CHAPTER

STORAGE FACILITIES 6

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6.1 Introduction

The traditional design of storm drainage systems has been to collect and convey storm runoff as rapidly as possible to a suitable location where it can be discharged. As areas urbanize, this type of design may result in major drainage and flooding problems downstream. Under favorable conditions, the temporary storage of some of the storm runoff can decrease downstream flows and often the cost of the downstream conveyance system. In addition to peak flow reductions, the temporary storage of storm runoff generally improves the quality of water discharged from the facility. Detention storage facilities can range from small facilities in parking lots or other on-site facilities to large lakes and reservoirs. This chapter provides general design criteria for detention/retention storage basins.

Urban stormwater storage facilities are often referred to as either detention or retention facilities. For the purposes of this chapter, detention facilities are those permanent stormwater management structures whose primary purpose is to temporarily store stormwater runoff and release the stored runoff at controlled rates. Therefore, detention facilities are designed to reduce the peak discharge and only detain runoff for some short period of time. These facilities can be designed to completely drain after the design storm has passed. Retention facilities are those permanent structures whose primary purpose is to permanently store a given volume of stormwater runoff. Release of the given volume is by infiltration, overflow structures, and/or evaporation. Since most of the design procedures are the same for detention and retention facilities, the term storage facilities will be used in this chapter to include detention and retention facilities. If special procedures are needed for detention or retention facilities, these will be specified. For information on infiltration systems, which also provide for runoff storage and control, the reader is referred to Chapter 7 of this manual on Water Quality Best Management Practices.

It should be noted that the location of storage facilities is very important as it relates to the effectiveness of these facilities to control downstream flooding. Small facilities will only have minimal flood control benefits and these benefits will quickly diminish as the flood wave travels downstream. Multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system which could decrease or increase flood peaks in different downstream locations.

6.2 Symbols And Definitions

To provide consistency within this chapter as well as throughout this manual, the following symbols will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this chapter, the symbol will be defined where it occurs in the text or equations.

Table 6-1 Symbols And Definitions

Symbol	Definition	Units
A	Cross sectional or surface area	ft^2

C	Weir coefficient	-
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
t	Routing time period	hrs
g	Acceleration due to gravity	32.2 ft/s^2
H	Head on structure	ft
H_c	Height of weir crest above channel bottom	ft
I	Inflow rate	cfs
L	Length	ft
Q	Flow or outflow rate	cfs
S, V_s	Storage volume	ft^3
t_b	Time base on hydrograph	hrs
T_i	Duration of basin inflow	hrs
t_p	Time to peak	hrs
V_s, S	Storage volume	ft^3
W	Width of basin	ft
Z	Side slope factor	-

6.3 Design Criteria

6.3.1 General Criteria

The utility of any storage facility depends on the amount of storage, its location within the system, and its operational characteristics. An analysis of storage facilities should consist of comparing the design flow at a point or points downstream of the proposed storage site with and without storage. As a rule of thumb, all proposed stormwater storage facilities should be routed through the downstream drainage system to a point where the size of the proposed development represents 10% or less of the total drainage area that contributes runoff to that point. This point is called the 10% point and was discussed in further detail in Chapter 1 - Introduction. In addition to the design flow, other flows in excess of the design flow that might be expected to pass through the storage facility should be included in the analysis (i.e., 100-year flood). The design criteria for storage facilities should include:

- release rate,
- storage volume,
- grading and depth requirements,
- safety considerations and landscaping,
- outlet works, and
- location.

Note: The same hydrologic procedure shall be used to determine pre- and post-development hydrology. Existing or pre-development land use data shall be taken from the March 1998 Horry County aerial photographs for the design criteria for storage facilities.

6.3.2 Release Rate

Control structure release rates shall approximate pre-development peak runoff rates for the 10-

year and 25-year storms, with emergency overflow capable of handling the 100-year discharge. Design calculations are required to demonstrate that the facility will limit runoff from the 10-year and 25-year post-development discharge rates to pre-development peak discharge rates.

