Town of Darien

Stormwater Management and Drainage Manual

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Stormwater Management and Drainage Manual

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Section 1 Introduction

1.1 Purpose

This manual establishes guidelines for the design and maintenance of drainage facilities within the Town of Darien. It presents guidelines for the best available criteria for drainage design and stormwater management. The Connecticut Department of Transportation Drainage Manual served as a basis for this Darien manual.

The objectives of good drainage are to:

1. Provide for the safety of the Darien general public, property owners, and those adjacent property owners.

2. Eliminate or minimize undesirable upstream or downstream flooding effects which may result from the construction and maintenance of the drainage facilities.

3. Eliminate or minimize nonpoint source pollution to improve the quality of stormwater sewer system discharges.

4. Balance the risk of flood damage related to the drainage facility and the cost of its construction.

5. Produce reliable and long-lasting drainage structures and facilities, capable of being satisfactorily operated and maintained.

6. Incorporate Best Management Practices (BMPs) to help offset the impacts of development through the proper management of pollutant sources and stormwater runoff.

It is the responsibility of the design engineer to develop the required hydrologic and hydraulic data for a project. Field investigations, data collection, data analysis and sound judgement are fundamentals of good drainage design and shall be used by the engineer while employing the criteria set forth in this manual.

A complete drainage report documenting the basis for the hydrology and hydraulic design is a requirement for any work to be reviewed for approval by the Darien Department of Public Works.

The criteria set forth herein shall be used in the design of drainage facilities generally encountered in Public Works projects carried out by the Department of Public Works, as well as in subdivisions, site developments and other projects over which the Town Health, Planning and Zoning, Environmental Protection Commission and/or other Commissions have a responsibility for review and approval. Projects over which the Connecticut Department of Transportation has responsibility for review and approval shall be designed in accordance with their Drainage Manual.

Because urban stormwater quantity and quality management is a relatively new practice, design guidelines will likely change in the future as new technologies are developed based on information obtained from long-term monitoring data, and changes that may occur in the regulatory requirements. The Town will strive to update this manual to allow for increased flexibility and efficiency in stormwater management.

The Darien Department of Public Works welcomes constructive suggestions or comments that would improve the contents of this manual.

1.2 Abbreviations

AHW	allowable headwater
BMP	Best Management Practice
CB	Catch Basin
DEP	Connecticut Department of Environmental Protection
DOT	Connecticut Department of Transportation
DPW	Darien Department of Public Works
CGS	Connecticut General Statutes
CN	SCS Method Curve Number
EGL	energy grade line
EPA	Environmental Protection Agency
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
HEC	United States Army Corps of Engineers Hydrologic Engineering Center
HGL	hydraulic grade line
MH	Manhole
MS4	Municipal Separate Stormwater Sewer System
NPDES	Nonpoint Discharge Elimination System
NEH	NRCS National Engineering Handbook
NRCS	Natural Resources Conservation Service (formerly SCS)
SCS	Soil Conservation Service (currently NRCS)
TSS	Total Suspended Solids
USDA	United States Department of Agriculture

1.3 Symbols

A	area
A_f	filter bed surface area
A_s	sedimentation basin surface area
C	land use coefficient
C_{FS}	safety factor for clogging
d	depth of water
d_c	critical depth
d_{max}	maximum depth of trench
d_{mid}	midpoint depth
d_r	filter bed depth
E'	sediment trapping efficiency constant
f	soil infiltration rate
G	width of grate
I	rainfall intensity
h_f	height of water above filter bed
H_w	wave height
K	relative manhole coefficient
k	permeability
L	flow length or length of reservoir
l	length of swale
n	Manning's roughness coefficient
Q	flow rate
Q_i	inflow rate
Q_o	discharge rate
Q_p	peak discharge
Q_v	treatment volume
P	perimeter of grate opening
P_2	rainfall depth, 2 year storm
p_w	wetted perimeter
R_{BMP}	total suspended solids removal rate of practice number n
R_n	radius, pipe n
r	hydraulic radius
RE_v	Recharge Volume
S	slope
S_c	critical slope
T	spread, width of flow (for gutter flow calculations)
T_c	time of concentration
t_f	filter bed drain time
T_m	travel time, segment m
T_s	maximum storage time
T_t	travel time
TSS_{inflow}	percentage of total suspended solids in inflow to practice number n

TSS_{prev}	cumulative percentage otal of suspended solids removed by previous practices
$TSS_{removed\ n}$	percentage of total suspended solids removed after practice number n
V	average velocity
V_p	volume accommodated by pretreatment
W	particle settling velocity
w_b	bottom width of swale or reservoir
w_{mid}	midpoint width
WQ_v	Water Quality Volume
z	horizontal ratio of sideslope for each unit of vertical rise
Δ	angle between pipe centerlines
η	void ratio of stone

Section 2 Design Practices

2.1 Hydrologic Analysis

Present formulas and models for estimating flood flows are based on statistical analyses of rainfall and runoff records. The recommended practice is for the engineer to select appropriate hydrologic estimating procedures, and obtain runoff data where available for purposes of evaluation, calibration and determination of the predicted values for the desired storm frequencies. The predicted value of the flood flows represents the engineer's best estimate, with varying degrees of error. The expected magnitude of this variation can be determined for some formulas or models as a part of the hydrologic design procedure.

2.2 Hydraulic Analysis

The next step in the design process is the selection of alternative designs that meet the site conditions and accommodate the design flows selected for analysis. The hydraulic analysis is made utilizing appropriate formulas, physical models or computer programs to define, calibrate and check the performance of the designs over a range of flows.

2.3 Stormwater Management Analysis

Changes in land use resulting from urbanization increases impervious surfaces, affecting the area hydrology and pollutant loading in stormwater runoff. Stormwater management design includes the selection, sizing, design and location of Best Management Practices (BMPs) to remove pollutants. The design and analysis of one or more of the particular BMP is made utilizing accepted formulas, or computer models.

2.4 Engineering Evaluation

The final step in the design process is the engineering evaluation of the preliminary designs and approval of the selected final design. This process involves consideration and balancing of number of factors. Some of these factors are:

- legal considerations
- flood hazards to neighboring property owners
- hydraulic efficiencies
- environmental and water quality
- other site specific concerns

2.5 Standard Practice

Hydrologic analysis, hydraulic analysis and stormwater management analysis set forth the design process representative of the present standard engineering practice. Engineering evaluation outlines the approach to be followed by a reasonably competent and prudent engineer in evaluating, selecting and approving a final design. The following considerations are made in regard to this design process.

- It is the engineer's responsibility to provide an adequate drainage design. The engineer is not required to provide a drainage system that will handle all conceivable flood flows under all possible site conditions.

- The detail of design should be commensurate with the risk associated with the development and with other economic, engineering, social, or environmental concerns.

- The overtopping and/or design floods may serve as criteria for evaluating the adequacy of a proposed design. The "overtopping flood" is the smallest recurrence interval flood that will result in flow over the neighboring property, roadway or other watershed boundary. The "design flood" is the recurrence interval of the flood for which the drainage system is sized. The overtopping flood and the design flood may vary widely depending on the characteristics of the watercourse and floodplain.

- The predicted value of the 100-year or base flood serves as the present engineering standard for evaluating flood hazards and as the basis for regulating floodplains under the National Flood Insurance Program. The engineer must make a professional judgement as to the degree of risk that is tolerable for the base flood on a case-by-case basis.

- The developed hydraulic performance curve of a drainage structure depicts the relationship between floodwater stage (or elevation) and flood flow and frequencies. With the performance curve, the engineer can evaluate the adequacy of the design for a range of flows and take into consideration variations caused by the hydrologic estimating procedure. It is standard engineering practice to use the predicted value of the 100-year flood as the basis for evaluating flood hazards for drainage structures which convey a watercourse from a watershed greater than 1 square mile; however, flows larger than this value may be considered for complex, high risk or unusual cases that require special studies or risk analyses.

2.6 Documentation

Frequently, it is necessary to refer to plans, specifications and analyses long after the actual construction has been complete. Documentation permits evaluation of the performance of

drainage systems and structures after flood events to determine if they performed as anticipated or to establish the cause of the unexpected behavior, if such is the case. In the event of a failure, it is essential that contributing factors be identified in order that recurring damage can be avoided.

Documentation can also be used as a basis for improvements upstream or downstream of the site.

The major purpose of providing good documentation is to define the design procedure that was used and to show how the design and decisions were arrived at.

Documentation can fall into three classifications, pre-construction, design and construction.

1. Pre-construction documentation may include:
   - Aerial Photographs
   - Contour mapping
   - Watershed Maps
   - Survey Data
   - FEMA Flood Insurance Studies
   - NRCS Soils Maps
   - Drainage Reports from other agencies or adjacent projects.
   - Town Mapping
   - Aquifer Protection Mapping
   - Impaired Waterbody Mapping

2. Design documentation shall include:
   - Hydrologic computations
   - Hydraulic Computations
   - Stormwater management computations
   - Drainage Report with watershed maps and plans

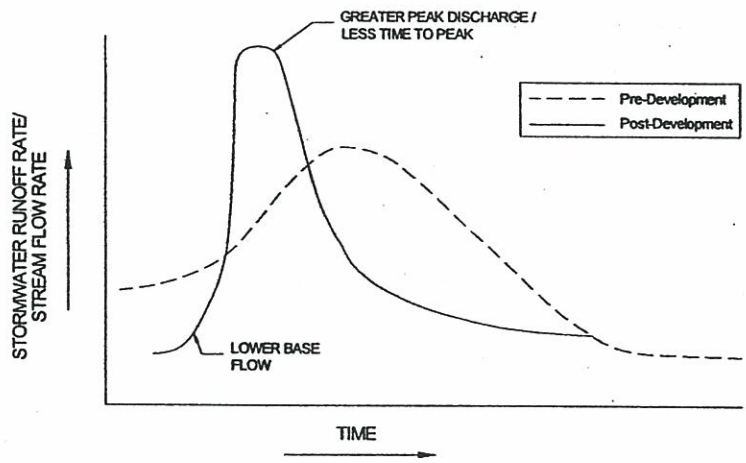
3. Construction and operation documentation shall include:
   - Plans, profiles and details.
   - As-builts plans with certification of conformance with the design intent.
   - Photographs
   - Records/reports of operation and maintenance.

Section 3 General Design Policies

3.1 Introduction

Changes in land use resulting from urbanization can affect the regional hydrology and pollutant loading in stormwater runoff. As urbanization increases, quality of runoff decreases. Development increases impervious surfaces, and decreases infiltration and groundwater recharge. As a result, much less water has a chance to infiltrate into the soil and therefore, the quantity and volume of runoff can increase significantly. (See Figure 3.1) The increased volume and quantity of runoff is also of poorer quality, and as a result, impairs the waterbodies of the Town.

Figure 3.1 Changes in Hydrology due to Urbanization



3.2 Runoff Rates

In general, the post development peak discharge rate shall not exceed the pre-development peak discharge rates, unless it can be demonstrated, to the satisfaction of the Director of Public Works, that the increased discharge will not exacerbate downstream flooding conditions or contribute to downstream erosion.

3.3 Stormwater Retention and Detention

Where it is anticipated that additional runoff incident to development or upstream drainage improvements will overload the existing downstream drainage facilities, or where downstream facilities are likely to become substandard in the future, then retention or detention of runoff should be given consideration as a means of providing hydrologic stability.

The concept of stormwater retention/detention is to temporarily store the excess stormwater runoff for subsequent release at a rate not to exceed pre-development conditions. Some or all of the stormwater may be infiltrated into the ground, depending upon the soil types at the site. Recharging the groundwater in this manner can further attenuate peak flows.

The location of the retention or detention facilities is very important as it relates to the effectiveness of these facilities to control downstream flooding. Small facilities have minimal flood control benefits and those benefits diminish as the flood wave travels downstream. Multiple storage facilities located in the same drainage basin affect the timing of the runoff through the conveyance system that may decrease or increase flood peaks in different downstream locations. Thus it is important for the engineer to design retention or detention facilities as drainage structures that both control runoff from a defined area and interact with other drainage structures within the drainage basin. Effective stormwater management must be coordinated on a basin-wide planning basis.

3.4 Downstream Property

Possible adverse effects of increased rates of runoff upon downstream property shall be investigated and appropriate measures taken, if necessary, to avoid possible damage.

3.5 Channels, Non-Structural Improvements

Except within the street right-of-way, open channels, ditches or swales are encouraged in lieu of pipes. In general, a non-structural approach to drainage improvements should be given preference, primarily through good site planning and management. Groundwater recharge should be investigated and utilized where appropriate as part of the overall stormwater management plan.

3.6 Stormwater Management Standards

The Town of Darien has developed Stormwater Management Standards. The Stormwater Management Standards are detailed in Section 7 of this manual.

Stormwater discharged from land uses or activities with higher potential for pollutant loadings than are found in typical stormwater runoff, may require the use of specific structural BMPs

and pollution prevention practices. These locations are known as "stormwater hotspots". Stormwater from a hotspot should not be infiltrated. Table 3.1 lists hotspot land uses.

Table 3.1 - Classification of Stormwater Hotspots
(Adapted from the Maryland Stormwater Design Manual, 2000)

The following land uses and activities are deemed stormwater hotspots:

- Vehicle salvage yards and recycling facilities*
- Vehicle fueling facilities, gas stations
- Vehicle service and maintenance facilities
- Vehicle and equipment cleaning facilities*
- Fleet storage areas (bus, truck, etc) *
-
- Marinas (service and maintenance)*
- Outdoor liquid container storage
- Outdoor loading/unloading facilities
- Public works storage areas
- Facilities that generate or store hazardous materials*
- Commercial container nursery
- Commercial parking lots with high intensity use or turnover
- Flat metal (galvanized metal or copper) rooftops of industrial facilities.
- Other land uses and activities as designated by an appropriate review authority

* Stormwater pollution prevention plan implementation is required for these land uses or activities under the EPA NPDES stormwater program.

3.7 Diversion of Drainage

The diversion of drainage from one watershed or watercourse to another may be permitted only after study of the hydraulic and hydrological impacts by a registered professional engineer and approval by the Director, Department of Public Works and other State or Federal Agencies having jurisdictional control.

3.8 Groundwater Discharge

The discharge of groundwater from sump pumps, subdrains, etc., onto sidewalks and roadways is prohibited.

3.9 Roof Leader Discharge

The discharge of roof leaders onto sidewalks and roadways is prohibited. For water quality purposes, rooftop runoff may be infiltrated into the ground without pretreatment. In other cases, roof leaders can discharge to the landscape provided that they will not exacerbate

flooding conditions or result in erosion of the landscape. Runoff from certain metal roofs must be treated before infiltration.

3.10 Tidal Considerations

1. **Drainage Systems**

In areas affected by tides, consideration must be given to the effects of tidal flooding.

For design purposes, unless otherwise approved by the Director, Department of Public Works, design elevations of tidal waters shall be equal to the elevation midway between mean high water, and the elevation of tidal flooding for the storm frequency of the facility being designed. Design elevations, in Darien, for common design storm frequencies are as follows:

Table 3.2 – Tidal Design Elevations

Design Storm Frequency	Design Elevation of Tidal Waters*
10 Years	6.6
25 years	7.2
50 years	7.6
100 Years	7.9

*Above mean sea level datum (NGVD29) in feet.

2. **Buildings**

The Town of Darien Zoning Regulations, Section 820, Flood Damage Prevention, require that:

"In the coastal high hazard zones, V, VE, and V1-30, the following provisions shall additionally apply:

a) All new construction shall be located landward of the reach of the mean high water line and landward of the high tide lines as defined by and regulated by the Connecticut Department of Environmental Protection;

b) All new construction and substantial improvements shall be elevated on adequately anchored pilings or columns, and securely anchored to such piles or columns so that the lowest portion of the structural members of the lowest floor (excluding the pilings or columns) shall be elevated to at least one foot above the base flood level;

c) A Connecticut registered professional engineer or architect shall certify that 1) the bottom of the lowest horizontal structural member of the lowest floor (excluding piling and columns) is elevated at least one foot above the base flood elevation; and 2) the pile or column foundation and structure attached thereto is anchored and resistant to floatation, collapse and lateral movement due to the affects of wind and water loads acting simultaneously on any building components;

d) All new construction and substantial improvements shall have the space below the lowest floor free of obstructions or be constructed with breakaway walls intended to collapse under stress without jeopardizing the structural support of the structure; such temporarily enclosed space shall not be used for human habitation. Such temporarily enclosed space shall be used solely for building access, parking of vehicles, and/or storage.

e) No use of fill for structural support of buildings shall be permitted.

f) Non-supporting breakaway walls, lattice work or mesh screening shall be allowed below the base flood elevation provided it is not part of the structural support of the building and is designed so as to breakaway under abnormally high tides or wave action, without damage to the structural integrity of the building on which it is to be used. Such enclosed space shall be used solely for building access, parking of vehicles, and/or storage."

In locations immediately along the shoreline where a building is subject to wave action (Zone V), floor elevations shall be a minimum of 1 foot above the base flood elevation unless the building has been designed as a Floodproof Building. In addition, permanent mechanical and electrical equipment shall be one foot above the base flood elevation.

3.11 Easements

Where new Town drainage facilities are proposed within private property, Town maintenance easements shall be provided and registered on a plan filed with the Town Clerk. Easements shall be a minimum of 20 feet in width. A swale, channel or ditch which passes drainage runoff from adjacent upstream public or private property shall be considered a drainage facility requiring an easement.

In subdivisions where roadway drainage discharges into private property, a note designating the private property owner as being responsible for properly passing drainage through his property shall be shown on the record plan.

3.12 Permits – Jurisdictional Control

Permits to construct drainage facilities shall be obtained from all Agencies that have jurisdictional control. There are a number of federal and state regulatory programs that are currently in place that would require permits and approvals. The following may be used as a guide.

Table 3.3 – Permitting Agencies

Area of Control	Agency
Watercourses	▪ Darien Environmental Protection Commission ▪ Connecticut Department of Environmental Protection (CGS 22a – 36 through 22a-45a) ▪ US Army Corps of Engineers
Inland Wetland Areas	▪ Darien Environmental Protection Commission ▪ Connecticut Department of Environmental Protection ▪ US Army Corps of Engineers
Coastal Area	▪ Connecticut Department of Environmental Protection – Office Long Island Sound Program ▪ Darien Planning and Zoning Commission ▪ US Army Corps of Engineers ▪ US Coast Guard – when work is in Navigable Waters
Stormwater Discharges from Construction Activities Disturbing more than 1 acre	▪ Connecticut Department of Environmental Protection (CGS 22a-416 through 22a-438)
Erosion and Sediment Control Guidelines	▪ Darien Environmental Protection Commission ▪ Darien Planning and Zoning Commission ▪ Connecticut DEP (CGS 22a-325 through 22a-329)
Town Highways, Sidewalks and/or Drainage Facilities	▪ Darien Department of Public Works (street opening permit, drainage connection permit) ▪ Connecticut Department of Transportation – Approval to construct on or near railroad
State Maintained Highways & Drainage Facilities	▪ Connecticut Department of Transportation

Section 4 Runoff Determination

4.1 General

A consideration of peak runoff rates for design conditions is generally adequate for the design of conveyance systems such as storm drains or open channels. However, if the design must include flood routing (e.g. detention/retention systems), a flood hydrograph is usually required. The development of runoff hydrographs (typically more complex than estimating peak runoff rates) is usually accomplished using computer programs.

4.2 Design Storm Frequencies

The average frequency of rainfall occurrence used for design largely determines the degree of protection afforded by a storm drainage system. This protection shall be consistent with the damage potential at full watershed development and the level of design required to protect the safety and convenience of the public. Other factors that may affect the choice of design frequencies rest on economic analysis and policy decisions of the Department of Public Works.

Tables 4.1 presents the minimum design storm frequencies to be used for design. Under exceptional circumstances, with the written approval of the Director, Department of Public Works, the design frequency may be reduced, on the basis of an engineering study.

Table 4.1
Design Storm Frequencies (Years)

Location	Design Frequency
Storm Drains	
Local Streets	25
Parking Lots	25
Collector & Arterial Streets	25
Collector & Arterial Streets at Sags[1]	50
Drainage Channels and Ditches	25
Culverts[2]	
Upstream watershed area < 1 square mile	50
Upstream watershed area > 1 square mile	100
Bridges	100
Detention Facilities	See Sect. 5.9

[1] Sags are defined as the lowest point in a vertical curve where water can only be removed through a storm system. The inlet at the sag shall be designed to accommodate a 50-year storm, while the remainder of the storm drainage system can be designed to accommodate the 25-year storm.

[2] Crossings of watercourses will require design for the 100-year storm, regardless of the size of the upstream watershed area. For those critical activities as defined in Section 25-68b-25-68h, CGS, the design storm frequency shall be 500 years.

4.3 Rational Method

The Rational Method of determining runoff is based on the premise that the peak rate of discharge from an area will be proportional to the rate of rainfall, modified by the size and characteristics of the area. The use of the Rational Method shall be limited to watersheds smaller than 200 acres.

In the Rational Method, runoff is related to rainfall intensity by the following equation:

$$Q = C \times I \times A \qquad \text{[Eq. 4.1]}$$

Where
- Q = flow rate (cfs)
- C = runoff coefficient, dependent upon ground cover
- I = rainfall intensity (in/hr)
- A = drainage area (acres)

Runoff Coefficient

The runoff coefficient, C, is dependent upon the type of cover on the site, as the total amount of impervious cover on the site increases, the value of C also increases. The value of C can be determined based upon either land use or soil type and slope, as shown in Tables 4.2 and 4.3, respectively. Alternatively, a composite C value can be calculated for a given area to reflect land use at a specific site as shown in Table 4.4. The selected method of determining C values must be used consistently throughout the entire hydrologic analysis to ensure a reliable basis for comparison.

Table 4.2
Rational Method Runoff Coefficients for Selected Land Uses

Area Type	Runoff Coefficient, C
Business: Downtown Areas	0.70 – 0.95
Residential:	
Single Family Areas	0.30 – 0.50
Multi units, detached	0.40 – 0.60
Multi units, attached	0.60 – 0.75
Suburban	0.25 – 0.40
Residential (lots > 1.2 acre)	0.30 – 0.45
Apartment dwelling areas	0.50 – 0.70
Light Industrial	0.50 – 0.80
Heavy Areas	0.60 – 0.90
Parks and Cemeteries	0.10 – 0.25
Playgrounds	0.20 – 0.40
Unimproved areas	0.10 – 0.30

Table 4.3
Rational Method Runoff Coefficients for Surfaces By Selected Hydrologic Soil Grouping[1] and Slope Ranges

Slope	Hydrologic Soil Grouping[1]			
	A	B	C	D
Flat (0 – 1%)	0.04 – 0.09	0.07 – 0.12	0.11 – 0.16	0.15 – 0.20
Average (2 – 6%)	0.09 – 0.14	0.12 – 0.17	0.16 – 0.21	0.20 – 0.25
Steep (Over 6%)	0.13 - 0.18	0.18 - 0.24	0.23 - 0.31	0.28 – 0.38
Impervious	0.80 – 0.90	0.80 – 0.90	0.80 – 0.90	0.80 – 0.90

Note

1. Hydrologic Soil Grouping as defined by the United States Department of Agriculture, Natural Resources Conservation Service in the "Soil Survey of Fairfield County, Connecticut", latest edition or by a site-specific soil survey by a Certified Soil Scientist.

Table 4.4
Rational Method Runoff Coefficients for Composite Analysis

Surface	Runoff Coefficients
Asphalt	0.70 – 0.95
Concrete	0.80 – 0.95
Brick Pavers (sanded joints only)	0.70 – 0.80
Roofs	0.75 – 0.95
Landscaped Areas	0.20 – 0.40
Wooded Areas	0.35 – 0.50
Packed Gravel	0.60 – 0.80

Typically, Tables 4.2 and 4.4 would be used to calculate composite values for larger drainage areas, while Table 4.4 would be used for smaller areas.

Rainfall Intensity

The rainfall intensity for the design storm is dependent upon the time of concentration, T_c. The time of concentration is the time it takes rainfall, falling upon the most hydraulically distant part of the watershed, to reach the location of the drainage facility being designed. The duration of rainfall is set equal to the time of concentration and is used to estimate the design average rainfall intensity, I.

Two common errors should be avoided when calculating T_c. First, in some cases, runoff from a portion of the drainage area that is highly impervious may result in a

greater peak discharge than would occur if the entire area were considered. In these cases, adjustments can be made to the drainage area by disregarding those areas where flow time is too slow to add to the peak discharge. Sometimes it is necessary to estimate several different times of concentration to determine the design flow that is critical for a particular application.

Second, when designing a drainage system, the overland flow path is not necessarily perpendicular to the contours shown on the available mapping. Often the land will be graded and swales will intercept the natural contours that may reduce the time of concentration. A field investigation of the existing drainage basins should be made in all cases to verify the assumed runoff patterns.

Time of Concentration

The time of concentration equals the sum of the travel times on each segment of the critical flow path. Sheet flow occurs in the upper reaches of a watershed. Such flow occurs over short distances and at shallow depths prior to the point where topography and surface characteristics cause the flow to concentrate in rills and swales. The depth of such flow is usually ¾ to 1 inch or less. Shallow concentrated flow is runoff that occurs in rills and swales and has depths on the order of 1.5 inches to 4 inches. Part of the principal flow path may include pipes or open channel flow such as through small streams. The travel time through open channel flow segments should be computed separately.

The following equation represents the time of concentration, which is the sum of the travel times (T_t) values for the various consecutive flow segments:

$$T_c = T_{t1} + T_{t2} + \ldots + T_{tm} \qquad \text{[Eq. 4.2]}$$

Where	T_c	=	time of concentration, hrs.
	m	=	number of flow segments
	T_{tm}	=	travel time, segment m, hrs.

Sheet Flow

Sheet flow is shallow runoff on a plane surface with the depth uniform across the sloping surface. Typically flow depths will not exceed 1 inch. Such flow occurs over relatively short distances, rarely more than about 300 feet. Sheet flow rates are commonly estimated using the NRCS TR-55 (1986) variation of the kinematic wave equation.

$$T_t = \frac{0.007}{P_2^{0.5}}\left(\frac{nL}{\sqrt{s}}\right)^{0.8} \qquad \text{[Eq. 4.3]}$$

Where	T_t	=	travel time, hrs
	n	=	Manning's roughness coefficient (values of n can be obtained from Table 4.5)
	L	=	flow length, ft
	I	=	rainfall intensity, (in/hr)
	S	=	slope of the hydraulic grade line (land slope), (ft/ft)
	P_2	=	2 year, 24 hour rainfall depth, (in)

Travel time is the ratio of flow length to flow velocity:

$$T_t = L/(3600V) \qquad \text{[Eq. 4.4]}$$

Where	T_t	=	travel time, hrs
	L	=	flow length, ft
	V	=	Average velocity, ft/s
	3600	=	Conversion factor from seconds to hours

Table 4.5
Manning's Roughness Coefficient (n)
for Overland Sheet Flow*

n	Surface Description
0.011	Smooth asphalt
0.012	Smooth concrete
0.050	Fallow (no residue)
	Cultivated Soils
0.060	Residue cover = 20%
0.170	Residue cover > 20%
0.130	Range (natural)
	Grass
0.150	Short Grass prairie
0.240	Dense grasses
0.410	Bermuda grass
	Woods
0.400	Light underbrush
0.800	Dense Underbrush

* Values obtained from NRCS TR-55 (1986) and McCuen (1989).

Shallow Concentrated Flow Travel Time

After a maximum of 300 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from Equations 4.5 and 4.6 in which average velocity is a function of watercourse slope and type of channel.

Unpaved $V = 16.1345(S)^{0.5}$ **[Eq. 4.5]**
Paved $V = 20.3284(S)^{0.5}$ **[Eq. 4.6]**

Where
V = Average velocity, ft/s
S = Watercourse slope, ft/ft

These two equations are based on the solution of Manning's equation with different assumptions for n (Manning's roughness coefficient) and r (hydraulic radius, (ft)). For unpaved areas, n is 0.05 and r is 0.4 ft, for paved areas, n is 0.025 and r is 0.2 ft.

After determining the average velocity, Equation 4.4 is used to estimate travel time, T_t for the shallow concentrated flow segment.

Open Channel Flow Travel Time

Manning's equation is

$$V = (1.49r^{2/3}s^{1/2})/n \quad \text{[Eq. 4.7]}$$
$$r = A/p_w \quad \text{[Eq. 4.8]}$$

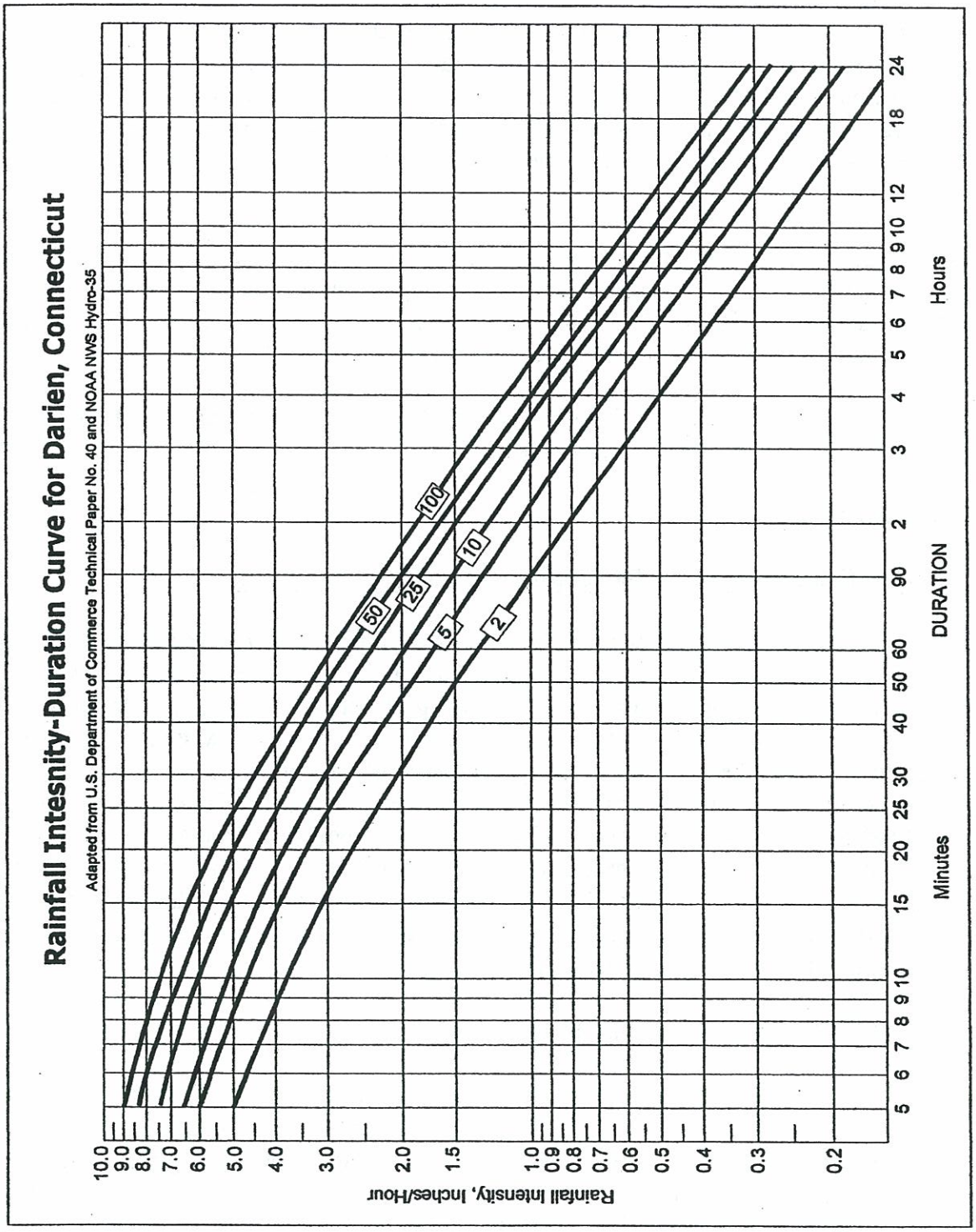
Where
V = Average velocity, ft/s
r = Hydraulic radius, ft
A = Cross sectional flow area, ft^2
p_w = Wetted perimeter, ft
s = Slope of watercourse, ft/ft
n = Manning's roughness coefficient

After average velocity for the channel segment is estimated using equation 4.7, the travel time is calculated using Equation 4.4

The minimum time of concentration to be used for a mostly impervious area shall be five minutes.

