (1.2 ha). The required nonpoint pollution management storage for each trench is 0.5 in (1.27 cm) runoff. The saturated soil infiltration rate is 0.5 in/hr (1.27 cm/hr). The minimum depth to bedrock or the seasonally high water table is 15 ft (4.58 m). Find the dimensions of the retention trench that will hold the required nonpoint pollution management storage.

Step 1. The nonpoint pollution storage requirement, Q_u, must be converted to units of feet:

Q_u = 0.5 in x 1 ft/12 in = 0.042 ft

Step 2. The infiltration rate is given in inches/hour. Convert to units of feet/hour.

f = 0.5 in/hr x 1 ft/12 in = 0.042 ft/hr

Step 3. Based upon table 11, an allowable dewatering time of 72 hrs will be assumed for rainfall zone 2:

T_d = 72 hr

Step 4. The assumed void ratio is:

V_r = 0.4

Step 5. The depth to bedrock or high groundwater is given.

d_{min} = 15 ft

Step 6. The required depth between the practice bottom and bedrock or high groundwater should ideally be 3 to 10 ft for water quality management purposes. Assume:

d_{req} = 5 ft

RETENTION TRENCH AND SHALLOW DRY WELL DESIGN WORKSHEET: PART A

Site Name and Location <u>Diamond Interchange Example Problem</u>

Subarea Number <u>1, 2, 3, 4</u>

---Design Data---

1.	Storage required for nonpoint pollution management, Q_u	0.042	ft
2.	Saturated soil infiltration rate, f	0.042	ft/hr
3.	Allowable dewatering time, T_d	72	hr
4.	Void ratio of trench aggregate, V_r	0.4	
5.	Depth to bedrock or seasonally high water table, d_{min}	15	ft
6.	Required distance between facility bottom and bedrock or seasonally high water table, d_{req}	5	ft
7.	Maximum allowable depth based on depth to bedrock or seasonally high water table [5 - 6], d_{max}	10	ft
8.	Maximum allowable depth based on infiltration rate and dewatering time [2 x 3/4], d_{max}	7	ft
9.	Final maximum trench depth; i.e., lesser value of [7] and [8], d_t	7	ft

Figure 46. Example - design of retention trench: Part A.

RETENTION TRENCH AND SHALLOW DRY WELL DESIGN WORKSHEET: PART B

Site Name and Location Diamond Interchange Example Problem

Subarea Number 1,2,3,4

TRENCH DESIGN

	BASIC PARAMETERS		SELECTED PARAMETERS		COMPUTATIONS		STORAGE COMPARISON		
Case	Area Controlled A_u (ft^2)	Depth d_t (ft)	Width W_t (ft)	Side Slopes Z (H:V)	Area A_t (ft^2) [eq. 15]	Length L_t (ft) [eq. 16 or 17]	Storage Required (ft^3) [eq. 18]	Actual Storage (ft^3) [eq. 19]	Meets Min. Reqt. ?
1	130,680	7	25	—	1,960	79	5,489	5,530	Yes

*Total storage includes the storage of upland runoff and of rainfall incident to the trench surface.

Figure 46. Example - Design of retention trench: Part B (continued).

Step 7. The maximum depth based on line 7 of the Part A worksheet is:

$$d_{max} = 15 \text{ ft} - 5 \text{ ft} = 10 \text{ ft}$$

Step 8. The maximum depth based on line 8 of the Part A worksheet is:

$$d_{max} = (0.042 \text{ ft/hr})(72 \text{ hr})/0.4 = 7.6 \text{ ft; say } 7 \text{ ft}$$

Step 9. Use the lower value resulting from Steps 7 and 8 for trench depth:

$$d_t = 7 \text{ ft}$$

The design data worksheet (Part A) has been completed. Now enter the Part B worksheet.

Step 10. The required upland drainage area is:

$$A_u = (3 \text{ ac})(43,560 \text{ ft}^2/\text{ac}) = 130,680 \text{ ft}^2$$

Step 11. The required surface area for each trench can be calculated by equation 15:

$$A_t = \frac{Q_u\ A_u}{(V_r\ d_t)} = \frac{(0.042 \text{ ft})(130,680 \text{ ft}^2)}{(0.4)\ (7 \text{ ft})}$$

$$A_t = 1,960 \text{ ft}^2$$

Step 12. To determine exact trench dimensions, a width W_t must be selected for each trench:

$$W_t = 25 \text{ ft}$$

Step 13. The required length of each trench can be computed by equation 16, assuming vertical sidewalls:

$$L_t = 1,960 \text{ ft}^2/25 \text{ ft} = 78.4 \text{ ft; use } L_t = 79 \text{ ft}$$

Step 14. In the storage comparison section, the required storage for each trench is calculated by equation 18:

$$\text{Storage Required} = (0.042 \text{ ft})(130,680 \text{ ft}^2) = 5,489 \text{ ft}^3$$

Step 15. The actual storage provided by each trench is calculated by equation 19:

$$\text{Actual Storage} = (79 \text{ ft})(25 \text{ ft})(8 \text{ ft})(0.4) = 5,530 \text{ ft}^3$$

Step 16. Since the actual storage of 5,530 ft^3 is greater than the required nonpoint pollution management storage of 5,489 ft^3, each trench is adequately sized. Therefore, provide one trench in each of the four quadrants of the interchange with the following dimensions for each:

Length = 79 ft
Width = 25 ft
Depth = 7 ft (vertical sidewalls)

The total storage provided in the four trenches is 22,120 ft^3.

6. WETLAND SYSTEMS FOR RUNOFF CONTROL

Wetland systems can potentially provide significant water quality treatment to highway runoff. Most of the available research and literature on wetlands pertains to the use of these systems to provide final treatment of municipal wastewaters. There are, however, a growing number of field studies and applications that focus on the use of wetlands for the treatment of urban stormwater runoff. Florida has recently adopted new regulations which promote the use of some wetlands for stormwater runoff treatment. Maryland recently developed guidelines for the construction of shallow wetlands in stormwater basins to reduce runoff pollution loadings. This chapter presents a summary of wetland planning and design experience to date which may be applied as guidelines for the use of wetland areas as mitigation measures for highway runoff pollution.

1. DESIGN CONSIDERATIONS

A. POLLUTANT REMOVAL MECHANISMS

Wetlands provide hydraulic resistance to surface runoff resulting in decreased velocities and increased deposition of suspended sediments. Toxicants (e.g., heavy metals) sorbed to suspended sediments can be deposited and retained within the wetland. The large surface area provided by surface soils and vegetation contributes to higher levels of adsorption, absorption, filtration, microbial transformation, and biological utilization than might normally occur in more channelized water courses.

Pollutants may be removed from the water column by physical, chemical, and biological means. Physical processes include sedimentation, emulsification, adsorption, and filtration. Chemical processes include chelation, precipitation, decomposition, and chemical adsorption. Biological processes are primarily vegetative uptake and removal, with some biological transformation and degradation occurring. Many of the processes are interrelated, and are variable for different pollutants.

The effectiveness of wetlands for pollutant removal varies with wetland type and a number of site specific parameters. The identification and quantification of the roles of individual mechanisms is difficult to assess. Kutash found that field studies of wetland treatment of stormwater generally produce the following conclusions: (Kutash, 1985)

- A wide disparity in the nonpoint pollution removal capabilities of wetlands, particularly with regard to nutrients.

- The greatest consistency in pollutant reduction appears to be for BOD, suspended solids, and heavy metals.

- The nature of flow and seasonal factors are major influences on pollutant removal capabilities in certain wetlands.

In general, hydrology tends to be the primary determinant of pollutant removal in wetlands as a result of its influence on processes of sedimentation, aeration, biological transformation, and adsorption onto bottom sediments. Wetlands with gradual gradients and low flow velocities that allow sedimentation of sediment-adsorbed pollutants will generally be more effective for treatment of stormwater runoff.

B. WETLANDS EVALUATIONS

A wetland is defined as an area that is inundated or saturated by surface water or groundwater at a frequency and duration sufficient to support, and that under normal circumstances do support a prevalence of vegetation typically adapted for life in saturated soils conditions; generally includes swamps, marshes, bogs, and similar areas (33 CFR 323.2C, 1986). Wetlands are characterized as having one or more of the following attributes: (Cowardin et al., 1979)

- The substrate is predominantly undrained hydric soil.
- The substrate is nonsoil and is saturated with water or covered by shallow water at some time during the growing season of each year.
- The land supports primarily hydrophytes (plants adapted to aquatic and semiaquatic environments).

Wetlands Evaluation Technique

A wetlands evaluation technique (WET) has been developed to provide a rapid initial assessment of wetland functions and values. (Adamus et al., 1987) The wetland functions pertaining to control of highway stormwater runoff that may be evaluated by WET include: sediment stabilization; sediment/toxicant retention; and nutrient removal and transformation. WET uses predictors based upon physical, chemical, and biological data that may be fairly easily collected to evaluate the capability of a wetland to perform a certain function. A qualitative probability rating of HIGH, MEDIUM, or LOW is assigned to each function.

Potential applications of WET to screen wetlands for use as highway runoff controls include:

- Comparison of different wetlands for pollutant removal effectiveness.
- Selection of priorities for wetland acquisition.
- Identification of priority wetlands unsuitable as highway runoff controls.
- Identification of options for permitting requirements.
- Comparison of created or restored wetlands with pre-impact wetlands for preliminary assessment of mitigation needs.

Wetlands Classifications

Classifying wetlands has been the subject of extensive study. Cowardin devised a widely used hierarchical classification system that is based on five distinct wetland systems: marine, estuarine, riverine, lacustrine, and palustrine. (Cowardin et al., 1979) These systems are further classified by subsystems, classes, subclasses, and dominant types. Classes, subclasses, and dominant types are typified by water regime, water chemistry, and soils. The major determinants of most classification systems are vegetation type, soils, and hydrology. A national list of plant species that occur in wetlands has been prepared by the U.S. Fish and Wildlife Service. (Reed, 1988) Guidance for wetland delineation can be found in a new Federal Manual for Identifying and Delineating Jurisdictional Wetlands. (Federal Interagency Committee for Wetland Delineation, 1989)

Both natural and constructed wetlands may be used to treat highway runoff pollution. Although the natural wetland may be the more desirable of the two from the standpoint of cost, there may be many practical and legal constraints that greatly limit the circumstances under which a natural wetland may be used. While constructed wetlands may be more expensive to construct and to operate, the design and operational constraints associated with the constructed wetland are fewer than for the natural wetland.

Natural Wetlands. There are several important constraints that must be addressed when screening a natural wetland for use as a highway runoff control. The first constraint is imposed whenever State or local laws, or Section 404 of the Clean Water Act, place some protection on the wetland of interest. Thus, it is important to determine early on: whether a candidate wetland is part of a park, or a wildlife refuge; whether it provides habitat for endangered species; or whether it connects with a sole source drinking water supply aquifer. Any of these factors may preclude water quality treatment applications.

Section 404 of the Clean Water Act is designed to regulate the discharge of dredge and fill materials to the waters of the United States, including wetlands. Regulatory aspects are the responsibility of the U.S. EPA, while operation of the permit program is the responsibility of the U.S. Army Corps of Engineers. Projects in which only a small wetland area will be affected may be treated generically by the permitting authority, where the application and approval process is straightforward. Where larger wetland areas are to be affected, greater scrutiny will generally be exercised by the permitting authority before issuance of a permit.

A second constraint is the proximity of the wetland to the highway site. If the wetland is not within the highway right-of-way, steps will be required either to acquire the wetland, or to obtain permission to incorporate the wetland into a treatment system. In addition, it may be necessary to obtain easements to assure access to the wetland area. The distance of the wetland from the source of runoff is also of importance, and will help determine whether an off-site wetland is a practical nonpoint pollution management alternative.

A third important consideration is whether the candidate wetland is large enough to handle the projected hydraulic load from the highway. If the hydraulic load is too large, damage to the resident vegetation could render the wetland useless from the standpoint of runoff pollution control. In weighing the effects that a change in hydraulic regime may cause, one must also consider the types of vegetation that prevail and the seasonal plant growth patterns.

If the wetland is not large enough, it will be necessary to determine whether the existing wetland may be altered or enlarged to increase its ability to assimilate pollutant loads in highway runoff. Alterations might include changes to natural channels to increase residence time, or the introduction of new plant species for improved treatment efficiency. Enlargements might be accomplished by forming connections between adjacent wetlands, or by the development of constructed wetlands.

Constructed Wetlands. Constructed wetlands offer many more options for the management of highway runoff than do natural wetlands. Apart from the obvious constraints discussed below that are imposed by terrain, native soils, local hydrology and climatology, and availability of areas in which to construct a wetland, considerable latitude is available to the designer of the constructed wetland. In particular, the constructed wetland can be: sized to accommodate a projected hydraulic load and to provide a specified residence time; constructed within the highway right-of-way, in median strips, in cloverleafs, or alongside the highway, and designed to facilitate operations and maintenance. The shape and depth of the constructed wetland can be designed to promote the growth of vegetation and to facilitate future maintenance.

The principal criteria that must be applied in the selection of the area to be converted to wetlands are: (Garbish, 1986)

- The land should have low fish and wildlife resource value in its present state.

- An adequate water supply should be available for connection to ensure successful wetlands development.

Constructed wetlands should be located in areas where sediment accretion occurs rather than where erosion and scouring are evident. Potential constructed wetland locations might include dredging or borrow areas, unvegetated or disturbed shorelines, or dredged material disposal areas.

C. WETLAND COMPONENTS

Soils

A natural wetland is characterized by its soil, its plant community, and its hydrologic regime. The hydric soils of the wetland are high in organic content and if not fully submerged, are frequently inundated. The soils are poorly aerated and are somewhat acid. The high organic content improves the ability of these soils to remove toxicants such as heavy metals by adsorption, so that the quality of runoff waters passing through wetland soils may be greatly improved.

Studies of metals removal rates in urban runoff detention basins show that the downward movement of trace metals through the soil profile is minimal. (Wigington et al., 1983) In comparisons of background concentrations of cadmium, zinc, copper, and lead at control sites with metal concentrations in a detention basin, significant differences between the concentrations in the control and basin soils were not detected below the surface soil layer (i.e., upper 1.97 in (5 cm)). The conditions that seemed to be most conducive to the downward migration of trace metals were seen at one site where large concentrations of cadmium, copper, lead, and especially zinc were present in the surface soil and a large part of the basin was submerged for long periods of time, a situation conducive to the production of organic compounds that can form soluble complexes with trace metals. Organic pollutants, in the form of petroleum products were also present and could have potentially increased metal solubility. Further, the soil profile was very sandy 9.84 in (25 cm) below the soil surface or deeper. Yet, there was no statistical evidence of the downward movement of cadmium. Copper and lead concentration differences between basin and control samples were small or did not exist below 5.91 in (15 cm). Also, the leaching of zinc was minor compared to the large surface accumulations.

The results of the soil depth investigation by Wigington were compatible with the research findings of Nightingale. (Wigington et al., 1983; Nightingale, 1975) He had found that large concentrations of zinc, lead, and copper were limited to the surface 1.97 in (5 cm) of soil in California urban stormwater detention basins. For these metals, background levels were reached at a soil depth of 5.91 to 11.81 in (15 to 30 cm).

Residence Time

The period of time that runoff is detained within a wetland is perhaps the most important factor in the removal of metals and other toxicants from highway runoff. With sufficient residence time, significant pollutant removal may be achieved by sedimentation processes, adsorption onto bottom sediments, and by the percolation of runoff waters through the soil. Sedimentation is aided by a suitable vegetative cover that serves to slow water velocities, diminish short circuiting of flows, and trap sediment particles. Many toxicants in runoff may be adsorbed to suspended sediment particles in the water column, and thereby subject to removal by solids settling. In summary, any wetland features that promote sedimentation will also improve the nonpoint pollution removal efficiencies.

The residence time that can be achieved in a constructed wetland depends upon such factors as slope, the ratio of open water to vegetated area, the hydraulic load, the area available for construction of a wetland, the frequency of rainfall events, and the extent to which water can percolate through the soils. Specific recommendations for residence time are discussed below.