6.3.3 Storage

Storage volume shall be adequate to attenuate the post-development peak discharge rates to pre-developed discharge rates for the 10-year and 25-year storms. Routing calculations must be used to demonstrate that the storage volume is adequate. If sedimentation during construction causes loss of detention volume, design dimensions shall be restored before completion of the project. For detention flood control basins, all detention volume shall be drained within 48 hours. Note: Parking lot detention storage facilities are not allowed in Horry County.

6.3.4 Grading And Depth

Following is a discussion of the general grading and depth criteria for storage facilities followed by criteria related to dry detention basins and wet detention ponds.

6.3.4.1 General

The construction of storage facilities usually require excavation or placement of earthen embankments to obtain sufficient storage volume. Vegetated embankments shall be less than 20 feet in height and shall have side slopes no steeper than 2:1 (horizontal to vertical) although 3:1 is preferred. Riprap-protected embankments shall be no steeper than 2:1. Geotechnical slope stability analysis is recommended for embankments greater than 10 feet in height and is mandatory for embankment slopes steeper than those given above. Procedures for performing slope stability evaluations can be found in most soil engineering textbooks, including those by Spangler and Handy (1982) and Sowers and Sowers (1970).

Impoundment depths greater than 25 feet or a storage volume greater than 100 acre-feet are subject to the requirements of the Safe Dams Act unless the facility is excavated to this depth. Other considerations when setting depths include flood elevation requirements, public safety, land availability, land value, present and future land use, water table fluctuations, soil characteristics, maintenance requirements, and required freeboard. Aesthetically pleasing features are also important in urbanizing areas.

6.3.4.2 Dry Detention Basins

The basin perimeter above the normal high water elevations should be sloped toward the basin to allow drainage and to prevent standing water. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of dry detention basins should be graded toward the outlet to prevent standing water conditions. A low flow or pilot channel constructed across the facility bottom from the inlet to the outlet is recommended to convey low flows, and prevent standing water conditions. Often a sediment collection forebay is provided with easy maintenance access.

6.3.4.3 Wet Detention Ponds

The maximum depth of wet detention ponds will be determined by site conditions, ground water elevation, design constraints, and environmental needs. In general, if the facility provides a permanent pool of water, a depth sufficient to discourage growth of weeds (without creating undue potential for anaerobic bottom conditions) should be considered. A depth of 4 feet is generally reasonable unless fishery requirements dictate otherwise. Aeration may be required in permanent pools to prevent anaerobic conditions. Where aquatic habitat is required, wildlife experts should be contacted for site-specific criteria relating to such things as depth, habitat, and bottom and shore geometry. In some cases a shallow bench along the perimeter is constructed to encourage emergent vegetation growth to enhance the pollution reduction capabilities or aesthetics of the pond.

6.3.5 Outlet Works

Outlet works selected for storage facilities typically include a principal spillway and an emergency overflow, and must be able to accomplish the design functions of the facility. Outlet works can take the form of combinations of drop inlets, pipes, weirs, and orifices. Slotted riser pipes are discouraged because of clogging problems. It should be noted that small outlets that will be subject to clogging or be difficult to maintain may not be acceptable to Horry County. The principal spillway is intended to convey the design storm without allowing flow to enter an emergency outlet. For large storage facilities, selecting a flood magnitude for sizing the emergency outlet should be consistent with the potential threat to downstream life and property if the basin embankment were to fail. The minimum flood to be used to size the emergency outlet is the 100-year flood, using fully developed land use conditions, with the outlet located at or above the 25-year ponding elevation. The sizing of the outlet works shall be based on results of hydrologic routing calculations.