Once a time of concentration has been calculated, the rainfall intensity for the storm having the appropriate frequency and a duration equal to the time of concentration can be determined using Figure 4.1.

Figure 4.1
Rainfall Intensity-Duration Curve for Darien, Connecticut



4.4 US Army Corps of Engineers Computer Models HEC-1 and HEC-HMS

These computer programs simulate rainfall and/or snowmelt runoff from sub-basins within a watershed, flow through stream reaches, flow from urban areas, flow diversions, and flow through reservoirs including a breached dam condition with a varying (degrading) failure section. There is no limitation related to basin size. The programs are suitable for use in the analysis of watersheds that vary in size from small urban catchments to large, multi-basin river systems. In order to transform rainfall values to runoff, a unit hydrograph or kinematic wave approach is employed.

Point rainfall amounts for synthetic storms shall be those quantities for the appropriate duration and return frequency. The computer model shall be used to carry out multiple flood analyses and shall include as a minimum the 2, 10, 25, 50, and 100-year storm events. The computer model should be calibrated to a stream gauge if one is available.

Times of concentration and travel times for use in the HEC-1 program shall be developed using the methodology described in this manual.

4.5 SCS Methods

1. **TR-55 Urban Hydrology for Small Watersheds, Version 2.0.**

   The US Soil Conservation Service, now known as the Natural Resources Conservation Service (NRCS), developed this rainfall runoff model. TR-55 presents simplified procedures to calculate storm runoff volume, peak rate of discharge, hydrographs, and storage volumes. These procedures are applicable in small watersheds, especially urbanizing watersheds. The primary functions of the program are for peak runoff computations using the Graphical Peak Discharge Method, the Tabular Peak Discharge Method and Temporary Storage. Support functions include computations for the runoff curve number (CN), the Time of concentration (T_c) and travel time through a subarea (T_t). TR-55 analysis is limited to use for only NRCS type rainfall distributions, 24-hour duration rainfall, a maximum of 10 subwatersheds, and a minimum 0.1 hour and maximum 10-hour time of concentration.

2. **TR-20 Technical Release No. 20**

   This is a computer program that computes direct runoff from a rainstorm, generates flood hydrographs from surface runoff, and routes flow through channel reaches or reservoirs. Output data includes peak discharges with related times of occurrence and water surface elevations at designated cross sections or structures. With the exception of the channel routing routine (Modified Att-Kin Method), TR-20 is based upon the methodologies contained in the NRCS National Engineering Handbook, Section 4, Hydrology, (commonly referred to as NEH 4).

Based on the assumption that rainfall depth is uniformly distributed throughout the watershed, the area limitations for subcatchment or overall basin size range from approximately 0.1 square miles to 200 square miles.

As required with HEC-1 program, the hydraulic engineer should utilize appropriate rainfall data for the design storm event. The rainfall distribution for all watersheds in Connecticut shall be the 24-hour "Type III" storm as recommended by the NRCS, and methodologies shall be used for time of concentration and travel time computations.

The computer model shall be calibrated to a gauge station, if one is available.

Section 5 Design of Drainage Facilities

5.1 General Requirements

The storm drainage system shall be designed to effectively transport storm water for the required storm frequency, assuring a minimum self-cleansing velocity of 3 feet per second, yet not attaining destructive velocities over 15 feet per second.

Storm drainage systems shall be designed on the basis of ultimate development of the tributary watershed. The latest edition of the Town of Darien Zoning Map shall be used as the basis for the future development to be expected.

Drainage systems shall be carried to a suitable outlet, preferably an existing river, stream, or street drainage system. Where a suitable outlet is not available within a developer's property or the adjacent street right-of-way, it will be necessary to obtain appropriate drainage easements and to construct new and/or improve existing drainage facilities.

Where there is no suitable outlet, with the written approval of the Director, Department of Public Works, drywells may be used for drainage from very small areas. For small sites which have frontage along Town streets, overland flow may be permitted to discharge into roadway gutters. The design of drywells and the discharge into roadway gutters shall be subject to the review and approval of the Director of Public Works. No point discharge will be allowed into roadway gutters.

5.2 Catch Basins

1. General

Catch basins are the means by which runoff enters the piped storm drainage system. Proper inlet analysis and design are required to define the hydraulic design capacity of the drainage system. The capacity of a catch basin depends on its geometry, surface cross slope, and longitudinal slope, total flow, depth of flow and surface roughness.

2. Location

Along roadways and in parking lots having curbs or shoulders raised above the pavement, catch basins shall be located to prevent gutter and/or catch basin capacities from being exceeded and to prevent ice from building up in the gutter.

On roadways which do not have curbs or shoulders raised above the edge of pavement, no provision will ordinarily be required to provide drainage of the pavement other than the cross slopes of the pavement and shoulder, except at low points which shall be

drained by catch basins located within the area normally clear after snow removal. In such cases, the curb head of the catch basin shall be set back two and one-half feet from the normal edge of pavement.

In residential areas where the design flow across a roadway intersection is 1 cfs or more, a catch basin shall be provided upstream of the intersection to intercept the flow. In business districts, catch basins shall be provided upstream of all intersections, irrespective of design flow.

At the downgrade end of all curbed or cut sections, catch basins shall be provided to prevent erosion of the adjacent shoulder or slope.

Within parking lots, catch basins shall be located a) at low points, b) to prevent gutter and/or catch basin capacities form being exceeded, and c) to drain gutters located along the high side of parking lots to prevent hazardous freezing of snow melt.

In business districts, catch basins shall be provided at driveway entrances where the design flow would significantly inconvenience pedestrians or where existing gutter capacity along the road is already exceeded or would become exceeded. Catch basins at driveway entrances shall be located outside the street right-of-way.

At all locations within public or private roadways, parking lots or driveway entrances, catch basins shall be provided with sumps. Catch basins shall also be equipped with bell traps on their pipe outlets.

3. **Inlet Capacity**

The amount of flow intercepted by a catch basin is dependent upon many factors including the type of grate, the amount of depression, potential for clogging, and velocity of flow. The primary factor is the type of grate that is used. Experience indicates that the portion of the flow within the width of the grate will usually be intercepted. Lateral flow over the side of the grate is negligible and should not be considered on normal installations. The remainder of the flow, outside the grate width, will bypass to the next catch basin.

4. **Inlet Flow**

The flow entering a catch basin shall be determined by gutter flow analysis based on the following design limits for the extent of flooding or ponding on streets and highways.

a) The shoulders of all roads may be flooded.
b) Two-lane bi-directional roads may have one-half of a lane in each direction flooded; or if superelevated; one-half of the lane on the low side of the superelevation.

c) Four-lane undivided roads may have a lane in each direction, or one lane on the low side of a superelevation flooded.

d) Depressed roads shall have a minimum of 12 feet of traveled way free of flooding.

Gutter flow analysis shall be carried out using the following procedure:

Procedure for Gutter Flow Analysis (See Figure 5.1)

1. **Inlet Station** – Identify inlet by station and offset from centerline or baseline.

2. **Area (A)** – Enter all areas of each surface type contributing runoff to the inlet in separate rows in the computation table, in acres.

3. **Runoff Coefficient (C)** – Runoff coefficient for each area of each surface type contributing to the inlet. Refer to Table 4.2.

4. **Time to Inlet** – Time required for surface flow to concentrate at the inlet in minutes. Refer to Section 4.3.

5. **Rainfall Intensity (I)** – The intensity determined by the Time to Inlet (Step #4), from Figure 4.1 in inches per hour.

6. **AC** – The product of the areas and the runoff coefficients, (Step #2 x Step #3).

7. **Total AC** – The AC bypassing the previous inlet and the AC for the inlet, (Step #6 + Step #14 from the upstream inlet).

8. **Q to Inlet** – The product of the Total AC, (Step #7), and the Rainfall Intensity, (Step #5), (Q=CIA), in cubic feet per second.

9. **Slope of Gutter** – Slope to be expressed in feet per foot.

10. **Cross Slope of Shoulder** (S_x) – Slope to be expressed in feet per foot.

11. **Spread** – Using Figure 5.2 or an approved computer model, determine the spread, T (width of flow) and calculate the depth, d, at the curb by multiplying T times the cross slope(s). Compare with the allowable spread. If the computed depth at the curb is less than the curb height,

Project:
Date:
Computed By:

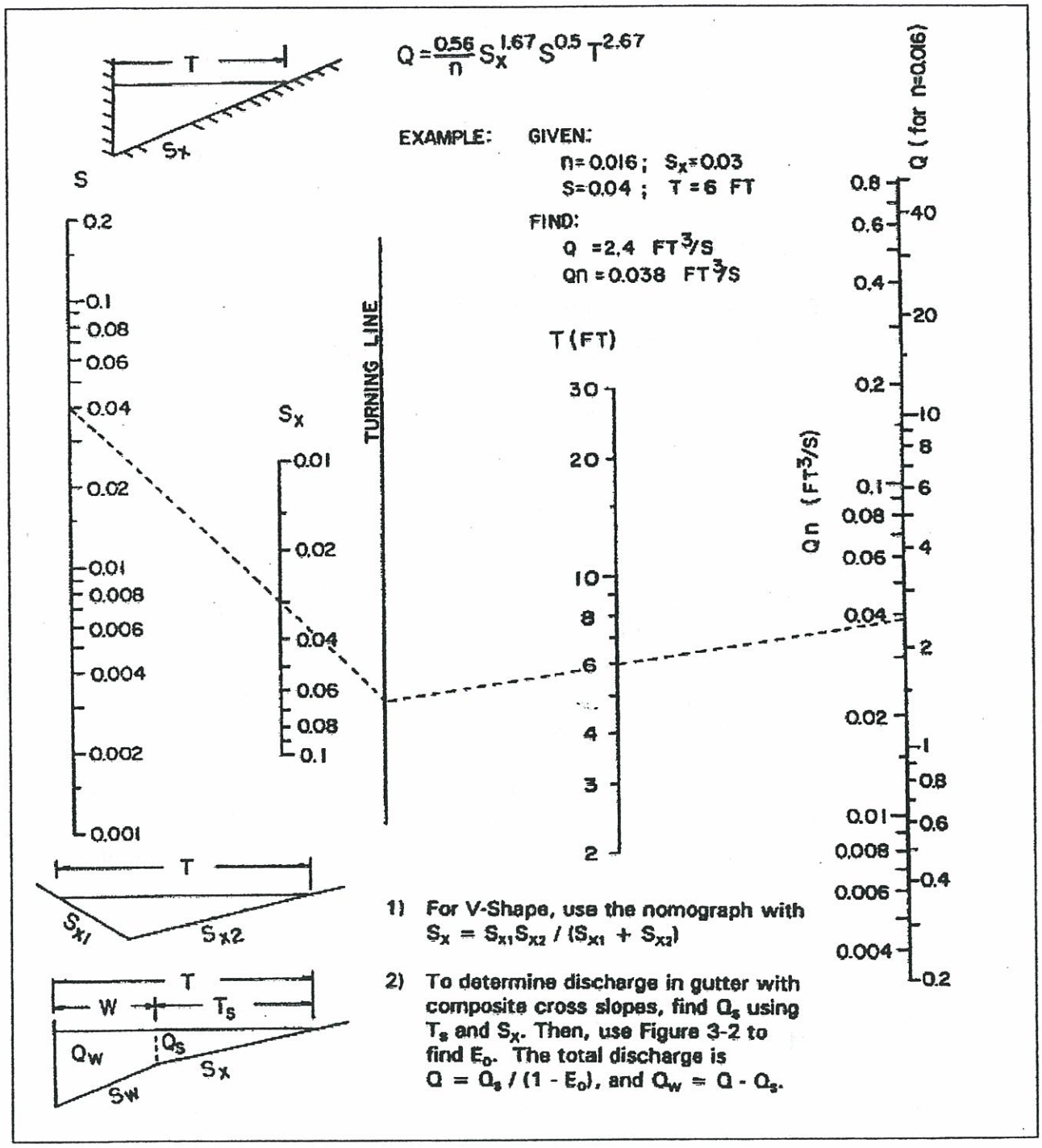
Gutter Flow Analysis

Inlet Station and Offset	Area in Acres (AC)	Runoff Coeff. (C)	Time to Inlet (min)	Rainfall Intensity (in/hr)	AC	Total AC	Q_T to Inlet (cfs)	Grade of Gutter (ft/ft)	Cross Slope of Shoulder (ft/ft)	d Depth of Flow of Gutter (ft)	T Width of Flow (ft)	Q Bypassing Inlet (cfs)	AC Bypassing Inlet	AC Entering CB
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15

Figure 5.1
Inlet Spacing Computation Sheet

Figure 5.2
Nomograph for Flow in Triangular Gutter Sections

(Source: U.S. Department of Transportation, Federal Highway Administration, Hydraulic Engineering Circular No. 12.)



and the computed spread is within the allowable spread, the flow entering the system (Step #15) and the flow bypassing the inlet (Step #14) may be computed. If the computed depth at the curb is greater than the curb height, expand or decrease the drainage area to meet the criteria and repeat this analysis until the computed spread is within the allowable spread.

12. **Q Bypassing Inlet** – The portion of flow that is beyond the width of the grate will be used to determine the bypass flow, Q. No consideration will be given to the minimal flow that enters along the longitudinal edge of the grate.

13. **Calculate Q Bypassing the Inlet.** The portion of flow that is beyond the width of the grate will be used to determine the bypass flow.

$$Q_{bypass} = Q_{total}\left(\frac{(T-G)^2}{T^2}\right) \qquad \textbf{[Eq. 5.1]}$$

Where G = Width of grate, ft
T = Spread, ft

14. **AC Bypassing Inlet** – Determined by dividing Q Bypassing Inlet, (Step #13), by Rainfall Intensity, (Step #5).

15. **AC Entering System** – The Total AC (Step #7), minus the AC Bypassing Inlet (Step #14).

Computerization of gutter flow analysis is acceptable. The computer printout shall contain the same information as is shown on the Gutter Flow Analysis Form, Figure 5.1. A detailed explanation of the method of analysis employed by the program shall be provided.

5. Sag Inlet Flow

The capacity of a catch basin in a sag or low point location shall be determined by the following method.

A grate inlet in a sag operates first as a weir having a crest length roughly equal to the outside perimeter along which the flow enters. Bars are disregarded and the side against the curb is not included in computing the perimeter. Weir operation continues to a depth of about 0.4 foot above the top of grate and the flow entering the grate is:

$$Q_i = 3.0\, P(d)^{1.5}\, C_{FS} \qquad \textbf{[Eq. 5.2]}$$

Where	Q_i	=	Flow entering the grate, cfs
	P	=	Perimeter of grate opening, ft Side against curb should not be included in perimeter
	d	=	Depth of water at grate, ft
	C_{FS}	=	Safety factor for clogging

When the depth at the grate exceeds about 1.4 feet, the grate begins to operate as an orifice and the flow entering the grate is:

$$Q_i = 5.37A(d)^{0.5}C_{FS} \qquad \text{[Eq. 5.3]}$$

Where	Q_i	=	Flow entering grate, cfs
	A	=	Clear opening of grate, ft^2
	d	=	Depth of ponded water above top of grate, ft
	C_{FS}	=	Safety factor for clogging

When the depth over the grate is between 0.4 ft and 1.4 ft, the operation of the grate inlet is indefinite due to vortices and other disturbances. The capacity of the grate is between that given by weir analysis and orifice analysis. The larger depth is used for design purposes.

Because of the vortices and the tendency of trash to collect on the grate, a factor of safety for clogging is included in the equation. For Type "C-L" catch basins with a high potential for clogging, a factor of safety of 2 should be used. Where danger of clogging is slight, a factor of safety less than two may be used. When a Type "C" catch basin is used, the curb opening provides the safety factor from clogging and a factor of safety of 1.0 may be used.

6. **Slotted Drains**

Slotted drains may be used to clear utilities where a standard catch basin cannot be installed, or to intercept sheet flow.

A method for determining the flow interception and length of slotted drains is found in the ConnDOT "Drainage Manual," pages 11.9-12 through 11.9-14.

5.3 Manholes

Manholes shall be provided where a change in horizontal and/or vertical alignment of pipes is required. In addition, manholes shall be located at not greater than 350 foot intervals for pipes up to 48" diameter and at not greater than 700 foot intervals for pipes larger than 48" diameter.

Within a roadway, Pipes shall not pass directly from catch basin to catch basin, but shall pass from catch basin to manhole, or manhole to manhole. However, within a building site, under conditions of limited flow, (less than 2.0 cfs) catch basins may be used as manholes, provided that a catch basin having a sump and a bell trap is installed prior to the discharge entering the Town's drainage system. In any case, no more than three catch basins shall be connected in series without a manhole.

In general, catch basins and manholes shall be precast. Concrete block may be used for the construction of catch basins with the written approval of the Director, Department of Public Works.

Connections to the Town drainage system shall be by means of a manhole. However, an isolated catch basin, within a building site, may discharge into an existing Town catch basin. Direct connection to existing pipes shall not be permitted.

Manholes are fabricated in different sizes to accommodate different pipe sizes and pipe geometries. The separation angle between two pipes inside a manhole is an important factor to consider when sizing manholes. There must be enough separation between the pipes so that the structural integrity of the manhole is not compromised by the pipe penetrations into the manhole. The following guidelines should be used as a "rule of thumb" when specifying manholes:

- When the storm drain pipe diameter is 30 inches or less, a 48-inch diameter manhole should be provided.
- When the storm drain pipe diameter is greater than 30 inches, up to 42 inches, inclusive, a 60-inch diameter manhole should be provided.
- When the storm drain pipe diameter is greater than 42 inches, up to 54 inches, a 72-inch diameter manhole should be provided.
- When the storm drain pipe diameter is larger than 54 inches, a reinforced pipe tee, a ConnDOT standard Type "C" or "C-L" catch basin with double grate, Type I or Type II, or if necessary, a specially designed junction box should be provided.

When determining the minimum manhole size required for various pipe sizes and locations, the manhole must be large enough to accommodate all pipes while maintaining an adequate separation between pipes. This determination can be calculated by determining the manhole coefficient, K, for each adjacent pair of pipes, and applying Table 5.1.

$$K = \frac{R_1 + T_1 + R_2 + T_2 + 14in}{\Delta} \qquad \text{[Eq. 5.4]}$$

Where R_1 = Interior radius of Pipe 1, in
T_1 = Wall thickness of Pipe 1, in
R_2 = Interior radius of Pipe 2, in.
T_2 = Wall thickness of Pipe 2, in.
Δ = Angle between centerlines of Pipes 1 and 2, degrees
K = Relative manhole coefficient, in/degree

Table 5.1 Manhole Sizing

MH Dia. (in)	K* (in/degree)	Max Pipe Size (in)
48	0.42	30
60	0.52	42
72	0.63	54

* When the computed K value exceeds (.63), a junction chamber or double catch basin will be required.

5.4 Storm Sewers

1. Pipe Selection

Circular pipes of less than 12 inches nominal diameter shall not be used for the transportation of surface drainage, except that roof leaders and single yard drains may be of smaller size. Elliptical pipe of equal hydraulic characteristics to the circular pipe may be substituted.

Reinforced concrete pipe, Class IV, shall be used except when otherwise approved in writing by the Director, Department of Public Works.

2. Pipe Installation

Pipe backfill and support conditions shall be in accordance with Standard Construction Details and Specifications, Department of Public Works. Refer to Appendix E for details. All trenches on slopes greater than 6 percent shall have cutoff walls.

Where drainage pipes cross sanitary sewers, water mains, gas mains or other utilities, minimum clearance shall be twelve inches, properly cushioned. Where it is not possible to provide the minimum twelve inches of clearance, provision shall be made for the protection of the pipes in accordance with the requirements of the utility company and subject to the written approval of the Director, Department of Public Works.

Pipes, except for underdrains, shall be installed on straight alignments, both horizontally and vertically, with manholes providing access to all deflection points and to all junctions of two or more lines. In special cases, with the approval in writing of the Director, Department of Public Works, pipes may be placed on curved alignments. Such curvature shall not exceed the pipe manufacturer's recommendations.

3. **Hydraulic Design**

Storm sewers shall be designed as open channels, where there is a free water surface (just full or less than full), or for pressure or pipe flow under surcharged conditions. The hydraulic design of a storm sewer system shall take into account the effect of backwater and shall allow for all energy losses in the system.

A hydraulic analysis shall be performed to determine whether the pipe will operate as an open channel or as a pressure system. Backwater calculations shall be made for each run of pipe. Friction losses as well as form losses shall be calculated. Form losses include losses due to manholes, bends, contractions, enlargements and transitions.

Manning's Formula shall be used for computing flow characteristics of conduits operating as open channels or under pressure:

$$V = \frac{1.486}{n}(r)^{2/3}(s)^{1/2} \qquad \text{[Eq. 5.5]}$$

$$Q = \frac{1.486}{n}A(r)^{2/3}(s)^{1/2} \qquad \text{[Eq. 5.6]}$$

Where V = Average velocity, ft/s
r = Hydraulic radius, ft
A = Cross sectional flow area, ft^2
s = Slope of watercourse, ft/ft
n = Manning's roughness coefficient
Q = Flow, cfs

Figure 5.3 provides an acceptable graphic solution of Manning's Formula for pipes flowing full and Figure 5.4 for pipes flowing partially full.

Figure 5.3
Nomograph for Manning's Equation for Storm Drain Flow
(Source: AASHTO Model Drainage Manual, 1991)

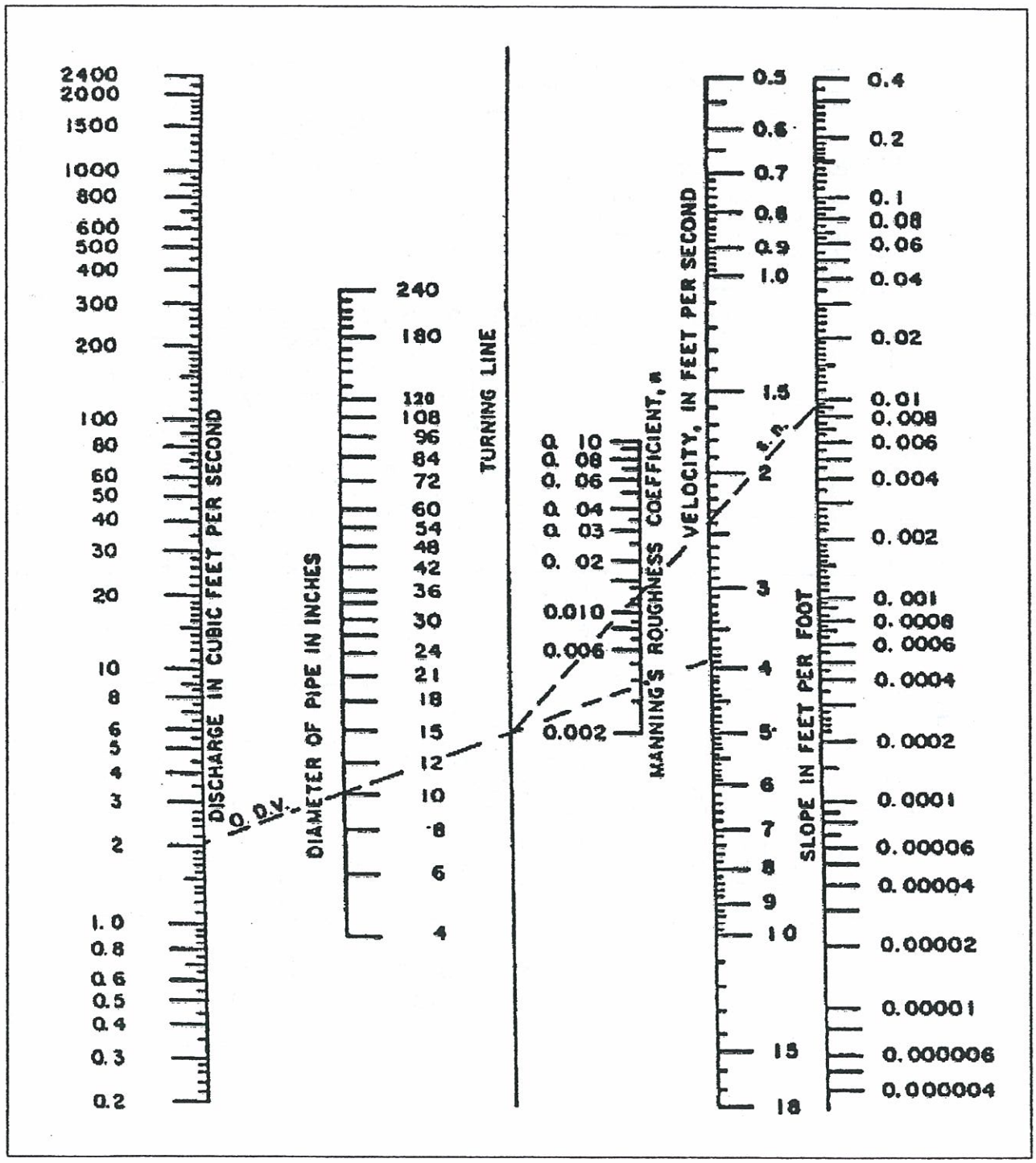


Figure 5.4
Chart for Partial Depth Flow
(Source: AASHTO Model Drainage Manual, 1991)

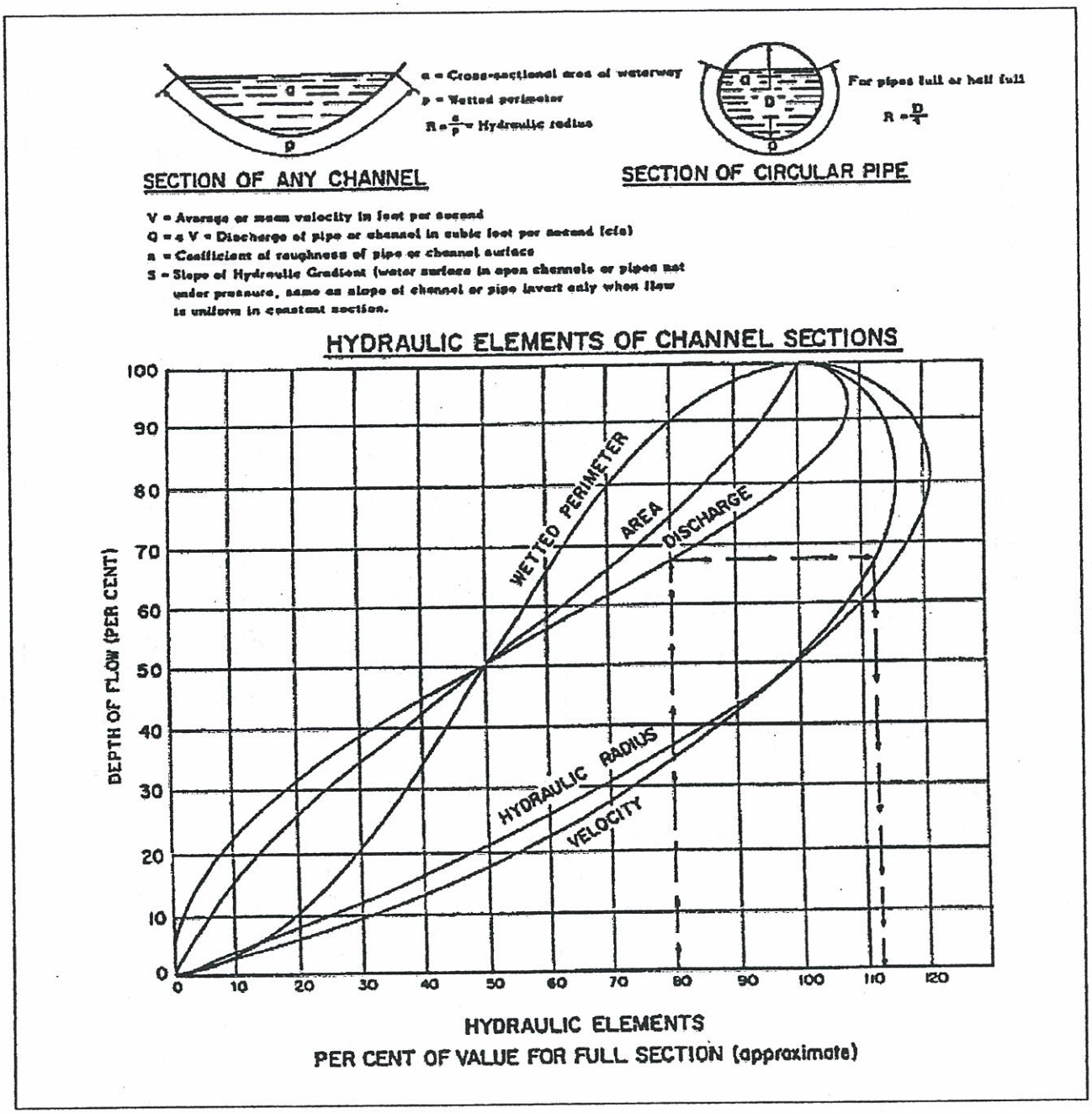
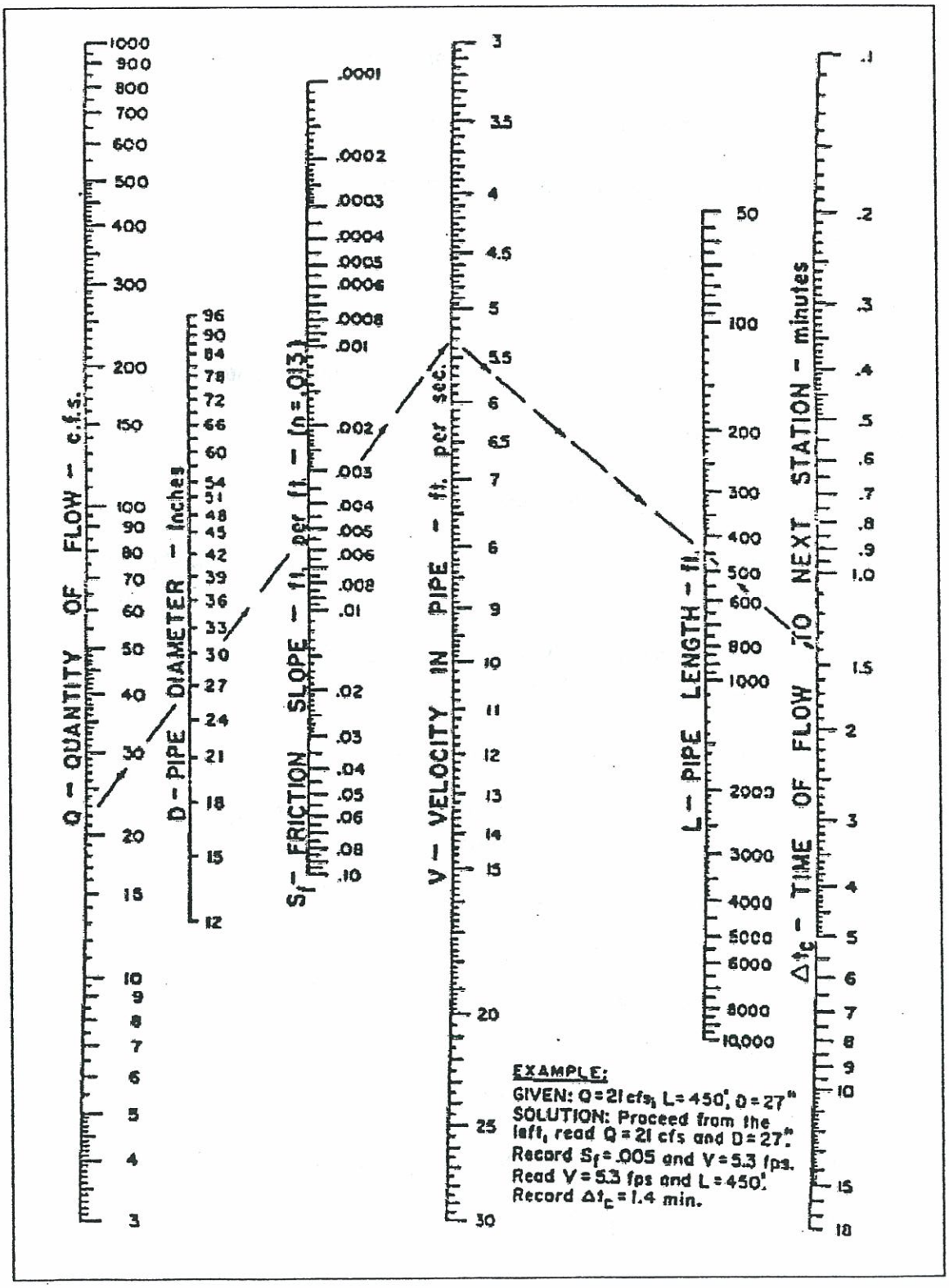


Figure 5.5
Nomograph for Concrete Pipe Flow
(Source: AASHTO Model Drainage Manual, 1991)



Procedure for Storm Sewer System Design (See Figure 5-6)

1. Line Segment - Identification of line, usually between inlets or manholes.

2. Length of Pipe - In feet measured between inside faces of structures.

3. Incremental area to pipe segment, acres.

4. Total area discharging to pipe segment, acres.

5. Runoff coefficient.

6. Incremental AC to pipe.

7. Total AC to pipe segment.8. Time to inlet, minutes.

9. Time in pipe, minutes.

10. Accumulated Time, minutes.

11. Rainfall Intensity - The intensity determined by the Accumulated Time (Step #10), inches /hour.

12. "Q" in System - The product of Sum of AC in System (Step #7), and the Rainfall Intensity (Step #11), in cubic feet per second.