Hydrology

An important objective of the constructed wetland system is to maintain a soil moisture profile that will support the vegetative cover that has been established. Whether a constructed wetland is a feasible alternative for the management of highway runoff depends strongly upon the frequency and intensity of rainfall, and the distribution of precipitation throughout the year.

If the chosen vegetative cover is lost, or cannot be established in the first place because of insufficient rainfall, the wetland will not perform as expected, and the following problems may be encountered. First, sedimentation may not be as extensive in a marginal wetland as in a wetland with a fully developed ground cover. Second, erosion and short circuiting may occur in a constructed wetland with a marginal ground cover. Third, if the surface soil in a constructed wetland desiccates completely, a hard, impervious surface may be formed that will not absorb runoff like a typical wetland. Thus, wetlands are not a suitable mitigation measure for the management of highway runoff in areas subject to lengthy dry seasons with insufficient rainfall to maintain the required basin soil and vegetative characteristics.

Climate

Due to the lengthy dry periods, wetlands are probably not suitable for the management of highway runoff in arid areas. The use of constructed wetland systems may be most practical in humid parts of the country that have a warm to mild winter. Wetlands located in areas that have highly erosive rainfall are most likely to receive larger pollutant loads from the upstream watershed area; therefore, wetlands in these areas will have a greater opportunity to provide pollutant removal benefits. A rainfall erosivity index of 300 has been proposed as an indicator of climatic areas where wetlands may be most effective for runoff pollution removal. (Adamus et al., 1987; Stewart et al., 1975)

Where native species are used, the plants may still go into a dormant period. Plants that undergo some form of dormancy will require a period of time in the spring for seeds to begin to grow, or for vegetative portions of the plant to produce new growth. During the winter or early spring when the ground is frozen and plants are inactive, the efficiency with which highway runoff is treated may be substantially less than in warmer periods when the vegetative cover is fully established. A wetland system may be hydraulically overloaded by snow melt or by heavy spring rains, which could jeopardize the vegetative cover, particularly so where the plants have died altogether and it is necessary to replant or reseed manually. Fewer operational problems will be encountered in warmer parts of the country where the vegetative cover remains active throughout the year.

D. WETLANDS DESIGN PARAMETERS

The selection of wetlands design parameters will often require consideration of site specific features or performance standards promulgated by regulatory agencies. Wetlands designs for management of highway runoff pollution should consider the following general design parameters:

- Relatively long retention time of runoff inflow.
- Shallow water with a low basin gradient resulting in slow-moving, well-spread sheet flow.

- Minimal direct open channels (where open channels exist, circuitous flow routes are preferred).
- Maximum contact between runoff inflows and wetland soils and vegetation.
- Irregular bottom morphology and bank edges.
- Constricted outlet or no surface outlet.
- Persistent emergent and/or floating aquatic vegetation forms.
- Sufficient storage volume for runoff.

Where information is available in the literature, additional details on wetland design parameters are outlined below.

Runoff Storage Volume

The Florida Department of Environmental Regulation has promulgated guidelines promoting the use of some wetlands for stormwater treatment. Chapter 17-25 of the Florida Administrative Code contains "performance standards" for the use of wetlands to control urban stormwater runoff. The 17-25 regulations state that the wetland treatment system may be used in combination with other BMPs and must provide storage/treatment for the initial 0.5 in (1.27 cm) of runoff. This storage parameter is a potential starting point for the design of a wetland system for highway runoff control.

Retention Time

The retention time is defined as the length of time a water particle remains in the wetland. For wetland treatment of municipal wastewaters, optimum retention times are reported to be on the order of 6 to 10 days and 7 to 14 days by various investigators. The use of wetlands for wastewater treatment involves a constant daily application rate which may be expressed in terms of inches per week. Treatment of highway stormwater runoff will be required only when a storm event occurs, a somewhat random phenomena. Therefore, wetland retention times for stormwater runoff management should consider the periodicity of storm events based on local climatologic conditions. Allowances should be made to drain the wetland system sufficiently between storm events so that the next storm event can be accommodated by available wetland storage capacity. However, the wetland must not be drained so much as to jeopardize the vegetative cover. The Florida 17-25 Regulations require that the wetland outfall structure be designed to bleed-down the specified treatment volume in no less than 120 hours with no more than one-half of the volume discharged within the first 60 hours. Statistical analyses of long-term runoff capture, such as those presented in earlier chapters, may be suitable for wetland designs in conjunction with assumed dewatering rates.

For wetland areas that include a shallow open water basin, the design procedures for wet detention basins outlined in chapter 3 can be used. These methods use estimates of runoff from the mean annual storm event.

For stormwater control systems that include wetlands, the controlled eutrophication method should be employed and an average hydraulic residence time of 2 to 3 weeks should be the design criterion.

Depth/Slope

The depth of inundation will vary with hydroperiod and hydraulic loading. Hydroperiod is the natural, seasonal fluctuation of wetland water levels. Important aspects of hydroperiod are timing, depth, and area of inundation. Long-term changes in depth may cause a shift in types of vegetation. Typical constructed wetland depths are about 1 to 3 ft (0.31 to 0.92 m). Side slopes should be on the order of 10:1 to 20:1 (H:V) to allow emergent vegetation to be established. Depth should vary to control the ratio of emergent vegetation to open water. Species diversity is typically greatest with a one-to-one ratio of emergent vegetation to open water.

For shallow wetland stormwater basins, Maryland guidelines (1987) recommend that 25 percent of the total wetland area be 2 to 3 ft (0.61 to 0.92 m) deep and 75 percent of the area less than 12 in (30.48 cm) deep. The outlet should be located in the deeper portion of the wetland to prevent blockage of the outlet structure. Of the wetland areas less than 12 in (30.48 cm) deep, approximately 25 percent should be less than 6 in (15.24 cm) deep. A length to width ratio of at least 2 to 1 is recommended to prevent short circuiting of flows between the inflow and outflow. Vegetation and grading should also be designed to maximize sedimentation and the mixing of runoff with shallower wetland areas.

Inflow

Highway runoff discharged into wetlands should be controlled to prevent erosion and scour and to ensure adequate distribution of flow throughout the wetland. Inflow should be at nonerosive velocities, preferably via overland flow, utilizing grassed channels or spreader swales. A baffle, skimmer, or grease trap should ideally be located immediately upstream to prevent oils and greases from entering the wetland.

A forebay area, located at the wetland inflow point is recommended to capture larger sediments. The depth of the forebay should be at least 3 ft (0.91 m) and should contain about 10 percent of the total basin volume. The forebay should be designed to facilitate future wetland maintenance activities (e.g., sediment excavation).

Outlet

The requirements for an outlet structure for a constructed wetland include: impounding the required storage volume to create the wetland; detention of the runoff storage volume for a specified treatment period; and prevention of blockage to the outlet structure interfering with the wetland functions. (Maryland WRA, 1987)

The most common type of outlet structure is the barrel and riser type coupled with a low dam. This type of outlet would retain baseflow for creation of a wetland. The structure may have both orifice(s) and weirs that permit runoff to leave the wetland at a design flow rate.

2. WETLAND DESIGN PROCESS

Given the limited experience with the use of wetlands for nonpoint pollution management, the design process must be viewed as an evolving procedure which will change as more experience is gained. The wetland design process consists of three primary steps:

1. Estimation of the hydraulic and runoff pollution loadings from the highway drainage area including loadings from land uses adjacent to the highway that will be discharged into the proposed wetland.

2. Assessment of the wetland area and feasibility of its use as a treatment area for highway runoff, and determination as to whether existing unsuitable wetland areas may be improved for use as a mitigation measure or whether wetlands can be constructed.

3. Selection of values for the following design parameters for the specific wetland application: detention time, depth of water, and slope. In addition, attention will have to be given to the soils to be used (native or imported) and the types of plants to use in the wetland.

These three steps are described in the following sections.

A. ESTIMATION OF NONPOINT POLLUTION LOADINGS

In the first step, estimates are made of the hydraulic and pollutant loading which can be expected from a specific highway drainage area. Whether the wetland system will be retrofitted to an existing highway, or incorporated into the design of a proposed highway, the hydraulic loading from the highway (e.g. the volume of runoff) is probably the single most important parameter because this will determine the detention time within the wetland system. Determination of hydraulic loading will involve:

- Delineation of the highway drainage area.

- Determination of the existing or proposed impervious acreage within the drainage area.

- Selection of an appropriate design storm or runoff volume (e.g., mean rainfall event volume) to be treated by the wetland system. This will be site specific and a function of local climatology and hydrology. General recommendations are presented in the chapter 3 on wet detention basins.

- Computation of the runoff volume from the highway drainage area.

It will also be useful to have an estimate of the nonpoint pollution loadings that will be discharged into the wetland. If extensive construction activities are anticipated within the highway drainage area, or high sediment loadings are projected for other reasons, pretreatment of stormwater runoff by an upland detention facility may be required to avoid excessive sedimentation within the wetland system.

Annual nonpoint pollution loading rates are usually satisfactory because deterioration of the wetland system will generally result from long term "chronic" loadings. Nevertheless, local climatology or hydrology combined with the importance of short-term shock loadings may dictate seasonal analyses. For example, in parts of Florida there is a distinct 4 month wet season (June through September) when as much as 60 to 70 percent of the annual precipitation occurs. Therefore, seasonal analyses may be appropriate for Florida, whereas annual analyses would be more appropriate to other areas where precipitation is more evenly distributed throughout the year.

B. NATURAL WETLAND SITE ASSESSMENT

These site assessment guidelines presume that there is a natural wetland available, or that construction of a wetland is considered feasible. An assessment should be undertaken of the existing or proposed wetland site to determine whether treatment of highway stormwater runoff is an appropriate use of the site. It would be appropriate to apply the Wetlands Evaluation Technique (WET) to assess the candidate site.

Existing Wetland Vegetation

For existing wetland sites, vegetation types should be characterized using the following three categories: (Chan et al., 1982)

- Emergent - rooted in sediments and growing through the water column above water level.
- Floating - aquatic roots with plant parts partly submerged or fully exposed on the water surface.
- Submerged - aquatic plants such as algae, or plants rooted in sediments, with all plant parts growing within the water column.

In general, pollutant removals improve with an increasing diversity of vegetation in the wetland.

Existing Wetland Type

The type of wetland may be characterized using the Cowardin or other classification systems. The uniqueness or sensitivity of the wetland and applicable regulatory protection should be ascertained (e.g., wetlands which are habitat for a protected species).

Various types of natural wetlands which have been used to treat wastewater stormwater runoff include:

- Mixed riverbottom hardwoods.
- Northern peatlands.

- Cattail/grass marshes.
- Southeastern swamplands.
- Cypress domes.
- Freshwater/tidal marshes.
- Open ponds.
- Meadow/seepage wetlands.

Little information is available on the advantages of one wetland type over another. Local conditions and requirements will be the primary factors in the selection of a wetland type.

Existing Wetland Hydrology

Assessment of the flow patterns and hydroperiod in the wetland is an important element of site assessment. Hydroperiods vary considerably from one type of wetland to another. For example, the hydroperiod of a coastal saltmarsh is controlled by daily tidal fluctuations that flood the marsh. In contrast, bottomland wetlands on a river may be inundated during the rainy season, or during spring melt, while the water table falls below ground level for the rest of the year.

Since the wetland hydroperiod defines the duration, frequency, and extent of inundation, alteration of this factor may have a profound effect on the vegetation types that will persist within a given wetland, and on the overall pollutant removal capability of a given wetland. Typically, wetlands that are permanently flooded, such as cypress domes and marsh-pond systems, can accommodate either intermittent or continuous polluted water discharges because their ecosystems are not dependent upon the polluted water inflow. Wetlands that are not permanently flooded, such as wet meadows or peatlands may be much more sensitive to hydrologic changes resulting from inflow of polluted waters.

Depth of inundation varies among wetland types as well as within a given wetland. The highest pollutant removal efficiencies are generally achieved with shallow water depths due to increased sedimentation and adsorption onto bottom sediments.

The water budget of an existing wetland should also be assessed. A water budget is a mass balance on the inflows and outflows from a wetland. The water budget should account for the following sources and sinks:

- Precipitation volume falling on the wetland surface area.
- Highway runoff into and from the wetland.
- Overland flow to the system from other sources.

- Volume of groundwater inflows and outflows.

- Evapotranspiration losses.

The water budget provides a means of estimating residence times within a wetland assuming that there is no change in storage. The water budget analysis may be performed on an annual or seasonal basis.

Flow patterns within an existing wetland should be assessed. Meandering channels with slow moving water and a large surface area will tend to show relatively high pollutant removal by sedimentation. Shallow, sheet flow patterns tend to enhance some assimilative processes. Deeper pools can sometimes improve the potential for denitrification. Overall, mixed flow patterns tend to provide higher pollutant removal efficiencies.

Geomorphology

Information about the soils and water table depths provides additional insight regarding the ability of an existing wetland to treat highway runoff. Identification of soil types for a wetland provides an indication of physical-chemical properties such as pH, cation exchange capacity, and permeability. The presence or absence of an impermeable pan layer can significantly influence groundwater impacts.

C. CONSTRUCTED WETLAND SITE ASSESSMENT

The assessment of a site for a constructed wetland system will require consideration of many of the factors described above for natural wetland systems.

The designer will have more leeway in terms of locating the wetland system, and sizing the wetland for hydraulic loading and detention time. Three major factors should be considered when assessing site suitability for constructed wetlands:

- Water table depth.

- Soil/Substrate.

- Space requirements.

The water table should be at or near the surface. The most effective wetland treatment systems are generally those receiving groundwater discharges. Ideally, constructed wetlands should have water at or above the ground surface most of the year. Wetlands that rely solely on runoff inflow for water supply typically experience significant fluctuations in water level, limiting establishment of vegetation. Locations for constructed wetland systems that may meet hydrologic requirements include natural depressions, borrow pits, flat terrain, and shorelines.

The soil or substrate of constructed wetland systems should consist of loose loam to clay textures, with a pH neutral to alkaline. Organic material is often helpful for pollutant removal and retention.

The space required for a constructed wetland system will be greater than a detention facility serving the same highway area. This is because the wetland system must be shallower with much flatter side slopes, while exhibiting similar storage capacities.

3. CONSTRUCTION CONSIDERATIONS

The construction of a wetland involves site preparation and establishment of the vegetative cover.

A. SITE PREPARATION

Site preparation involves site grading and substrate treatment. Because the elevation of the substrate and water levels are of critical importance to a wetland, site grading activities should be carefully planned and executed. Water elevation tolerances of of the selected plant species must be carefully monitored as part of final grading.

Topsoiling a wetland site is generally not required. However, topsoil from upland areas associated with the site can be removed and stockpiled during grading, and respread at the final grade. The substrate is important for vegetative establishment and pollutant removal, and should consist of clean materials (inorganic/organic) at least 1 ft (0.31 m) in depth, most of which can pass a No. 10 sieve. An organic substrate with fine grained clay particles is ideal for many constructed wetlands.

Other, less desirable, substrates may require treatments to improve their capabilities. Lime can be added to reduce substrate acidity. Organic material can be incorporated into the substrate to increase its ability to retain pollutants and support plants. Peats are generally acid, and should therefore be avoided. Gravel can be added to fine-grained unconsolidated substrates to provide physical support and anchoring for emergent vegetation. If necessary, substrates can be sealed underneath with a clay layer to retain water. If subsoil fertility is extremely low (e.g., sand), fertilizer may be incorporated into the substrate. Low levels of a slow release fertilizer are recommended. (Kobriger et al., 1982)

B. VEGETATIVE ESTABLISHMENT

The establishment of a suitable vegetative cover is an important concern in the development of a wetland treatment system. The ability of a wetland's vegetation to absorb and retain pollutants, anchor sediment, and reduce velocities to enhance sedimentation has a significant effect on pollutant removal. Key aspects of vegetative establishment are species selection, plant propagation, and the planting methods and spacing.