6.3.6 Location

In addition to controlling the peak discharge from the outlet works, storage facilities will change the timing of the entire hydrograph. If several storage facilities are located within a particular basin, it is important to determine what effects a particular facility may have on combined hydrographs in downstream locations. The following procedure is recommended to determine downstream effects. For all storage facilities, channel routing calculations should proceed downstream to a confluence point where the drainage area being analyzed represents ten percent of the total drainage area. At this point, the effect of the hydrograph routed through the proposed storage facility on the downstream hydrograph can be assessed and shown not to have detrimental effects on downstream hydrographs.

6.3.7 Safe Dams Act

Under the Safe Dams Act, a regulated a dam is an artificial barrier that does or may impound water and that is 25 feet or greater in height or has a maximum storage volume of 100 acre-feet or more. A number of exemptions are allowed from the Safe Dams Act and any questions concerning a specific design or application should be addressed to the South Carolina Department of Health and Environmental Control.

6.4 General Procedure

6.4.1 Data Needs

The following data will be needed to complete storage design and routing calculations.

- Inflow hydrograph for all selected design storms for pre- and post-development land use conditions outlined in the Horry County Stormwater Management and Sediment Control Ordinance, Division 2, Section 17.7-36. Existing or pre-development land use data shall be taken from the March 1998 Horry County aerial photographs.

- Stage-storage curve for proposed storage facility (see Figure 6-1 below for an example).
- Stage-discharge curve for all outlet control structures (see Figure 6-2 below for an example).

Using these data a design procedure is used to route the inflow hydrograph through the storage facility with different basin and outlet geometry until the desired outflow hydrograph is achieved. Although hand calculation procedures are available for routing hydrographs through storage facilities, they are very time consuming especially when several different designs are evaluated. Standard textbooks on Hydrology and Hydraulics give examples of hand-routing techniques (See Debo & Reese, 1995). For this manual, it assumed that designers will be using one of the many computer programs available for storage routing and thus other procedures and example applications will not be given in this chapter. Procedures for preliminary detention calculations will be given in this chapter since they provide simple procedures that can be used to estimate storage needs and also provide a quick check on the results of using different computer programs.

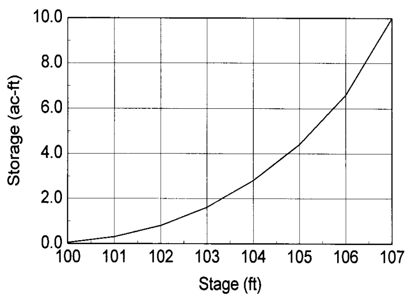


Figure 6-1
Example Stage-Storage Curve

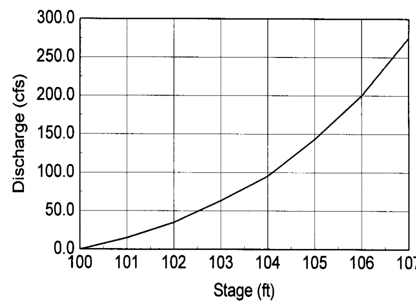


Figure 6-2
Example Stage-Discharge Curve

6.4.2 Stage-Storage Curve

A stage-storage curve defines the relationship between the depth of water and storage volume in a reservoir. The data for this type of curve are usually developed using a topographic map and the double-end area, frustum of a pyramid, prismoidal formulas or circular conic section. The double-end area formula is expressed as:

$$V_{1,2} = [(A_1 + A_2)/2]d \qquad (6.1)$$

Where: $V_{1,2}$ = storage volume, ft^3, between elevations 1 and 2
A_1 = surface area at elevation 1, ft^2
A_2 = surface area at elevation 2, ft^2
d = change in elevation between points 1 and 2, ft

The frustum of a pyramid is expressed as:

$$V = d/3\ [A_1 + (A_1 \times A_2)^{0.5} + A_2]/3 \qquad (6.2)$$

Where: V = volume of frustum of a pyramid, ft^3
d = change in elevation between points 1 and 2, ft