13. Pipe Size - Inside diameter, inches.

14. Full Capacity - Flow that would be carried by pipe of size and type specified, flowing full, cubic feet per second.

15. Full Velocity - Velocity for pipe of size and type flowing full, feet per second.

16. Design Velocity - The velocity that will occur in pipe of size, type and slope specified at design flow, in feet per second.

17. Invert elevation at upper end of pipe, feet.

18. Invert elevation at lower end of pipe, feet.

19. Slope - Feet per foot.

Figure 5.6
Storm Drain Computation Sheet

Project:
Date:
Computed By:

Storm Drain Computation Sheet

Slope of Drain, S_o (ft/ft)											
Invert Elevation (ft)	Lower End										
	Upper End										
Velocity (fps)	Design Flow, V										
	Flowing Full, V_F										
Capacity Full, Q_F (cfs)											
Pipe Diameter, in											
Total Runoff, Q= CIA (cfs)											
Rainfall Intensity, I (in/hr)											
Flow Time, T_o (min)	Accumulated										
	In Pipe										
	To Inlet										
A x C	Total										
	Increment										
Runoff Coefficient, C											
Drainage Area, A (acres)	Total										
	Increment										
Length, L (ft)											
Station	To										
	From										

4. **Hydraulic Grade Line**

The hydraulic grade line (HGL) is the last important feature to be established relating to the hydraulic design of storm drains. The hydraulic grade line determines the acceptability of the proposed system by establishing the elevations along the system to which the water will rise from the design flow.

The detailed methodology employed in calculating HGL through the system begins at the system outfall with the tailwater elevation. If the outfall is an existing storm drainage system, the HGL calculation must begin at the outlet end of the existing system and proceed upstream through this in-place system, then upstream through the proposed system to the upstream inlet.

For most design applications, the tailwater will either be above the crown of the outlet or can be considered to be between the crown and critical depth (d_c). To determine the tailwater elevation (EGL) or $(d_c+D)/2$, whichever is higher, add the velocity head for full flow and proceed upstream to compute all losses such as exit losses, friction losses, junction losses, bend losses and entrance losses as appropriate.

Computer models such as Hydraflow can be used to calculate hydraulic grade lines for piped drainage systems.

Acceptable method for calculating hydraulic grade lines is also found in the ConnDOT "Drainage Manual" pages 11.2-1 through 11.2-10.

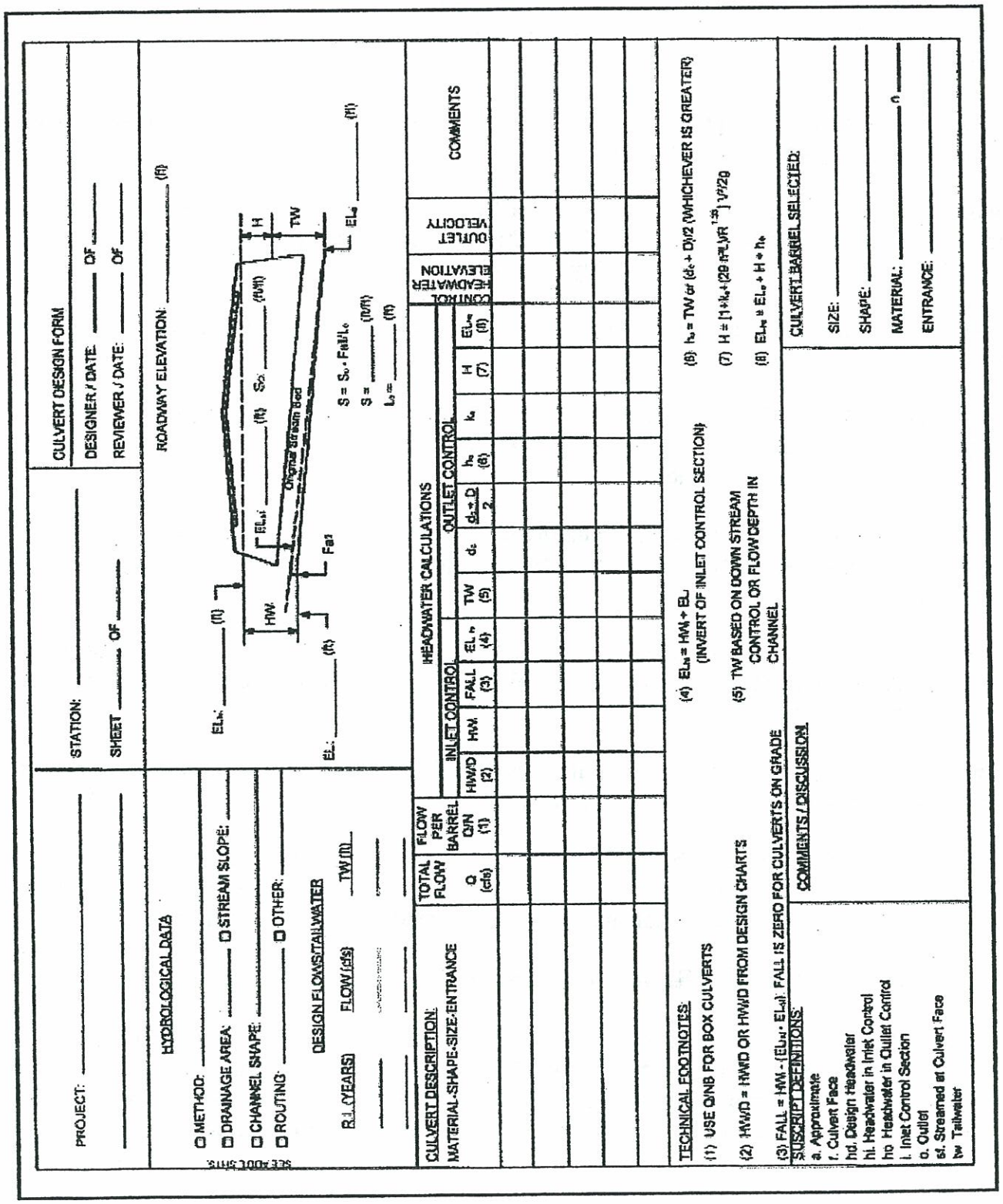
All outlets shall be designed to prevent erosion. Outlets shall be carried down steep slopes to lesser slopes where outlet erosion will not occur. Riprap or modified riprap shall be used at all outlets not flowing over exposed rock or directly into watercourses or ponds.

Where pipes are installed on steep grades and it is necessary to provide for energy dissipation at the outlet, the methods described in FHWA, Hydraulic Engineering Circular No. 14 (HEC-14) shall be used.

5.5 Culverts

Culverts shall be designed to carry discharges resulting from the storm frequencies defined in Table 4.1. The water surface used at the inlet to determine the culvert size shall generally be based on the allowable headwater (AHW) minus one foot for freeboard. The AHW shall be indicated on the culvert computation forms. (Figure 5.7), The AHW is determined or limited by the elevation of the roadway, parking lot or building sills for present as well as future development. Each culvert design must be considered individually based on the AHW and the topography adjacent to its inlet.

Figure 5.7
Culvert Design Form
(Source: HDS No. 5, 1985)



PROJECT: _____

STATION: _____

SHEET _____ OF _____

CULVERT DESIGN FORM

DESIGNER / DATE: _____ OF _____

REVIEWER / DATE: _____ OF _____

HYDROLOGICAL DATA

☐ METHOD: _____ ☐ STREAM SLOPE: _____

☐ DRAINAGE AREA: _____

☐ CHANNEL SHAPE: _____

☐ ROUTING: _____ ☐ OTHER: _____

SEE ADDITIONS

DESIGN FLOWS/TAILWATER

R.I. (YEARS) FLOW (cfs) TW (ft)

ROADWAY ELEVATION: _____ (ft)

EL$_{hi}$: _____ (ft) HW EL$_{ui}$ _____ (ft) S$_o$ _____ (ft/ft) H TW Original stream bed Fall EL$_o$ _____ (ft)

EL: _____ (ft) S = S$_o$ + Fall/L$_o$

S = _____ (ft/ft)

L$_o$ = _____ (ft)

CULVERT DESCRIPTION: MATERIAL-SHAPE-SIZE-ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW/D (2)	HW (3)	FALL (3)	EL$_{hi}$ (4)	TW (5)	OUTLET CONTROL d_c	$\frac{d_c + D}{2}$	h_o (6)	k_e	H (7)	EL$_{hw}$ (8)	HEADWATER ELEVATION CONTROL	OUTLET VELOCITY	COMMENTS

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) HW/D = HW/D OR HW/D FROM DESIGN CHARTS

(3) FALL = HW - (EL$_{hi}$ - EL$_{ui}$): FALL IS ZERO FOR CULVERTS ON GRADE

(4) EL$_{hi}$ = HW + EL$_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL

(6) h_o = TW or (d_c + D)/2 (WHICHEVER IS GREATER)

(7) H = [1+k$_e$+(29 n^{2}L$_o$/R$^{1.33}$)] V^2/2g

(8) EL$_{hw}$ = EL$_o$ + H + h$_o$

SUBSCRIPT DEFINITIONS:

a. Approximate
f. Culvert Face
hd. Design Headwater
hi. Headwater in Inlet Control
ho. Headwater in Outlet Control
i. Inlet Control Section
o. Outlet
st. Streamed at Culvert Face
tw. Tailwater

COMMENTS / DISCUSSION

CULVERT BARREL SELECTED:

SIZE: _____

SHAPE: _____

MATERIAL: _____ n _____

ENTRANCE: _____

Consideration shall be given to the effect of the selected design water surface on abutting private property. No increase in offsite flooding conditions will be permitted. Backwater effects should be contained within the street right-of-way or in a drainage easement. Engineering judgement shall be exercised to ensure that the headwater level selected is reasonable.

At all culvert inlets, the approximate limits of the flooded area covered by the headwater shall be shown on the plans submitted for review.

Where a culvert will carry a discharge reduced by detention the Department of Public Works reserves the right to require that the design be based on zero storage. In that case, the culvert will be designed as if the detention facility or facilities did not exist.

The location and alignment of culverts should be consistent with the flow tendency of existing streams.

Culverts should be designed with the aid of the FHWA, "Hydraulic Charts for the Selection of Highway Culverts" as found in Hydraulic Engineering Circulars Nos. 5 and 10 and should be checked for inlet and outlet control.

Where successive culverts are utilized and the flow in upper culverts is affected by headwaters in the lower culverts, a water surface profile and appropriate computations shall be submitted for review.

The effects of tidal action shall be investigated to insure that scour or erosion resulting from high velocities due to the movement of the tidal prism will not endanger the structure, roadway or adjacent property.

The need for energy dissipation at all outlets shall be investigated. Outlet dissipation methods are outlined in Hydraulic Engineering Circular No. 14

Design calculations shall be submitted in a clear, neat and understandable manner. The sample computation sheets and/or information provided on Figure 5.6 may be used as a guide. Where other forms are used, they shall contain all of the information shown in Figure 5.7.

5.6 Headwalls, Endwalls, and End Sections

A headwall, or other approved protection, shall be provided wherever an open drainage system discharges into or receives discharge from an enclosed system. An increase in culvert efficiency can be realized by the design of "Improved Inlets." An outline for the hydraulic design of improved inlets is detailed in FHWA, Hydraulic Engineering Circular No. 13.

Safety of the structure location must be considered in relation to pedestrian and vehicular traffic. Culvert ends are preferred to endwalls. Safe recovery areas shall be provided where vehicles may impact the structure, if guide rail is not used.

The need for trash racks shall be established by field investigation. If used, they shall be designed so as to allow for overflow when clogged. Access for maintenance of trash racks should be considered.

Headwalls shall conform to Town of Darien Standard Details.

5.7 Channels

1. Hydraulic Design

Channels shall be designed with a section and grade which will carry the design discharge, in its flattest section, under the controlling conditions, providing freeboard as needed, and with channel lining protection to prevent erosion. A change in the type of channel lining, with all other conditions unchanged, may require a change in channel dimensions unless the two linings have the same roughness coefficient. Open channels shall be designed to carry runoff consistent with the storm frequencies defined in Table 4.1. The ConnDOT "Drainage Manual" and the FHWA, Hydraulic Engineering Circular No. 15 shall be used for the design of channels.

Channels shall be designed to convey the design discharge within their banks unless an engineering study is submitted and approved by the Director, Department of Public Works, which demonstrates that the proposed solution is acceptable.

Location, linings and cross slope of banks are important safety considerations. Along streets and highways, motorists' safety generally improves with increasing distance between the traveled way and the channel.

All channels shall be designed so as to minimize erosion. Flattening channel grades, where uniform flow conditions exist, may reduce erosion velocities, otherwise an appropriate channel lining shall be used. To prevent erosion all channels, ditches or swales shall be lined, in accordance with their design, as soon as they are excavated. Where possible, swales should have vegetative buffers.

Consideration should be given in the design to maintenance requirements. The design should reduce maintenance to a minimum.

Freeboard should be provided where overflow would cause considerable damage by providing extra height of lining above the design depth of flow.

Superelevation of the flow surface will occur at channel bends or curves and requires an additional height of lining. Subcritical flow around a curve causes the water to rise on the outside of the curve. Supercritical flow around a curve will cause the water to

rise alternatively on the outside and inside of the curve requiring additional height throughout the curve length. The height of the water surface may be computed by using the formula provided in FHWA, Hydraulic Engineering Circular No. 15.

In large watercourses wave action may occur. Wave action will require additional height of lining. Where channel slopes are between 0.7 S_c and 1.3 S_c it is recommended that an allowance for waves be added to the normal depth of flow based on the following formula:

$$H_w = 0.25d_c\left[1-(11.1)(\frac{S}{S_c}-1)^2\right] \qquad \text{[Eq. 5.7]}$$

Where H_w = Wave height, ft
d_c = Critical depth, ft
S = Slope of channel, ft/ft
S_c = Critical slope, ft/ft

2. **Rigid Linings**

The hydraulic design of riprap revetments is described in FHWA, Hydraulic Engineering Circular No. 11.

The following design features should be considered in the design of rigid channel linings.

a. A riprap pad or energy dissipater at the outlet end of the rigid lining to reduce scour and undermining. An energy dissipater may be needed in the channel itself.

b. The use of back drainage where ground water is intercepted.

c. A cut-off wall at the outlet end to reduce undermining.

d. The channel should be kept below existing ground, if possible.

e. The use of sod, one foot wide, adjacent to the top of the lining to prevent surface water infiltration behind the lining material.

3. **Flexible Linings**

A flexible lining is generally more economical and is aesthetically preferable since it blends in with the landscape. There are several types of flexible linings. These include grass, riprap, keyed riprap and gabions. Design procedures for flexible linings are described in FHWA, Hydraulic Engineering Circular No. 15. Preference should be given to grass-lined swales where possible.

5.8 Energy Dissipaters

Energy dissipaters serve to reduce the kinetic energy of flowing water to minimize scour and erosion in channels and at the outlets of structures. There are several types, including check dams, drop structures, baffles, stilling basins, scour holes and structures designed to create a hydraulic jump.

In some situations, channel lining requirements may be reduced downstream of an energy dissipater, because of the reduced water velocity, providing an economic benefit.

Energy dissipaters shall be designed to prevent overtopping and causing damage, when carrying the design flow.

The design procedure for energy dissipaters is found in FHWA, Hydraulic Engineering Circular No. 14.

5.9 Detention/Retention Facilities

Detention refers to a storm water storage facility that is normally dry but is designed to temporarily hold (detain) water during and immediately after a runoff event. Examples of detention facilities are, natural swales with berms, subsurface tanks, pipes or reservoirs, rooftop storage and parking lots. Play equipment and sheds shall not be placed in areas designated for stormwater retention or detention. In general, detention/retention areas should be in place before construction of new impervious surfaces.

In contrast, a retention facility always contains (retains) a substantial volume of water to serve recreational, aesthetic, water supply or other functions. Storm water is temporarily stored above the normal water surface elevation (stage) during and immediately after runoff events. Examples of retention facilities include ponds, galleries and other underground storage that recharges water, lakes, streams, and rivers.

1. **Hydrological Study**

   A registered professional engineer shall prepare a hydrological study of the impact of proposed changes in runoff on the entire watershed. The study should include:

   a. An investigation of the adequacy of the existing downstream drainage system.

   b. A review of existing flooding problems.

   c. An estimate of the impact of additional runoff from development.

d. A review of the probability and impact of further development within the watershed that may cause increased runoff.

e. An analysis of the effect of time lag between drainage systems of developments and the surrounding watershed at critical points.

The study should be based on various storm durations and frequencies to ensure that the retention or detention facility will function properly and prevent increased downstream flooding for the design storm as well as more frequent storms.

Detailed engineering computations should be made although, with the approval of the Director, Department of Public Works, qualitative judgement will be accepted where conclusions are obvious or where an inordinate amount of computations would be required to demonstrate impacts known to be insignificant and not accumulative in the watershed under consideration. A report of the results shall be submitted to and approved by the Director, Department of Public Works.

Retention and detention facilities shall not be provided where discharge is directly into Long Island Sound or in areas where the engineering study indicates retention or detention is not required such as in the lower reaches of large streams or rivers.

The design engineer shall make provision for safety and for the maintenance of the proposed facility. Please refer to the standard Maintenance Agreement in Appendix A.

2. **Design Procedure**

Detention and retention facilities shall be designed in accordance with the following procedures:

1. Develop an in-flow Hydrograph for the detention or retention facility for the design storm using one of the following methods:

   a. **Modified Rational Method**

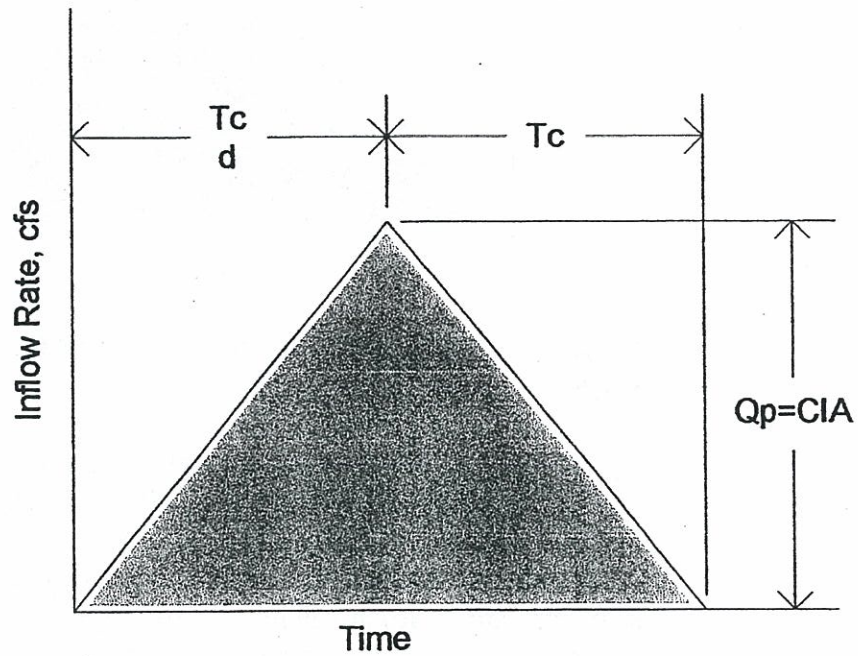
   Application – The Modified Rational Method may be used to develop an in-flow hydrograph and is based on the Rational Method of estimating peak rates of runoff. The use of this method shall be limited to watersheds smaller than 10 acres. The basic assumption inherent in the Rational Method is that the rainfall occurs at a uniform rate throughout the entire rainfall period. This is a highly improbably assumption and therefore the method should be applied only to relative small drainage areas.

Procedure – In the Rational Method, the peak rate of runoff is determined by the formula Q=CIA (see Section 4 – Rational Method, for application and procedure).

The in-flow hydrograph shall fall into one of the following cases:

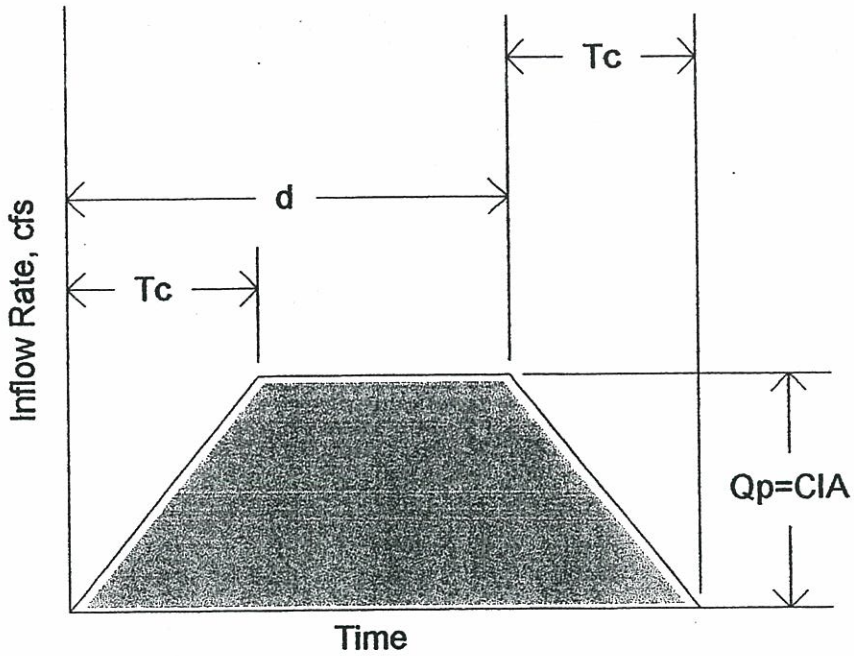
Case 1: The duration of the storm (d) is equal to or less than the time of concentration (T_c) and the peak discharge (Q_p) is equal to CIA. This hydrograph shape is shown in Figure 5.8.

Figure 5.8
Hydrograph where Duration ≤ Tc



Case 2: The duration of the storm (d) is greater than the time of concentration (T_c) and the peak discharge Q_p is equal to CIA. This hydrograph shape is shown in Figure 5.9 below.

Figure 5.9
Hydrograph where Duration > Tc



b. **US Army Corps of Engineers Computer Models HEC-1 and HEC HMS**

Application – In-flow hydrographs may be determined by methods outlined in the HEC-1 and HEC-HMS Hydrologic Modeling System User's Manual. There is no limitation related to basin size. The programs are suitable for use in the analysis of watersheds that vary in size from small urban catchments to large, multi-basin river systems. In order to transform rainfall values to runoff, a unit hydrograph or kinematic wave approach is employed.

Point rainfall amounts for synthetic storms shall be those quantities for the appropriate duration and return frequency.

Procedure – The procedures described in the above referenced User's Manual shall be followed.

c. **Soil Conservation Service Tabular Hydrograph Method**

Application – An in-flow hydrograph may be determined by methods outlined in SCS Technical Release No. 55 "Urban Hydrology for Small Watersheds." Use of this method shall be limited to watersheds of less

than 20 square miles having a 24-hours duration and a storm pattern with a Type III distribution.

Procedure – The design procedure described in the above referenced publication shall be followed.

d. **Soil Conservation Service Unit Hydrograph Method**

Application – The Soil Conservation Service has developed a method of hydrograph synthesis that may be used to determine an in-flow hydrograph. Use of this method shall be limited to watersheds of less than 20 square miles and is outlined in SCS National Engineering Handbook, Section 4, "Hydrology" (NEH-4).

Procedure – The design procedure described in the above referenced publication shall be followed.

2. Determine the allowable discharge rate. This shall be not greater than the undeveloped discharge rate.

3. Compute the Storage/Stage Curve for the proposed detention or retention facility indicating the volume of storage at various water surface elevations.

4. Select a specific outlet control structure, and compute the outflow discharge rates for various water surface elevations to produce the Outflow/Stage Curve.

5. The time interval selected shall be no greater than five minutes for a time of concentration less than one hour and no less than 10 minutes for a time of concentration equal to or greater than one hour.

6. Route the design In-flow Hydrograph through the detention or retention facility, utilizing the Storage-Indication Working Curve Method or Modified Puls Method. This routing is accomplished by the use of the standard flood routing equation.

$$\frac{[I_1 + I_2]}{2}\Delta T - \frac{[Q_1 + Q_2]}{2}\Delta T = S_2 - S_1 = \Delta S \qquad \textbf{[Eq. 5.8]}$$

Where Q_1 & Q_2 = Outflow rates at times 1 and 2, cfs
I_1 & I_2 = Inflow rates at times 1 and 2, cfs
S_1 & S_2 = Storage values at times 1 and 2
T = $time_2 - time_1$ (Time interval)

The standard flood routing equation can be rewritten in the Storage-Indication Working Curve format:

$$I_1 + I_2 + [\frac{2S}{\Delta T}1 - Q_1] = [\frac{2s}{\Delta T}2 + Q_2] \qquad \text{[Eq. 5.9]}$$

At the beginning of the rainfall event I_1, S_1 & Q_1 are equal to zero, so that

$$I_2 = \frac{2S}{\Delta T}2 + Q_2 \qquad \text{[Eq. 5.10]}$$

The maximum outflow shall not exceed the allowable discharge rate and the freeboard at the peak water surface elevation shall be not less than one foot.

7. Once the detention or retention facility has been designed to offset peak runoff from the design frequency storm, storms of other durations and frequencies should be checked to ensure that downstream flooding does not occur for these storm events.

8. The Hydrological Report shall document the method of generating the in-flow hydrograph and all assumptions for the routing. The routing curves and tables shall be included. The routing output shall provide, but not be limited to, the time, the in-flow rate, the out-flow rate, and the maximum storage volume.

Section 6 Water Quality Overview

6.1 Introduction

Stormwater runoff carries pollutants that have been deposited on land into rivers, streams, lakes, ponds, wetlands, marine waters, and groundwater. In addition to exacerbating flooding, pollutant contaminated runoff significantly degrades water quality and aquatic habitat.

Development increases the coverage of impervious surfaces, which prevent the infiltration of rainfall into the ground. Since less rainfall infiltrates, more rainfall flows across the ground, potentially increasing flooding, and groundwater recharge necessary to support wetlands and maintain base flows in streams is reduced. In addition to increasing impervious cover, development also tends to create uniformly graded slopes. Uniformly graded slopes eliminate natural depressions that serve to temporarily pond water. Over time, the atmosphere deposits pollutants upon impervious surfaces. During storms, these pollutants are quickly carried away to watercourses by stormwater runoff. These pollutants include nutrients, suspended solids, organic carbon, bacteria, hydrocarbons, and trace metals. Pollutant transport is especially critical at the hotspots shown in Table 3.1. Impervious surfaces have a greater affinity for heat absorption, and runoff generated from these surfaces could also increase the temperature of receiving waters.