The selection of appropriate plant species (and species mix) is largely dependent upon the site and wetland type. Recommendations for plant species include:

- Select a minimum number of species adaptable to various water elevations and prevalent in wetlands throughout the region.

- Select primarily persistent perennial species.

- Select species known to remove nutrients and other pollutants from sediments and the water column.

- Select species characterized by sufficiently dense growths (as individuals and stands) to enhance filtration, stabilize the substrate, and provide habitat (but are not foraged by wildlife).

Garbisch presents a list of selected perennial wetlands plants that are commonly encountered throughout the U.S. (Garbisch, 1986) These plants have been successfully utilized in artificial wetland establishment projects, and are commercially available from selected nurseries. The Maryland WRA guidelines recommend that a constructed wetland include two primary species that spread aggressively that are planted in quantity within the wetland area. (Maryland Department of Natural Resources, 1987) In addition, a smaller quantity of three secondary species should be included to provide diversity for wildlife.

A variety of species should be used for practical and aesthetic reasons. Because many plants are selective in their accumulation and biomagnification of various heavy metals, mixed stands of vegetation provide the best overall heavy metals removal rates. Monocultures are unstable and can be devastated by disease, herbivore activity, or fluctuations in environmental conditions.

Plant propagation is accomplished by seeds, rootstocks, rhizomes, tubers, cuttings, seedlings, and transplants. Seeding is the most economical means for establishing wetland vegetation; however, seeding is likely to have a higher risk of failure. Seed germination and seedling development may be limited by water temperature, siltation, turbidity, or salinity. Transplants, although the most labor intensive, are the most effective, particularly for persistent emergents. Transplants should originate from within the region. Plant propagation should be limited to species common to natural wetlands within the region. Plant spacing is highly site and species specific, and is governed by substrate, type of propagule, growing season, and desired density of cover.

Transplant spacings of 1 to 4 ft (0.3 to 1.2 m) in parallel rows are recommended for persistent emergent plants. This spacing should achieve a relatively uniform cover by the second growing season, while making efficient use of machinery and labor. Most constructed wetlands will require two growing seasons for the vegetation to be fully established. During this time, replacement planting may be required.

C. CONSTRUCTION TECHNIQUES

General guidelines for the construction of wetlands are as follows:

- Minimize all construction slopes to reduce erosion potential.

- Avoid compacting soil where not required.

- Revegetate disturbed wetland areas with water-tolerant species.

- Construct levees at least 10 ft (3.05 m) wide and 1 ft (0.31 m) above the highest water level for ease of access.

- Maintain strict control of water entering and leaving the site during installation to avoid unnecessary soil erosion and inhibition of installation activities.

- Install sediment traps in areas that receive runoff from upstream where pretreatment is advisable.

- Avoid the installation of pipelines or facilities directly adjacent to a wetland during ecologically-sensitive period (e.g., during reproductive periods for sensitive wetland species).

4. OPERATIONS AND MAINTENANCE

General O&M guidelines have been developed for constructed wetlands systems used for wastewater treatment. Typical O&M activities include harvesting and other activities to maintain a suitable vegetative cover, spraying for the control of mosquitos and other pests, monitoring, and periodic replacement of substrate and vegetation if the assimilative capacity of the system should be depleted.

However, the hydraulic regime of a wetland used to treat a wastewater effluent is essentially different from the wetland that is used to treat highway runoff. For the former there is a constant flow of water whose range of variation is predictable, and can be accommodated in the design of the wetland. In the case of highway runoff, inflow will: (1) coincide with rainfall, (2) be intermittent and random, and (3) depend upon the intensity and duration of a particular storm. Excessive rainfall may cause erosion in a wetland, or may damage the vegetative cover if inundation should last longer than can be tolerated. If there is too little rainfall, plants may die and require replacement.

An O&M program should be established for wetland systems designed to provide treatment of highway runoff. The O&M program may include the following activities:

- Periodic sediment removal within wetland. Sediment removal may be required to maintain flow patterns, decrease benthic oxygen demand, and remove accumulated nutrients, metals, and other pollutants.

- Harvesting or burning of vegetation. Periodic harvesting may allow for greater vegetative diversity, especially if a vegetative monoculture has been established. Harvesting will also remove nutrients and other pollutants from the wetland system while they are bound up in the vegetative structure. Burning is also used to control vegetative monocultures and to provide the "burn" environment needed periodically by some wetlands as part of their natural regeneration process.

- Toxic monitoring. Since highway runoff may contain high concentrations of heavy metals and other toxicants, it may also be necessary to monitor the wetland periodically and report the findings to the appropriate regulatory agencies. Finally, should the wetland require occasional dredging and other activity to maintain its function, or should the assimilative capacity of the wetland be exhausted and require complete replacement, it may be necessary to conduct additional testing for toxics in order to determine the proper mode of disposal of dredged materials.

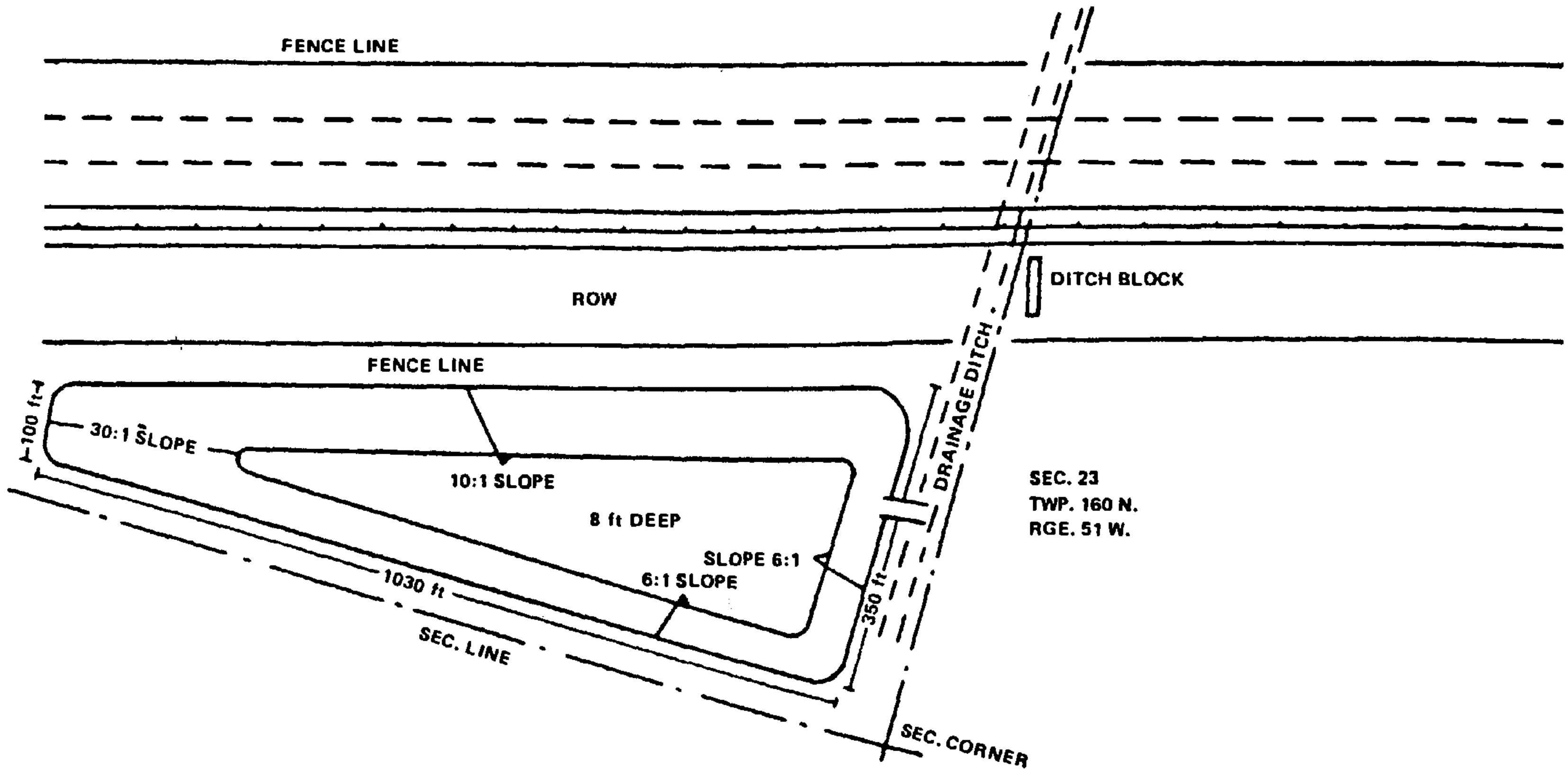
- Mosquito control. Control of mosquitos can be accomplished by introduction of mosquitofish (*Gambusia affinis*) and encouragement of selected bird species. Mosquitofish are tolerant of low oxygen conditions, breed prolifically, and are voracious consumers of mosquito larvae. Insecticide applications or manipulation of water level at certain times of the year, if feasible, may also help to control mosquitos. Biological larvacides (i.e., *Bacillus thuringiensis*) are also available which attack the immature stages of the insect.

Other maintenance concerns include repair of channels, berms, and hydraulic control structures located at the inlet to and outlet from the wetland.

5. HIGHWAY APPLICATIONS

The development of wetland treatment systems within the highway right-of-way requires special consideration of wetland shape. Highways and the land available for wetland construction in the rights-of-way are linear systems. Therefore, constructed wetlands for highway runoff control will often be linear in design, as their shape is constrained by the available land. Wetlands within the right-of-way may be constructed as a single system with a relatively high length to width ratio or as a series of smaller systems. Several small wetland systems linked together can often provide a greater variety of hydrologic and vegetative conditions, enhancing potential pollutant removal mechanisms. However, large individual wetland systems often provide greater retention of highway runoff as well as greater permanence.

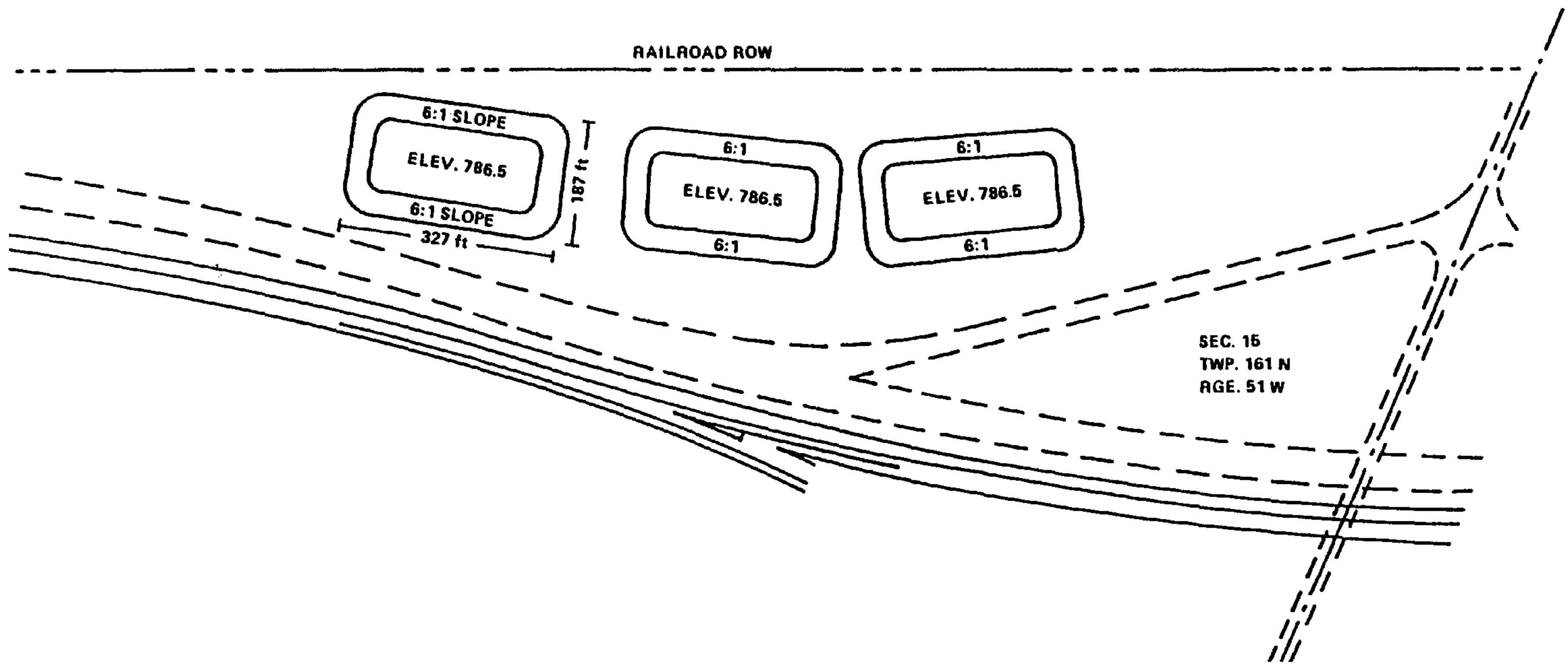
Examples of triangular, rectangular, and hexagonal wetlands constructed adjacent to highways are shown in figures 47 through 50. These wetlands were constructed in North Dakota for the purpose of replacing wildlife (specifically waterfowl) habitat lost during highway construction. They are presented here as examples of how wetlands may be designed into linear highway systems. The hexagonal wetland system of three linked wetlands in figure 50 appears to have the greatest potential for pollutant removal.



Source: Rossiter and Crawford, 1983.

1 ft = 0.305 m

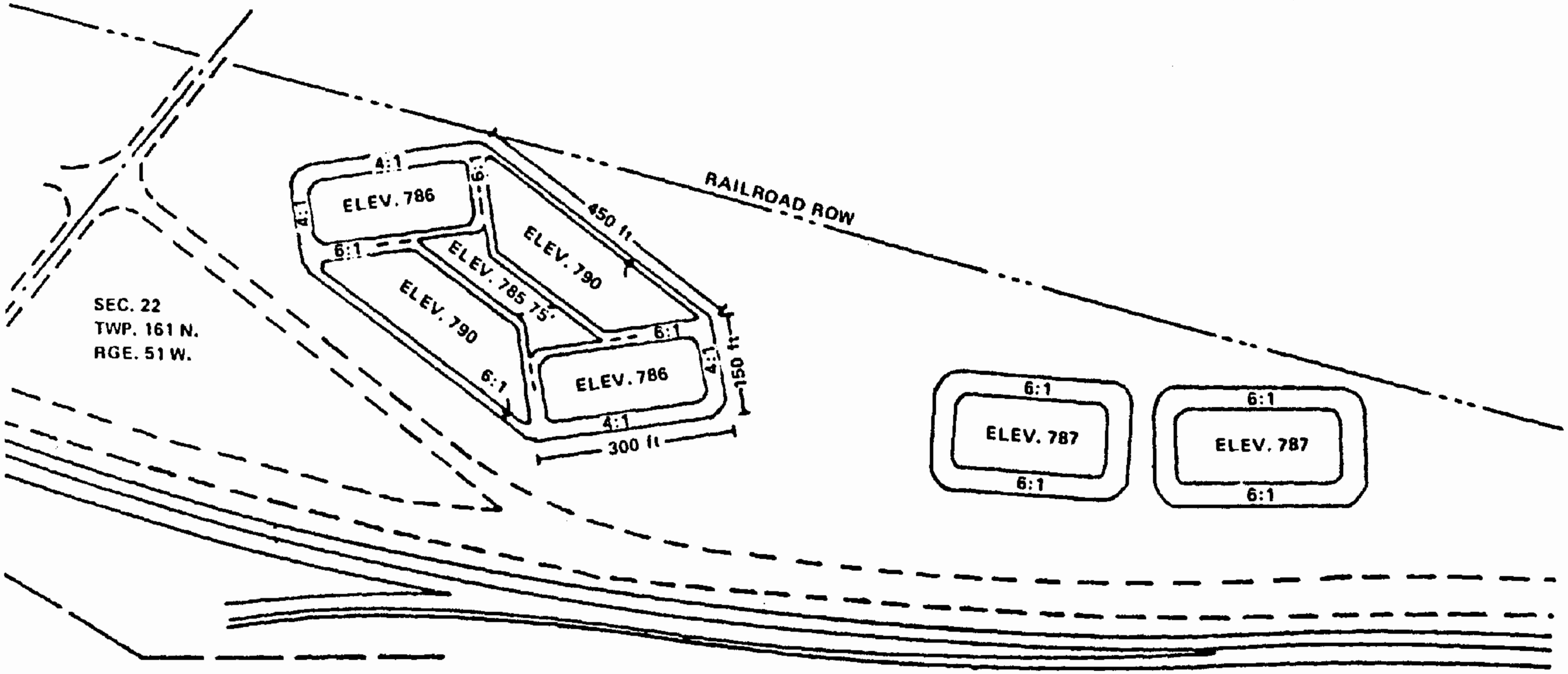
Figure 47. Triangular-shaped constructed wetland with surface area approximately 3.7 acres (1.5 ha).