A_1 = surface area at elevation 1, ft^2

A_2 = surface area at elevation 2, ft^2

The prismoidal formula for trapezoidal basins is expressed as:

$$\mathbf{V = LWD + (L + W)\, ZD^2 + 4/3\, Z^2\, D^3} \qquad \textbf{(6.3)}$$

Where:
- V = volume of trapezoidal basin, ft^3
- L = length of basin at base, ft
- W = width of basin at base, ft
- D = depth of basin, ft
- Z = side slope factor, ratio of horizontal to vertical

The circular conic section formula is:

$$\mathbf{V = 1.047\, D\, (R_1^2 + R_2^2 + R_1R_2)} \qquad \textbf{(6.4)}$$

$$\mathbf{V = 1.047\, D\, (3R_1^2 + 3ZDR_1 + Z_2D^2)} \qquad \textbf{(6.5)}$$

Where: R_1 and R_2 = bottom and surface radii of the conic section, ft
- D = depth of basin, ft
- Z = side slope factor, ratio of horizontal to vertical

6.4.3 Stage-Discharge Curve

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility has two spillways: principal and emergency. The principal spillway is usually designed with a capacity sufficient to convey the design flood without allowing flow to enter the emergency spillway. A pipe culvert, weir, or other appropriate outlet can be used for the principal spillway or outlet. The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal spillway. This spillway should be designed taking into account the potential threat to downstream life and property if the storage facility were to fail. The stage-discharge curve should take into account the discharge characteristics of both the principal spillway and the emergency spillway.

6.4.4 Procedure

A general procedure for using the above data in the design of storage facilities is presented below:

Step 1 - Compute inflow hydrograph for runoff from the 10-year, 25-year, and 100-year design storms using the procedures outlined in Chapter 2 - Hydrology of this manual. Both pre- and post-development hydrographs are required for the 10-year and 25-year design storms. Only the post-development hydrograph is required for runoff from the 100-year design storm.

Step 2 - Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1 (see Section 6.6).

Step 3 - Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 should be used. From the selected shape, determine the maximum depth in the storage facility.

Step 4 - Select the type of outlet and size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage.

Step 5 - Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using a storage routing computer model. If the routed post-development peak discharges from the 10- and 25-year design storms exceed the pre-development peak discharges, or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise the available storage volume, outlet device, etc., and return to step 3.

Step 6 - Perform routing calculations using the 100-year hydrograph to determine if any increases in downstream flows from this hydrograph will cause damages and/or drainage and flooding problems. If problems will be created (e.g., flooding of habitable dwellings, property damage, or public access and/or utility interruption), then the storage facility must be designed to control the increased flows from the 100-year storm. If not, then consider emergency overflow from runoff due to the 100-year (or larger) design storm and established freeboard requirements.

Step 7 - Evaluate the downstream effects of detention outflows for the 25- and 100-year storms to ensure that the routed hydrograph does not cause downstream flooding problems. The exit hydrograph from the storage facility should be routed though the downstream channel system until a confluence point is reached where the drainage area being analyzed represents 10 percent or less of the total drainage area.

Step 8 - Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

6.5 Outlet Hydraulics

6.5.1 Outlets

Sharp-crested weir flow equations for no end contractions, two end contractions, and submerged discharge conditions are presented below, followed by equations for broad-crested weirs, v-notch weirs, proportional weirs, and orifices, or combinations of these facilities. If culverts are used as outlets works, procedures presented in Chapter 4 - Design of Culverts should be used to develop stage-discharge data.