In Connecticut, stormwater runoff and the resultant discharges from stormwater drain pipes are significant contributors to water quality problems in the State's rivers, streams, and marine waters. Stormwater discharges were a major impediment to aquatic life and recreation to one quarter of the state's 893 miles of major rivers and streams according to the State's 2002 list of impaired waterbodies.

The prevention of infiltration by impervious cover reduces the recharge of groundwater. Groundwater is a critical resource that residents depend upon for drinking water. Groundwater is also critical to maintaining the hydrology of non-tidal wetlands.

6.2 Stormwater Regulation

Initial efforts to improve water quality in Connecticut focused on regulating direct discharges of wastewater from municipal and industrial facilities. These point sources of pollution have been regulated under the National Pollutant Discharge Elimination System (NPDES) Program. This program sets limits for pollutant discharge concentrations and requires municipalities and industries to obtain permits for relevant discharges.

In 1987, Congress amended the Clean Water Act, creating a phased National Pollution Discharge Elimination Program (NPDES) to address stormwater impacts. The first phase of

the NPDES stormwater permit program established controls for storm discharges resulting from certain industrial activities, land disturbances over five acres, and municipal storm drains in highly populated areas. In 1990, Congress reauthorized the Coastal Zone Management Act, amending it to include new stormwater pollution control measures for coastal waters (Section 6217) in states with coastal zone management programs.

In 1999, EPA published its NPDES Phase II Final Rule. Phase II of the program expands the scope of regulated discharges to include certain regulated small municipal separate storm sewer systems (MS4s), and construction activity disturbing between one and five acres of land.

6.3 Sedimentation and Erosion Control

This manual is to be used in conjunction with the State of Connecticut, Department of Environmental Protection's Bulletin No. 34, "Connecticut Guidelines for Soil Erosion and Sediment Control," dated 2002. Erosion results in increased total suspended solids in runoff. The general permits established under Phase 1 of the NPDES regulations address this problem by requiring sedimentation and erosion controls during construction.

6.4 Critical Areas

Certain discharge locations are more sensitive to stormwater pollutants than others. These areas are called critical areas. They include:

❶ Aquifer Protection Zones
❷ Shellfish Beds
❸ Shorefront
❹ Other watercourses deemed sensitive by the Town

The Appendix of this manual includes a map illustrating areas that the Town has determined to be critical in terms of stormwater runoff quality. These critical areas are subject to more stringent water quality treatment requirements, as further outlined in Section 6.5.

6.5 Water Quality Volume

A sufficient amount of stormwater from a particular site is to be treated for pollutant removal. This amount of water is known as the Water Quality Volume. The water volume to be treated is dependent upon where the water will be discharged:

1. **Critical Areas**

   The required Water Quality Volume is 1.0 inch of runoff multiplied by the total impervious area of the post-development site. (Source: Massachusetts Stormwater Management Manual)

2. **All Other Areas**

The required Water Quality Volume is 0.5 inches of runoff multiplied by the total impervious area of the post-development site. (Source: Massachusetts Stormwater Management Manual)

For redevelopment projects, it may not be feasible to treat the prescribed water quality volume. In these instances, the project proponent must implement stormwater quality controls to the maximum extent practicable, and must demonstrate why full treatment is not feasible, and that the proposal will improve stormwater over existing conditions.

Runoff from roofs, except certain metal roofs, is considered uncontaminated and may be infiltrated directly into the ground. Infiltrated runoff may be subtracted from the water quality volume.

Water Quality Volume deals with the runoff generated by a site. For sites where off-site areas also contribute to the site's drainage, the off-site area need not be included in the calculations of Water Quality Volume.

6.6 Recharge Volume

In order to preserve base flows in groundwater, it is good stormwater management practice to infiltrate captured stormwater runoff into the natural soil. This volume is referred to as the Recharge Volume. The Recharge Volume should be determined using the existing site soil hydrologic groups as defined in the United States Natural Resources Conservation Service (NCRS, formerly SCS) Soil Survey of Fairfield County, Connecticut. Refer to the Appendix of this manual for a map of the Hydrologic Soil Groups in Darien.

Stormwater runoff from "hotspot" areas shall not be directly infiltrated into the soil without appropriate pretreatment.

Table 6.2 below lists the depth of runoff to be recharged. To determine the Recharge Volume, multiply the depth by the area of impervious cover.

Table 6.2
Groundwater Recharge Depth

Hydrologic Group	Depth, Inches
A	0.40
B	0.25
C	0.10
D	No recharge requirement

Depth to groundwater and bedrock also play a role in the suitability of infiltration systems to a particular site. These restrictions are described in further detail in Section 9. Where infiltration measures are used, geotechnical investigations of the subsoils may be required.

Where more than one hydrologic soil group is present on a site, a composite value shall be used based upon the proportions of the soil groups. The Recharge Volume shall be infiltrated in the most permeable soil group on the site.

The Recharge Volume may be subtracted from the total Water Quality Volume required to be treated at a site.

Section 7 Stormwater Management Standards

7.1 Purpose of Standards

Stormwater management standards address both water quality and water quantity by establishing the level of required controls that can be achieved through the use of site planning, non-structural measures, and Best Management Practices. Darien's standards have been designed to conform to current stormwater management requirements, with the following goals in mind:

❶ Prevent untreated discharges;
❷ Preserve hydrologic conditions that closely resemble pre-development conditions;
❸ Reduce or prevent flooding by managing the peak discharge rate and the total volume of runoff;
❹ Minimize erosion and sedimentation;
❺ Reduce suspended solids and other pollutants to improve water quality;
❻ Provide increased protection of sensitive natural resources.

7.2 Applicability

The Darien Stormwater Management Standards apply to development activities disturbing one-half acre or more.

Project proponents must demonstrate that they are implementing the highest practicable level of stormwater treatment when all the standards cannot be met.

The Darien Stormwater Management Standards do not apply to emergency repairs which affect drainage systems, although application of the standards is still encouraged.

7.3 Stormwater Management Standards

The following standards must be met at development sites, as described in Section 7.2 above.

1. New stormwater outfalls may not discharge untreated stormwater directly to, or cause erosion in, wetlands or watercourses.

2. Post-development peak discharge rates shall not exceed pre-development peak discharge rates, unless documentation can demonstrate, to the satisfaction of the Director of Public Works, that the proposed development would not exacerbate downstream flooding conditions. Refer to Section 3.2.

3. Annual groundwater recharge rates should be maintained through the use of infiltration measures to the maximum extent practicable. The annual recharge from the post-development site should approximate the annual recharge from the pre-development or existing site conditions, based on soil types.

4. Stormwater management systems for new development shall be designed to remove 80% of the average annual post-development conditions load of Total Suspended Solids. It is presumed that this standard is met when:

   - Suitable nonstructural practices for source control and pollution prevention are implemented.

   - Stormwater management best management practices are used to capture the prescribed water quality volume; and

   - Stormwater management facilities are constructed and maintained as designed.

5. Stormwater discharges from areas with higher potential pollutant loads and from stormwater hotspots as defined in Table 3.1 require the use of specific stormwater management and pollution prevention practices. The use of infiltration without pretreatment is prohibited.

6. Stormwater discharges to Critical Areas, as defined in Section 6, are subject to additional criteria and may need to utilize or restrict certain management practices.

7. Redevelopment of previously developed sites must meet the Stormwater Management Standards to the maximum extent practicable. All reasonable efforts must be made to meet the standards, including evaluation of alternative designs and their locations. If it is not practicable to meet all the standards, new (retrofitted or expanded) stormwater management systems must be designed to improve existing conditions. Redevelopment is defined as any construction, alteration or improvement exceeding one acre of land disturbance on sites where existing land use is commercial, industrial, institutional, or multi-family residential.

8. Erosion and sediment controls must be implemented to prevent impacts during construction.

9. All stormwater management systems shall have an enforceable operation and maintenance plan to ensure that systems function as designed.

7.4 Explanation of Standards

Standard 1 – Untreated Stormwater

Stormwater runoff not meeting the stormwater quality standards is considered to be untreated. Runoff from rooftops, except certain metal roofs, is considered to be uncontaminated and is eligible for direct infiltration into the ground.

Standard 2 – Post-Development Peak Discharge Rates

Refer to Section 3.2 Runoff Rates

Standard 3 – Groundwater Recharge

The volume of stormwater to be recharged to the groundwater should be determined using the existing site soil hydrologic groups as defined in the US Natural Resources Conservation Service (NCRS, formerly SCS) Soil Survey of Fairfield County, Connecticut.

Refer to Section 6.6 for the calculation of Recharge Volume.

Standards 4, 5 & 6 - Water Quality Volume

As stated in Section 6.5, the runoff volume to be treated is based on the following:

1. For discharges to critical areas, the treatment volume is calculated as 1.0 inch of runoff multiplied by the total impervious area of the post-development project site.

2. For all other discharges, treatment volume is calculated as 0.5 inches of runoff multiplied by the total impervious area of the post-development project site.

The total impervious area of a developed site is determined based on final project site plans, not on pre-existing site conditions. Roof runoff, except from certain metal roofs, may be infiltrated. Infiltrated volumes may be subtracted from the total Water Quality Volume.

Standard 4 - Removal of 80% Total Suspended Solids

A single or a combination of Best Management Practices must be used to remove eighty percent of Total Suspended Solids. Table 7.1 shows removal rates for various Best Management Practices. The column showing the Design Rate may be used to estimate for the project removal rate, unless there is a demonstration that a higher or lower value within the range of average removal rates should be used. Best Management Practices not listed in Table 7.1 may be used, provided acceptable data on removal efficiencies is provided. The 80% removal requirement applies to post-development conditions after the site is stabilized.

Table 7.1
Total Suspended Solids Removal Rates
(adapted from Schueler, 1996 & EPA, 1993)

Best Management Practice	Design Rate	Range of Average Removal Rates	Brief Design Requirements
Extended Detention Pond	70%	60-80%	Sediment forebay
Wet Pond (a)	70%	60-80%	Sediment forebay
Constructed Wetland (b)	80%	65-80%	Designed to infiltrate or retain
Water Quality Swale	70%	60-80%	Designed to infiltrate or retain
Infiltration Trench	80%	75-80%	Pretreatment critical
Infiltration Basin	80%	75-80% (predicted)	Pretreatment critical
Dry Well	80%	80% (predicted)	Rooftop runoff (uncontaminated only)
Sand Filter (c)	80%	80%	Pretreatment
Organic Filter (d)	80%	80%+	Pretreatment
Sediment Trap (Forebay)	25%	25% w/cleanout	Storm flows for 2 year event must not cause erosion; 0/1" minimum WQV storage
Drainage Channel	25%	25%	Check dams; non-erosive for 2yr.
Deep Sump and Hooded Catch Basin	25%	25% w/cleanout	Deep sump general rule = 4xpipe diameter or 4.0' for pipes 18" or less.
Mechanical Oil/Grit Separators	Varies	Varies	Pretreatment required
Street Sweeping	10%	10%	Discretionary non-structural credit must be part of approved plan.

Notes: a) Includes wet extended detention ponds, wet ponds.
b) Includes shallow marsh extended detention wetlands, pocket wetland, and pond/wetland design.
c) Includes surface, underground, pocket, and perimeter designs.
d) Includes compost, peat/sand, and biofiltration designs.
e) Mechanical oil/grit separator efficiencies vary by manufacturer.

Standard 5 - Land Uses With High Potential Pollutant Loads

Certain land uses generate higher concentrations of pollutants than found in typical stormwater runoff. These locations are called stormwater "hotspots", and are listed in Table 3.1.

Infiltration trenches, infiltration basins, and dry wells are prohibited within these "hotspots". Sand or organic filters, detention basins, wet ponds, or constructed wetlands may be used if they are sealed or lined.

Standard 6 - Critical Areas

Best Management Practices which are used for discharge to Critical Areas shall generally be limited to:

- Extended detention basins
- Wet ponds

- Constructed wetlands
- Water quality swales
- Sand filters
- Infiltration basins
- Infiltration trenches
- Deep sump and hooded catch basins (used with other Best Management Practices)
- Mechanical oil/grit separators

Stormwater management systems near public water supplies and other critical resources shall incorporate designs that allow for bypass and containment in the event of an emergency spill or other unexpected contamination event.

For the purpose of this standard, "near" means those locations where there is a strong likelihood of a significant impact occurring to a critical area, taking into account site-specific factors.

Standard 7 – Redevelopment

Redevelopment projects are defined as follows:

1. Maintenance and improvement of existing pavements, including widening less than a single lane, adding shoulders, correcting substandard intersections and drainage, and repaving; and

2. Development, rehabilitation, expansion, and phased projects on previously developed sites, provided the redevelopment results in no net increase in impervious area.

Components of redevelopment projects that include development of previously undeveloped sites do not fall under Standard 7.

Standard 8 - Erosion and Sedimentation Controls

Comprehensive sedimentation and erosion controls must be in place before any construction begins. Sedimentation and erosion controls shall be in conformance with Connecticut DEP Bulletin No. 34, "Connecticut Guidelines for Soil Erosion and Sediment Control."

Standard 9 - Operation and Maintenance Plans

An Operation and Maintenance Plan should, at a minimum, identify:

❶ Stormwater management system(s) owner (s);
❷ The party or parties responsible for operation and maintenance;
❸ A schedule for inspection and maintenance; and

❹ The routine and non-routine maintenance tasks to be undertaken.

The owner is generally considered to be the landowner of the property on which the stormwater management facility is located, unless other legally binding agreements are established with another entity.

7.5 Stormwater Management Checklist

Appendix B contains a stormwater management checklist that should be submitted.

Section 8 Site Planning Techniques

8.1 Introduction

A project component may comply with the Darien Stormwater Management Standards, by utilizing three basic methods, in the order shown below:

1. Utilize site planning techniques to minimize stormwater runoff from the development;

2. Utilize nonstructural techniques, such as pollution prevention and source reduction to minimize the extent of treatment the stormwater needs; and

3. Construct and maintain structural Best Management Practices to capture and treat the stormwater runoff.

Generally, the most cost-effective means to comply with the Stormwater Management Standards is through site planning and nonstructural approaches.

8.2 Site Planning

Site planning that integrates stormwater management with site development is the most effective approach to reduce and prevent potential pollution and flooding problems.

Site Planning Techniques That Minimize Runoff

Site planning techniques that will minimize the creation of new runoff and provide removal of some suspended solids include:

1. Minimization of Impervious Surfaces

   Replacing natural cover and soils with impervious surfaces increases runoff volume, velocity, and pollutant loads. Increased runoff may lead to flooding and channel erosion, which adversely affect long-term hydrology and natural systems.

   As the impervious surface area increases, the size and cost of the stormwater management facilities also increase.

   These site planning methods will minimize impervious surfaces and reduce the volume of runoff.

- Shorter road lengths.
- Permeable fill over parking areas.
- Smaller parking demand ratios.
- Smaller parking stalls.
- Subdivisions with open space.
- Shared parking and driveways.
- Narrower sidewalks.
- Minimize placement of new structures or roads over porous soils.
- Use shallow grassed roadside swales and parking lot islands instead of curbs.
- Utilization of turf pavers for overflow parking.

2. Fit the Development to the Terrain

Preserve natural drainageways by interrupting and bending the road grid around them. Construct grassed waterways, vegetated drainage channels, or water quality swales along street right-of-ways or on the back of lots to channel runoff without abrupt changes in the direction of flow. Site planning methods to fit development to the terrain include:

❶ Maintain natural buffers and drainageways.
❷ Minimize the creation of steep slopes.
❸ Maintain as much of the pre-development vegetation as possible.

3. Select Appropriate Best Management Practices

Infiltration techniques have very specific site and construction requirements. Site constraints, such as depth to groundwater, nearby septic systems, or wells, must be identified during the planning process so the technique will not fail, or cause the septic system or well to malfunction. Proper site planning will locate the most appropriate point on the site to direct the stormwater discharge.

Discharge points should be located on low slopes and stable soil, away from wetland edges. Infiltration trenches for surface runoff and dry wells for roof runoff should be used where appropriate. Separate collection and treatment of contaminated and uncontaminated runoff should be adopted. Using careful planning, a system of multiple technologies that collectively meet the Stormwater Management Standards should be designed. This approach will reduce the cost of stormwater management, and reduce long-term maintenance requirements.

Section 9 Best Management Practices

9.1 Non-Structural Practices

Nonstructural practices reduce the type and concentration of contaminants in stormwater runoff. These practices include materials management at industrial sites, fertilizer management in residential areas, erosion and sediment controls at construction sites, and comprehensive snow management. Since nonstructural practices reduce stormwater pollutant loads, the size and expense of stormwater management facilities can be reduced.

Non-Structural practices include:

1. Street and Parking Lot Sweeping

   A regular program of street sweeping removes pollutants that accumulate on road and parking lot surfaces. The effectiveness of sweeping is dependent upon the sweeping equipment, and the amount of pollutants present.

   Projects subject to the Stormwater Management Standards may incorporate a street sweeping plan, provided that sweeping will be carried out on a regular basis. A street sweeping program may receive a 10% credit towards the 80% removal standard for Total Suspended Solids

2. Catch Basin Cleaning

   Removing the accumulated sediment in catch basins from the winter months as soon as possible, before heavy and frequent spring precipitation.

3. Stream Buffers

   Stream buffers, such as undisturbed natural vegetation, protect the overall stream quality by providing shade for the stream and wildlife habitat. Buffers remove pollutants through infiltration and biofiltration and can provide a setback from the stream which may prevent damage due to flooding.

4. Disconnection of Rooftop Drainage

   The disconnection of rooftop drainage involves directing rooftop drainage toward pervious surfaces where it can filter through the grass or other vegetative cover or infiltrate into the ground. This practice may not be used for certain metal roofs, or where the disconnected runoff would result in erosion.

5. Turf Paving

Turf paving consists of concrete or polyethylene cells filled with pervious material to carry vehicular loading. The cells are usually covered with sod to conceal their appearance. This practice is best used for infrequently trafficked areas. Special consideration should be given regarding the proposed level of use and to winter freezing. Stormwater can infiltrate into the soil through the pervious voids of the cells.

9.2 Structural Practices

Numerous structural methods are available for meeting the Stormwater Management Standards. While a single practice may not meet the Stormwater Management Standards alone, a combination of practices may meet the standards.

Structural practices fall into the following basic categories:

- **Detention/Retention** Extended detention to removes pollutants from the stormwater.
- **Filtration** The pollutants in the stormwater are removed by filtering it through sand or other media.
- **Infiltration** The stormwater is temporarily stored and allowed to infiltrate into the native soil. Runoff from stormwater hotspots shall not be infiltrated.
- **Pretreatment** These practices alone do not meet stormwater management standards, but are critical for the maximum performance of other practices in that they help remove coarse particulates and debris that may adversely impact other stormwater structures.
- **Mechanical** These BMPs consist of specially designed units that employ mechanical methods to remove pollutants, such as oil/grit separators.

Table 9.1 on the following page summarizes the structural practices applicable to the categories listed above.

Table 9.1
Management Practices for Stormwater Management

Category	Practice	Description
Detention/Retention	Extended Detention Basin	Pond or basin which holds stormwater for 24 hours, allowing pollutants to settle out.
	Wet Pond	Pond or basin with a permanent pool of water to allow stormwater pollutants to settle out.
	Stormwater Wetlands	Constructed wetlands that utilize shallow pools and wetland vegetation for pollutant removal.
	Water Quality Swales	Swale with check dams to maximize pollutant removal.
	Underground Storage	Underground pipe or chamber which detains water for 24 hours.
Filtration	Sand Filter	Stormwater pollutants are removed by filtering the stormwater through a bed of sand.
Infiltration	Infiltration Trenches	Stormwater is detained underground and allowed to exfiltrate into the soil, removing pollutants.
	Infiltration Basins	Stormwater is impounded in a basin, and the impounded water infiltrates into the soil.
	Drywells	Excavated pit or underground chamber backfilled with aggregate for the infiltration of uncontaminated stormwater, such as that from roofs.
Pretreatment	Deep Sump Catch Basin	Catch basin with a deep sump to allow pollutants to settle out.
	Sediment Trap/Forebay	Small basin to reduce the velocity of stormwater runoff and allow solids to settle out.
	Drainage Channels	Channels to convey stormwater runoff, allows small quantities of runoff to infiltrate
Mechanical	Oil/Grit Separators	Structures which employ mechanical methods to remove pollutants from stormwater

9.3 Alternative Designs

New stormwater management technologies are constantly emerging. A management practice that is not listed in this manual, may be used provided that it is shown that the alternative design would meet the Darien Stormwater Management Standards. Final determination of the adequacy of the alternative design shall be by the Director, Department of Public Works.

9.4 Selection of Management Practices

When possible, the site planning and non-structural approaches presented in Section 8 and this section should be considered in the design prior to the adoption of structural approaches, because structural approaches cost more to construct and maintain. Each structural technology has limitations which make certain practices more desirable for a site, while making others less desirable for the same site.

Selection needs to consider the following:

❶ Watershed Factors
❷ Stormwater Treatment Suitability
❸ Physical Feasibility Factors
❹ Community Acceptance
❺ Locational and Permitting Factors

1. Watershed Factors

Certain watersheds may have specific constraints, such as an aquifer protection zone, a shellfish/beach protection zone, or may contain a watercourse designated as environmentally sensitive. Refer to Section 6 of this manual..

Table 9.2 on the following page describes the applicability of Best Management Practices for use in Critical Areas. Critical Areas are defined in Chapter 6.

Table 9.2
Best Management Practices for Critical Areas

BMP Group	Critical Area		
	Sensitive Watercourse	Aquifer Protection	Beach Shellfish
Detention/Retention	Additional storage may be required to prevent channel erosion.	Liner required for Group A Soils. Pretreat hotspots. 24"-48" vertical separation distance*	Provide 48 hours detention for maximum bacterial mortality. Consider designs to discourage geese.
Filtration	Useful for pretreatment.	Acceptable without exfiltration.	Acceptable.
Infiltration	Acceptable, depending upon soils.	Pretreat hotspots. Rooftop runoff may be infiltrated. 24" – 48" vertical separation distance*	Acceptable, provided 24" – 48" vertical separation distance is provided.
Pretreatment	Required in all cases.		
Mechanical	Pretreatment in all cases.		

* Separation is measured from the bottom of the structure to groundwater and/or bedrock.

2. Stormwater Treatment Suitability

Table 9.3 outlines the ability of the various Best Management Practices to meet the Darien Stormwater Management Standards for peak discharge control and recharge. If a certain practice does not meet a Stormwater Management Standard, that does not automatically eliminate its eligibility for use, but is an indication that additional practices may be needed.

Controlling a site's post-development hydrology can be achieved through impoundment and infiltration, or a combination of both. Controlled post-development runoff prevents downstream flooding and channel degradation.

Table 9.3 includes a general indication as to how much space a technique occupies, and of the ability of a specific practice to accept runoff from hotspots.

Table 9.3
Stormwater Treatment Suitability

Best Management Practice	Recharge Ability	Peak Control Ability	Space Requirements	Accept Hotspot Runoff
Extended Detention Basin	No	Yes	Large	Yes[1]
Wet Pond	No	Yes	Large	Yes[1]
Stormwater Wetland	Varies[2]	Yes	Large	Yes[1]
Water Quality Swale	Varies	Varies	Small	No
Underground Storage	Varies	Yes	Varies	Yes[1]
Sand Filter	Varies[2]	Varies	Varies	Yes[3]
Infiltration Trenches	Yes	Varies	Varies	Yes[1]
Infiltration Basin	Yes	Varies	Large	Yes[1]
Drywells	Yes	Yes	Small	Yes[1]
Deep Sump Catch Basin	No	No	Small	Yes
Sediment Trap/ Forebay	No	No	Varies	Yes
Drainage Channel	Yes	No	Small	Yes[3]
Mechanical Oil/Grit Separators	No	No	Small	Yes

Notes:
1. Not allowed unless runoff is pretreated to remove hydrocarbons, trace metals, and other toxicants.
2. May be provided by exfiltration.
3. Bottom of facility must be lined with impermeable filter fabric.

The peak flow control ability of a practice is dependent upon the design storm frequency. While some techniques may be effective at controlling more frequent events, they may be ineffective for larger events. Table 9.4 provides guidance regarding the adequacy of a particular technique to control the quantity of typical 25-year design storms and larger 100-year events. Infiltration of stormwater provides some degree of water quantity control.

Table 9.4
Stormwater Quantity Control

BMP	Peak Discharge 25 Year	Peak Discharge 100 Year	Volume Control Through Recharge
Extended Detention Basin	●	◉	◉
Wet Pond	●	◉	○
Stormwater Wetland	●	◉	○
Water Quality Swale	◉	◉	◉
Underground Storage	●	◉	◉
Infiltration Trench	◉	○	●
Infiltration Basin	◉	○	●
Drywells	◉	○	●
Sand Filters	○	○	◉
Sediment Trap/Forebay	◉	◉	◉
Drainage Channel	◉	○	○
Deep Sump Catch Basin	○	○	○
Oil/Grit Separator	○	○	○

● = Usually provided
◉ = Sometimes provided
○ = Rarely provided

3. **Physical Feasibility Factors**

Soils, water table, drainage area and slope also impact the suitability of a management practice. In general, detention/retention practices are applicable to a broad range of soil conditions, although wet ponds may be difficult to establish in dry, sandy soils. Infiltration practices are directly dependent upon the permeability of soils, and are not suitable for areas where the soil has very low permeability.

Water-table is critical to the performance of management practices which are dependent upon soil infiltration. High water tables act as a barrier to infiltration. High water tables, at or near the surface, tend to be more suitable for constructed wetlands, provided that the stormwater runoff is not from a hotspot.

Table 9.5 provides details regarding the suitability of Best Management Practices for selected site conditions. The limits given on drainage areas should be considered as guidelines. The limits may be increased or decreased depending upon groundwater availability, mechanisms that will be employed to prevent clogging, or the ability to assume an increased maintenance burden.

Table 9.5
Physical Feasibility

Best Management Practice	Soils	Min. Depth to Water Table (in)	Drainage Area (acres)	Slope Restrictions
Extended Detention Basin	Group A soils require liner. Group B soils require testing.	48" if hotspot or aquifer	10 Min.	None
Wet Pond	Group A soils require liner. Group B soils require testing.	48" if hotspot or aquifer	7 Min.	None
Stormwater Wetlands	Group A soils may require liner.	48" if hotspot or aquifer	7 Min.	None
Water Quality Swales	Not restricted	24"	Any	4% Max. Slope
Underground Storage	Not restricted, Except where infiltraton used	None, Except where infiltration used	Any	No More than 15%
Sand Filter	Not restricted	24"	10 Max.	None
Infiltration Trenches	$f \geq 0.52$* in/hr	48"	5 Max.	No More than 15%
Infiltration Basin	$f \geq 0.52$* in/hr	48"	10 Max.	No More than 15%
Drywells	$f \geq 0.52$* in/hr	48"	2 Max.	No More than 15%
Deep Sump Catch Basin	Not restricted	Not restricted	2 Max.	Not restricted
Sediment Trap/Forebay	Not restricted	Not restricted	5 Max.	Not restricted
Drainage Channel	Not restricted	24"	5 Max.	4% Max. Slope
Mechanical Oil/Grit Separator	Not restricted	Not restricted, anchoring may be required	Varies depending on unit size	None

* f = infiltration rate

Depth to bedrock should also be another consideration when locating and selecting management practices. In general, bedrock should be 2 feet or more below the bottom of any infiltration-based facility.

4. Locational and Permitting Factors

Certain areas of a site may be regulated, and the construction of stormwater management facilities within these areas may not be allowed. In other instances, implementation of certain practices will result in the creation of embankments. Embankments exceeding a certain height, or the creation of impoundments over a certain size may be subject to review by the Connecticut Department of Environmental Protection.

Table 9.6 provides guidance regarding location and permitting considerations

Table 9.6
Locational and Permitting Guidance

Wetlands	• Use of existing natural wetlands for stormwater treatment is prohibited. • BMPs within the 50-foot wetland setback are subject to regulation and restriction. The setback is 100 feet for Holly Pond, Gorham's Pond, the Noroton River, the Five Mile River, and the Goodwives River. • Wetland buffers may be used as a non-structural filter strip for sheet flow. • Discharges to wetlands must be treated.
Stream Channel	• In-stream ponds are prohibited.
100 Year Floodplain	• Placement of fill or grading within the 100 year floodplain is strongly discouraged. • Floodplain fill cannot raise the floodplain water surface elevation by more than 1 foot. This limitation is 0.01 feet on the Noroton River.
Utilities	• Placement of stormwater management facilities within utility easements is discouraged.
Roads	• Consult DPW or DOT for any right-of-way width requirement.
Septic Systems	• Infiltration facilities setbacks shall meet local and state health department codes.
Water Wells	• Infiltration facilities setbacks shall meet local and state health department codes. • A water diversion permit may be required for the use of well water to establish an impoundment.

Dam Safety	• Embankments of a certain height, or the creation of impoundments of a certain size may be subject to regulation by the Connecticut DEP.

9.5 Detention/Retention Facilities

Stormwater management facilities falling under the detention/retention category are designed to impound stormwater for a period of time. The pool of water allows pollutants to settle out. Embankments over a certain height, or the creation of impoundments of a certain size may be subject to a review may be required from the Connecticut DEP. In addition, all embankments should be constructed to provide a minimum of 1 foot of freeboard above design high water. Examples of management practices falling under the Detention/Retention category include:

❶ Extended Detention Basins
❷ Wet Ponds
❸ Stormwater Wetlands
❹ Water Quality Swales
❺ Underground Storage Facilities

1. Extended Detention Basin

Extended detention basins are similar to conventional dry ponds, except that the extended detention basin is designed to hold stormwater for a minimum of 24 hours. The holding of water has two benefits, first, it allows solids to settle out, and second, it reduces downstream flooding.