1 ft = 0.305 m

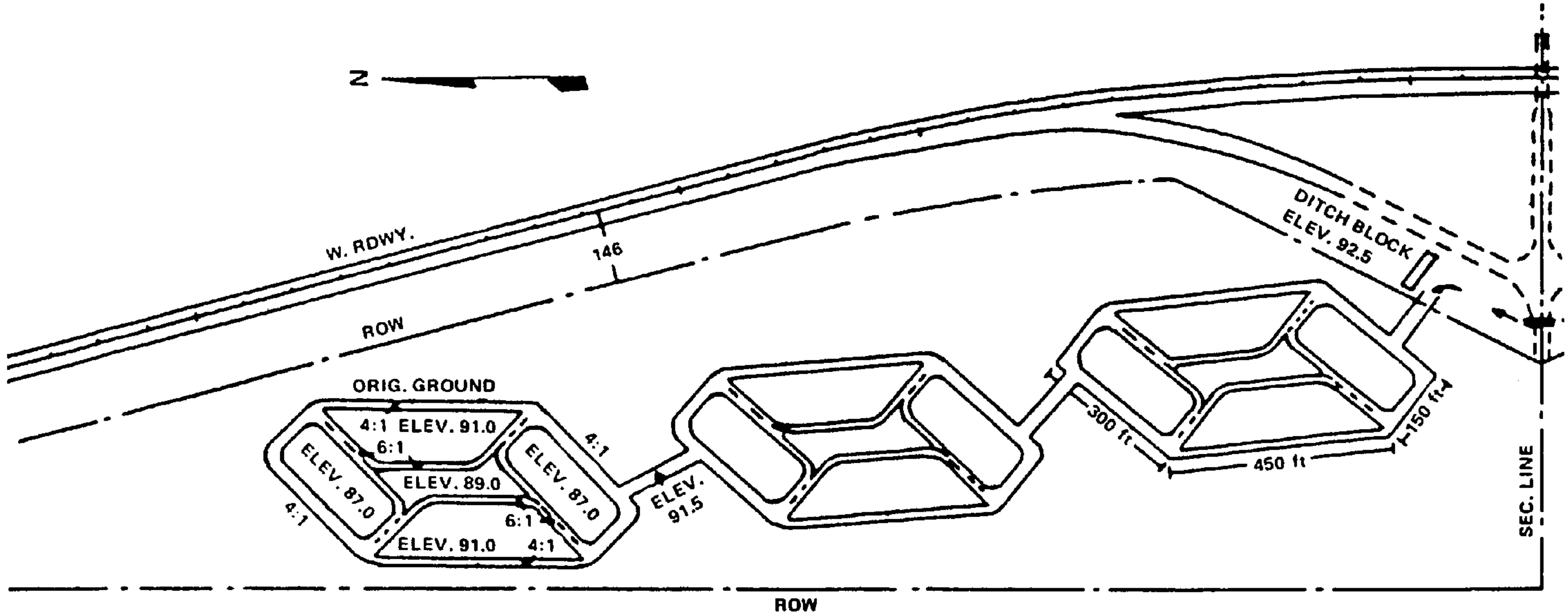
Source: Rossiter and Crawford, 1983.

Figure 48. Rectangular constructed wetlands.



Source: Rossiter and Crawford, 1983. 1 ft = 0.305 m

Figure 49. Rectangular constructed wetlands with mean surface area of approximately 1.5 acres (0.6 ha).



Source: Rossiter and Crawford, 1983. 1 ft = 0.305 m

Figure 50. Hexagonal-shaped constructed wetlands with mean surface area of approximately 3.3 acres (1.3 ha).

7. COMBINATIONS OF EFFECTIVE MEASURES

In applying management measures to specific highway runoff situations, it may be desirable to combine two or more measures. Combinations of measures may increase pollutant removal effectiveness, allow for filtration of suspended solids, or be used to overcome site factors which limit the effectiveness of a measure. Although each of the five cost-effective measures discussed in chapters 2 through 6 may be used alone, combinations of measures are recommended where practicable.

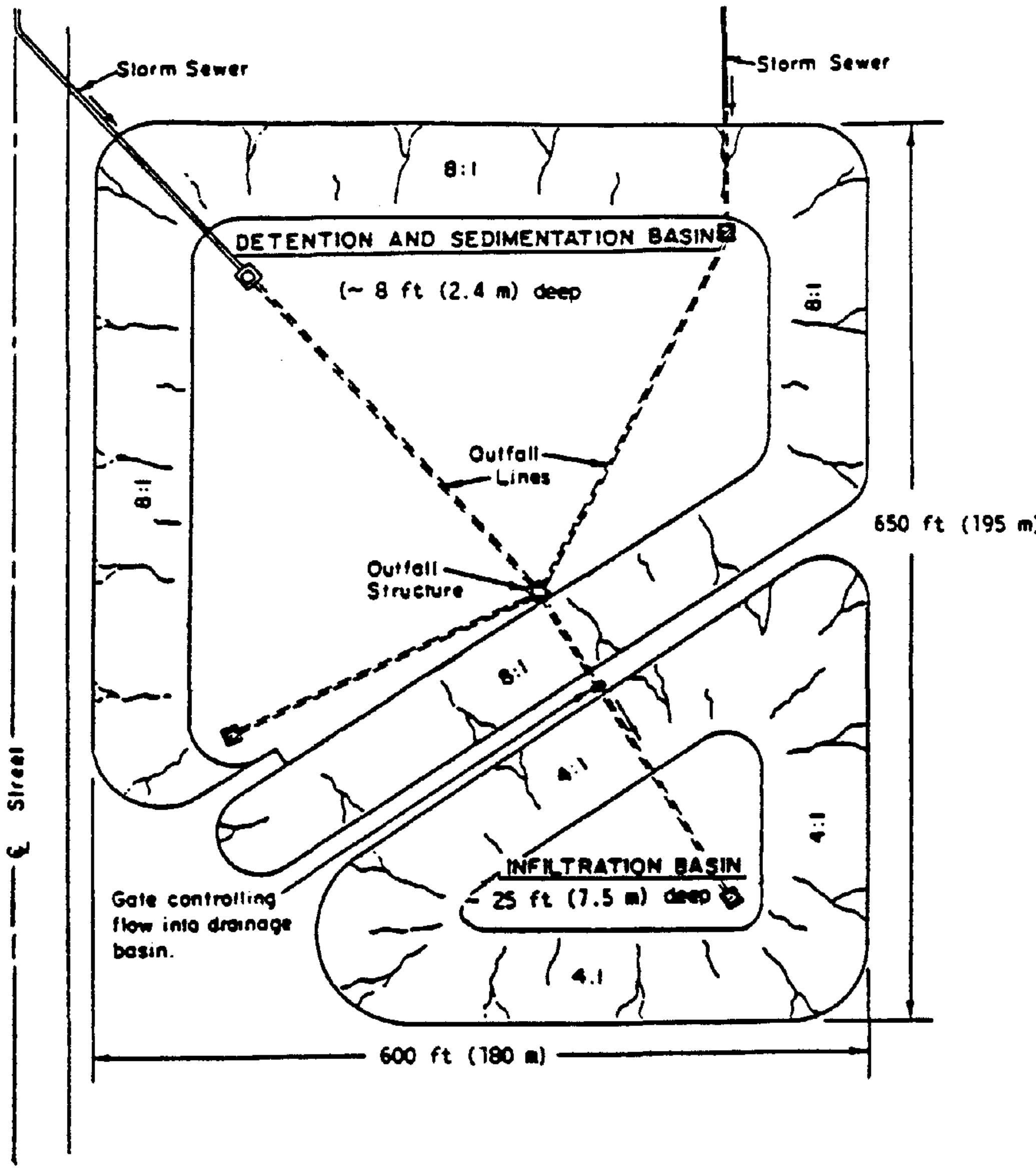
Vegetative controls are the only management measure providing pollutant abatement while the runoff is conveyed from point to point. Therefore, vegetative controls should be used to convey highway runoff wherever possible. Such controls should serve as the runoff collection and conveyance system, both a single management measure and as a link between different measures.

Vegetative controls can be used in combination with other effective management measures to increase pollutant removal, provide filtering of suspended solids for infiltration systems, and reduce erosion and scour at inflow discharges to infiltration basins, detention basins, and wetlands. In addition, the vegetative controls will extend the life of the downstream management measures and reduce the potential for resuspension of particles trapped in these measures. Combinations are particularly advantageous where the desired length of grassed channel or width of overland flow is unobtainable. Vegetative controls include overland flow over grassed channels and vegetated areas. Suggestions for management measure combinations involving vegetative controls include:

1. Highway runoff → overland flow → grassed channel → receiving water
2. Highway runoff → vegetative control(s) → detention basin → receiving water
3. Highway runoff → vegetative control(s) → infiltration trench → receiving water
4. Highway runoff → vegetative control(s) → infiltration basin → receiving water
5. Highway runoff → vegetative control(s) → wetlands → receiving water

Detention basins may be used in combination with vegetative controls to provide storage of runoff or sediment removal prior to infiltration basins or wetlands. Figure 51 presents an example of a combined detention basin and an infiltration basin. Note that the infiltration basin is relatively small and deep in comparison with the detention basin, affording increased side wall infiltration area as well as increased head.

The primary consideration in the use of infiltration systems for pollutant removal from highway runoff is the vulnerability of the systems to



Source: Jackura, 1980.

Figure 51. Detention basin and infiltration basins combinations.

sediment. Except for basins receiving relatively sediment-free runoff, infiltration systems require additional highway runoff management measures (vegetative controls or detention basins) to provide adequate runoff storage and sediment removal prior to infiltration. In addition, infiltration systems require more frequent maintenance. Thus, infiltration systems are usually an add-on feature to other management measures. Additional potential combinations of infiltration systems and other management measures are:

1. Highway runoff → detention basin → infiltration basin

2. Highway runoff → detention basin → vegetative control(s) → infiltration basin

3. Highway runoff → vegetative controls → detention basin → infiltration basin

Wetlands can be used in combination only with vegetative controls or detention basins, not with infiltration. Wetlands should not be used prior to infiltration basins, as accumulated sediment and/or decaying plant matter are often flushed from wetlands in the spring. The sediments and particulate matter could clog the infiltration basin. In addition, conditions favorable to wetlands, such as a high water table and impervious soils, are unfavorable to infiltration measures.

The sequence of combinations of effective management measures should be developed such that the least costly measure with the least maintenance requirements, such as vegetative controls, is first to receive the highway runoff. (NVPDC, 1987) The upstream facility will reduce the maintenance cost and prolong the effective life of the downstream facility.

The type of detention basin included with the combined system will depend upon whether detention is installed upstream or downstream of the other BMP. For example, if the detention basin is to be installed downstream of a vegetative control, a wet detention basin designed for removal of dissolved nutrients is preferable to supplement the removal of suspended pollutants within the vegetative control. If the detention basin is to be installed upstream of an infiltration BMP, either a wet or dry detention basin should suffice to reduce discharges of sediment and suspended pollutants into the infiltration basin.

Calculations of the combined pollutant removal efficiency should reflect removal rates for both dissolved and suspended pollutants. For example, assuming that a grass channel exhibits a 70 percent removal rate for TSS and a wet detention basin exhibits a 90 percent TSS removal rate when each BMP is evaluated individually, the composite efficiency of a grass channel/detention basin combination will be less than the sum of the individual BMP efficiencies (i.e., sum = 1.0 - ((1.0 - 0.7)(1.0 - 0.9)) = 0.97 or 97%). This is because the grass channel removes a considerable amount of the TSS (probably larger particle sizes) which would otherwise be removed in the wet detention basin. Therefore, it would be double-counting to combine the individual BMP efficiencies without adjusting for the overlapping pollutant removal. To calculate the composite pollutant removal efficiency for a combined BMP system, the methods presented in earlier chapters should be applied sequentially with the pollutant outflow from the upstream BMP serving as inflow for efficiency calculations in the downstream BMP.

Appendix A. Modifications of design curves for wet detention basins.

Driscoll reported on the analysis of NURP data to evaluate long-term average TSS removal efficiencies of wet detention basins. (Driscoll, 1983) A probabilistic analysis methodology was used to compute performance directly from the statistical properties of detention basin inflows. The analysis assumes that overall performance is due to the combined effect of removals which take place under dynamic conditions as flows move through a basin and under quiescent conditions during the intervals between successive storms. Results of the analysis were tested against observed performance and monitored storm events from the NURP data base of 5 to 30 or more separate storm events at each of 13 detention basins.

Results of the analysis have been used to develop the design procedure presented in chapter 3. This appendix provides the step-by-step method used to develop the design curves (figures 20 through 24) and a procedure which may be used to develop design curves where rainfall statistics have been developed for a locality. The procedure may also be used to develop design curves where particle size distribution differs from the assumed distribution used in the development of figures 20 through 24.

Figures 52 and 53 were developed from the analysis of NURP data. (Driscoll 1983) The figures may be used for design, but the procedure is tedious and time-consuming.