6.5.2 Sharp-Crested Weirs

A sharp-crested weir with no end contractions is illustrated in Figure 6-3. The discharge equation for this configuration is (Chow, 1959):

$$\mathbf{Q = [(3.27 + 0.4(H/H_C)]\ LH^{1.5}} \qquad \textbf{(6.6)}$$

Where: Q = discharge, cfs
H = head above weir crest excluding velocity head, ft

H_C = height of weir crest above channel bottom, ft
L = horizontal weir length, ft

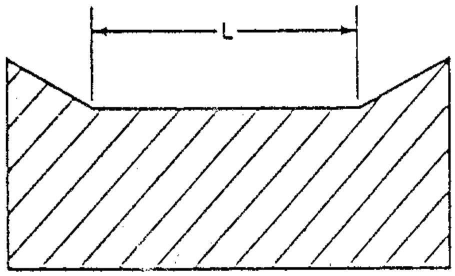


Figure 6-3
Sharp-Crested Weir
No End Contractions

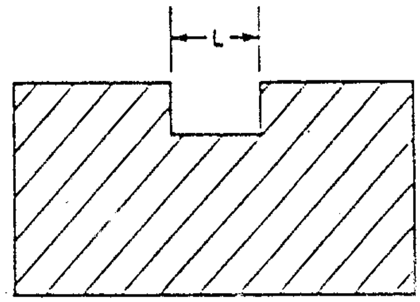


Figure 6-4
Sharp-Crested Weir,
Two End Contractions

A sharp-crested weir with two end contractions is illustrated in Figure 6-4. The discharge equation for this configuration is (Chow, 1959):

$$Q = [(3.27 + .04(H/H_C)] (L - 0.2H) H^{1.5} \quad (6.7)$$

Where: Variables are the same as equation 6.6.

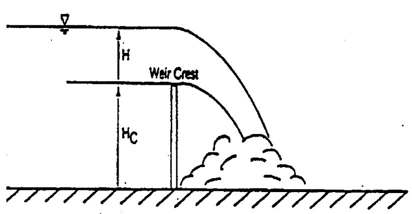


Figure 6-5 Sharp-Crested Weir And Head

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$$Q_S = Q_f(1 - (H_2/H_1)^{1.5})^{0.385} \quad (6.8)$$

Where: Q_S = submergence flow, cfs
Q_f = free flow, cfs
H_1 = upstream head above crest, ft
H_2 = downstream head above crest, ft

Note: H_1 and H_2 should be located far enough upstream and downstream to ensure that normal depth flow has been established.

6.5.3 Broad-Crested Weirs

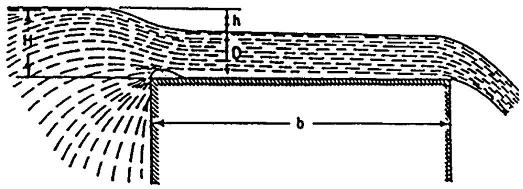
The equation for the broad-crested weir is (Brater and King, 1976):

$$Q = CLH^{1.5} \tag{6.9}$$

Where:
- Q = discharge, cfs
- C = broad-crested weir coefficient
- L = broad-crested weir length, ft
- H = head above weir crest, ft

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Information on C values as a function of weir crest breadth and head is given in Table 6-2.

Table 6-2
Broad-Crested Weir Coefficient C Values As A
Function Of Weir Crest Breadth (b) And Head (H)



Measured Head, H[1] (ft)	Weir Crect Breadth (ft) 0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.27	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

[1]Measured at least 2.5H upstream of the weir.

Reference: Brater and King (1976).

6.5.4 V-Notch Weirs

The discharge through a v-notch weir can be calculated from the following equation (Brater and King, 1976).

$$Q = 2.5 \tan(\theta/2)H^{2.5} \quad (6.10)$$

Where: Q = discharge, cfs
θ = angle of v-notch, degrees
H = head on apex of notch, ft