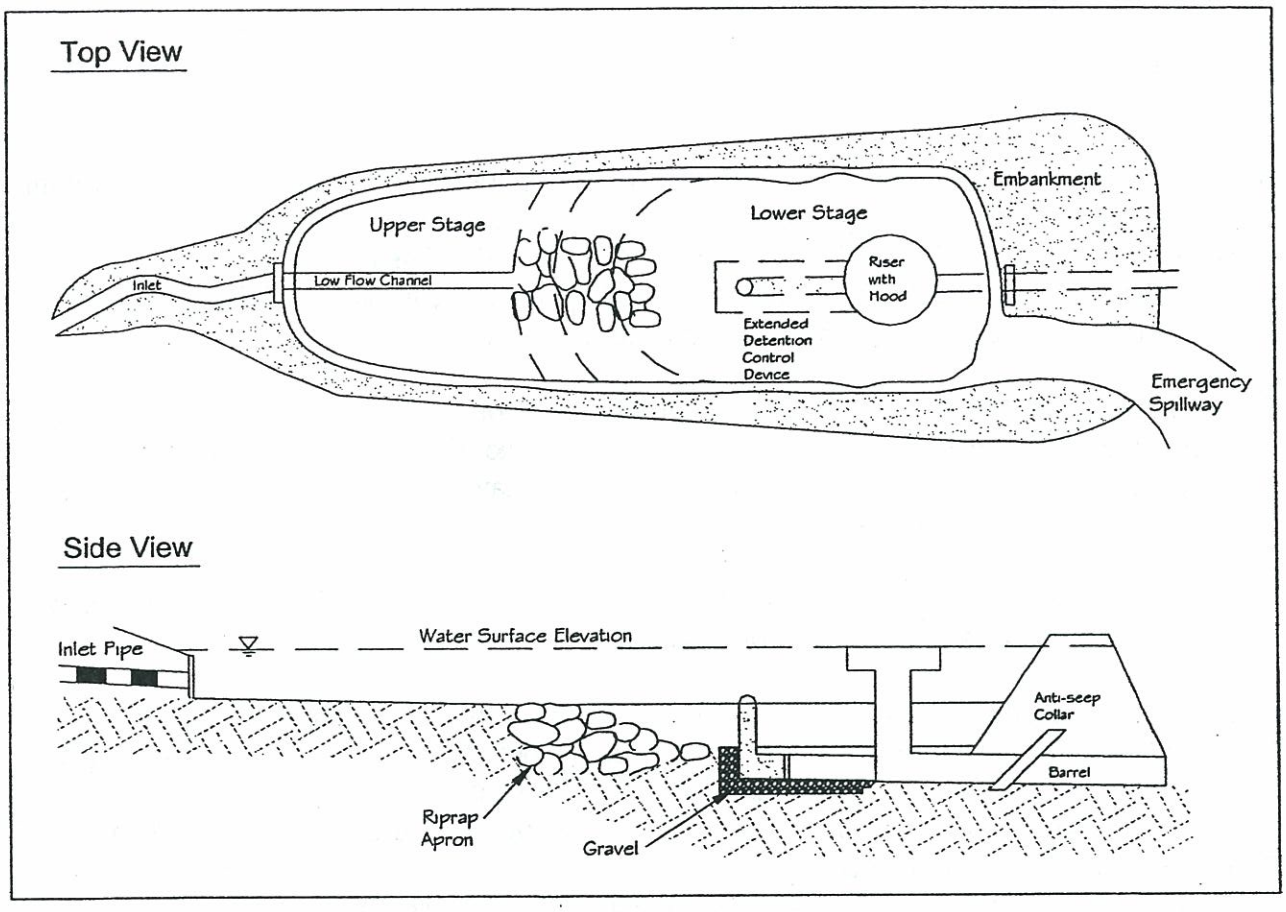
Additional amenities such as a micropool or shallow marsh may be included in the design to enhance pollutant removal.

Pretreatment of stormwater entering an extended detention basin is required to remove trash and debris.

An extended detention basin consists of a constructed embankment or excavated pit. These basins generally have two distinct stages. The upper stage has the capacity to regulate peak flows, providing stormwater quantity control for design storm events and remains dry under normal conditions. The lower stage is designed to hold stormwater long enough so that pollutants can settle out.

Figure 9.1 shows a typical schematic of an extended detention basin.

Figure 9.1
Extended Detention Basin Schematic



Advantages

- Relatively inexpensive
- Good retrofit option for existing basins
- Significant levels of sediments and absorbed pollutants removed.
- Potential for habitat
- Fewer hazards than deeper permanent pools

Disadvantages

- Negligible infiltration and groundwater recharge
- Minimal removal of soluble pollutants
- Requires large land area
- Maintenance is moderate to high
- May contribute to downstream warming
- Sediment may be resuspended after large storms if pond is not properly maintained or pretreated.

Design Guidelines

The minimum contributing area should be 10 acres. In general, four acres of drainage area are recommended for each acre-foot of storage in the basin.

Where bedrock is near the surface, extended detention basins may be infeasible due to the cost associated with rock excavation.

If an extended detention basin is situated on relatively impermeable soils, or in an area with a high water table, the basin may experience issues with standing water since the soil will not assist in drawing down the level of the basin. Wet detention basins may be more appropriate in these locations. Similarly, if the soils are highly permeable, it may be difficult to establish a shallow marsh component in the basin unless a liner is used.

Basins may not be located within a jurisdictional watercourse or wetland. Their use may be restricted within the wetland setback area. Basin embankments exceeding a certain height, or the creation of impoundments over a certain size may be subject to regulation by the Connecticut Department of Environmental Protection.

The separation from the basin bottom to the water table should not be less than 24 inches when it is situated within an aquifer protection zone.

The basin is typically located at the lowest point on a site, where its interception of stormwater is maximized. The design of a basin must account for both large infrequent storms for quantity control, and smaller, more frequent storms for quality control. Since the first flush of runoff contains the highest concentrations of pollutants, detention basins shall be designed to provide 24 hours of detention for storms of a recurrence interval of 2 years or less. The incorporation of multiple outlet structures allows longer detention times to be achieved for smaller storms. For larger storms, the peak outflow is typically limited so that the total peak runoff from the site is equal to or less than the pre-development peak level.

To maximize the removal of pollutants, flow paths should be as long as possible. One way to achieve this is to place the inlet and outlet points as far from each other as possible. Other methods to increase the flow path include the use of low-flow meandering pilot channels, or the use of baffles. Long, narrow basin configurations are preferred, with a minimum length to width ratio of 1.5:1. Irregular pond shapes with varying side slopes are preferred to give a more natural appearance. Shallow basins with large surface areas are generally more efficient for the purposes of pollutant removal than deep, compact basins. The low flow channel shall be adequately protected from erosion. The type of protection depends upon the volume and velocity of the runoff flowing through the channel.

Detention basin side slopes shall not exceed 3:1. The use of intermittent benches is recommended for safety and vegetation. Flatter slopes make maintenance and access easier,

and also promote the establishment vegetation while reducing erosion. It is recommended that the side slopes vary to give a more natural appearance.

Low flow underdrains, connected to the outlet structure or other downstream discharge point are recommended to dry the channel and basin bottom after the design detention period.

Inlet pipes should be placed so that they are not fully submerged when the basin is at normal levels. Inlets shall be designed with riprap or another type of energy dissipator, such as a sediment forebay. Likewise, the outlet for the pond shall be protected from erosion.

The outlet structure will be dependent upon the type of spillway used, basin configuration and extended detention outflow rate. The outlet should be designed to discourage clogging, and should be located such that it is easily accessible for maintenance. Provisions should be made for a draw-down valve which allows the entire basin to be drained within 24 hours. The draw-down valve shall be protected to prevent unauthorized draw-downs.

Establishing native wetland vegetation in a shallow marsh component or on an aquatic bench in the lower stage of the basin will enhance the removal of soluble nutrients, facilitate sediment trapping, reduce sediment re-suspension and provide wildlife and waterfowl habitat. Refer to the Appendix for a list of native wetland species. Where shallow marshes are provided, the multi-stage outlet should allow for a shallow pool of water to remain at all times. Woody vegetation shall not be placed on embankments.

Embankments shall have an emergency spillway capable of bypassing runoff from the 100-year storm without damaging the structure. Embankments shall have a minimum crest width of 10 feet for maintenance purposes.

Maintenance

Operation and maintenance plans shall identify the owner, as well as parties responsible for maintenance along with an inspection and maintenance schedule. Detention basins shall be inspected at least once per year to verify that they are operating as designed. Inspections should also be conducted during storm events to ensure that a particular basin can meet the required detention times. Necessary repairs shall be made immediately. Inspection shall examine, at a minimum, the following items:

❶ Subsidence
❷ Erosion
❸ Cracking or woody vegetation growth on the embankment
❹ Damage to the emergency spillway
❺ Sediment accumulation at the outlet
❻ Adequacy of erosion controls at the inlet and outlet
❼ Condition of the pilot channel
❽ Erosion within the basin and on the banks

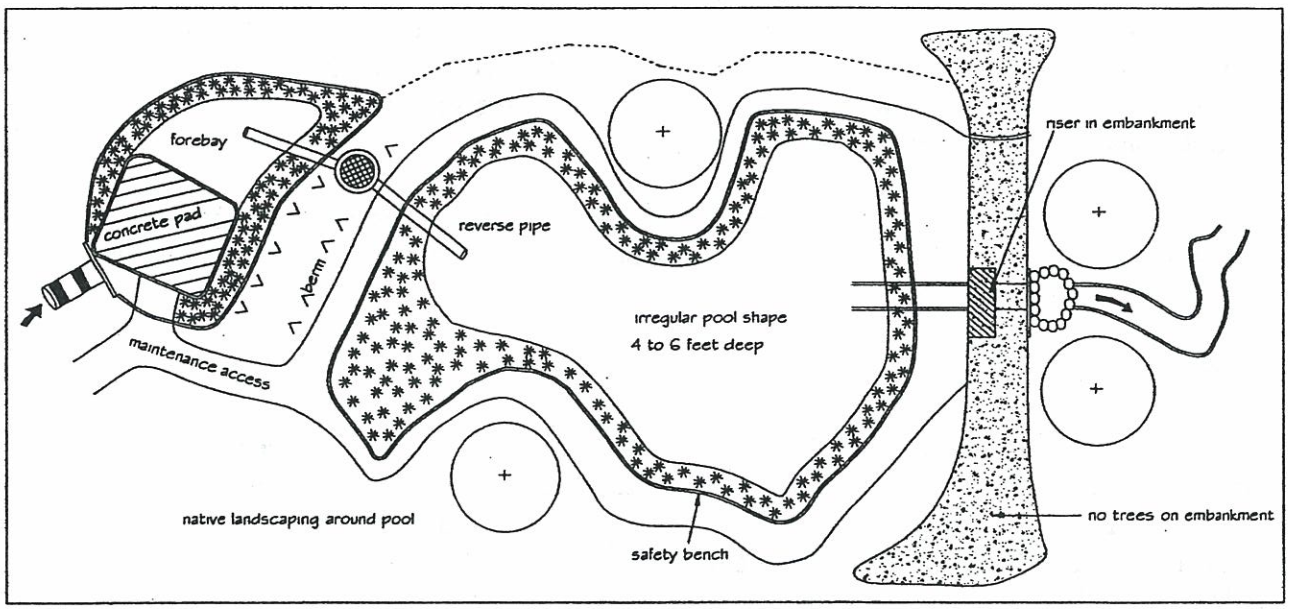
❾ Changes within the upstream watershed

Sediment should be removed from the basin as needed, and at a minimum of once every 10 years. Sediment in extended detention basins receiving runoff from hotspot areas may need to be tested prior to disposal. At least twice during the growing season, the upper stage, side slopes, embankment and emergency spillway shall be mowed, and accumulated trash and debris removed.

2. Wet Ponds

Wet ponds operate under the same general principle as extended detention basins, except that wet ponds use a permanent pool of water as opposed to a temporary one. The permanent pool allows the settlement of fine sediments. Wet ponds shall have additional dry storage capacity to control peak discharge rates, as shown in Figure 9.2.

Figure 9.2
Wet Pond Schematic



Wet ponds can be created with an embankment or through excavation, or by a combination of both. The primary element of a wet pond is the permanent pool of water, however, wet ponds can also incorporate other components such as a shallow marsh or sediment forebay to enhance pollutant removal.

Incoming stormwater runoff displaces water is already in the pool. This allows for an increased settling time that promotes the deposition of particulates, including fine sediments. The deep pool also protects the deposited settlements from re-suspension.

As with extended detention basins, wet ponds may be designed with multi-stage outlet structures to control discharges from various sized storms.

Advantages

- Can remove solid and soluble pollutants
- Aesthetically pleasing
- May have positive impact on property values
- Less frequent sediment removal necessary

Disadvantages

- More costly than extended detention basins
- Requires more land area than other stormwater management facilities
- Runoff volume control is limited
- Moderate to high maintenance
- May contribute to downstream warming.

Design Guidelines

Dry weather base flow is required to maintain the permanent pool elevation. Minimum contributing drainage area should be 10 acres, but not more than one square mile. Sites less than 10 acres may still be amenable to the use of wet ponds provided there is sufficient groundwater flow. In general, 4 acres of drainage area are recommended for each acre-foot of storage.

Wet ponds are particularly suitable for sites where nutrient loadings are expected to be high.

Soils, depth of bedrock, and depth of water table all need to be investigated before siting a wet pond. Where bedrock is close to the surface, the excavation cost may make the wet pomd economically infeasible. Permeable and well-drained soils may drain so quickly that it is difficult to maintain a permanent pool without a liner. Acceptable liner types include a 6-inch layer of clay, a polyvinyl liner, bentonite, or a 6-inch layer of silt loam.

A wet pond is typically located at the lowest point on a site, where its interception of stormwater is maximized. The design of ponds must account for both large infrequent storms for quantity control, and smaller, more frequent storms for quality control. For larger storms, the peak outflow is typically limited so that the total runoff from the site is equal to or less than the pre-development peak level.

Ponds may not be located within a jurisdictional watercourse or wetland. Their use may be restricted within the wetland setback area. Embankments exceeding a certain height, or the creation of impoundments over a certain size may be subject to regulation by the Connecticut Department of Environmental Protection.

To maximize the removal of pollutants, flow paths should be as long as possible. One way to achieve this is to place the inlet and outlet points as far from each other as possible. Long, narrow pond configurations are preferred, with a minimum length to width ratio of 2:1, with

4:1 being preferred. Irregular pond shapes with varying side slopes are preferred to give a more natural appearance. Intermittent benches provide safety and promote vegetation. A safety bench shall be a minimum of 10 feet wide and shall be located above normal pool elevations. An aquatic bench shall be a minimum of 10 feet wide and shall be 12 to 18 inches below the normal pool elevation.

Pool depth is critical for sediment deposition. An average depth of 3 to 6 feet is recommended. Re-suspension of deposited sediments is problematic in ponds shallower than 24 inches. Depths exceeding 8 feet may result in thermal stratification, which contributes to anoxic conditions. Shallow depths at the inlet concentrate sediment deposition where it is more accessible, while greater depths at the outlet mitigate downstream thermal loading.

The recommended minimum pool surface area is 0.25 acres. Pollutant removal performance of a wet pond may be enhanced by increasing the surface area as opposed to the depth. Increased surface area tends to increase temperatures and increase evaporation rates.

The inlet pipe invert should be set at or below the surface of the permanent pool. Pipes discharging above the pool surface may cause erosion. Inlets shall be designed with riprap or another type of energy dissipator, such as a sediment forebay.

The outlet pipe should be placed approximately 12 inches below the normal pool elevation. The outlet should be designed to discourage clogging, and should be located such that it is easily accessible for maintenance. Provisions should be made for a valve to allow for complete pond draw-down for emergency situations or maintenance. The draw-down valve shall be protected to prevent unauthorized draw-downs. The outlet discharge point shall be protected from erosion through the use of an energy dissipator.

Embankments shall have an emergency spillway capable of bypassing runoff from very large storms without damaging the structure. Embankments shall have a minimum crest width of 10 feet for maintenance purposes.

Wet ponds should not be utilized to treat runoff generated from construction activity.

Maintenance

Operation and maintenance plans shall identify the owner, as well as parties responsible for maintenance along with an inspection and maintenance schedule. Wet ponds shall be inspected at least once per year to verify that they are operating as designed. Inspections should also be conducted during storm events to ensure that a particular pond can meet the required detention times. Necessary repairs shall be made immediately. Inspection shall examine, at a minimum, the following items:

❶ Subsidence
❷ Erosion

❸ Cracking or woody vegetation growth on the embankment
❹ Damage to the emergency spillway
❺ Sediment accumulation at the outlet
❻ Adequacy of erosion controls at the inlet and outlet
❼ Growth of deleterious aquatic vegetation
❽ Erosion within the pond and on the banks
❾ Changes within the upstream watershed

Sediment should be removed from the basin as needed, and at a minimum of once every 10 years. Sedimentation in wet ponds basins receiving runoff from hotspot areas may need to be tested prior to disposal. At least twice during the growing season, the upper stage, side slopes, embankment and emergency spillway shall be mowed, and accumulated trash and debris removed.

3. Constructed Stormwater Wetlands

Constructed stormwater wetlands are designed to maximize the removal of pollutants from stormwater runoff through biological uptake, retention, and settling. The constructed wetlands temporarily store runoff in shallow pools. Constructed wetlands may employ sediment forebays.

Stormwater wetlands shall not be located within natural wetlands, as they do not have the full function of natural wetlands.

There are four different types of stormwater wetlands:

❶ Shallow Marsh Systems	Consist of a series of pools ranging from 6 to 18 inches in depth. The pools are connected by winding pathways to increase detention times and contact area. See Figure 9.3a.
❷ Pond/wetland systems	Utilize a wet pond in conjunction with a shallow marsh. The wet pond removes particulates and reduces runoff velocity. The shallow marsh provides biological uptake for soluble pollutants. Requires less space than shallow marsh systems. See Figure 9.3b.
❸ Extended Detention Wetlands	Substitute vertical storage for shallow marshes. The vertical storage allows for greater runoff detention. Requires less space than shallow marsh systems, but wetland vegetation must be able to tolerate intermittent flooding and dry periods. See Figure 9.3c.

❹ Pocket Wetlands

Utilized for sites from 1 to 10 acres, where adequate groundwater exists. This type of wetland is a smaller version of the shallow marsh system, containing just a single cell and a single winding pathway from the inlet to the outlet. Sensitive to changes in upstream development. See Figure 9.3d.

Figure 9.3a
Shallow Marsh System Schematic

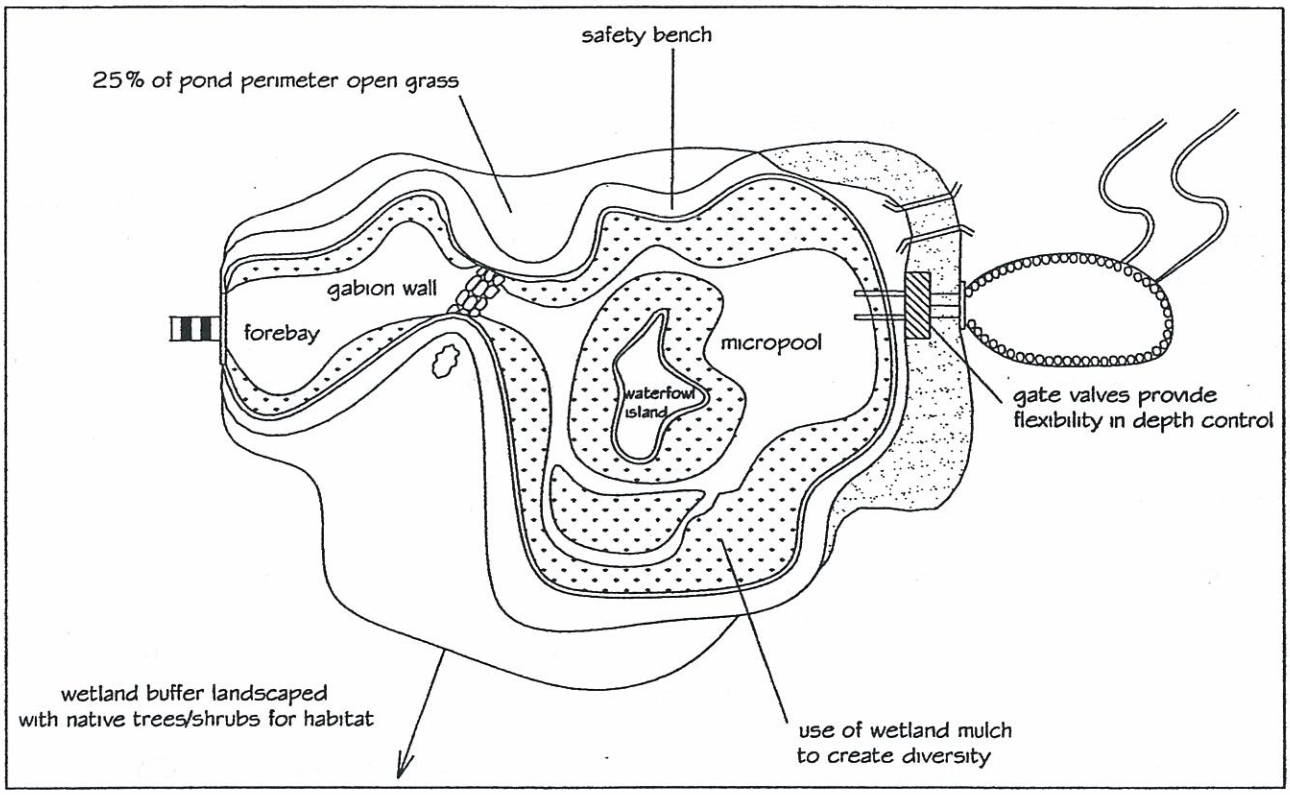


Figure 9.3b
Pond/Wetland System Schematic

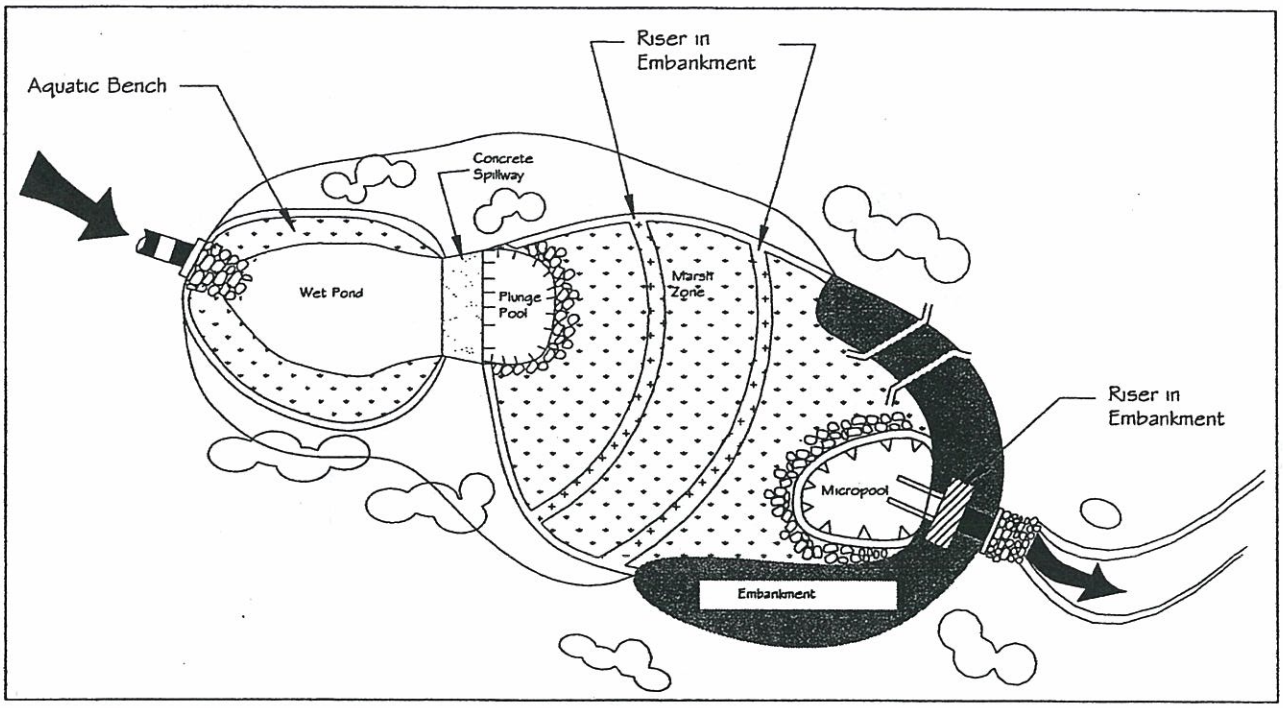


Figure 9.3c
Extended Detention Wetlands Schematic

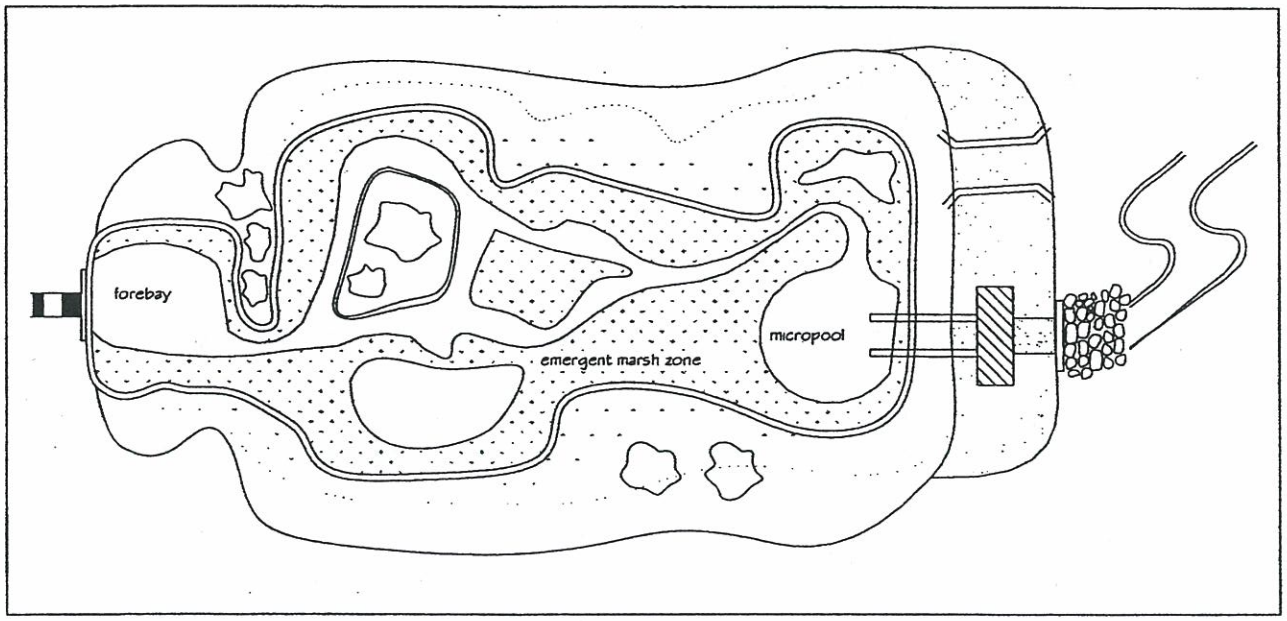
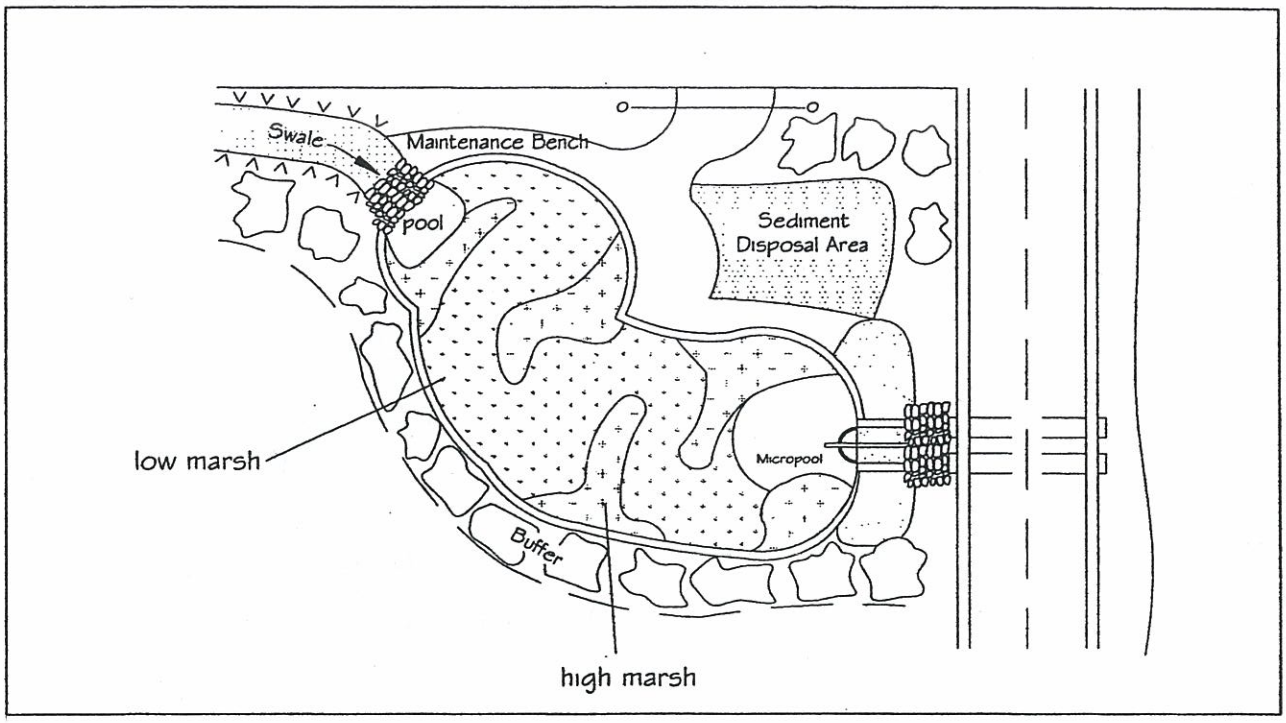


Figure 9.4d
Pocket Wetlands Schematic



Advantages

- Relatively low maintenance costs
- High pollutant removal efficiency.
- May have aesthetic benefits.

Disadvantages

- Large consumers of space
- Vegetation must be well-established for full benefit.
- Relatively high construction cost

Design Guidelines

Stormwater wetlands are inappropriate to be used to treat runoff from construction activity.

The use of constructed wetlands may be limited by soil type, groundwater depth, drainage area, and available land area. Several different types of wetlands may be used in combination.

Soil type, depth to bedrock and depth to groundwater are all important considerations. Medium to fine textured soils are better to establish vegetation, retain surface water, permit groundwater discharge and capture pollutants. Where the soils are well-drained, an impermeable liner may be required. Liners may also be required where the potential for groundwater contamination is high, such as for stormwater runoff from hotspots. Bedrock

close to the surface may result in high excavation costs that could make construction economically infeasible.

Stormwater wetlands generally have slightly better removal efficiencies when compared to pond systems. The pond/wetland system is the most effective of the four wetland practices. Pocket wetlands are the most sensitive, and may be subject to particulate re-suspension, especially when forebays are not used.

Since stormwater wetlands generally employ shallow depths which do not store much volume, peak reduction is best accomplished by using another management practice.

Wetland designs require pondscaping plans, which detail the location, quantity and propagation methods for stormwater plants, and a water budget analysis to ensure that the proper pool elevations will be maintained, and that the wetland will not be allowed to dry for more than a 2 month period. Refer to the Appendix for a list of native wetland species.