The long-term average TSS removal efficiency, E, of a wet detention basin is equal to the removal efficiency during the period in which storm runoff is passing through the basin, or dynamic efficiency, plus removal efficiency between storm runoff events, or quiescent efficiency. This is stated in equation (20):

$$E = E_D(D/\Delta) + E_Q(1- D/\Delta) \tag{20}$$

where:

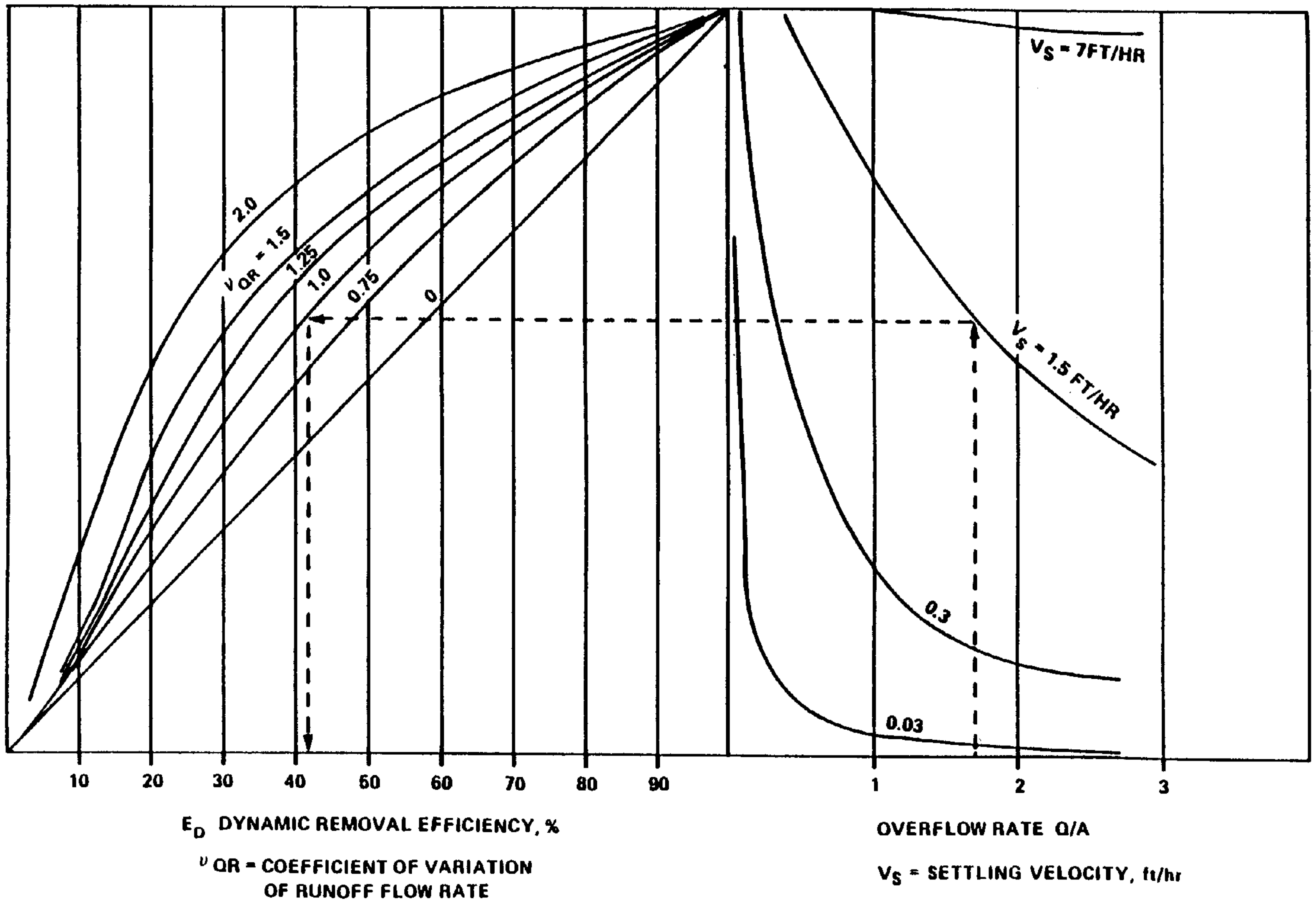
E_D = dynamic removal efficiency

D = mean storm duration, hrs

Δ = mean time interval between storm midpoints, hrs

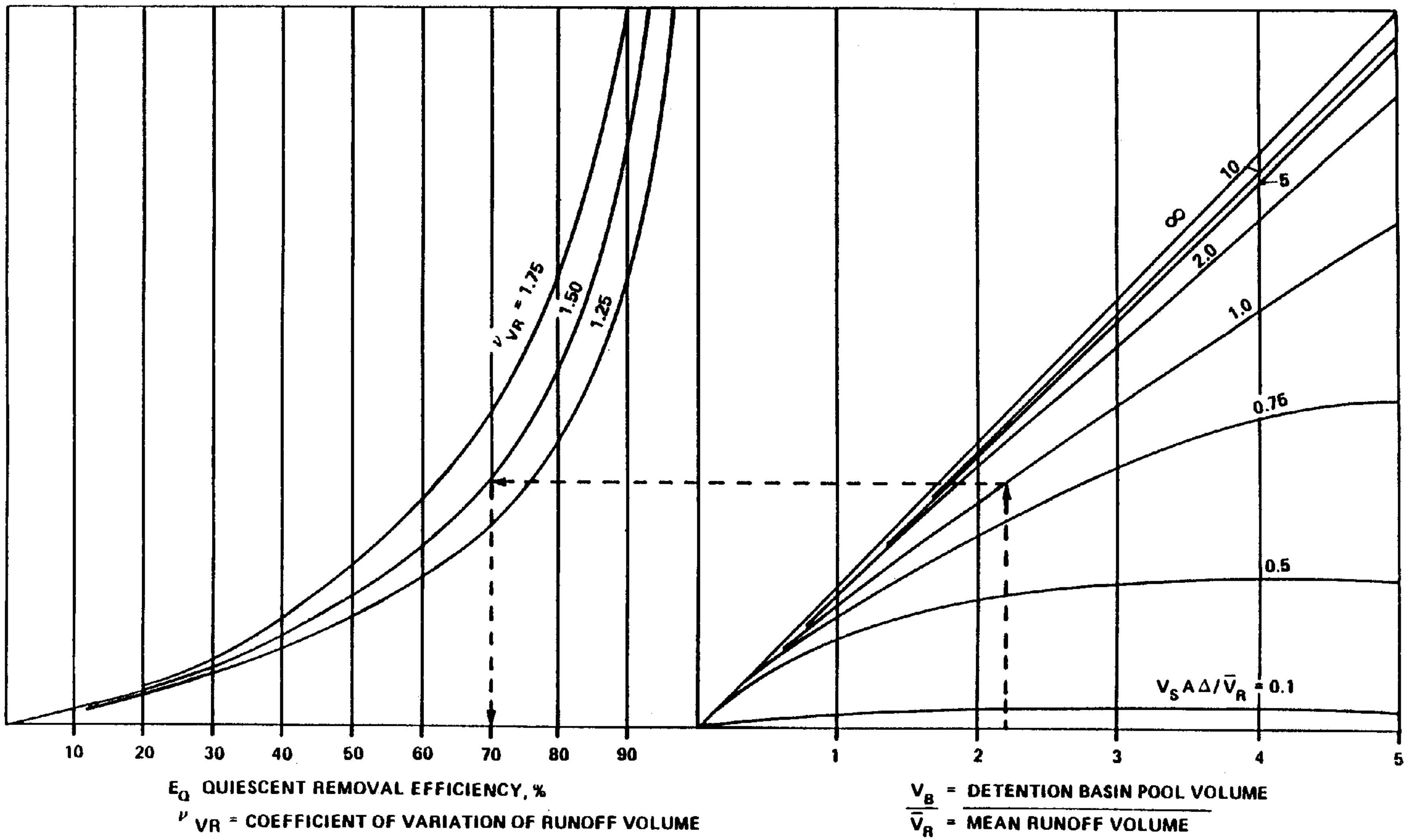
E_Q = quiescent removal efficiency

Values for dynamic and quiescent TSS removal efficiency can be read from figures 52 and 53. The following is to define relationships between the parameters used in the figures.



Derived from Driscoll, 1983.

Figure 52. Dynamic removal efficiency for TSS.



Derived from Driscoll, 1983.

Figure 53. Quiescent removal efficiency.

The overflow rate in figure 52 is given as Q/A, where Q is the flow rate, in this case, ft^3/hr; and A is the surface area of the basin. Assuming that the mean duration of the stormwater runoff is equal to the mean duration of rainfall, D, then:

$$Q = V_R/D \tag{21}$$

where:

V_R = mean volume of runoff, ft^3

Neglecting the side slopes on the banks of a basin, surface area is equal to the volume of the basin divided by the depth:

$$A = V_B/H \tag{22}$$

where:

V_B = basin volume, ft^3

H = basin depth, ft

Therefore:

$$Q/A = (V_R H)/(V_B D) \tag{23}$$

Also, the coefficients of variation of rainfall volume, runoff flow, and runoff volume are assumed to be equal:

$$\nu V = \nu QR = \nu VR \tag{24}$$

where:

νV = coefficient of variation of rainfall volume

νQR = coefficient of variation of runoff flows

νVR = coefficient of variation of runoff volumes

The procedures used in the development of figures 20 to 24 are illustrated by an example for rainfall zone 7 using a 2 ft (0.61 m) basin depth. The following rainfall statistics for zone 7 are from table 7:

D = 20 hrs

Δ = 101 hrs

νV = 1.61

Step 1: Solve equation (23) in terms of V_B/V_R.

$Q/A = (V_R H)/(V_B D)$

$H/D = 2/20 = 0.1$

$Q/A = 0.1/(V_B/V_R)$

Step 2: Compute values for Q/A for values of VB/VR from 0.2 to 10.

V_B/V_R	Q/A	V_B/V_R	Q/A
0.2	0.5	2	0.05
0.4	0.25	4	0.025
0.6	0.17	6	0.017
1	0.1	10	0.01

Step 3: Read values of E_D from figure 52

		V_s=0.03	V_s=0.3	V_s=1.5	V_s=7	V_s=65
V_B/V_R	Q/A	E_D	E_D	E_D	E_D	E_D
0.2	0.50	5	21	84	100	100
0.4	0.25	7	40	99	100	100
0.6	0.17	9	59	100	100	100
1	0.10	12	84	100	100	100
2	0.05	21	99	100	100	100
4	0.025	40	100	100	100	100
6	0.017	59	100	100	100	100
10	0.01	84	100	100	100	100

Step 4: Compute values of $E_{D}(\frac{D}{\Delta})$.

$$E_{D}(\frac{D}{\Delta}) = (20/101)E_D = 0.2\ E_D$$

	V_B/V_R							
	0.2	0.4	0.6	1	2	4	6	10
V_S	$0.2E_D$	$0.2E_D$	$0.2E_D$	$0.2E_D$	$0.2E_D$	$0.2E_D$	$0.2E_D$	$0.2E_D$
0.03	1	1	2	2	4	8	12	17
0.3	4	8	12	17	20	20	20	20
1.5	17	20	20	20	20	20	20	20
7	20	20	20	20	20	20	20	20
65	20	20	20	20	20	20	20	20

Step 5: Compute TSS removal efficiency.

TSS removal efficiency is equal to the percentage of particle sizes in a size range multiplied by the dynamic efficiency of the wet basin in removing that size range.

For convenience, the assumed TSS distribution is repeated here.

Settling Velocity, V_S, ft/hr	TSS distribution (percent)
0.03	20
0.3	20
1.5	20
7	20
65	20

Let the dynamic efficiency in removing particles in each size range equal E_D.

		V_B/V_R															
	Dist	0.2		0.4		0.6		1		2		4		6		10	
V_S	%	$0.2E_D$	E'_D	$0.2E_D$	E'_D	$0.2E_D$	E'_D	$0.2E_D$	E'_D	$0.2E_D$	E'_D	$0.2E_D$	E'_D	$0.2E_D$	E'_D	$0.2E_D$	E'_D
0.03	20	1	0.2	1	0.2	2	0.4	2	0.4	4	0.8	8	1.6	12	2.4	17	3.4
0.3	20	4	0.8	8	1.6	12	2.4	17	3.4	20	4.0	20	4.0	20	4.0	20	4.0
1.5	20	17	3.4	20	4.0	20	4.0	20	4.0	20	4.0	20	4.0	20	4.0	20	4.0
7	20	20	4.0	20	4.0	20	4.0	20	4.0	20	4.0	20	4.0	20	4.0	20	4.0
65	20	20	4.0	20	4.0	20	4.0	20	4.0	20	4.0	20	4.0	20	4.0	20	4.0
Total			12.4		13.8		14.8		15.8		16.8		17.6		18.4		19.4

Step 6: Compute values of $V_S A\Delta/V_R$ for use in figure 52.

$$V_S A\Delta/V_R = V_S \frac{(V_B)}{H}\Delta/V_R$$

$$= V_S \frac{(V_B)}{V_R} \Delta/H$$

$$\Delta/H = 101/2 = 50.5$$

$$V_S A\Delta/V_R = 50.5 \frac{(V_B)}{V_R} V_S$$

	Values of $V_S A\Delta/V_R$				
V_B/V_R	$V_S = 0.03$	$V_S = 0.3$	$V_S = 1.5$	$V_S = 7$	$V_S = 65$
0.2	0.3	3	15	>15	>15
0.4	0.6	6	>15		
0.6	0.9	9			
1	1.5	>15			
2	3				
4	6				
6	9				
10	>15				

Step 7: Read values of E_Q from figure 52.

$\nu R = 1.6$

V_B/V_R	$V_s = 0.03$ E_Q	$V_s = 0.3$ E_Q	$V_s = 1.5$ E_Q	$V_s = 7$ E_Q	$V_s = 65$ E_Q
0.2	10	18	18	18	18
0.4	25	28	28	28	28
0.6	32	38	38	38	38
1	48	50	50	50	50
2	70	73	73	73	73
4	87	88	88	88	88
6	100	100	100	100	100
10	100	100	100	100	100

Step 8: Compute values of E_Q $(1-D/\Delta)$ and quiescent removal efficiencies. Let the quiescent efficiency in removing particles in each size range equal E_Q.

$$E_Q' = E_Q(1 - D/\Delta) = E_Q(1 - 20/101) = 0.8E_Q$$

		V_B/V_R															
	Dist	0.2		0.4		0.6		1		2		4		6		10	
V_S	%	$0.8E_Q$	E'_Q	$0.8E_Q$	E'_Q	$0.8E_Q$	E'_Q	$0.8E_Q$	E'_Q	$0.8E_Q$	E'_Q	$0.8E_Q$	E'_Q	$0.8E_Q$	E'_Q	$0.8E_Q$	E'_Q
0.03	20	8	1.6	20	4.0	26	5.2	38	7.6	56	11.2	70	14.0	80	16.0	80	16.0
0.3	20	14	2.8	22	4.4	30	6.0	40	8.0	58	11.6	70	14.0	80	16.0	80	16.0
1.5	20	14	2.8	22	4.4	30	6.0	40	8.0	58	11.6	70	14.0	80	16.0	80	16.0
7	20	14	2.8	22	4.4	30	6.0	40	8.0	58	11.6	70	14.0	80	16.0	80	16.0
65	20	14	2.8	22	4.4	30	6.0	40	8.0	58	11.6	70	14.0	80	16.0	80	16.0
Total			12.8		21.6		29.2		39.6		57.6		70.0		80.0		80.0

Step 9: Compute TSS removal efficiency, $E = E_D + E_Q$

V_B/V_R	E_D'	E_Q'	E
0.2	12	13	25
0.4	14	22	36
0.6	15	29	44
1	16	40	56
2	17	58	75
4	18	70	88
6	18	80	98
10	19	80	99

Step 9: Return to step 1 and repeat procedure for other basin depths.

Step 10: Plot results as in figures 20 to 24.

Appendix B. Design worksheets.

The design worksheets in this appendix are a compilation of all the worksheets that appear in the body of the report. The worksheets are presented here for ease of reference and photocopying by the reader/user.

GRASSED CHANNEL DESIGN WORKSHEET
(for triangular or trapezoidal cross sections)

*******************************DESIGN DATA********************************

Design Runoff Flow	____________________	(Q)	ft^3/s
Channel Slope	____________________	(S)	ft/ft
Channel Length	____________________	(L)	ft
Trial Bottom Width	____________________	(B)	ft
Side Slope Ratio (horizontal/ vertical)	____________________	(Z)	
Grass Species	____________________		
Vegetation Height	____________________		in
Soil Type (Circle One)	Erodible/Intermediate/Nonerodible		

********************DESIGN PROCEDURES FOR STABILITY*********************

Retardance (i.e., A, B, C) (from table 6)	____________________		
Maximum Depth of Flow (from figures 5 to 7)	____________________	(D_{max})	ft
Flow Area ($A = D_{max} \times (B + (Z \times D_{max}))$)	____________________	(A)	ft^2
Wetted Perimeter ($P = B + (2 \times D_{max}) \quad 1 + Z^2$)	____________________	(P)	ft
Hydraulic Radius ($R = A/P$)	____________________	(R)	ft
Velocity (from figures 8 to 12)	____________________	(V)	ft/s
Maximum Flow ($Q_{max} = A \times V$)	____________________	(Q_{max})	ft^3/s

If $Q_{max} > Q$ then channel will be stable. If not , increase bottom width, decrease longitudinal slope or increase side slope to increase the area of flow or to reduce velocity.

GRASSED CHANNEL DESIGN WORKSHEET
(CONTINUED)

**********DETERMINATION OF LONG-TERM POLLUTANT REMOVAL EFFICIENCY**********

Long-term Mean Runoff Event Flow	____________________	(Q_a)	ft^3/s
Trial Centerline Flow Depth	____________________	(D)	ft
Flow Area (A = D x (B + (Z x D)))	____________________	(A)	ft^2
Wetted Perimeter (P = B+ (2 x D) 1 + Z^2)	____________________	(P)	ft
Hydraulic Radius (R = A/P)	____________________	(R)	ft
Manning's n (n = 1.49 x A x $R^{2/3}$ x $S^{1/2}/Q_a$)	____________________	(n)	
Velocity (V = Q_a/A)	____________________	(V)	ft/s

If calculated Manning's n is in reasonable agreement with Manning's n from figure 13 as a product of V and R, then go on to next step; otherwise, revise trial depth and repeat calculations.