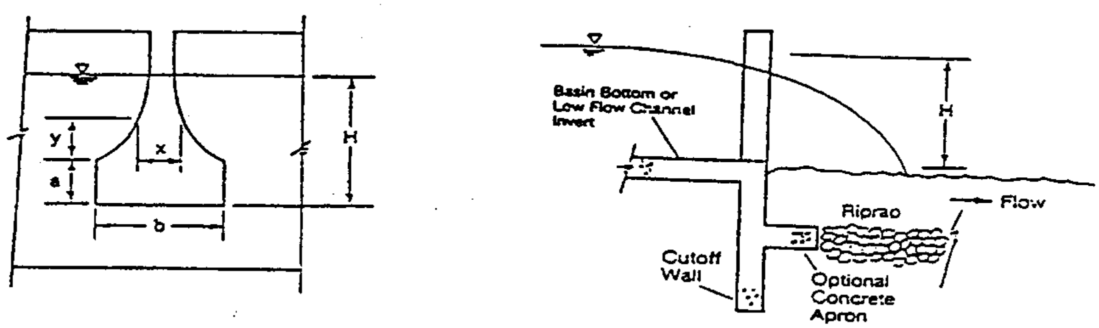
6.5.5 Proportional Weirs

Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head. Design equations for proportional weirs are (Sandvik, 1985):

$$Q = 4.97\, a^{0.5}\, b(H - a/3) \quad (6.11)$$

$$x/b = 1 - (1/3.17)\,(\arctan\,(y/a)^{0.5}) \quad (6.12)$$

Where: Q = discharge, cfs
arctan (y/a) is in radians
Dimensions a, b, h, x, and y are shown in Figure 6-6



Elevation **Section**

Figure 6-6 Proportional Weir Dimensions

6.5.6 Orifices

Pipes smaller than 12" may be analyzed as a submerged orifice if H/D is greater than 1.5. For square-edged entrance conditions,

$$Q = 0.6A(2gH)^{0.5} = 3.78D^{2}H^{0.5} \quad (6.13)$$

Where: Q = discharge, cfs
A = cross-section area of pipe, ft^2
g = acceleration due to gravity, 32.2 ft/s^2
D = diameter of pipe, ft
H = head on pipe, from the center of pipe to the water surface

6.5.7 Combination Outlets

Combinations of weirs, pipes and orifices can be put together to provide a variable control stage-discharge curve suitable for control of multiple storm flows. They are generally of two types: shared outlet control and separate outlet controls. Shared outlet control is typically a number of individual outlet openings, weirs or drops at different elevations on a riser pipe or box which all flow to a common larger conduit or pipe. Separate outlet controls are less common and normally consist of a single opening through the dam of a detention facility in combination with an overflow spillway for emergency use. For a complete discussion of outlets and combination outlets see Municipal Storm Water Management by Debo and Reese.

6.6 Preliminary Detention Calculations

6.6.1 Storage Volume

For small drainage areas, a preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in Figure 6-7.

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_s = 0.5T_b(Q_i - Q_o) \qquad \textbf{(6.14)}$$

Where: V_s = storage volume estimate, ft^3
Q_i = peak inflow rate, cfs
Q_o = peak outflow rate, cfs
T_b = duration of basin inflow, sec

Any consistent units may be used for Equation 6.14.

6.6.2 Alternative Method

An alternative preliminary estimate of the storage volume required for a specified peak flow reduction can be obtained by the following regression equation procedure (Wycoff & Singh, 1986).

1. Determine input data, including the allowable peak outflow rate, Q_o, the peak flow rate of the inflow hydrograph, Q_i, the time base of the inflow hydrograph, t_b, and the time to peak of the inflow hydrograph, t_p.