Wetland systems have distinct depth zones, as follows:

❶ Deepwater Zone	From 1.5 feet to 6 feet deep, supports submerged or floating vegetation. May be further divided into micropool, forebay, and deepwater components.
❷ Low Marsh Zone	6 to 18 inches deep, suitable for several emergent wetland species.
❸ High Marsh Zone	From 0 to 6 inches deep. Supports a broader range of wetland species than low marsh zone, and typically has a higher surface area to volume ratio than a low marsh zone.
❹ Semi-Wet Zone	Areas above the permanent water level that may be inundated to the extent that they can support wetland vegetation.

Table 9.7 shows the recommended allocation of surface area amongst the depth zones for the four wetland systems to meet Darien Stormwater Standards.

Table 9.7
Stormwater Wetland Zone Allocations

Target Allocation	Wetland Type			
	Shallow Marsh	Pond/Wetland	ED Wetland	Pocket Wetland
% of Surface Area				
Forebay	5	0	5	0
Micropool	5	5	5	0
Deepwater	5	40	0	5
Low Marsh	40	25	40	50
High Marsh	40	25	40	40
Semi-Wet	5	5	10	5
% of Treatment Volume				
Forebay	10	0	10	0
Micropool	10	10	10	0
Deepwater	10	60	-	20
Low Marsh	45	20	20	55
High Marsh	25	10	10	25
Semi-Wet	0	0	50	0

A wetland should be designed so that the ratio of its length to width is approximately 2:1. Cells should be located such that the interconnection between cells is as long and winding as possible.

Each wetland should have a separate cell at the inlet to function as a sediment forebay, with a minimum capacity of 10% of the water quality volume. The forebay should be 4-6 feet deep and be accessible for clean-out. Outlets should have a micropool, meeting the same requirements to prevent clogging. Generally, the outlet should be located 12 inches below the normal pool surface. Provisions should be made for complete draw-down for maintenance and emergencies. The draw-down valve shall be protected to prevent unauthorized draw-downs.

Deepwater cells require a 10-foot wide safety bench all around, and shall be no deeper than 18 inches below the normal pool elevation.

Embankments exceeding a certain height or the creation of impoundments over a certain size may be subject to regulation by the Connecticut Department of Environmental Protection.

Maintenance

Operation and maintenance plans shall identify the owner, as well as parties responsible for maintenance along with an inspection and maintenance schedule.

Wetlands require considerable routine maintenance, but do not require intense activities such as sediment removal, like ponds and basins. Inspections shall be performed at six month intervals to verify that they are operating as designed. Inspections should also be conducted during storm events to ensure that a particular wetland can meet the required detention times. Necessary repairs shall be made immediately. Inspection shall examine, at a minimum, the following items:

❶ Types and distribution of dominant plants
❷ Types and distribution of invasive species
❸ Percentage of unvegetated standing water
❹ Stability of original depth zones
❺ Accumulation of sediment in the forebay and micropool
❻ Survival rate of wetland buffer plantings

Sediment shall be removed from forebays yearly. In the case of a wetland/pond system, pond maintenance procedures shall be followed for the pond component.

4. Water Quality Swales

Water quality swales are designed as a water quality control technique, whereas drainage channels are primarily designed for conveyance purposes. Water quality swales include dry swales, wet swales, and grassed swales, sometimes known as biofilters.

Figures 9.4a, 9.4b and 9.4c show schematics of dry swales, wet swales and grassed swales, respectively.

Figure 9.4a
Dry Swale

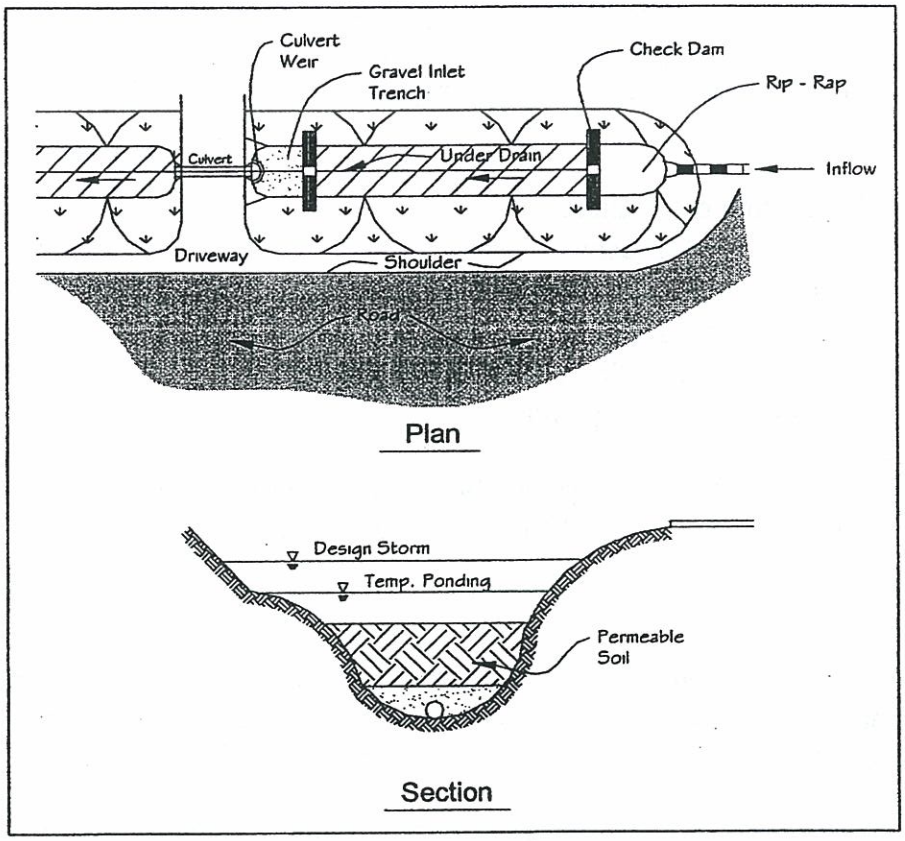


Figure 9.4b
Wet Swale Schematic

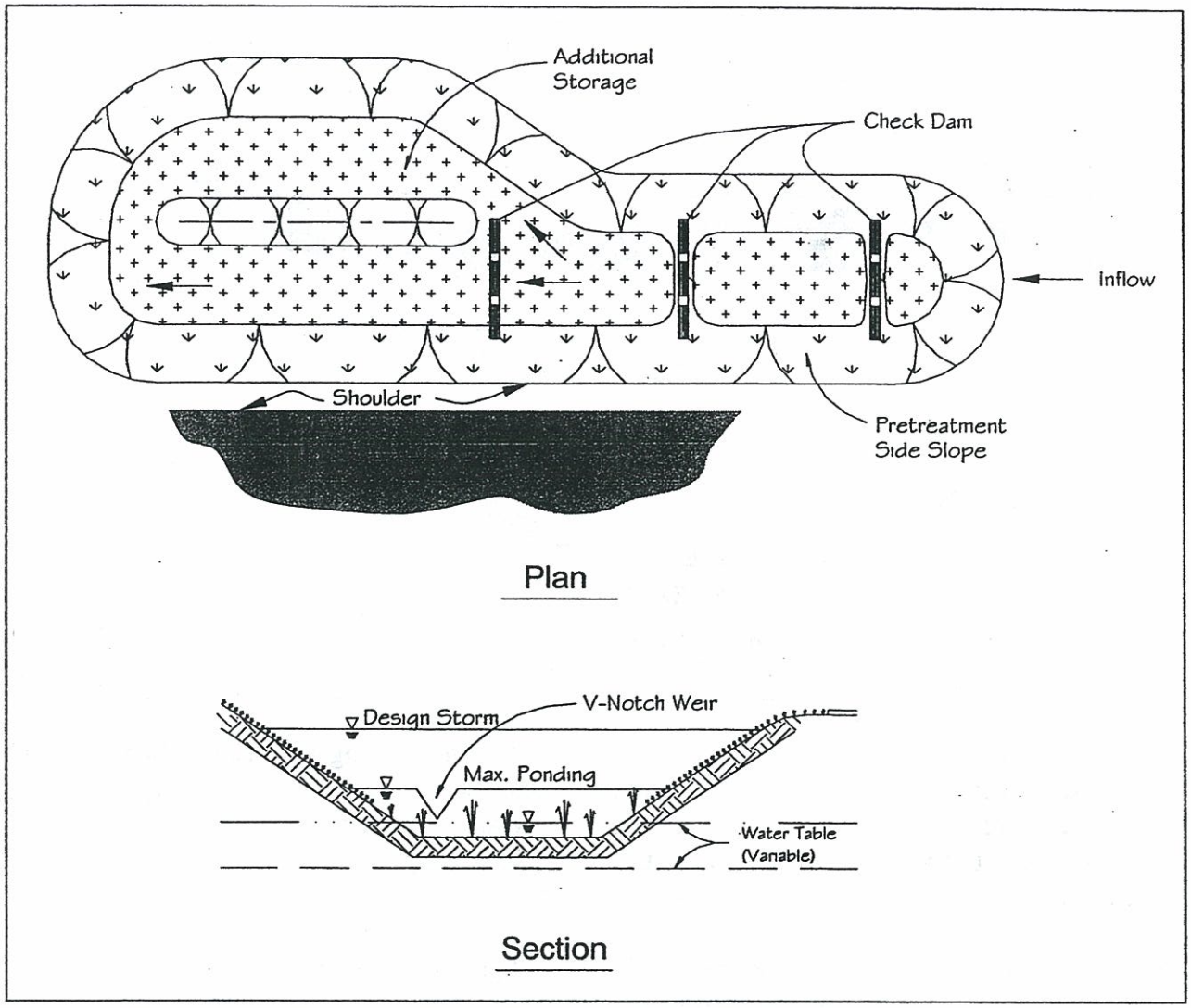
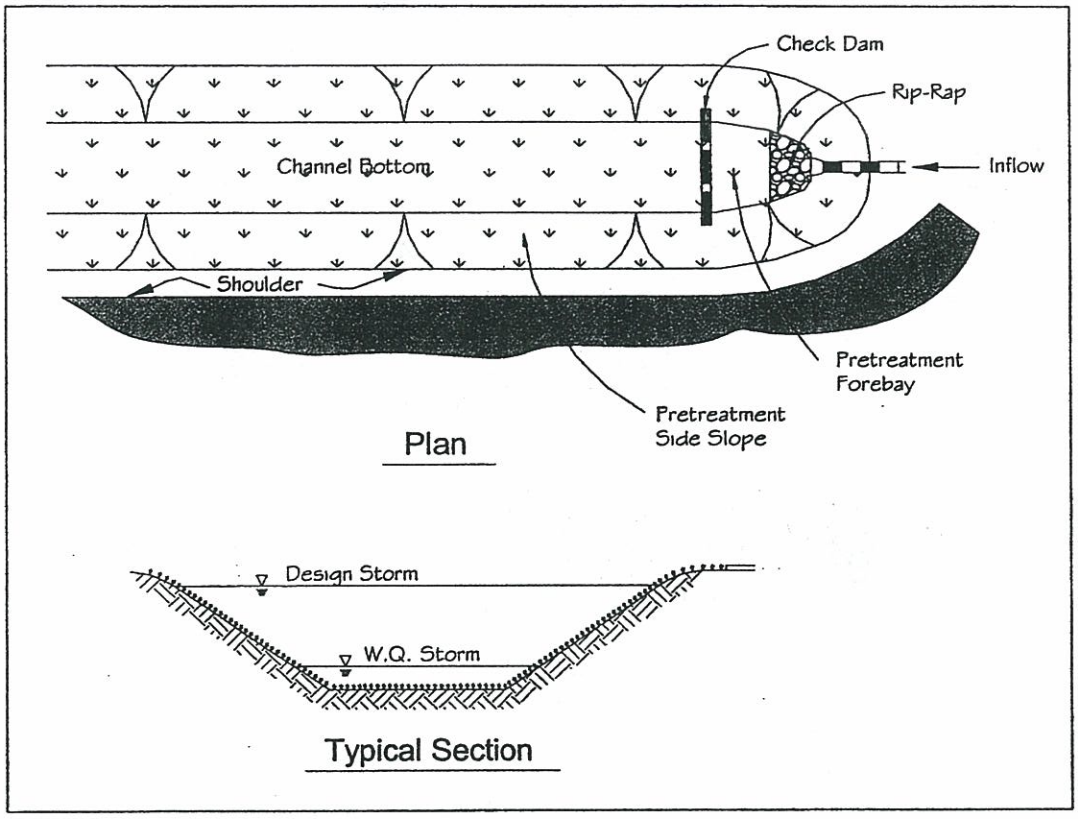


Figure 9.4c
Grassed Biofilter Swale



Swales provide control for certain peak runoff rates by impounding stormwater behind check dams and reducing erosive velocities. Impoundment of the runoff also provides for some volume control through infiltration, while allowing pollutants to settle out through sedimentation, filtration, nutrient uptake and infiltration. Dry swales tend to rely more upon infiltration while wet swales rely on sedimentation and biological uptake.

Advantages

- Reduces peak volumes and provides volume control.
- Provides pretreatment
- Less expensive construction than curb and gutter systems

Disadvantages

- Requires more maintenance than curb and gutter systems
- Roadside swales may be subject to damage

Design

Swales are most appropriate in low to moderate density developments. The percentage of impervious cover contributing to an individual swale should be relatively small. Roadside swales may not be appropriate where there are many driveway and/or sidewalk crossings.

Swales may be used in conjunction with other stormwater management practices.

When a swale is vegetated, the plants should be selected so that they are suited to the soil and water conditions.

Soil conditions will have an impact on the effectiveness of a particular swale. Porous soils lend themselves well to the use of dry swales and biofilter swales, while wet swales are better geared toward more poorly drained soils.

Dry swales should be sized to infiltrate the Water Quality Volume through the bottom of the swale. The soil bed should be a minimum of 30 inches deep, composed of 50% sand and 50% loam. Pretreatment is required, since this management practice relies upon infiltration. Pretreatment can be in the form of a sediment forebay or a mechanical oil/grit separator. Underdrains may be incorporated into the design to ensure complete drainage. Cross-sections are typically trapezoidal, with side slopes not exceeding 3 horizontal to 1 vertical. Bottom width vary depending upon the volume of water treated and conveyed, but are typically between 2 and 8 feet. Swales should be designed to convey the 25-year design storm. Groundwater should not be within 2-4 feet of the swale bottom. As with other management techniques, the inlet and outlet must be protected from erosion and scour.

Wet swales, like dry swales, should be sized to treat the Water Quality Volume and convey the 10-year storm event. They should be used where the water table is high, and where the soil is poorly drained, so that the bottom of the swale is constantly wet. Pretreatment is required, and is usually in the form of a sediment forebay or mechanical oil/grit separator. Check dams may be used to achieve multiple cells. Shape and sideslope requirements are the same for dry swales. Bottom widths shall be from 2 to 8 feet. Outlets shall be properly protected from erosion and scour. The bottom of the swale should be planted with emergent vegetation.

Grassed swales shall be sized to treat the Water Quality Volume, and convey the 25 year storm event. Grassed swales shall be trapezoidal in shape, with sideslopes not exceeding 3 horizontal to 1 vertical. Bottom widths shall not exceed 8 feet. Soils should be sandy loam, with 10 to 20 percent organic matter. Clay content shall not exceed 20%. Grass species shall produce fine, dense and uniform cover that can survive moisture conditions. Outlets shall be properly protected from scour and erosion.

Maintenance

Operation and maintenance plans shall identify the owner, as well as parties responsible for maintenance along with an inspection and maintenance schedule.

Inspections shall be performed at six month intervals to verify that the swale is operating as designed. Inspections should be done at more frequent intervals during the first year of action to ensure that the vegetation establishes itself.

Swales should be mowed a minimum of twice per year, and grass clippings should not be deposited within the swale. The grass shall not be cut to less than four inches in height.

Sediment and debris shall be removed manually, at a minimum of once per year.

5. Underground Storage Facilities

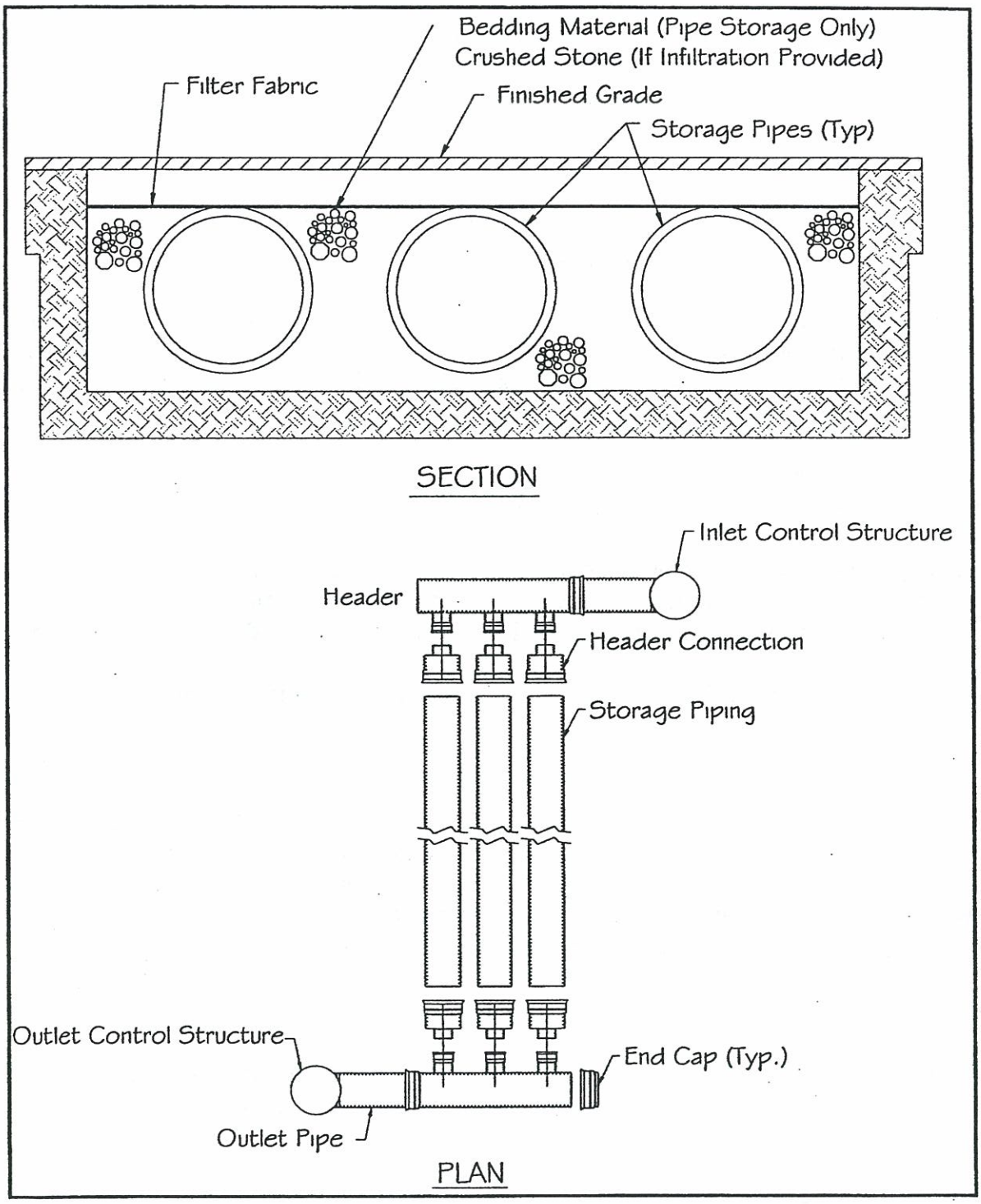
Underground detention facilities are similar to conventional dry ponds, except that the water is stored below grade for stormwater quantity control. The underground storage facility should hold stormwater for a minimum of 24 hours. The holding of water has two benefits, first, it allows solids to settle, and second, it reduces downstream flooding.

Pretreatment for these systems shall be provided, and they shall be cleaned frequently.

The underground storage is usually accomplished with pipes, chambers, or box culvert sections.

Figure 9.5 shows a typical schematic of a typical underground storage facility.

Figure 9.5
Underground Storage Schematic



Advantages

- Provides quantity control
- Does not consume surface area
- Fewer hazards than deeper permanent pools

Disadvantages

- Requires underground area
- Maintenance cost is moderate to high

Design Guidelines

Where bedrock is near the surface, underground storage facilities may be economically infeasible due to the cost of rock excavation.

Underground storage facilities can be designed for volume control as well, if they employ a system for infiltrating some of the runoff into the ground. When designed to provide infiltration, the underground facility operates as an infiltration trench, as described in Section 9.6, and the criteria for that type of practice shall apply.

Underground storage facilities may not be located within a jurisdictional watercourse or wetland. Their use may be restricted within a wetland setback area.

An underground storage facility should have a multi-stage outlet control structure to accommodate a variety of storms. For larger storms, the peak outflow is typically limited so that the total peak runoff from the site is equal to or less than the pre-development peak level.

All underground storage facilities shall have access ports for maintenance and inspection.

Maintenance

Operation and maintenance plans shall identify the owner, as well as parties responsible for maintenance along with an inspection and maintenance schedule. Underground storage facilities shall be inspected at six month intervals to verify that they are operating as designed. Inspections should be conducted during storm events to ensure that a particular facility can meet the required detention times. Necessary repairs shall be made immediately.

9.6 Infiltration Facilities

Infiltration facilities infiltrate impounded runoff into the soil to meet the recharge volume and/or water quantity requirements. Infiltration facilites remove both soluble and particulate pollutants.

Examples of infiltration facilities include:

❶ Infiltration Trenches
❷ Infiltration Basins

❸ Drywells

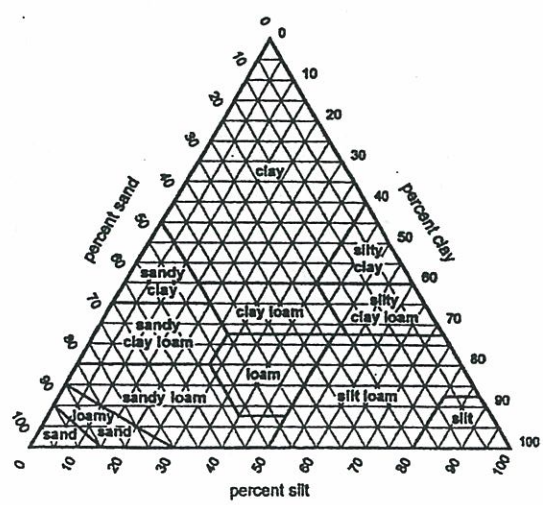
Table 9.8 shows infiltration rates for various soil textures. The values presented in this table are intended for planning purposes only. The design of infiltration facilities on any given site shall be based upon the results of field testing to determine actual infiltration rates. Generally, soils with infiltration rates of less than 0.52 in/hr are unsuitable for infiltration systems.

Table 9.8
Soil Infiltration Rates

Texture Class	Minimum Infiltration Rate (inches/hour)	Hydrologic Soil Grouping
Sand	8.27	A
Loamy Sand	2.41	A
Sandy Loam	1.02	A
Loam	0.52	B
Silt Loam	0.27	B
Sandy Clay Loam	0.17	C
Clay Loam	0.09	D
Silty Clay Loam	0.06	D
Sandy Clay	0.05	D
Silty Clay	0.04	D
Clay	0.04	D

The texture class descriptions in Table 9.8 are based upon the United States Department of Agriculture Soil Textural Triangle, shown in Figure 9.6.

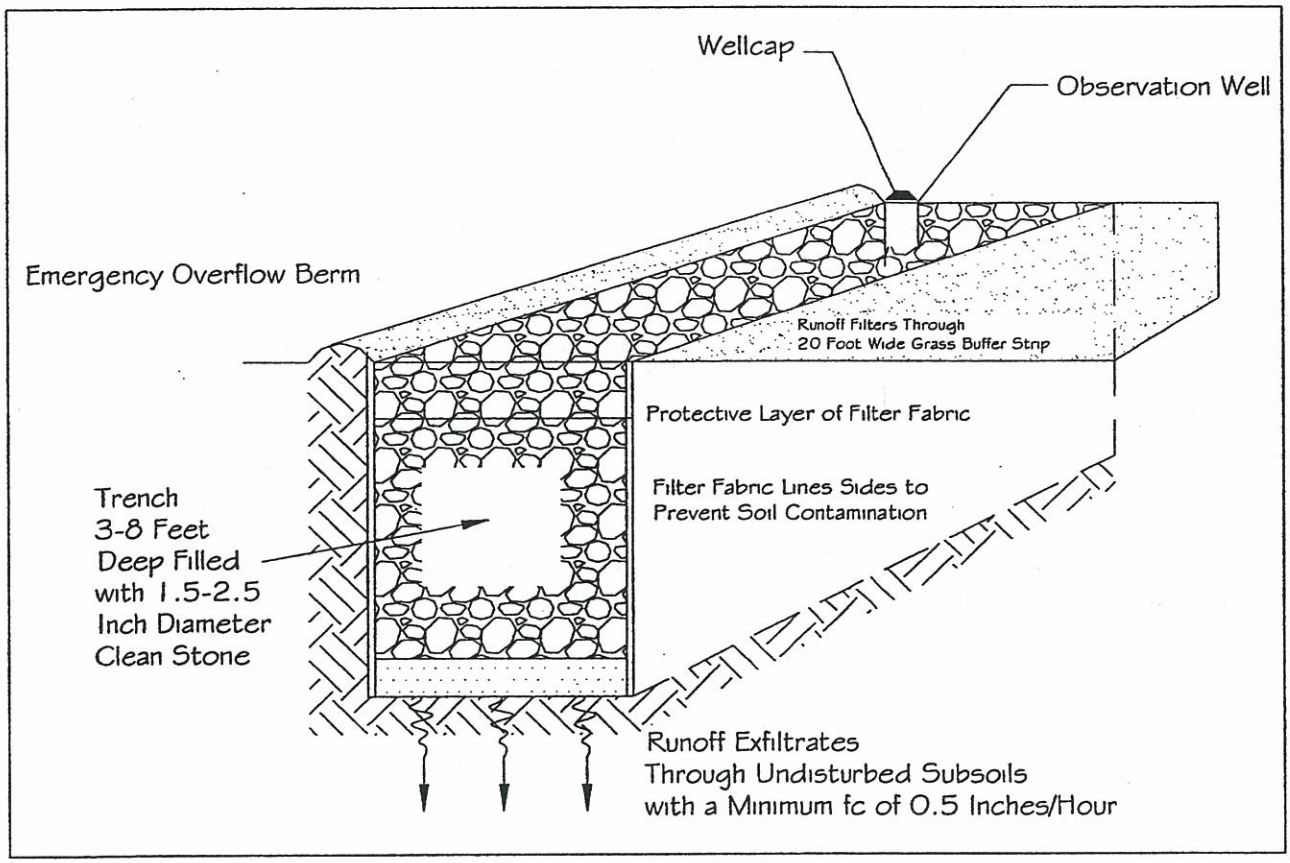
Figure 9.6
USDA Soils Textural Triangle



1. Infiltration Trenches

Infiltration trenches consist of shallow, linear excavations filled with crushed stone, and may employ perforated pipe. The voids in the crushed stone create an underground impoundment for stormwater runoff. The impounded runoff slowly filters through the bottom of the trench into the soil. Figure 9.6 illustrates a typical infiltration trench.

Figure 9.6
Schematic of Infiltration Trenches



Infiltration trenches may be used in combination with other stormwater management practices. A portion of the runoff impounded within an infiltration trench may be routed to other facilities. Such combination facilities are designed to meet the Recharge Volume, while relying on other techniques for water quantity control. One method of partial infiltration is an off-line method that routs the "first flush" of the runoff into the infiltration trench through an overflow weir. A second method is an on-line method that employs a perforated pipe at the top of the trench. Once the trench is filled to capacity, the excess runoff discharges into other stormwater management facilities.

Full infiltration systems, where the system is designed to store and infiltrate the entire runoff from a storm, provide water quantity control and water quality control for the prescribed Water Quality Volume. Full infiltration systems shall be equipped with an emergency overflow channel to convey storms exceeding the design storm. The application of full infiltration systems may be limited by physical or economic constraints. Generally, full infiltration systems are not used to provide storage for large infrequent events, such as the 100-year storm.

Advantages

- Promotes groundwater recharge
- Reduces downstream flooding
- Preserves the natural site water balance
- Provides a high degree of runoff pollution control if properly maintained
- Reduces the size and cost of downstream stormwater management facilities and storm drainage systems

Disadvantages

- Failure rates often high
- Use is generally restricted to small drainage areas
- May cause groundwater contamination
- Frequent maintenance required
- Prone to clogging by sediment

Design

Infiltration trenches are appropriate at sites with permeable soils, gentle slopes, and where both bedrock and groundwater are a minimum of 48 inches below the surface. The bottom of the trenches shall be a minimum of 24 inches above groundwater or bedrock. Contributing drainage areas should not exceed 5 acres. Infiltration trenches are suitable for runoff from parking lots, rooftops, and small residential developments. They shall not be used to treat runoff from stormwater hotspots. In water supply recharge areas, infiltration trenches shall be limited to runoff from uncontaminated rooftops. Generally, it is most practical to separate contaminated runoff from uncontaminated runoff and treat each with the appropriate management practices.

Pretreatment of runoff is a prerequisite to infiltration trenches since they do not remove coarse particulates. Infiltration trenches have a high failure rate, which is mostly attributable to clogging by particulates. Once clogged, they are difficult to repair.

Soils should have a minimum percolation rate of 0.5 inches per hour, and longitudinal slopes should not exceed 5%. Infiltration trenches should not be used at sites where soils have a 30% or greater clay content or greater than 40% silt/clay content. Trenches should not be placed over fill materials. Trenches should be sited so that they are convenient for maintenance activities.

If designing for water quantity control, the maximum drain time should be 72 hours, with a minimum retention time of 48 hours.

Since an infiltration trench is filled with stone, only the void space and the space within the perforated pipe is available for storage. Typically, the stone should be clean and washed, with a size ranging from 1.5 to 3 inches. The soil infiltration rate should be determined by a geotechnical study, with a minimum of two borings taken for each infiltration trench, and an additional boring for each 50 foot segment of trench in excess of 100 feet. The infiltration design rate should be based upon the most restrictive infiltration rate obtained from the study.

The lowest part of the trench shall be 24 inches above seasonal high groundwater or bedrock, and below the frost line. The maximum depth of the trench shall be governed by the following equation:

$$d_{max} = \frac{fT_s}{\mu} \qquad \textbf{[Eq. 9.1]}$$

Where d_{max} = Maximum depth of trench, ft
T_s = Maximum storage time, hours
f = Soil infiltration rate, in/hr
η = Void ratio of stone

A 20-foot vegetated buffer around surface trenches is recommended. Filter fabric should be placed 6 to 12 inches below the surface, above the trench, along the sides, and at the bottom to help prevent clogging. Trenches should have inspection ports with a lockable above ground cap.