Average Depth D_{avg} = A/(B+(2 x D x Z)))	____________________	(D_{avg})	ft
Travel Time T = L/(60 x V))	____________________	(T)	min
Long-term TSS Removal from figure 2)	____________________	(E_{TSS})	%
Long-term Pb Removal (90% of TSS Removal)	____________________	(E_{Pb})	%
Long-term Cu Removal (60% of TSS Removal)	____________________	(E_{Cu})	%
Long-term Zn Removal (50% of TSS Removal)	____________________	(E_{Zn})	%

OVERLAND FLOW SYSTEM DESIGN WORKSHEET

********************************DESIGN DATA**********************************

Slope	______________	(S)	ft/ft
Width	______________	(B)	ft
Length	______________	(L)	ft
Vegetation Type	______________		
Vegetation Height	______________		in
Soil Type (Circle One)	Erodible/Intermediate/Nonerodible		

**********DETERMINATION OF LONG-TERM POLLUTANT REMOVAL EFFICIENCY**********

1-year, 24-hour rainfall depth	______________	(P)	in
Travel Time (from figure 15)	______________	(T)	min
Assumed Flow Depth (typically .01-.02 ft)	______________	(D)	ft
Long-term TSS Removal (from figure 2)	______________	(E_{tss})	%
Long-term Pb Removal (90% of TSS removal)	______________	(E_{pb})	%
Long-term Cu removal (60% of TSS removal)	______________	(E_{cu})	%
Long-term Zn removal (50% of TSS removal)	______________	(E_{zn})	%

If pollutant removal efficiency is less than desired, increase length (L)

WET DETENTION BASIN DESIGN WORKSHEET
FOR SOLIDS SETTLING MODEL APPROACH

*********************************DESIGN DATA*********************************

Drainage Area	______	(A_D) ac
Location ______ (City, State)		
Rainfall Zone (from figure 19)	______	
Mean Rainfall Event Volume (from table 7)	______	(R_V) ft
Runoff Coefficient		
Pervious Area Coefficient	______	(C_P)
Impervious Area Coefficient	______	(C_I)
Impervious Area/Total Area	______	(X)
Runoff Coefficient ($C = C_P + (C_I - C_P)X$)	______	(C)

*****************************DESIGN PROCEDURE*****************************

Average Runoff Volume ($V_R = CR_VA_D$)		______	(V_R) ac-ft
Surface Area of Permanent Pool		______	ac
Permanent Pool Volume	H = 2 ft	______	(V_{B2}) ac-ft
	H = 4 ft	______	(V_{B4}) ac-ft
	H = 6 ft	______	(V_{B6}) ac-ft
	H = ___ ft	______	($V_{B\_\_}$) ac-ft
Ratio of Permanent Pool Volume to Average Runoff Volume	H = 2 ft	______	(V_{B2}/V_R)
	H = 4 ft	______	(V_{B4}/V_R)
	H = 6 ft	______	(V_{B6}/V_R)
	H = ___ ft	______	($V_{B\_\_}/V_R$)

WET DETENTION BASIN DESIGN WORKSHEET
FOR SOLIDS SETTLING MODEL APPROACH
(CONTINUED)

***********************DESIGN PROCEDURE (Continued)**********************

TSS Removal Efficiency, (from figures 20 to 24)	H = 2 ft	____________	(E_2)	%
	H = 4 ft	____________	(E_4)	%
	H = 6 ft	____________	(E_6)	%
	H = ___ ft	____________	($E_{\_}$)	%
Selected Design Depth of Permanent Pool	H = ___ ft			
Selected Design TSS Removal Efficiency (from figures 20 to 24)		____________	(E_{TSS})	%

Long-term Average Pollutant Removal Efficiencies:

Lead (90% of TSS)	____________	(E_{Pb})	%
Copper (60% of TSS)	____________	(E_{Cu})	%
Zinc (45% of TSS)	____________	(E_{Zn})	%
TKN,BOD (10%-30% of TSS)	____________	($E_{TKN,BOD}$)	%

WET DETENTION BASIN DESIGN WORKSHEET
FOR LAKE EUTROPHICATION MODEL APPROACH

********************************DESIGN DATA********************************

Drainage Area ________________ (A_D) ac

Location ____
(City, State)

Rainfall Zone
(from figure 19) ________________

Mean Rainfall Event Volume ________________ (R_V) ft
(from table 7)

Runoff Coefficient

Pervious Area Coefficient ________________ (C_P)

Impervious Area Coefficient ________________ (C_I)

Impervious Area/Total Area ________________ (X)

Runoff Coefficient ________________ (C)
($C = C_P + (C_I - C_P)X$)

Phosphorus Values

Total Phosphorus Inflow ________________ (P)mg/L
(0.4 mg/L for Urban Highways and or
0.20 mg/L for Rural Highways) ________________ (P)ug/L

Orthophosphorus/Total Phosphorus ________________ (F)
(use 0.25)

******************************DESIGN PROCEDURE******************************

Runoff Volume ________________ (V_R) ac-ft
($V_R = CR_VA_D$)

Surface Area of Permanent Pool ________________ ac

Average Hydraulic Residence Time ________________ (T) yr
(____ wks/52)

Number of Storms ________________ (NS)
(365 x 24/____ interval from table 7)

WET DETENTION BASIN DESIGN WORKSHEET
FOR LAKE EUTROPHICATION MODEL APPROACH
(CONTINUED)

**********************DESIGN PROCEDURE (Continued)*************************

Permanent Pool Volume ($V_B = T \times V_R \times NS$)	________________	(V_B) ac-ft
Mean Depth of Permanent Pool (V_B/A_S)	________________	(Z) ft or
X 0.3048 =	________________	(Z) m
Mean Overflow Rate (Q_S = Z/T)	________________	(Q_S) m/yr
Decay Rate ($K_2 = (0.056)(Q_S)(F)^{-1}/(Q_S+13.3)$)	________________	(K_2)
N Factor ($N = K_2 \times P \times T$)	________________	(N)
Total Phosphorus Removal Efficiency ($R = 1 + (1 - (1+4N)^{0.5})/(2N)$)	________________	(R) or
	________________	%

Additional Pollutant Removal Efficiencies:

Basin Volume/Runoff Volume	________________	(V_B/V_R)
TSS Removal Efficiency (from figures 20 to 24)	________________	(E_{TSS}) %
Lead Removal Efficiency (90% of TSS)	________________	(E_{Pb}) %
Copper Removal Efficiency (60% of TSS)	________________	(E_{Cu}) %
Zinc Removal Efficiency (45% of TSS)	________________	(E_{Zn}) %
TKN, BOD (10%-30% of TSS)	________________	($E_{TKN,BOD}$) %

DRY EXTENDED DETENTION BASIN DESIGN WORKSHEET

*****************************DESIGN DATA***********************************

Drainage Area	______________	(A_D) ac
Location ______________________ (City, State)		
Rainfall Zone (from figure 19)	______________	
Mean Storm Event Volume (from table 7)	______________	(R_V) ft
Runoff Coefficient		
Pervious Area Coefficient	______________	(C_P)
Impervious Area Coefficient	______________	(C_I)
Impervious Area/Total Area	______________	(X)
Runoff Coefficient ($C = C_P + (C_I - C_P)X$)	______________	(C)

*****************************DESIGN PROCEDURE*****************************

Runoff Volume from Mean Storm Event ($V_R = CR_VA_D$)	______________	(V_R) ac-ft
Area of Space Available for Basin	______________	ac
Selected Mean Depth of Extended Detention Pool	______________	ft
Surface Area of Extended Detention Pool	______________	ac
Outlet Structure		
Release Rate	______________	ft^3/sec
Type and Size of Opening	______________	
	______________	

STORAGE REQUIREMENT FOR RETENTION MEASURES

**********************************DESIGN DATA**********************************

Location ____
(City, State)

Rainfall Zone
(from figure 19) ____________________

Mean Rainfall Event Volume
(from table 7) ____________________ (R_V) in

Runoff Coefficient

Pervious Area Coefficient ____________________ (C_p)

Impervious Area Coefficient ____________________ (C_I)

Impervious Area/Total Area ____________________ (X)

Runoff Coefficient
($C = C_p + (C_I + C_p)X$) ____________________ (C)

Saturated Soil Infiltration Rate
(from table 10) ____________________ (I_R) in/hr

Dewatering time
(from table 11) ____________________ (T_d) hrs

*******************************DESIGN PROCEDURE*******************************

Runoff Volume from mean rainfall
event volume
$V_{ro} = CR_V$ ____________________ (V_{ro}) in

Volume Released by Dewatering
$V_d = I_R T_d$ ____________________ (V_d) in

Ratio of V_d/V_{ro} ____________________ V_d/V_{ro}

Fraction Captured
(from figure 37)

For Basin Volume to Runoff
Volume Ratios (Trial Values)

V_b/V_{ro} = ________ E = ______ %

V_b/V_{ro} = ________ E = ______ %

V_b/V_{ro} = ________ E = ______ %

Selected Ratio ________

Design Storage ________ in
$(V_b/V_{ro})(V_{ro})$

RETENTION BASIN DESIGN WORKSHEET: PART A

Site Name and Location ______________________________

Subarea Number ______

------------------------Design Data------------------------

1. Storage required for nonpoint pollution management, Q_u	________	ft
2. Design rainfall volume for nonpoint pollution management, P	________	ft
3. Saturated soil infiltration rate, f	________	ft/hr
4. Allowable dewatering time, T_d	________	hr
5. Depth to bedrock or seasonally high water table, d_{min}	________	ft
6. Required distance between facility bottom and bedrock or seasonally high water table, d_{req}	________	ft
7. Maximum allowable depth based on depth to bedrock or seasonally high water table [5 - 6], d_{max}	________	ft
8. Maximum allowable depth based on infiltration rate and dewatering time [3 x 4], d_{max}	________	ft
9. Final maximum basin depth; i.e., lesser value of [7] and [8] ,d_b	________	ft

RETENTION BASIN DESIGN WORKSHEET: PART B

Site Name and Location ________________________________

Subarea Number _________

BASIN DESIGN

	BASIC PARAMETERS		SELECTED PARAMETERS		COMPUTATION	STORAGE COMPARISON			
Case	Area Controlled A_u (ft^2)	Depth d_b (ft)	Top Width W (ft)	Side Slopes Z (H:V)	Top Length L (ft) [eq. 11]	Storage Required (ft^3) [eq. 12]	Actual Storage (ft^3) [eq. 13]	Meets Min. Reqt.?	Total* Storage (ft^3) [eq. 14]

*Total storage includes the storage of upland runoff and of rainfall incident to the basin surface.

RETENTION TRENCH AND SHALLOW DRY WELL DESIGN WORKSHEET: PART A

Site Name and Location ______________________________

Subarea Number _______

------------------------------Design Data------------------------------

1.	Storage required for nonpoint pollution management, Q_u	________	ft
2.	Saturated soil infiltration rate, f	________	ft/hr
3.	Allowable dewatering time, T_d	________	hr
4.	Void ratio of trench aggregate, V_r	________	
5.	Depth to bedrock or seasonally high water table, d_{min}	________	ft
6.	Required distance between facility bottom and bedrock or seasonally high water table, d_{req}	________	ft
7.	Maximum allowable depth based on depth to bedrock or seasonally high water table [5 - 6], d_{max}	________	ft
8.	Maximum allowable depth based on infiltration rate and dewatering time [2 x 3/4], d_{max}	________	ft
9.	Final maximum trench depth; i.e., lesser value of [7] and [8], d_t	________	ft

RETENTION TRENCH AND SHALLOW DRY WELL DESIGN WORKSHEET: PART B

Site Name and Location ______________________________

Subarea Number ________

TRENCH DESIGN

	BASIC PARAMETERS		SELECTED PARAMETERS		COMPUTATIONS		STORAGE COMPARISON		
Case	Area Controlled A_u (ft^2)	Depth d_t (ft)	Width W_t (ft)	Side Slopes Z (H:V)	Area A_t (ft^2) [eq. 15]	Length L_t (ft) [eq. 16 or 17]	Storage Required (ft^3) [eq. 18]	Actual Storage (ft^3) [eq. 19]	Meets Min. Reqt. ?

*Total storage includes the storage of upland runoff and of rainfall incident to the trench surface.

GLOSSARY

Abiotic process: a chemical or physical process occurring without the aid of living organisms.

Adsorption: the attraction of ions or compounds to the surface of a solid.

Agglomeration: an indiscriminately formed cluster of particles.

Alkaline soils: soils with a relatively high pH, (e.g., pH > 7.0).

Anoxic: lacking oxygen, anaerobic conditions.

Aquifer: earth material containing sufficient groundwater that the water can be pumped out. Highly fractured rocks and unconsolidated sands and gravels make good aquifers.

Biotic process: a process resulting from the actions of living organisms.

Bog: wetlands formed in deep, steep-sided lakes with small watershed areas and poor drainage. High acidity is typical.

Bucket cleaning: use of clamshell bucket to remove materials from a catchbasin.

Catchbasin: a chamber for the admission of stormwater runoff to a sewer or subdrain, having at its base a sediment sump designed to catch grit and sediment.

Cation exchange capacity: the sum total of exchangeable cations that a soil can adsorb.

Cation: a positively charged ion.

Chelation: the formation of complex ions called chelates, the process in which an organic reactant combines with a metal ion to form a cyclic compound in which chains of atoms in the reactant are closed by coordination with unshared pairs of electrons to form a product containing five- or six-membered rings. The resulting compound is usually very stable.

Coordinated chemical reactions: reactions producing a covalent bond consisting of a pair of electrons supplied by only one of the two atoms joined.

Denitrification: removing nitrogen from a material or chemical compound, as by bacterial action in the soil.

Design runoff event: the maximum surface runoff event (e.g., maximum surface runoff flow) that a specified management measure can pass safety, typically expressed by its probability of occurence.

Detention basin: a permanent dam, basin, or excavation to detain water.

Drainage area: the area contributing to the point for which channel capacity is to be determined.

Drop structure: a structure of nonerodible materials used to effect a change in elevation in a very short distance, thereby reducing the effective gradient in an open channel.

Dry detention basin: a normally dry detention basin. A device that temporarily stores water from stormwater runoff events and remains dry between events.

Dry extended detention basin: a detention basin designed to release stored stormwater over an extended period of time and then remain dry between events.

Eductor cleaning: use of a vacuum device in which the vacuum is created by a water pump to remove the solids-water mixture from a catchbasin.

Estuarine: Deep water tidal habitats and adjacent tidal wetlands that are usually semienclosed by land but have access to the open ocean.

Fetch: the extent of open water across which wind blows, generating waves.

Filter course: pervious material used between a porous material and the reservoir course to prevent materials from the reservoir course from clogging the porous pavement; must be less pervious than the pavement and more pervious than the reservoir course; may be of filter cloth, such as woven plastic.

Filtration system: any of a number of devices used to filter suspended materials from stormwater runoff.

Flocculants: a chemical or mix of chemicals added to a dispersion of solids in a liquid to bring together the fine particles to form flocs.

Flow alteration system: any of a number of measures used to reduce the erosive potential of runoff by changing the character of the flow, e.g., from channel flow to sheet flow.

Hydric soil: soil that is wet long enough to periodically produce anaerobic conditions, thus influencing establishment and growth of vegetation.

Hydrophytes: any plant growing in water or on a substrate that is at least periodically deficient in oxygen as a result of excessive water content; plants typically found in wet habitats.

Infiltration: the movement of surface water from the soil/air interface into the soil.

Ion exchange: a chemical reaction in which mobile hydrated ions of a solid are exchanged, equivalent for equivalent, for ions of like charge in solution.