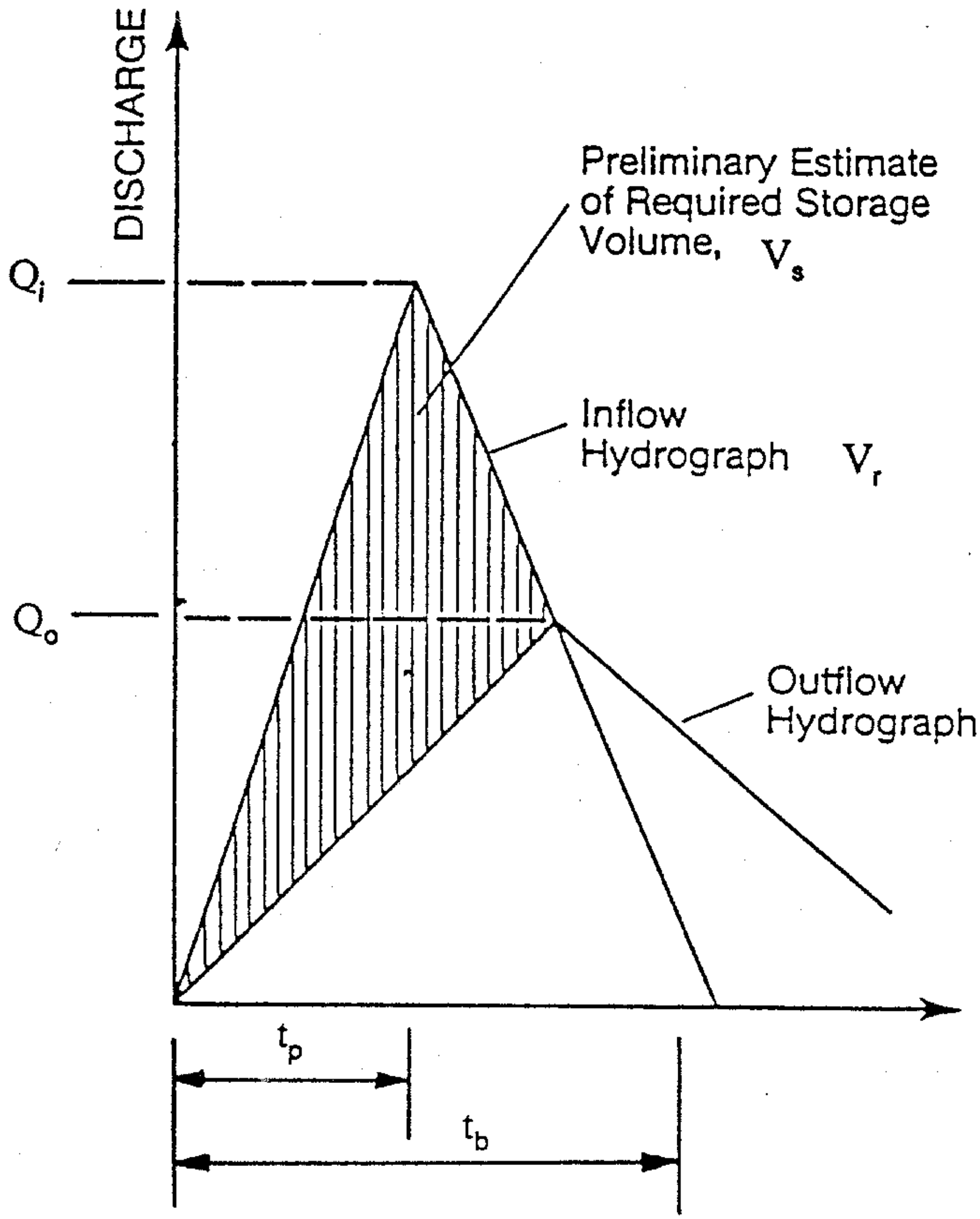


Figure 6-7 Triangular Shaped Hydrographs
(For Preliminary Estimate Of Required Storage Volume)

2. Calculate a preliminary estimate of the ratio V_S/V_r using the input data from Step 1 and the following equation:

$$V_S/V_r = [1.291(1-Q_o/Q_i)^{0.753}]/[(t_b/t_p)^{0.411}] \quad \textbf{(6.15)}$$

Where: V_s = volume of storage, ft^3
V_r = volume of runoff, ft^3
Q_o = outflow peak flow, cfs
Q_i = inflow peak flow, cfs
t_b = time base of the inflow hydrograph, hrs (Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak.)
t_p = time to peak of the inflow hydrograph, hrs

3. Multiply the volume of runoff, V_r, times the ratio V_s/V_r, calculated in Step 2 to obtain the estimated storage volume V_s.