During construction, heavy equipment shall not operate on the footprint of the trench, as compaction will adversely impact infiltration capability. Infiltration trenches are inappropriate for runoff controls during construction activity.

Maintenance

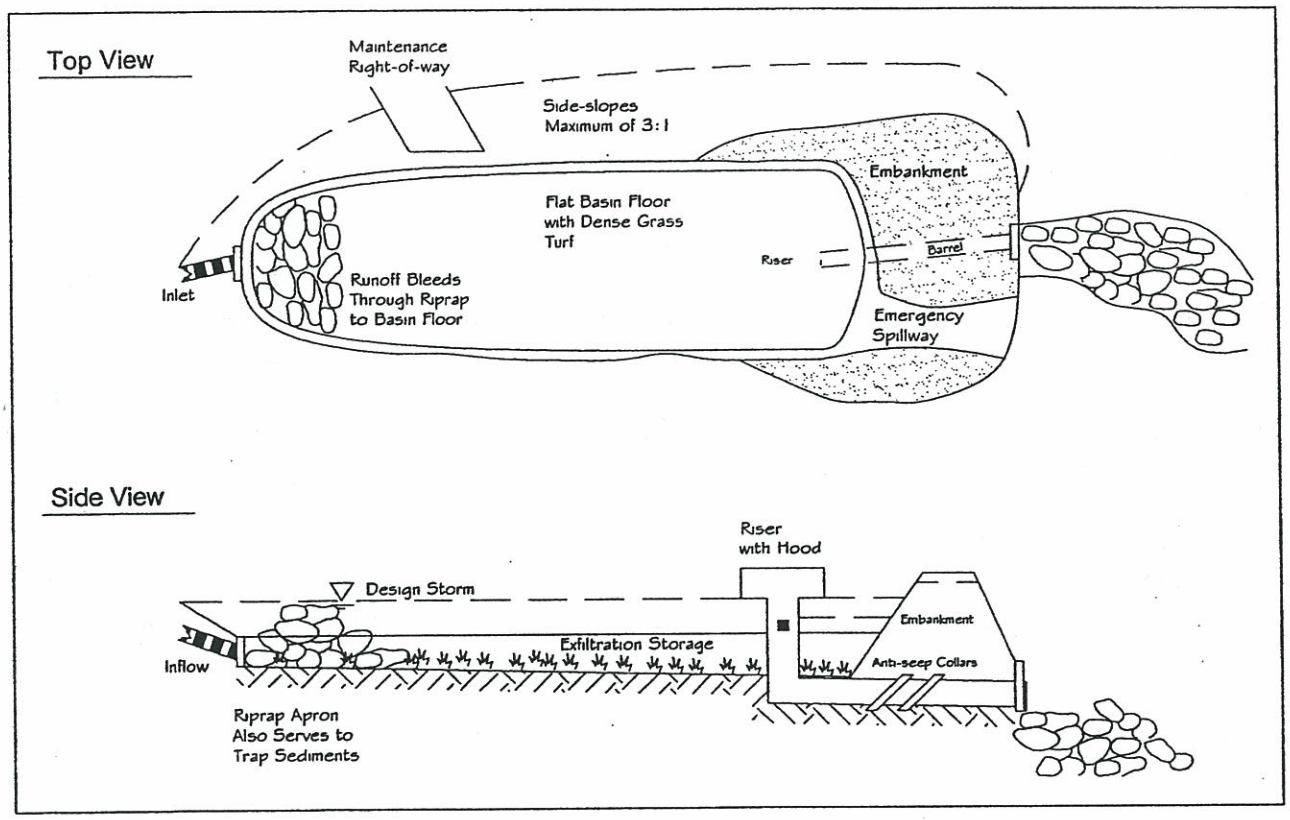
Operation and maintenance plans shall identify the owner, as well as parties responsible for maintenance along with an inspection and maintenance schedule.

Infiltration trenches shall be inspected at six month intervals. Sediment should be removed from the pretreatment facilities as needed, and at a minimum, at six month intervals. Ponded water observed within a trench after several days indicates that the trench bottom is clogged. Corrective maintenance shall be carried out immediately.

2. Infiltration Basins

Infiltration basins operate on the same premise as infiltration trenches, but employ an above-ground impoundment instead of excavated trenches. Embankments over a certain height, or the creation of impoundments over a certain size may be subject to regulation by the the Connecticut Department of Environmental Protection. The stormwater runoff is stored in a basin, where it is allowed to infiltrate into the soil, as shown in Figure 9.7.

Figure 9.7
Infiltration Basin Schematic



Full infiltration basin systems, where the basin is designed to store and infiltrate the entire runoff from a storm, provide water quantity control and water quality control for the prescribed Water Quality Volume. Full infiltration systems shall be equipped with an emergency overflow channel to convey storms exceeding the design storm. The application of full infiltration systems may be limited by physical or economic constraints. Since infiltration basins have a limited capacity for controlling runoff quantity for large storms, it may be necessary to use infiltration basins in combination with other techniques.

Partial or water quality infiltration trench systems are designed to meet the Recharge Volume standard, while relying on other management practices for water quantity control. One method

of partial infiltration is an off-line method that routs the "first flush" of the runoff into an infiltration basin through an overflow weir.

Advantages

- Promotes groundwater recharge
- Reduces downstream flooding
- Preserves the natural site water balance

Disadvantages

- Failure rates are often high
- Use is generally restricted to small drainage areas
- May cause groundwater contamination
- Frequent maintenance required
- Inappropriate for large sediment loads

Design

Infiltration basins are appropriate at sites with permeable soils, gentle slopes, and where both bedrock and groundwater are a minimum of 48 inches below the surface. Contributing drainage areas should be between 2 and 15 acres. Infiltration basins are suitable for runoff from parking lots and small residential developments provided that the runoff is pretreated. Uncontaminated rooftop runoff may be accepted without pretreatment. Infiltration basins shall not be used to treat runoff from stormwater hotspots. In water supply recharge areas, infiltration basins shall be limited to runoff from uncontaminated rooftops. Generally, it is more practical to separate contaminated runoff from uncontaminated runoff and treat each with the appropriate management practices.

Pretreatment of runoff is a prerequisite to infiltration basins since they do not remove coarse particulates. Infiltration basins have a high failure rate which is mostly attributable to clogging by particulates. Once clogged, they are difficult to repair. Infiltration basins should have a sediment forebay or other pretreatment device to trap coarse pollutants.

To prevent erosion at the inlet and outlet, appropriate erosion and scour controls should be provided.

The required storage of the basin is the volume of runoff entering the basin added to the amount of rainfall over the basin.

Soils should have a minimum percolation rate of 0.5 inches per hour, and slopes should not exceed 5%. Infiltration basins should not be used at sites where soils have a 30% or greater clay content or greater than 40% silt clay content. Basins should not be placed over fill materials. Basins should be sited so that they are convenient for maintenance activities.

If designing for water quantity control, the maximum drain time should be 72 hours, with a minimum retention time of 48 hours.

The soil infiltration rate should be determined by a geotechnical study, with a minimum of one boring plus one boring for each 5,000 square feet of basin area. The infiltration design rate should be based upon the most restrictive rate obtained from the study.

The bottom of the basin should be as flat as possible to allow for uniform infiltration. The side slopes should not exceed a ratio of 3 horizontal to 1 vertical, so that maintenance and mowing is facilitated. After construction, the bottom and side slopes shall be stabilized with dense turf or grass. Woody vegetation shall not be planted within a basin or on its embankments.

A basin liner may be used in lieu of turf. The liner shall consist of 6 to 12 inches of fill material such as coarse sand. The material may be cleaned and replenished as necessary

The lowest part of the basin shall be 24 inches above seasonal high groundwater or bedrock.

A 20-foot vegetated buffer around the perimeter of the basin is recommended to assist in erosion control and nutrient removal.

The maximum depth of the infiltration basin is governed by the following equation:

$$d_{max} = f \times T_p \qquad \textbf{[Eq 9.2]}$$

Where d_{max} = Maximum allowable basin depth, ft
T_p = Maximum ponding time, hrs
f = Soil infiltration rate, in/hr

During construction, heavy equipment shall not operate on the footprint of the basin, as compaction will adversely impact infiltration capability. Generally, infiltration basins are inappropriate for runoff controls during construction activity.

Embankments shall have an emergency spillway capable of bypassing runoff from very large storms without damaging the structure. Embankments shall have a minimum crest width of 10 feet for maintenance purposes.

Maintenance

Operation and maintenance plans shall identify the owner, as well as parties responsible for maintenance along with an inspection and maintenance schedule.

Infiltration basins shall be inspected after all storms of 2-year or greater frequency. Sediment should be removed from pretreatment facilities as needed and, at a minimum, at six month intervals. Ponded water observed within the basin after several days indicates that the trench bottom is clogged. Corrective maintenance shall be carried out immediately.

Inspection should also evaluate the following:

❶ Subsidence
❷ Erosion
❸ Cracking or woody vegetation growth on the embankment
❹ Damage to the emergency spillway
❺ Sediment accumulation at the outlet
❻ Adequacy of erosion controls at the inlet and outlet
❼ Growth of deleterious aquatic vegetation
❽ Erosion within the basin and on its banks
❾ Changes within the upstream watershed

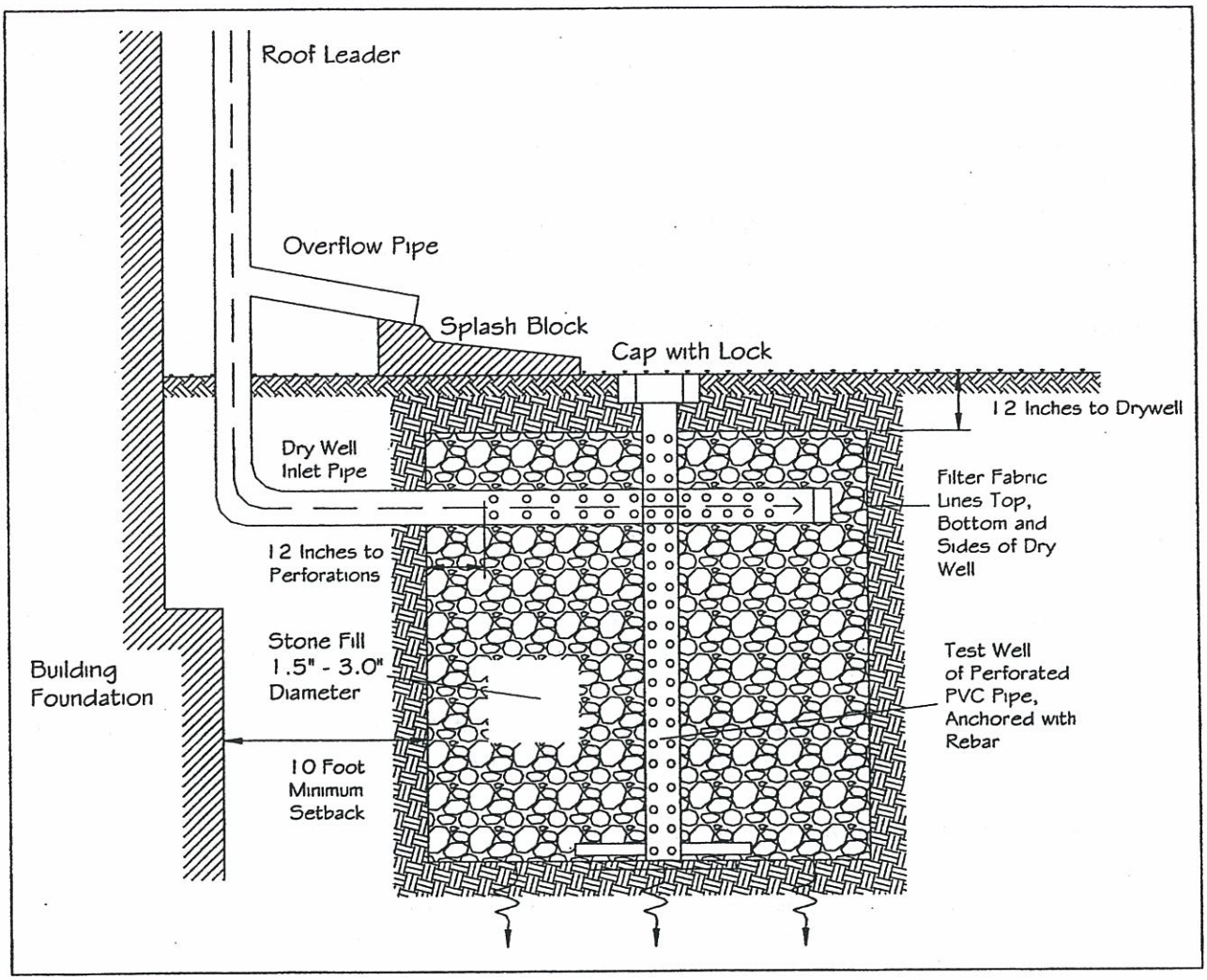
At least twice a year the buffer area, side slopes and basin bottom shall be mowed. Grass clippings shall be removed to avoid adverse impacts to infiltration.

Sediment may be removed when the basin is dry, but only with light equipment that will not compact the infiltration surface. Deep tilling may restore clogged surfaces.

3. Drywells

Drywells are small excavated pits, or buried perforated precast concrete structures backfilled with aggregate. Drywells are used to infiltrate uncontaminated runoff, such as that from roofs. They shall not be used to infiltrate runoff that is significantly contaminated with sediment or other pollutants, such as runoff from stormwater hotspots. Figure 9.8 shows a typical drywell.

Figure 9.8
Drywell Schematic



Advantages

- Can be used where storm drains are not available
- Can reduce the size and cost of downstream stormwater management facilities
- Good retrofitting option
- Provide groundwater recharge for uncontaminated runoff

Disadvantages

- Limited application for stormwater quality and quantity control
- May fail due to clogging
- Only applicable for drainage areas of one acre or less

Design

Drywells are appropriate at sites with permeable soils, gentle slopes, and where both bedrock and groundwater are a minimum of 48 inches below the surface. Contributing drainage areas

should not exceed 1 acre. Drywells should not be used as a stand-alone solution, but in conjunction with other stormwater management facilities. Runoff from stormwater hotspots shall not be infiltrated. Drywells shall not be used to infiltrate runoff that could transport pollutants and sediment, such as parking lot runoff without appropriate pretreatment. Drywells shall not be used to infiltrate stormwater runoff from construction sites.

Drywells are small systems that can be used to infiltrate rooftop runoff, reducing the flows, and in turn, the size of downstream stormwater management facilities. If used in this manner, roof gutters should use screens to trap leaves and other debris. Drywell infiltration capabilities also help contribute to volume reduction and recharge of groundwater.

Soils should have a minimum percolation rate of 0.5 inches per hour, and a maximum of 5 inches/hour. Drywells should not be used at sites where soils have a 30% or greater clay content or greater than 40% silt clay content. Drywells should not be placed over fill materials. The soil infiltration rate should be determined by a geotechnical study, with a minimum of one soil sample per drywell.

Drywells should be located no closer than 10 feet to any building foundation.

Since the infiltration trench is filled with stone, only the void space is available for storage. Typically, the stone should be clean and washed, with a size ranging between 1.5 to 3 inches.

Minimum storage is 48 hours, maximum is 72 hours. The bottom of the drywell shall be no less than 24 inches above bedrock or seasonal high water, but must be below the frost line.

A 20-foot vegetated buffer around drywells with surface inlets is recommended.

During construction, heavy equipment shall not operate on the footprint of a drywell, as compaction will adversely impact infiltration capability. Within the excavation, large roots should be cut to avoid tearing or puncturing the filter fabric surrounding the stone. If the drywell has no surface inlet, an inspection port shall be installed, with a lockable cap at grade.

Maintenance

Operation and maintenance plans shall identify the owner, as well as parties responsible for maintenance along with an inspection and maintenance schedule.

Drywells shall be inspected after every storm of a 2-year or greater frequency for the first three months after construction, and once per year thereafter. Water depth within the inspection port should be measured during a storm, and 24 and 48 hours following a storm to ensure that the drywell is exfiltrating properly.

If evidence of clogging is revealed by the inspection, corrective action shall be taken immediately.

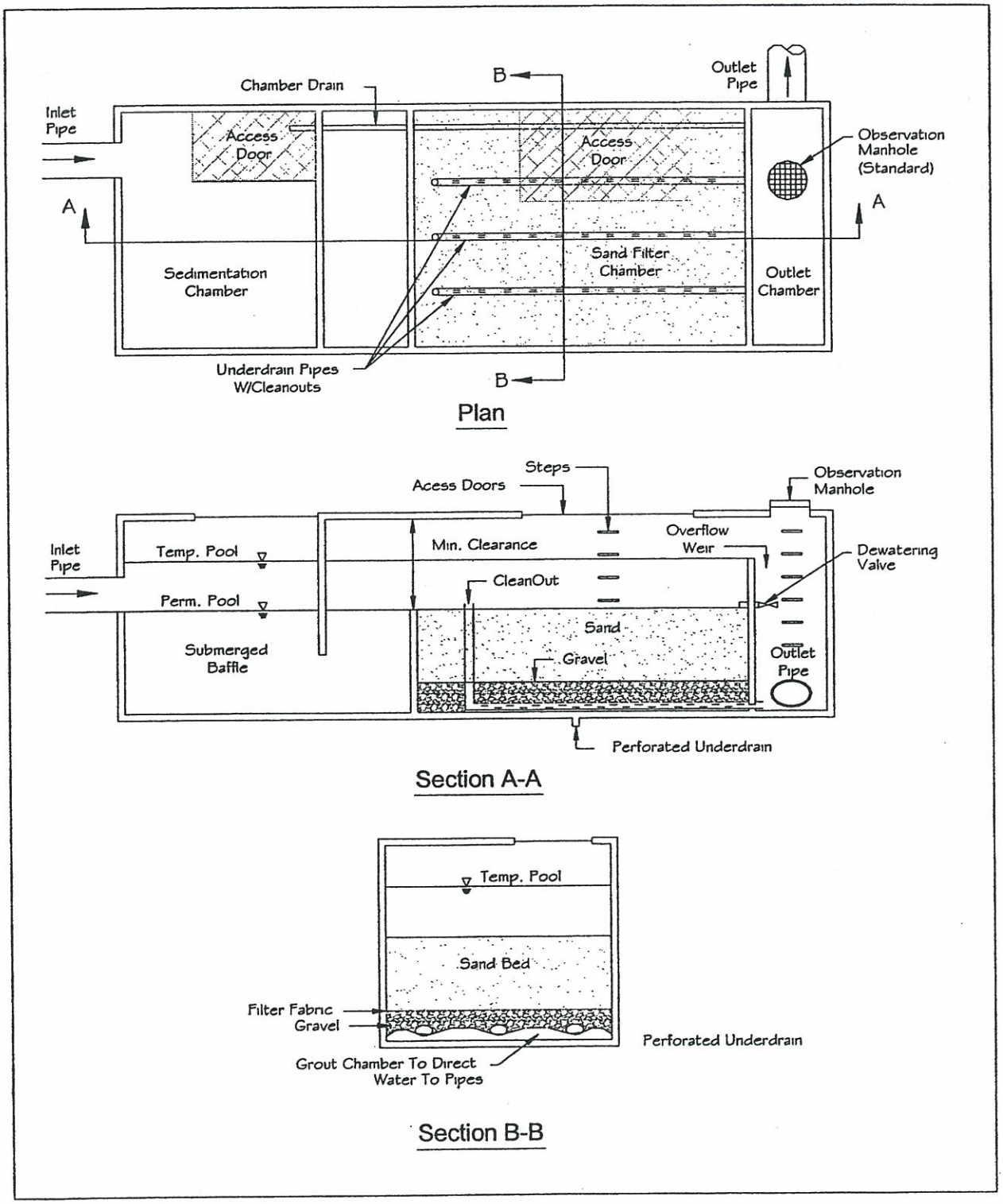
9.7 Filters

Filters rely upon filtering media, such as sand, to remove pollutants. The filtered stormwater runoff is then discharged to other stormwater management facilities for additional treatment. Filters are designed for water quality control, and have limited application for quantity control. Under certain conditions, where proper soils are present and there is low risk of groundwater contamination, filters may have an infiltration component. Filters tend perform best under head. Where the contributing drainage area to a filter is less than 75% impervious, the stormwater shall be pretreated with a sedimentation basin.

1. Sand Filters

A schematic of a typical sand filter is shown in Figure 9.9. Sand filters can also be used in a trench arrangement around the perimeter of parking lots to treat stormwater runoff from those locations. The stormwater is strained through the media so that pollutants settle at the top of the sand bed.

Figure 9.9
Sand Filter Schematic



Advantages

- Appropriate for small drainage areas of 1 to 10 acres
- Good retrofit option
- Long term longevity
- Can provide groundwater recharge under appropriate conditions

Disadvantages

- Pretreatment required
- Frequent maintenance
- May be unattractive without grass cover
- May have odor problems

Design

Sand filters can be used where space is limited, and can treat completely impervious surfaces. However, since the pollutant removal mechanism relies upon filtering through media, particulates may clog the system. As a result, pretreatment is a necessary requirement for sand filters. Due to the risk of clogging, sand filters shall not be used to treat runoff from sites with high particulate pollutant loads, such as construction sites.

Sand filters are typically an off-line system utilizing a diversion structure, such as a weir, to route a portion of the runoff into the facility, while larger flows bypass the facility.

For aesthetic reasons, it may be necessary to provide a grass cover to conceal surface sand filters. However, exposed sand filters are easier to maintain.

Sand filter designs often employ a sedimentation component, and a filtration component. The sedimentation component is designed to remove particulates that may clog the filtration component. Usually the sedimentation component is a vegetated swale or a forebay. The volume of the two components should be similar. The filtration component traps finer particulates and other pollutants. Sand filters shall also have an underdrain in a gravel layer.

Pretreatment is required for sand filters. Pretreatment is usually in the form of a sedimentation basin. The minimum pretreatment sedimentation basin area shall be computed using the Camp-Hazen equation, as follows:

$$A_s = \frac{Q_o}{W} x E' \qquad \text{[Eq. 9.3]}$$

Where A_s = Sedimentation basin surface area, ft^2

Q_o = Discharge rate from basin, cfs

W = Particle settling velocity, ft/s

- If Impervious Area $\leq$ 75% of Contributing Area:
  $W = 0.0004$ ft/s
- If Impervious Area > 75% of Contributing Area:
  $W = 0.0033$ ft/s

E' = Sediment trapping efficiency constant, for 90% efficiency, $E'=2.30$.

The filter area should be sized based upon Darcy's Law. A coefficient of permeability of 3.5 ft/day is used for sand:

$$A_f = \frac{Q_v \times d_r}{k(h_f + d_f)(t_f)} \qquad \text{[Eq. 9.4]}$$

Where A_f = Filter bed surface area, ft^2
Q_v = Treatment volume, ft^3
d_r = Filter bed depth, ft
k = Permeability of filter media, ft/day.
h_f = Average height of water above filter bed, ft
t_f = Design filter bed drain time, days (1.67 for sand)

The sand filter bed should be protected from contamination with native soil, by either providing a concrete chamber or protecting it with filter fabric.

The sand bed should be a minimum of 18-inches thick and consist of 0.02 – 0.04 inch diameter sand. Provisions should be made to maintain the sand bed, such as an access grating. It may be necessary to fence off the sand bed for safety reasons.

Maintenance

Operation and maintenance plans shall identify the owner, as well as parties responsible for maintenance along with an inspection and maintenance schedule.

For the first three months, sand filters shall be inspected after every storm event of 2-year or greater frequency and at six month intervals thereafter. If a problem is detected, it shall be corrected immediately.

Typical maintenance of the sand filter is to clear trash and debris and to remove the top layer of discolored sand and replace it with clean sand.

2. <u>Organic Filters</u>

Should organic filters be proposed, they should be designed, installed, and maintained in accordance with the manufacturer's recommendation. Their use shall be subject to the review and approval of the Director of Public Works.

9.8 Pretreatment Facilities

Pretreatment devices, by themselves, cannot achieve stormwater management standard compliance, but when used in combination with other stormwater management facilities and practices, they may help a site comply with the standards. In many instances, pretreatment is

required for other facilities to remove coarse particulates that may adversely impact their performance.

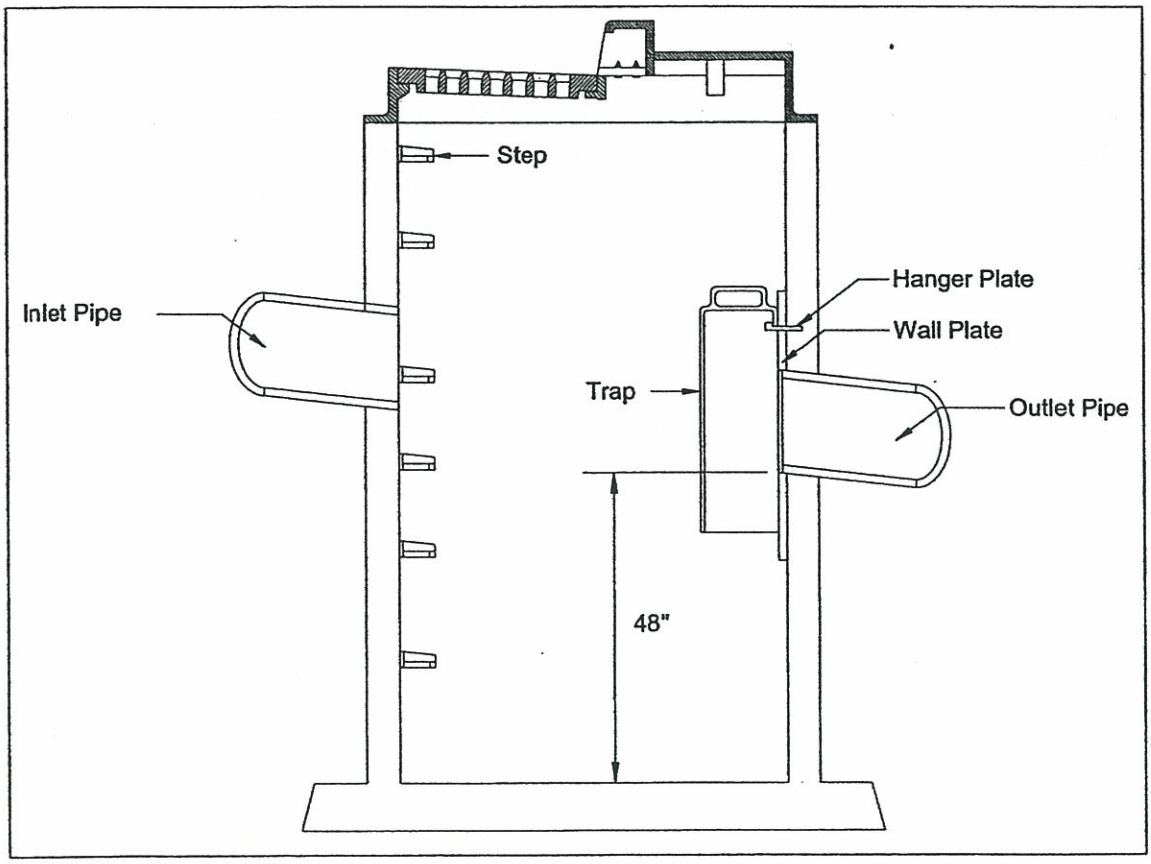
Pretreatment devices include:

❶ Deep Sump Catch Basin
❷ Sediment Traps/Forebays
❸ Drainage Channels

1. Deep Sump Catch Basins

Figure 9.10 shows a typical deep sump catch basin. A deep sump catch basin consists of a catch basin with a minimum 4-foot deep sump below the outlet of the pipe. The outlets shall be equipped with a hood or bell trap to confine floatable debris to the catch basin. The hood traps oil and grease inside the catch basin, where the oil and grease float on top of the pool of water, removing hydrocarbons from the stormwater runoff.

Figure 9.10
Deep Sump Catch Basin Schematic



Advantages

- Located underground
- Can be used as components of a storm drainage system
- Good retrofit option
- Good maintenance access
- High longevity

Disadvantages

- Limited pollutant removal
- No volume control
- Frequent maintenance
- Trapped oil and grease must be disposed of properly.

Deep sump catch basins are well suited for parking lots, including those with high traffic turnover such as convenience stores and gas stations. The contributing area to each deep sump catch basin should be limited to one acre of impervious cover.

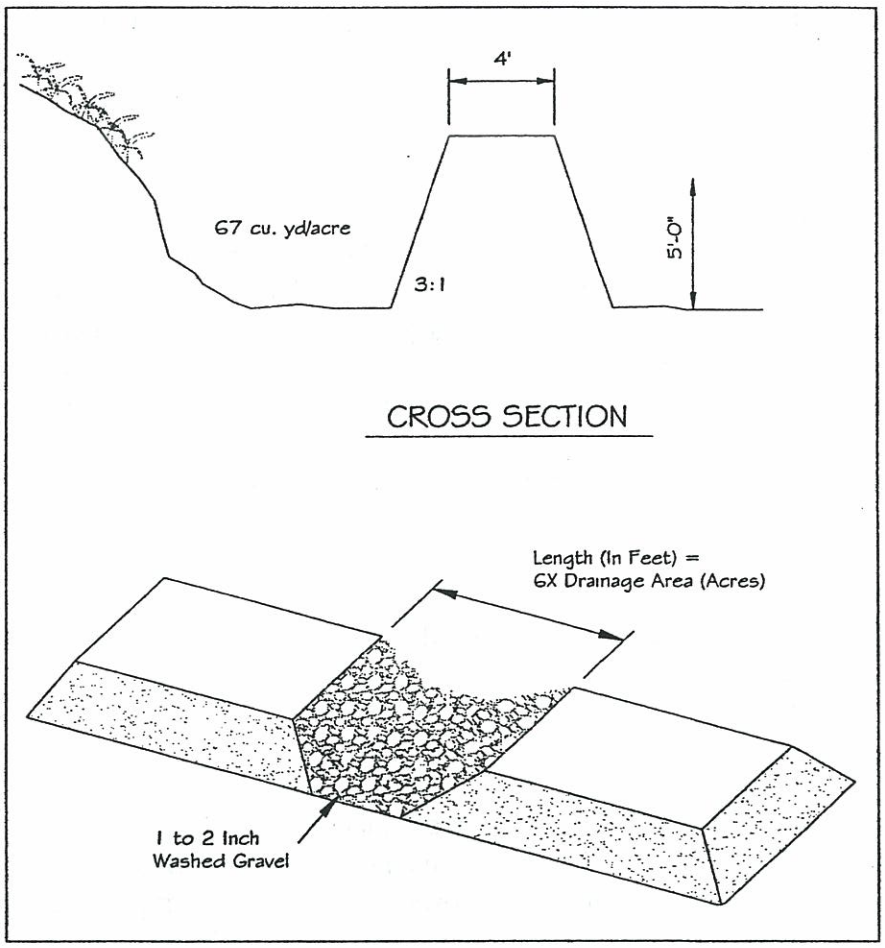
Maintenance

Since pollutants accumulate inside deep sump catch basin, they are only removed when maintenance takes place. As a result, regular maintenance is a necessity.