Lacustrine systems: term used to describe wetlands and deep-water habitats with all of the following characteristics: (1) situated in a topographic depression or a dammed river channel; (2) lacking trees, shrubs, persistent emergents, nonaquatic masses or lichens with greater than 30 percent areal coverage; and (3) greater than 20 ac (8ha) in size.

Laminar flow: streamline flow in a fluid occurring near a solid boundary.

Level spreaders: an excavated channel constructed at a zero gradient across a slope to convert channel flow to sheet flow.

Macronutrient: a dissolved inorganic substance required in greater than trace amounts for normal growth by a plant (e.g., Ca, K, Mg, N, P, S).

Marshes: treeless wetlands characterized by shallow water and abundant emergent, floating, and submergent hydrophytes.

Micronutrient: a dissolved inorganic substance required by plants in only trace amounts for normal growth (e.g., B, Cu, Fe, Mn, Mo, Z).

Molecular diffusion: transfer of material by molecular movement under the influence of a concentration gradient.

Overland flow: stormwater runoff flowing over grass as sheet flow.

Palustrine: includes all nontidal wetlands dominated by trees, shrubs, persistent emergents, emergent mosses or lichens, and all such tidal wetlands where salinity from ocean-derived salts is below 0.5 percent.

Percolation: the movement of water within soil or rock.

Persistent emergents: emergent wetland plants that typically remain standing at least until the beginning of the next growing season.

Phytotoxicity: toxic to plant life.

Pocosin: an upland evergreen shrub bog of the coastal plain of the southeastern U.S.

Pollutant: any substance that renders the air, soil, water or other natural resource harmful or unsuitable for its designated purpose.

Pool: the water retained within a detention basin below the level of the outlet structure or spillway. The pool can be temporary (e.g., dries between runoff events from evaporation/infiltration) or permanent (e.g., remains wet between runoff events).

Post-deposition controls: practices used to remove deposits from the highway environment between stormwater runoff events, thereby reducing the potential pollutant loads in runoff.

Post-runoff pollution mitigation: practices used to reduce the pollutant loads and concentrations in runoff; sometimes termed "end-of-pipe" measures.

Potholes: wetlands in depressions or pits, common in prairie regions.

Propagule: a shoot of a plant capable of propagating the plant.

Redox reaction: refers to oxidation-reduction processes in which a substance is said to be oxidized when it loses electrons, while a substance that gains electrons is said to be reduced.

Retardance: the relative hydraulic resistance of grasses; a psuedo-Manning's roughness coefficient used because the hydraulic resistance of grasses varies with species, density, and height.

Retrofitting: developing stormwater management measures for existing highways by designing the measures to fit existing highway situations.

Rhizomatons: plants with rootlike horizontal stems growing under or along the surface of the ground, and sending out roots from its lower surface and leaves or shoots from its upper surface.

Riverine: wetlands and deepwater habitats contained within a channel, not including wetlands dominated by trees or persistent emergents, or habitats containing 0.5% salt.

Senescence: deterioration which occurs with time; reduced effectiveness over time, aging.

Short-circuiting: the failure of influent to thoroughly mix with water in a basin or wetland before discharge through the outlet structure.

Solute: a substance dissolved in another, the solvent.

Sorption: the process of sorbing, of one substance being taken up and becoming attached to another substance.

Sprigging: establishing new plants by planting sprigs, e.g., small shoots or twigs.

Stoloniferous: bearing stolons, or stems growing along or under the ground and taking root at the nodes or apex to form new plants.

Stone riprap: broken rock, cobbles, or boulders placed on earth surfaces for erosion control.

Subsidence: a sinking, settling, or otherwise lowering of an area.

Swale: an open channel conveyance structure, usually shallow with gentle side slopes and longitudinal gradient.

Swamp: forested wetland with a shallow water table.

Vacuum cleaning: use of equipment that creates a partial vacuum to remove materials accumulated in a catchbasin.

Vacuum sweeper: equipment that utilizes a partial vacuum to remove deposits from the highway pavement.

Water table: the upper limit of the ground saturated with water.

Wet detention basin: a stormwater runoff storage device in which a "permanent" pool is maintained by placing the outlet structure above the bottom of the basin.

Wet meadow: low, level, moist wetland composed primarily of grasses and sedges.

Wetted perimeter: the area within an infiltration basin in which percolation and infiltration occurs.

REFERENCES

ABAG, "Treatment of Stormwater Runoff by a Marsh/Flood Basin," Association of Bay Area Governments, Interim Report to U.S. Environmental Protection Agency, August 1979.

Adamus, Paul, and L. T. Stockwell, "A Method for Wetland Functional Assessment: Volume I, Critical Review and Evaluation Concepts," Report No. FHWA-IP-82-23, Center for Natural Areas, Department of Transportation, Federal Highway Administration, Washington, DC, March 1983.

Adamus, P.R., E.J. Clairain, R.D. Smith, and R.E. Young. Wetland Evaluation Technique (WET) Volume II: Methodology Operational Draft. Federal Highway Administration Report No. FHWA-IP-88-029. 206 pp. 1987.

American Public Works Association, "Urban Stormwater Management," Special Report No. 49, Chicago, IL, 1981.

Amimoto, Perry Y., "Erosion and Sediment Control Handbook," California Department of Conservation, Report No. EPA 440/3-78-003, U.S. Environmental Protection Agency, Water Planning Division, Washington, DC, May 1978.

Aronson, G. L., D. S. Watson, and W. C. Pisano, "Evaluation of Catchbasin Performance for Urban Stormwater Pollution Control," Report No. EPA-600/2-83-643, U.S. Environmental Protection Agency, Office of Research and Development, Municipal Environmental Research Laboratory, Cincinnati, OH, June 1983.

Becker, Burton C., Michael L. Clar, and Robert R. Kautzman, "Approaches to Stormwater Management," U.S. Department of Interior, Office of Water Resources Research, Washington, DC, November 1973.

Bell, John H., and Martin P. Wanielista, "Stormwater Management Using Overland Flow on Interstate Highways," Fifty-eighth Annual Meeting of the Transportation Research Board, Washington, DC, January 15, 1979.

Bertram, P. E., and J. L. Kaster, "Biological Assays of Highway Runoff Water," Draft, Federal Highway Administration, Washington, DC, April 1982.

Boto, K. G., and William H. Patrick, Jr., "Role of Wetlands in the Removal of Suspended Sediments," National Symposium on Wetlands, Lake Buena Vista, FL, American Water Resources Association, November 7 to 10, 1978.

Boyt F. L., S. E. Bayley, and J. Zoltek, Jr., "Removal of Nutrients from Treated Municipal Wastewater by Wetland Vegetation," Journal WPCF, May 1977.

Breeding, N. K., Jr., and J. W. Dawson, "Pros and Cons of Storm Water Recharge Wells," Water and Sewage Works, February 1977.

Broome, S. W., W. W. Woodhouse, and E. D. Seneca, "The Relationship of Mineral Nutrients to Growth of Spartina alterniflora in North Carolina," in Nutrient Status of Plants and Soils in Natural Stands, Soil Science Society of America 39(2): 295, 1975.

Bursztynsky, Terry A., "Wetlands and Pollutant Treatment Mechanisms," Technical Memorandum No. 69, Association of Bay Area Governments, Berkeley, CA, September 1981.

Carlstrom, Scott, "Use of Detention Basins and Wetlands to Treat Urban Runoff," Minnesota Department of Transportation, Office of Environmental Services, December 1981.

Chan, Emy, T. A. Bursztynsky, Norman Hantzche, and Yoram J. Litwin, "The Use of Wetlands for Water Pollution Control," U.S. Environmental Protection Agency, Municipal Environmental Research Laboratory, Cincinnati, OH, November 1981.

Chan, Emy, Gary Silverman, and Terry Bursztynsky, "San Francisco Bay Area Regional Wetlands Plan for Urban Runoff Treatment-Volume I Plan and Amendments to the Environmental Management Plan," Draft report, Association of Bay Area Governments, Berkeley, CA, December 1982.

Cowardin, L. W., V. Carter, F. C. Golet, and E. T. LaRoe, "Classification of Wetlands and Deepwater Habitats of the United States," U. S. Department of Interior, Fish and Wildlife Service, Office of Biology Service, Washington, D. C. #FWS/OBS-79/31.

Cramer, George H., II, "A Mitigation Technique for Highway Runoff," Louisiana Department of Transportation and Development, Baton Rouge, LA, undated.

Day, Gary E., "Investigation of Concrete Grid Pavements," in Stormwater Management Alternatives, Water Resources Center, University of Delaware, 1980.

Day, Gary E., David R. Smith, and John Bowers, "Runoff and Pollution Abatement Characteristics of Concrete Grid Pavements," Office of Water Research and Technology, Washington, DC, October 1981.

DiToro, D. M. and M. J. Small, "Stormwater Interception and Storage," Journal of Environmental Engineering Division, ASCv, Vol. 105, No. EE1, February 1979, pp. 43-54.

Donigian, A.S., and Crawford, N.H., "Modeling Nonpoint Pollution from the Land Surface, "EPA-600/3-76-083, U.S. Environmental Protection Agency, Environmental Research Laboratory, Athens, GA, July 1976.

Dorman, M.E.; J.P. Hartigan, J. Johnson and B. Maestri, "Retention, Detention and Overland Flow for Pollutant Removal from Highway Stormwater Runoff: Interim Guidelines for Management Measures," Publication No. FHWA/RD-87/056, Federal Highway Administration, McLean, Virginia, 1987.

Driscoll, E.D., "Lognormality of Point and Nonpoint Source Pollutant Concentrations" (in) Urban Runoff Quality: Impact and Quality Enhancement Technology. B. Urbonas and L. A. Roesner, eds. American Society of Civil Engineers (ASCE). New York, NY, 1986. pp. 158-459.

Driscoll, Eugene D., "Performance of Detention Basins for Control of Urban Runoff Quality," 1983 International Symposium on Urban Hydrology, Hydraulics, and Sediment Control, University of Kentucky, 1983.

Driscoll, Eugene D., Personal communication with Frank Johnson, letter dated March 11, 1985.

Dupuis, T. V., J. Kaster, P. Bertram, J. Meyer, M. Smith, and N. Kobriger, "Effects of Highway Runoff on Receiving Waters, Volume II. Results of Field Monitoring Program," Draft, FHWA/RD-84/063, Federal Highway Administration, Washington, DC, November 1984.

Dupuis, T. V., and N. P. Kobriger, "Effects of Highway Runoff on Receiving Waters, Volume IV. Procedural Guidelines for Environmental Assessments", Draft report, Report No. FHWA/RD-84/065, Federal Highway Administration, Washington, DC, January 1985.

Driver, Charles H., Bjorn F. Hrutfiord, Demetrios E. Spyridakis, Eugene B. Welch, and David D. Wooldridge, "Assessment of the Effectiveness and Effects of Land Disposal Methodologies of Waste Water Management," U.S. Army Corps of Engineers, Office of the Chief of Engineers, Washington, DC, January 14, 1972.

Environmental Laboratory, "Wetland Habitat Development with Dredged Material. Engineering and Plant Propagation," Technical Report DS-78-16, U.S. Engineer Waterways Experiment Station, Vicksburg, MS, December 1978.

Erickson, Paul, George Camougis, and Norman Miner, "Highways and Wetlands, Volume II, Impact Assessment, Mitigation, and Enhancement Measures," Report No. FHWA-IP-80-11, U.S. Department of Transportation, Federal Highway Administration, Washington, DC, July 1980.

Ettinger, Charles E., "Development of Methods to Improve Performance of Surface Mine Sediment Basins," Report No. EPA-600/7-80-072, U.S. Environmental Protection Agency, Office of Research and Development, Cincinnati, OH, April 1980.

Fair, G. M., and J. C. Geyer, Water Supply and Waste Water Disposal, John Wiley and Sons, 1954.

FHWA, "Volume 1, Design Procedures for Predicting Pollutant Loads and Receiving Water Quality Impacts," FHWA/RD-87/085, Federal Highway Administration, Washington, DC, 1987.

Federal Interagency Committee for Wetland Delineation, "Federal Manual for Identifying and Delineating Jurisdictional Wetlands," U.S. Army Corps of Engineers, U.S. Environmental Protection Agency, U.S. Fish and Wildlife Service, and U.S.D.A. Soil Conservation Service, Washington D.C., cooperative technical publication, 1989.

Fenn, D., et al., "Procedures Manual for Ground Water Monitoring at Solid Waste Disposal Facilities," Report No. EPA/530/SW-611, U.S. Environmental Protection Agency, Washington, DC, 1977.

Field, Richard, Hugh Masters, and Melvin Singer, "Porous Pavement: Research; Development; and Demonstration," Transportation Engineering Journal of ASCE, Vol. 108, No. TE3, May 1982.

Fitzsimmons, D. W., J. R. Busch, R. B. Long, D. H. Lindeborg, and G. M. McMaster, "Evaluation of Measures for Controlling Sediment and Nutrient Losses from Irrigated Areas," Report No. EPA/600/2-78/138, Environmental Research Lab, Source Management Branch, Ada, OK, July 1978.

Florida Department of Transportation, "Stormwater Management Manual," Tallahassee, FL, 1982.

Garbisch, Edgar N., "Highways and Wetlands, Compensating Wetland Losses," Final report, FHWA-IP-86-22, U.S. Department of Transportation, Federal Highway Administration, McLean, VA, 1986.

Goforth, G. F., J. P. Heaney, and W. C. Huber, "Comparison of Basin Performance Modeling Techniques, "Journal of Environmental Engineering Division, ASCE, Vol. 109, No. 5, October 1983, pp. 1082-1098.

Grizzard, T. J., C. W. Randall, B.L. Weand and K. L. Ellis, "Lognormality of Point and Nonpoint Source Pollutant Concentrations" (in) Urban Runoff Quality: Impact and Quality Enhancement Technology. B. Urbonas and L. A. Roesner, eds. American Society of Civil Engineers (ASCE). New York, NY, 1986. pp. 158-459.

Gupta, M. K., "Constituents of Highway Runoff Volume VI-Executive Summary," Report No. FHWA/RD-81/047, Federal Highway Administration, Washington, DC, 1981.

Gupta, M. K., R. W. Agnew, D. Gruber, and W. Kreutzberger, "Constituents of Highway Runoff Volume IV, Characteristics of Highway Runoff from Operating Highways," Report No. FHWA/RD-81/045, Federal Highway Administration, Washington, DC, 1981.

Gupta, M. K., R. W. Agnew, and T. L. Meinholz, "Constituents of Highway Runoff Volume II, Procedural Manual for Monitoring of Highway Runoff," Report No. FHWA/RD-81/043, Federal Highway Administration, Washington, DC, 1981.

Gupta, M. K., R. W. Agnew, and N. P. Kobriger, "Constituents of Highway Runoff Volume I, State-of-the-Art Report," Report No. FHWA/RD-81/042, Federal Highway Administration, Washington, DC, 1981.

Hammer, David E., and Robert H. Kadlec, "Design Principles for Wetland Treatment Systems," Report No. EPA-600/52-83-026, U.S. Environmental Protection Agency, Environmental Research Laboratory, Ada, OK, May 1983.

Hannon, Joseph B., "Underground Disposal of Storm Water Runoff,"Design Guidelines Manual, Report No. FHWA-TS-80-218, Federal Highway Administration, Washington, DC, February 1980.

Hartigan, J. P., "Basis for Design of Wet Detention Basin BMP's," ASCE/ Engineering Foundation Conference on Current Practice and Design Criteria for Urban Runoff Water Quality Control, Potosi, MO, 1988.

Heaney, James P., and Richard H. Sullivan, "Source Control of Urban Water Pollution," Journal Water Pollution Control Federation, Vol. 43, No. 3, April 1979, pp. 571 to 579.

Heaney, James P., and Stephen J. Nix, "Storm Water Management Model: Level I--Comparative Evaluation of Storage-Treatment and Other Management Practices," Report No. EPA-600/2-77-083, U.S. Environmental Protection Agency, Cincinnati, OH, April 1977.

Hickok, Eugene A., Marcus C. Hannaman, and Norman C. Welch, "Urban Runoff Treatment Methods Volume I, Non-Structural Wetland Treatment," Report No. EPA-600/2-77-217, U.S. Environmental Protection Agency, Cincinnati, OH, December 1977.

Hittman Associates Inc., "Processes, Procedures, and Methods to Control Pollution Resulting from All Construction Activity," Report No. EPA 430/9-73-007, U.S. Environmental Protection Agency, Office of Air and Water Programs, Washington, DC, October 1973.

Huber, Wayne C., and Robert E. Dickinson, "Storm Water Management Model, Version 4: User's Manual," U.S. Environmental Protection Agency, Environmental Research Laboratory, Athens, Georgia, August 1988.

Hydrologic Engineering Center, "Urban Storm Water Runoff, STORM," U.S. Army Corps of Engineers, Davis, California, 1977.

Hydroscience, Inc., "Water Quality Management Planning Methodology for Urban and Industrial Stormwater Needs," prepared for the Texas Water Quality Board, December 1976.

Jackura, Kenneth A., "Infiltration Drainage of Highway Surface Water," Report No. FHWA/CA/TL-80/04, California Department of Transportation, Office of Transportation Laboratory, Sacramento, CA, July 1980.

Jens, Stifel, W., "Design of Urban Highway Drainage, The State of the Art," Report No. FHWA-TS-79-225, Federal Highway Administration, Washington, DC, August 1979.

JBF, "An Analysis of the Functional Capabilities and Performance of Silt Curtains," T.R. D-78-39, Office, Chief of Engineers, U.S. Army, Washington, DC, 1978.

Kadlec, Robert H., and John A. Kadlec, "Wetlands and Water Quality," Proceedings of the National Symposium on Wetlands, Lake Buena Vista, FL, November 7 to 10, 1978.

Kamedulski, Gregory E., and Richard H. McCuen, "The Effect of Maintenance on Storm Water Detention Basin Efficiency," Water Resources Bulletin, Vol. 15, No. 4, August 1979, pp. 1146 to 1152.

Kathuria, D. Vir, Michael A. Nawrocki, and Burton C. Becker, "Effectiveness of Surface Mine Sedimentation Ponds," Report No. EPA-600/2-76-117, U.S. Environmental Protection Agency, Cincinnati, OH, August 1976.

Kobriger, N. P., M. K. Gupta, and A. Geinopolos, "Sources and Migration of Highway Runoff Pollutants," Report No. FHWA/RD-821, Federal Highway Administration, Washington, DC, October 1982.

Kobriger, N. P., T. L. Meinholz, M. K. Gupta, and R. W. Agnew, "Constituents of Highway Runoff Volume III, Predictive Procedure for Determining Pollution Characteristics in Highway Runoff," Report No. FHWA/RD-81/044, Federal Highway Administration, Washington, DC, 1981.

Kobriger, N. P., and T. M. Krischan, "Constituents of Highway Runoff Volume V, Highway Runoff Data Storage Program and Computer User's Manual," Report No. FHWA/RD-81/046, Federal Highway Administration, Washington, DC, 1981.

Kussy, Edward F. A., "Wetland and Floodplain Protection and the Federal-Aid Highway Program," 13 Environmental Law, Lewis and Clark Law School, Portland, OR, 1982.

Kutash, W., "Effectiveness of Wetland Stormwater Treatment, Some Examples," Stormwater Management: An Update, University of Central Florida, Environmental Systems Engineering Institute, Orlando, Florida, July 1985, pp. 145-151.

Lager, John A., William G. Smith, and George Tchobanoglous, "Catchbasin Technology Overview and Assessment," Report No. EPA-660/2-77-051, U.S. Environmental Protection Agency, Cincinnati, OH, May 1977.

Lee, C. R., and R. E. Peters, "Overland Flow Treatment of a Municipal Lagoon Effluent for Reduction of Nitrogen, Phosphorus, Heavy Metals, and Coliforms," Prog. Wat. Tech., Vol. II, Nos. 4/5, 1979, pp. 175 to 183.

Litwin, Y. J., and T. Bursztynsky, "Treatment of Stormwater Runoff by a Marsh/Flood Basin," Interim Report, Association of Bay Area Governments, Berkeley, CA, August 1979.

Lynard, William G., John Finnemore, Joseph A. Loop, and Robert M. Finn, "Urban Stormwater Management and Technology: Case Histories," Report No. EPA 600/8-80-035, U.S. Environmental Protection Agency, Cincinnati, OH, August 1980.

Mar, Brian W., Richard B. Horner, John F. Ferguson, Dimitris E. Spyridakis, and Eugene B. Welch, "Highway Runoff Water Quality," Draft final report, Washington State Department of Transportation, Seattle, WA, March 1982.

Maryland Department of Natural Resources, "Guidelines for Constructing Wetland Stormwater Basins," Sediment and Stormwater Division, Water Resources Administration, Annapolis, Maryland, March 1987.

Maryland Water Resources Administration (WRA), "Standards and Specifications for Infiltration Practices, "Maryland Department of Natural Resources, Annapolis, Maryland, February 1984.

Masch, Frank D., "Hydrology," Report No. FHWA IP-84-15, Hydraulic Engineering Circular No. 19, Federal Highway Administration, Washington, DC, October 1984.

Metropolitan Washington Council of Governments (MWCOG), "Urban Runoff in the Washington Metropolitan Area - Final Report Washington, DC Area Urban Runoff Project," prepared for U.S. EPA and WRPB, 1983.

Metropolitan Washington Council of Governments, "Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs," July 1987.

Mills, W.B., et al., "Water Quality Assessment: A Screening Procedure for Toxic and Conventional Pollutants," EPA-600/6-82-004, U.S. EPA, Environmental Research Laboratory, Athens, Georgia, 1982.

Morris, F. A., M. K. Morris, T. S. Michaud, and L. R. Williams, "Meadowland Natural Treatment Process in the Lake Tahoe Basin: A Field Investigation," Report No. EPA-600/4-81-026, U.S. Environmental Protection Agency, Las Vegas, NV, April 1981.

Nightingale, H. J., "Lead, Zinc, and Copper in Soils of Urban Storm-Runoff Retention Basin," American Water Works Association Journal, 67:443-446, 1975.

Norman, J. M., "Design of Stable Channels with Flexible Linings," Hydraulic Engineering Circular No. 15, Federal Highway Administration, Washington, DC, October 1975.

Northern Virginia Planning District Commission (NVPDC), "Guidebook for Screening Urban Nonpoint Pollution Management Strategies," prepared for Metropolitan Washington Council of Governments, 1979.

O'Brien, A. L, and W. S. Motts, "Hydrogeologic Evaluation of Wetland Basins for Land Use Planning," Water Resources Bulletin, 1980, pp. 785 to 789.

Occoquan Watershed Monitoring Laboratory (OWML), "Final Contract Report: Washington Area NURP Project," prepared for Metropolitan Washington Council of Governments, 1983.

Occoquan Watershed Monitoring Laboratory (OWML), "Final Report London Commons Extended Detention Facility Urban BMP Research and Demonstration Project," prepared for Northern Virginia Soil and Water Conservation District, December 1987.

Pitt, Robert, "Demonstration of Non-point Pollution Abatement Through Improved Street Cleaning Practices," Report No. EPA-600/2-79-161, U.S. Environmental Protection Agency, Cincinnati, OH, August 1979.

Poland, Claudius, "Grassed Waterway Design," in U.S. Environmental Protection Agency "Non-point Source Pollution Seminar Section 108(a) Demonstration Projects Progress Reports," Report No. EPA 905/9-75-007, U.S. Environmental Protection Agency, Chicago, IL, 1975.

Portele, G. J., B. W. Mar, R. R. Horner, and E. B. Welch, "Effects of Seattle Area Highway Stormwater Runoff on Aquatic Biota," WA-RD-39.11, Washington State Department of Transportation, January 1982.

Reed, P.B., Jr., "National List of Plant Species that Occur in Wetlands: National Summary," Biological Report 88 (24), U.S. Fish and Wildlife Service.

Richards, William G., John E. Shwop, and Roy Romano, "Evaluation of Urban Stormwater Quality and Non-Structural Best Management Practices," Symposium on Non-point Pollution Control—-Tools and Techniques for the Future, Office of Water Research and Technology, Washington, DC, January 1981, pp. 82 to 99.

Rivers, Grover E., and Charles J. Allen, "Silt Barriers as Erosion Pollution Control in a Large Recreational Lake," Transportation Research Record 551, 54th annual meeting of the Transportation Research Board, National Research Council, Washington, DC, 1975.

Rosenbaum, Nelson, "Statutory Stringency and Policy Implementation: The Case of Wetlands Regulation," Policy Implementation Workshop, Pomona College, Claremont, CA, November 1978.

Rossiter, J. A. and R. D. Crawford, "Evaluation of Artificial Wetlands in North Dakota: Recommendations for Future Design and Construction," Wetlands, Floodplains Erosion and Storm Water Pumping, PB85-102291, Transportation Research Board, Washington, D.C., 1983.

Sartor, James D., and Gail B. Boyd, "Water Pollution Aspects of Street Surface Contaminants," Report No. EPA-R2-72-081, U.S. Environmental Protection Agency, Washington, DC, November 1972.

Soil Convervation Service, "Urban Hydrology for Small Watersheds," Technical Release No. 55, U.S. Department of Agriculture, Washington, D.C., June 1986.

Stenstrom, Michael K., Gary Silverman, and Taras A. Bursztynsky, "Oil and Grease in Stormwater Runoff," Association of Bay Area Governments, Berkeley, CA, February 1982.

Stewart, B.A., D.A. Wollhiser, J.H. Wischmeier, Caro, and M.H. Frere, "Control of Pollution from Cropland," U.S. EPA Report No. 600/2-75-026 or USDA Report No. ARS-H-5-1, Washington, D.C., 1975.

Taylor J. S., P. Moore, V. Godlewski, and B. A. Snyder, "Coagulation and Detention of Stormwater Runoff," Annual Meeting of the American Society of Civil Engineers, October 27 to 31, 1980, Hollywood, Florida, University of Central Florida, 1980.

Thelen, Edmund, Wilford C. Grover, Arnold J. Hoiberg, and Thomas I. Haigh, "Investigation of Porous Pavements for Urban Runoff Control," Project No. 11034 DUY, U.S. Environmental Protection Agency, Washington, DC, March 1972.

Thronson, Robert E., "Comparative Costs of Erosion and Sediment Control, Construction Activities," Report No. EPA-430/9-73-016, U.S. Environmental Protection Agency, Washington, DC, July 1973.

Tourbier, J. Toby (ed.), and Richard Westmacott (ed.), "Stormwater Management Alternatives," University of Delaware, Water Resources Center, Newark, DE, April 1980.

Transportation Research Board, "Design of Sedimentation Basins," NCHRP Synthesis 70, National Research Council, Washington, DC, June 1980.

University of California, Davis, Department of Civil Engineering, "The Use and Potential of Aquatic Species for Wastewater Treatment - Appendix A: The Environmental Requirements of Emergent and Submerged Aquatic Plants," Publication No. 65, California State Water Resources Control Board, 1980.
U.S. Department of Agriculture, "National Engineering Handbook, Hydraulics," U.S. Department of Agriculture, Soil Conservation Service, Washington, DC, 1975.

U.S. Department of Agriculture, Soil Conservation Service, "Technical Release 55: Urban Hydrology for Small Watersheds," Washington, D.C., June 1986.

U.S. Department of Commerce, Weather Bureau, "Technical Paper No. 40: Rainfall Frequency Atlas of the United States," Washington, D.C., May 1961.

U.S. Department of Transportation, "Design of Urban Highway Drainage, The State of the Art," Report No. FHWA-TS-79-225, Department of Transportation, Federal Highway Administration, Washington, DC, 1979.

U.S. Environmental Protection Agency, "Fate of Priority Pollutants in Publicly Owned Treatment Works," EPA 440/1-82/303, Effluent Guidelines Divison, Washington, D.C., September 1982.

U.S. Environmental Protection Agency, "Results of the Nationwide Urban Runoff Program: Volume I: Final Report," Water Planning Division, Washington, D.C., December 1983.

U.S. Environmental Protection Agency, Region IV, "Freshwater Wetlands for Wastewater Management: Environmental Assessment Handbook," prepared by CTA Environmental, Inc., and Gannett Fleming Corddry and Carpenter, Inc., September 1985.

Van der Valk, Arnold G., Craig Davis, James Baker, and Craig Beer, "Natural Fresh Water Wetlands as Nitrogen and Phosphorus Traps for Land Runoff," National Symposium on Wetlands, Lake Buena Vista, FL, November 7 to 10, 1978.

Virginia State Water Control Board, "Best Management Practices Handbook, Hydrologic Modifications," Planning Bulletin 319, Richmond, VA, 1979.

Virginia Soil and Water Conservation Commission, "Virginia Erosion and Sediment Control Handbook," Second Edition, Virginia Soil and Water Conservation Commission, Richmond, VA, 1980.

Walker, W.W., "Phosphorus Removal by Urban Runoff Detention Basins," Lake and Reservoir Management: Volume III, North American Lake Management Society, Washington, D.C., 1987, pp. 314-326.

Wang et al., Transport Deposition and Control of Heavy Metals in Highway Runoff, prepared for the Washington State Department of Transportation, January 1981.

Wang, Tzn-Siang, Dimitris E. Spyridakis, Brian W. Mar, and Richard R Horner, "Transport, Deposition and Control of Heavy Metals in Highway Runoff," Washington State Department of Transportation, Seattle, WA, January 1981.

Wanielista, Martin P., Robert N. Gennaro, John H. Bell, and Jesse W. Johnson, "Shallow-Water Roadside Ditches for Stormwater Purification," Florida Department of Transportation, March 1978.

Weaver, Robert J., "Recharge Basins for Disposal of Highway Storm Drainage," Research Report 69-2, New York State Department of Transportation, Albany, NY, May 1971.

Westerman, Philip W., Michael R. Overcash, and Samuel C. Bingham, "Reducing Runoff Pollution Using Vegetated Borderland for Manure Application Sites," Report No. EPA-600-52-83-022, U.S. Environmental Protection Agency, Ada, OK.

Whigham, Dennis F., and Suzanne E. Bayley, "Nutrient Dynamics in Freshwater Wetlands," National Symposium on Wetlands, Lake Buena Vista, FL, November 1978.

Whipple, William Jr., and Joseph V. Hunter, "Settleability of Urban Runoff Pollution," Journal WPCF, Vol. 53, No. 12, December 1981, pp. 1726 to 1731.

Wigington, P. W., Randall, C. W., Grizzard, T. J., "Accumulation of Selected Trace Metals in Soils of Urban Runoff Detention Basins," Water Resources Bulletin, American Water Resources Association, Vol. 19, No. 5, October 1983.

Winters, G. R., and J. L. Gidley, "Effects of Roadway Runoff on Algae," Report No. FHWA/CA/TL-80/24, Federal Highway Administration, Washington, DC, 1980.

Young, R.A., Terry Huntrods, and Wayne Anderson, "Effectiveness of Vegetated Buffer Strips in Controlling Pollution from Feedlot Runoff," Journal of Environmental Quality, Vol. 9, No. 3, 1980, pp. 483 to 487.

Yousef, Y, et al., Best Management Practices: Removal of Highway Contaminants by Roadside Swales, prepared for Florida Department of Highway Transportation, July 1985.

Yousef Y, et al., "Design and Effectiveness of Urban Retention Basins," Proceedings of an Engineering Foundation Conference, Henniker, NH, June 23-27, 1986, Ed. Ben Urbonas and Larry Roesner.

Yousef, Y.A., et al., "Fate of Pollutants in Retention/Detention Ponds," Stormwater Management: An Update, Publication #85-1, University of Central Florida, Environmental Systems Engineering Institute, Orlando, Florida, July 1985, pp. 259-275.

Yu, S.L., Norris, W.K., and Wyant, D.C., "Urban BMP Demonstration Project in the Albemarle/Charlottesville Area," submitted to the Virginia Department of Conservation and Historic Resources, Division of Soil and Water Conservation, December 1987.