2. Sediment Traps/Forebays

Sediment traps and forebays are excavated pits or cast structures which slow stormwater runoff by creating a small impoundment, which provides some pollutant removal. They are always used as a pretreatment device. Figure 9.11 shows a schematic of one of these structures.

Figure 9.11
Sediment Trap/Forebay Schematic



Advantages

- Compatible with most other stormwater management techniques.
- Good retrofit option
- Slows velocity of stormwater
- Good maintenance access
- High longevity

Disadvantages

- Limited pollutant removal
- Does not remove soluble pollutants
- No volume control
- Frequent maintenance
- Trapped sediment, oil and grease must be disposed of properly.

Design

Typically, the forebay volume should approximate 0.1 inches of rainfall over each contributing acre. Due to their limited pollutant removal capabilities, forebays should only be used as a pretreatment device for other stormwater management facilities. Forebays rely upon settling to remove pollutants, usually only coarse particulates. Because of the small size of the pool, resuspension of pollutants is possible.

Forebays should be designed to facilitate maintenance. One way to accomplish this is to have a concrete pad at the bottom of the forebay to allow for easy clean up of accumulated sediments. Generally, the forebay should not be more than 3 to 6 feet deep. Side slopes should not exceed 3 horizontal to 1 vertical. Embankments exceeding a certain height, or the creation of impoundments over a certain size may be subject to a review by the Connecticut DEP. Inlets and outlets should be protected from erosion.

Forebays should have underdrains to dewater accumulated sediment.

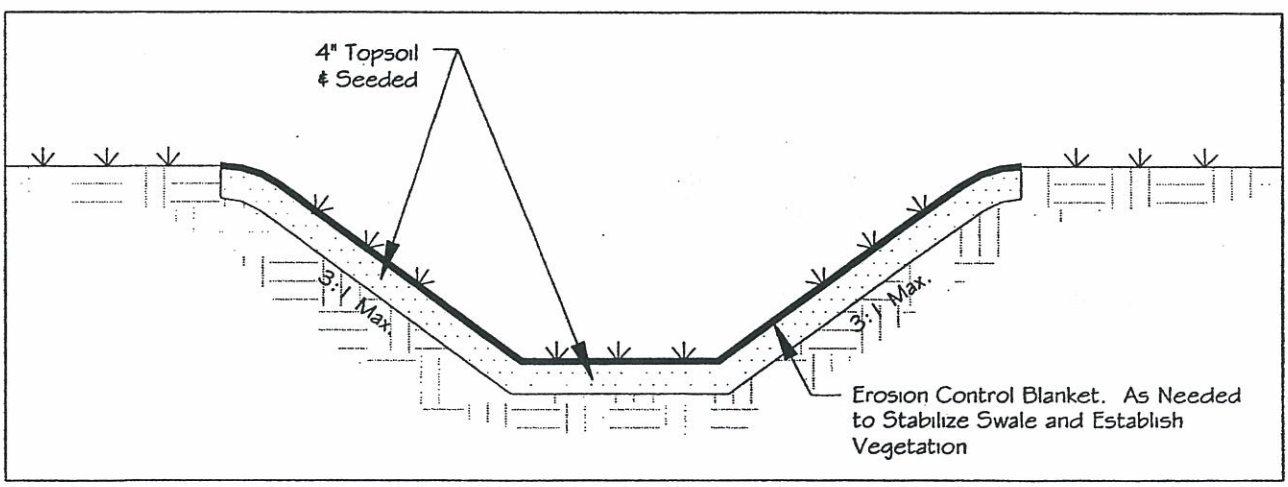
Maintenance

Forebays shall be inspected once per year. Regular maintenance to remove sediments, pol and grease is a necessity, at three month intervals.

3. Drainage Channels

Drainage channels convey runoff, whereas water quality swales impound runoff in order to improve the quality of the water. Drainage channels have a limited capacity for pollutant removal, if properly stabilized. Figure 9.12 shows a schematic of a typical drainage channel.

Figure 9.12
Drainage Channel Schematic



Advantages

- Limited infiltration capabilities
- Reduces runoff velocities

Disadvantages

- Requires semi-frequent maintenance
- Limited water quality application

Design

Drainage channels should convey runoff from relatively small impervious areas. The longitudinal slope of the channel should be as flat as possible, and should not exceed 5%. Since one of the primary functions of the channel is to reduce velocities, it is important that vegetation be established inside the channel. In general, trapezoidal or parabolical cross-sections should be used in favor of triangular shapes to maximize the velocity reduction and for ease of maintenance.

Channel length should be at least 100 feet to provide for appropriate pretreatment. Side slopes should not exceed 3 horizontal to 1 vertical. Outlets should be properly protected from erosion and scour.

Maintenance

Maintenance and inspection shall be carried out at six month intervals. During the first three months of operation, inspections shall be made after every storm event with a recurrence interval of 2-years or more, to verify that the channel is operating in accordance with the design intent. Regular maintenance includes mowing, fertilizing, liming, watering, weeding and pest control, as well as removing accumulated trash and debris. Grass, when mowed, shall be kept no less than 4 inches high to maintain the velocity retardance of the vegetation.

9.9 Mechanical Separators

Mechanical separators are proprietary products designed to separate pollutants from stormwater. The most popular variety of mechanical separator is the oil/grit separator, which utilizes baffles to create a vortex action inside a chamber as stormwater flows in. The vortex causes particulates to settle out at the bottom, while oil and grease rise to the top. The cleaned stormwater exits into a separate chamber, from which it is discharged. In general, the maintenance requirements for mechanical separators are more stringent than for the other stormwater management techniques discussed, and maintenance activities tend to be costlier. However, separators are ideally suited for sites where space is at a premium.

Designers which propose using mechanical separators shall be accompanied by field testing data describing the pollutant removal efficiency of the proposed unit and shall be submitted for the approval of the Director of Public Works.

9.10 Landscaping and Buffers

Landscaping plans shall be developed for pond and wetland stormwater management facilities, so that it can be demonstrated how the facilities will be stabilized and established.

During construction, the soils may be compacted by the use of heavy construction equipment. Heavily compacted soils are not beneficial to plant establishment. It is recommended that the area around landscaping zones be over-excavated and backfilled with loose topsoil. Mulch is recommended to assist in plant establishment and for weed suppression during the first year of operation. After the first year, the mulch shall be removed.. If 50% coverage is not achieved in the landscaping zone after two years, reinforcement planting will be required.

Buffers of 20 feet should be established around the perimeter of ponds or wetlands. The buffers provide non-structural treatment for overland flow.

Refer to Appendix D for native plantings suitable in wetland areas.

Section 10 Operation & Maintenance Plans

10.1 Introduction

Operation and maintenance practices are critical to ensure that stormwater management facilities continue to function properly and that runoff meets the prescribed Stormwater Management Standards. A stormwater facility operation and maintenance plan shall be developed for each development.

10.2 Operation and Maintenance Plans

Each operation and maintenance plan shall, at a minimum, identify the owner, the party responsible for maintenance, and inspection and maintenance schedules. Generally, the owner is the property owner upon which the stormwater management facility is situated, but there may be other legal agreements regarding ownership.

Required maintenance and inspection schedules, as well as required components of inspection, are described in Section 9.

A copy of the operation and maintenance plan shall be kept onsite at all times for maintenance and emergency purposes. A second copy shall be filed on the Darien Land Records.

10.3 Remedy of Deficiencies

Where inspections reveal that a stormwater management facility is not operating properly and/or not meeting the Darien Stormwater Management Standards, corrective action will be taken immediately.

Appendix

Appendix A

Standard Maintenance Agreement

A.1 Purpose

The attached Standard Maintenance Agreement is attached for the maintenance of drainage structures by project proponents. The Agreement shall be required at the discretion of the Director of Public Works.

Appendix

B Stormwater Management Checklist

B.1 Purpose

The Stormwater Management Checklist on the following pages is recommended for submission on projects which the Department of Public Works would review. Inclusion of the stormwater management checklist helps to facilitate the Department's review of the project.

Town of Darien
Department of Public Works

Stormwater Management Checklist

This checklist is to help an applicant ensure that a proposed development meets the Stormwater Management Standards set forth in the Town of Darien Drainage Manual. Applicants are strongly encouraged to include this checklist as part of their application in order to facilitate review.

Project: _____

Address: _____

The Water Quality Volume for this project is based upon the following depth of runoff over the post-development site impervious area:

☐ **1 inch of runoff.** (Critical areas as defined in Section 6 of the manual.)
☐ **0.5 inches of runoff.** (All other areas)

Standard 1: Untreated stormwater.

☐ The project has been designed so that new stormwater discharges do not discharge untreated stormwater into watercourses and wetlands and these new stormwater discharges do not contribute to erosion of watercourses or wetlands. *If not, please attach a sheet to explain.*

Standard 2: Post-development peak discharge rates.

☐ The post-development peak discharge from the site does not exceed the existing conditions discharge from the site.
☐ The post-development stormwater discharge will not exacerbate offsite flooding impacts for storms up to a 100-year recurrence interval.

Standard 3: Recharge to groundwater.

☐ Groundwater recharge for the post-development site approximates the annual recharge from existing conditions.
☐ On-site soil hydrologic groups have been identified. Recharge volume calculations are based upon a soil hydrologic group of _____ and a total impervious area of _____ acres.

Standard 4: 80% TSS Removal.

☐ The proposed stormwater management systems will remove 80% of the post-development site's average annual load of total suspended solids. (TSS)

Standard 5: Stormwater hotspots.

☐ This project does not involve land uses consistent with stormwater hotspots.
☐ This project involves land uses that are consistent with stormwater hotspots.

Standard 6: Critical area protection.

☐ This project does not discharge to critical areas.
☐ This project does discharge to critical areas. Critical area: _____

Standard 7: Redevelopment projects.

☐ This project is not a redevelopment project.
☐ This project is a redevelopment project. If checked, please address the following on a separate sheet of paper:

1. Identify the Stormwater Management Standards that have been met.
2. Identify the Stormwater Management Standards that have not been met.
3. Briefly explain how the project will improve stormwater quality over existing conditions.

Standard 8: Erosion/sediment control.

☐ Erosion and sediment controls have been incorporated into the design as prescribed by state and local regulations.

Standard 9: Operation and maintenance plans

☐ An operation and maintenance plan has been developed, and includes contact information for the BMP owner, parties responsible for maintenance, and inspection and maintenance schedules.

Appendix C Stormwater Management Sample Problem

C.1 Total Suspended Solids Calculations

Table 7.1 shows removal rates for various Best Management Practices. The column showing the Design Rate may be used to estimate for the project removal rate, unless there is a demonstration that a higher or lower value within the range of average removal rates should be used.

The percentage Total Suspended Solids removed from stormwater by water quality practices is calculated as follows:

$$TSS_{removed\ n} = (TSS_{inflow} \times R_{BMP}) + TSS_{prev} \qquad \text{[Eq. C.1]}$$

Where

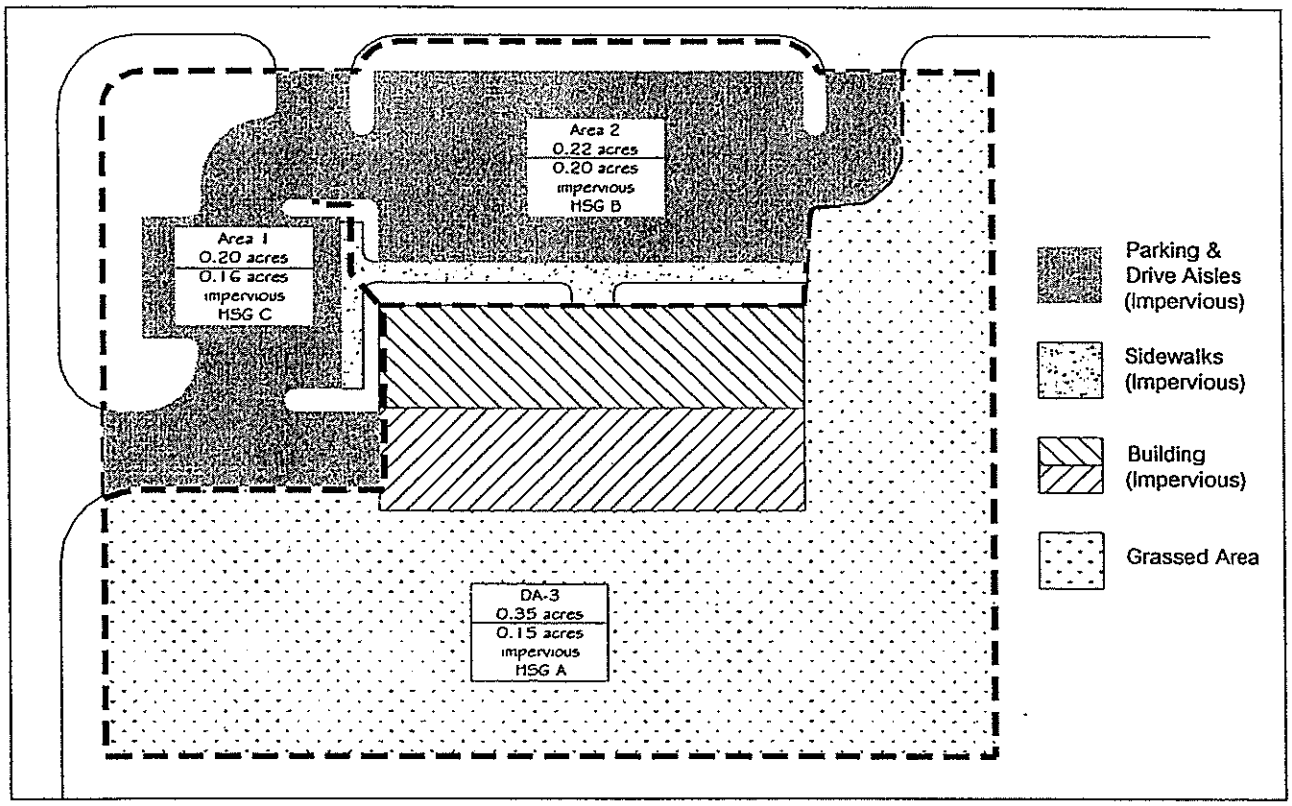
$TSS_{removed\ n}$	=	Percentage of Total Suspended Solids removed after practice number n in a treatment chain.
TSS_{inflow}	=	Percentage of Total Suspended Solids in inflow into practice number n
R_{BMP}	=	Removal rate of practice number n, from Table 7.1, unless demonstrated otherwise
TSS_{prev}	=	Cumulative percentage total of Total Suspended Solids removed by previous practices, 1 through n-1, in a treatment train.

The Total Suspended Solids inflow into the first practice in the treatment train shall be assumed to be 100 percent. The Total Suspended Solids flowing into the next practice shall be the percentage of Total Suspended Solids remaining in the stormwater after treatment by the previous practice. This process shall be repeated sequentially for each subsequent practice, until 80% of the Total Suspended Solids have been removed, in compliance with the Stormwater Management Standards.

C.2 Sample Problem

A 0.77 acre wooded site is to be developed to include a small commercial office building and parking lot. The developed site can be divided into three drainage areas as shown in Figure C.1. The site is not within a Critical Area.

Figure C.1
Sample Problem Site



Water Quality Volume (WQ_v)

The first step is to determine the Water Quality Volume (WQ_v) for each of the three drainage subareas, using the procedures described in Section 6.5. Since the site is not within a Critical Area, a 0.5-inch depth over the impervious area may be used to calculate the WQ_v.

Area 1 Impervious Area = 0.16 acres (given)

WQ_v = 0.16 acres x 0.5 inches = 0.08 acre-inches = 291 ft^3

Area 2 Impervious Area = 0.20 acres (given)

WQ_v = 0.20 acres x 0.5 inches = 0.10 acre-inches = 363 ft^3

Area 3 Impervious Area = 0.15 acres (given)

WQ_v = 0.15 acres x 0.5 inches = 0.075 acre-inches = 273 ft^3

Recharge Volume (Re_v)

The next step is to calculate the recharge volume for the entire site, as described in Section 6.3, by multiplying the Hydrologic Soil Group recharge depth by the impervious area:

Area 1: Hydrologic Soil Group C soils
Re_v = 0.10 inches x 0.16 acres = 0.016 acre-inches = 58 ft^3

Area 2: Hydrologic Soil Group B soils
Re_v = 0.25 inches x 0.20 acres = 0.05 acre-inches = 182 ft^3

Area 3: Hydrologic Soil Group A soils
Re_v = 0.40 inches x 0.15 acres = 0.06 acre-inches = 218 ft^3

Since the impervious component of Area 3 is rooftop runoff, the rooftop runoff may be infiltrated directly into the ground using drywells.

Selection of BMPs and Verification of Total Suspended Solids Removal

<u>Area 1</u>

Drainage from Area 1 will be collected by deep sump catch basins, and conveyed to a mechanical oil/grit separator. The impervious surfaces within this area will be swept on a regular basis.

Groundwater Recharge. The storm drainage network will funnel all the collected runoff into a single storm drain. The storm drain will flow into a splitter chamber that has a low-flow diversion to a drywell to accommodate the 58 ft^3 recharge volume requirement for the drainage area.

Water Quality. The mechanical oil/grit separator's manufacturer must be consulted for field test data and to verify that the selected unit can treat the contributing WQ_v. In this case, the manufacturer has supplied field test data supporting a 75% TSS removal rate and can that its units handle the contributing WQ_v

The next step would be to calculate the Total Suspended Solids removal of the first practice. Consultation of Table 7.1 reveals that street sweeping removes 10% of Total Suspended Solids. Since sweeping is the first practice in the chain, the stormwater is assumed to contain 100% of Total Suspended Solids prior to any treatment practice.

$$TSS_{removed} = (TSS_{inflow} \times R_{BMP}) + TSS_{prev}$$
$$= (1 \times 0.10) + 0 = 0.10$$

The next structural practice in the treatment chain is the deep sump catch basin, which removes 25% of TSS in the stormwater inflow, based on Table 7.1. For the purpose of this exercise, assume that the catch basins are not connected in series. First, the percentage of Total Suspended Solids in the stormwater inflow must be calculated. Street sweeping removed 10% of the Total Suspended Solids as calculated above, so 90% of Total Suspended Solids remain in the stormwater inflow to the deep sump catch basins.

$$TSS_{removed} = (0.90 \times 0.25) + 0.10 = 0.325$$

Lastly, this stormwater is discharged to the mechanical oil/grit separator which removes 75% of Total Suspended Solids. If 32.5% of Total Suspended Solids in the stormwater inflow have been removed, 67.5% of Total Suspended Solids remain.

$$TSS_{removed} = (0.675 \times 0.75) + 0.325 = 0.83125$$

The Stormwater Management Standard of 80% Total Suspended Solids removal has been achieved for Area 1.

<u>Area 2</u>

Drainage from Area 2 will be collected by deep sump catch basins and conveyed into a dry swale for water quality treatment. The impervious areas here will also be swept regularly, as they were for Area 1. The swale will have a longitudinal slope of 3% and will be 50 feet long, with sideslopes of 3 horizontal to 1 vertical.

Water Quality. The first step would be to determine if the proposed treatment train is sufficient for Total Suspended Solids removal. Since a regular sweeping program will be employed, the 10% sweeping credit can be applied against the Total Suspended Solids:

$$\begin{aligned} TSS_{removed} &= (TSS_{inflow} \times R_{BMP}) + TSS_{prev} \\ &= (1 \times 0.10) + 0 = 0.10 \end{aligned}$$

The next structural practice in the treatment chain is the deep sump catch basin, which removes 25% of TSS in the stormwater inflow. Again it shall be assumed that the catch basins are not connected in series. Street sweeping removed 10% of the Total Suspended Solids as calculated above, so 90% of Total Suspended Solids remain in the stormwater inflow to the deep sump catch basins.

$$TSS_{removed} = (0.90 \times 0.25) + 0.10 = 0.325$$

Lastly, this stormwater is discharged to the water quality swale which removes 70% of Total Suspended Solids. If 32.5% of Total Suspended Solids in the stormwater inflow have been removed, 67.5% of Total Suspended Solids remain.

$$TSS_{removed} = (0.675 \times 0.70) + 0.325 = 0.7975$$

Since the percentage of Total Suspended Solids removed is less than 80%, another practice must be employed within the train. In this case, a sediment trap/forebay will be added upstream of the inlet to the water quality swale, so that the stormwater discharges from the deep sump catch basins, into the sediment trap, where it is pretreated before discharging to the swale.

Revise the Total Suspended Solids calculations from the point after the calculations for the deep sump catch basins. After treatment by the deep sump catch basins, it was calculated that 67.5% of Total Suspended Solids remained in the stormwater runoff. The next step in the treatment train is now the sediment forebay, which has a 25% removal rate:

$$TSS_{removed} = (0.675 \times 0.25) + 0.325 = 0.49375$$

As the stormwater flows into the drainage swale, 49.375% of Total Suspended Solids have been removed, meaning that 50.625% of Total Suspended Solids are still present in the stormwater discharge. Water quality swales remove 70% of Total Suspended Solids:

$$TSS_{removed} = (0.50625 \times 0.70) + 0.49375 = 0.848125$$

The Stormwater Management Standard of 80% Total Suspended Solids removal has been achieved for Area 1.

Sizing the Structural Practices. Once it has been verified that the proposed treatment train will remove pollutants, the next step is to size the structural practices.

Sediment Trap

Sediment traps/forebays shall be designed to treat 0.1 inches of runoff over the impervious area:

Sediment Trap Volume = 0.1 inches x 0.20 acres impervious
= 0.02 acre-feet = 72.6 ft^3

Water Quality Swale

For this design, assume that three check dams would be used, one at the outlet, one at the swale midpoint and one at the swale inlet. This arrangement creates a ponding area within the swale. At a slope of 3%, the depth of the ponding area at the downstream checkdam relative to the upstream checkdam is 50 feet multiplied by 3%, which is 1.5 feet.

The first step is to size the swale to accommodate the WQ_v. The depth at the midpoint of the ponding area, d_{mid}, is 0.75 feet. This value can be used to calculate the width of the swale at the midpoint depth for the entire WQ_v.

$$WQ_v - V_p = w_{mid} \times l \times d_{mid} \qquad \text{[Eq. C.2]}$$

Where	WQ_v	=	Water Quality Volume
	V_p	=	Volume accommodated by pretreatment
	w_{mid}	=	Width of swale at the midpoint depth
	l	=	Length of swale
	d_{mid}	=	Depth of swale at midpoint

This equation can be rearrange to solve for the width at the midpoint:

$$w_{mid} = \frac{WQ_v - V_p}{l \times d_{mid}} \qquad \text{[Eq. C.3]}$$

w_{mid} = 363 ft^3 - 72.6 ft^3 / 50 ft x 0.75 ft = 7.74 feet

The bottom width can be determined using the midpoint width and depth, as well as the side slope relationship:

$$W_b = w_{mid} - (2 \times z \times d_{mid}) \qquad \text{[Eq. C.4]}$$

Where	w_b	=	Bottom width of swale
	w_{mid}	=	Width of swale at the midpoint depth
	z	=	Horizontal ratio of sideslope for each unit of vertical rise
	d_{mid}	=	Depth of swale at midpoint

= 7.74 feet – (2 x 3 x 0.75 feet)

= 3.24 feet

Groundwater Recharge. Recharge storage will be provided by a stone-filled reservoir beneath the dry swale underdrain system. Assume the reservoir will cover the same area as the bottom of the swale, then determine the depth of the reservoir required to store the Re_v volume. Assume the stones have 30% void space.

$$D = \frac{Re_v}{l \times w \times \eta} \qquad \text{[Eq. C.5]}$$

Where	Re_v	=	Recharge volume
	w_b	=	Bottom width of reservoir

L	=	Length of reservior
η	=	Void space in stone

$$D = 182\ ft^3 / (50\ ft \times 3.24\ ft \times 0.3) = 3.74\ ft$$

The depth of the recharge reservoir beneath the swale must be a minimum of 3.74 feet to meet the recharge requirements of the Stormwater Management Standards.

The 25-year conveyance capacity of the water quality swale may be checked using Manning's Equation as described in Section 4.3.

Area 3

The entire impervious area of Area 3 is rooftop runoff that will be infiltrated into the soil through drywells, meeting the recharge Stormwater Management Standards. Since drywells remove 80% of Total Suspended Solids, the water quality requirement component of the Stormwater Management Standards is also met.

Appendix D Native Plantings

D.1 Herbaceous Species

Blue Flag Iris	*Iris versicolor*
Cardinal Flower	*Lobelia cardinalis*
Cinnamon Fern	*Osmunda cinnamonmea*
Sensitive Fern	*Onoclea sensibilis*
Royal Fern	*Osmunda regalis*
Marsh Fern	*Dryopteris thelypteris*
Smartweed	*Polygonum* spp.
Trout Lily	*Erythronium americanum*
Soft Rush	*Juncus effusus*
Swamp Milkweed	*Asclepias incarnata*
Tussock Sedge	*Carex stricta*

D.2 Shrubs

Spicebush	*Lindera benzoin*
Red Osier Dogwood, Red Twig Dogwood	*Cornus sericea*
Silky Dogwood	*Cornus amomum*
Swamp Azalea	*Rhododendron viscosum*
Sweet Pepperbush, Summersweet	*Clethra alnifolia*
Inkberry	*Ilex glabra*
Winterberry	*Ilex verticillata*
Highbush Blueberry	*Vaccinium corymbosum*
Arrowwood	*Viburnum dentatum*
Elderberry, American Elder	*Sambucus canadensis*
Buttonbush	*Cephalanthus occidentalis*
Shadbush, Serviceberry	*Amelanchier canadensis*
Speckled Alder	*Alnus rugosa*

D.3 Trees

Red Maple	*Acer rubrum*
Silver Maple	*Acer saccharinum*
Black Ash	*Fraxinus nigra*
River Birch	*Betula nigra*
Tupelo, Black Gum	*Nyssa sylvatica*
Sweetgum	*Liquidambar styraciflua*
Pin Oak	*Quercus palustris*

Willow	*Salix* spp.

D.4 Emergent Plants

Arrow-Arum	*Peltandra virginica*
Cattail	*Typha latifolia*
Sweet Flag	*Acorus calamus*
Arrowhead	*Sagittaria latifolia*
Soft Stem Bulrush, Great Bulrush	*Scirpus validus*
Three-Square Rush	*Scirpus americanus*
Pickerelweed	*Pontederia cordata*
Bur-reed	*Sparganium americanum*

Appendix E

Maps

E.1 Critical Areas Map

E.2 Soils Map

Appendix F

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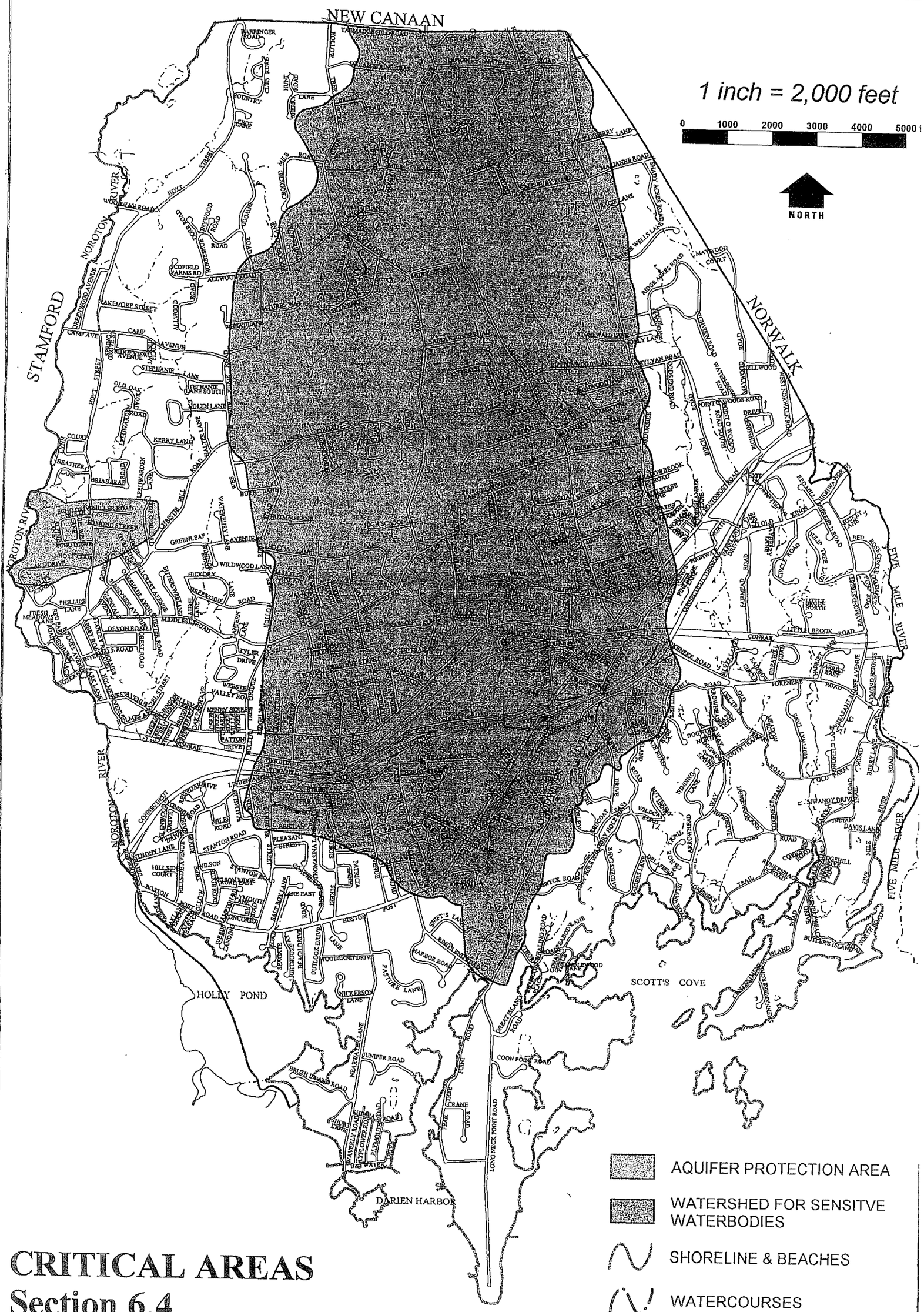

CRITICAL AREAS
Section 6.4

SOILS MAP
Section 6.6