6.6.3 Peak Flow Reduction

A preliminary estimate of the potential peak flow reduction for a selected storage volume can be obtained by the following procedure.

1. Determine the following:
   - volume of runoff, V_r,
   - peak flow rate of the inflow hydrograph, Q_i,
   - time base of the inflow hydrograph, t_b,
   - time to peak of the inflow hydrograph, t_p, and
   - storage volume, V_s.

2. Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using the following equation (Singh, 1976):

$$\mathbf{Q_o/Q_i = 1 - [0.712(V_s/V_r)^{1.328}(t_b/t_p)^{0.546}]} \qquad \mathbf{(6.16)}$$

Where: Q_o = outflow peak flow, cfs
Q_i = inflow peak flow, cfs
V_s = volume of storage, ft^3
V_r = volume of runoff, ft^3
t_b = time base of the inflow hydrograph, hrs (Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak.)
t_p = time to peak of the inflow hydrograph, hrs

3. Multiply the peak flow rate of the inflow hydrograph, Q_i, times the potential peak flow reduction calculated from step 2 to obtain the estimated peak outflow rate, Q_o, for the selected storage volume.

6.7 Routing Calculations

6.7.1 General

Routing of hydrographs through storage facilities is critical to the proper design of these facilities. Although storage design procedures using inflow/outflow analysis without routing have been developed, their use in designing detention facilities has not produced acceptable results in many areas of the country including the southeast. Thus, the methods like the Modified Rational Formula for storage design is not recommended for design calculations in Horry County.

Using the procedures given in Chapter 2 - Hydrology of this manual and this chapter, the designer should develop an inflow hydrograph for the design storms, stage-discharge curve, and stage-storage curve for the proposed storage facility. Then using a storage routing computer program, proceed with the design of the storage facility to determine the required storage volume and characteristics of the outlet device(s) that will be needed to control the 10-year and 25-year storms and accommodate the 100-year storm.

6.7.2 Downstream Effects

For all storage facilities, channel routing calculations should proceed downstream to a suitable location (normally to the point where the controlled land area is less than 10% of the total drainage to that point). At this point, the effect of the hydrograph routed through the proposed storage facility on the downstream hydrograph should be assessed and shown not to have detrimental effects on downstream areas. This Downstream Analysis at the 10% location is described in further detail in Chapter 1 - Introduction.

6.8 Trash Racks And Safety Grates

Trash racks and safety grates serve several functions:

- they trap larger debris well away from the entrance to the outlet works where they will not clog the critical portions of the works;
- they trap debris in such a way that relatively easy removal is possible;
- they keep people and large animals out of confined conveyance and outlet areas;
- they provide a safety system whereby persons caught in them will be stopped prior to the very high velocity flows immediately at the entrance to outlet works and persons will be carried up and onto the outlet works allowing for them to climb to safety; and
- Well-designed trash racks can have an aesthetically pleasing appearance.

When designed well trash racks serve these purposes without interfering significantly with the hydraulic capacity of the outlet (or inlet in the case of conveyance structures) (ASCE, 1985, Allred-Coonrod, 1991). The location and size of the trash rack depends on a number of factors including: head losses through the rack, structural convenience, safety, and size of outlet.

Trash racks at entrances to pipes and conduits should be sloped at about 3H:1V to 5H:1V to allow trash to slide up the rack with flow pressure and rising water level, the slower the approach flow the flatter the angle. Rack opening rules-of-thumb abound in the literature. Figure 6-8 gives opening estimates based on outlet diameter (UDFCD, 1992). Judgment should be used in that an area with higher debris (e.g. a wooded area) may require more opening space.

The bar opening space for small pipes should be less than the pipe diameter. For larger diameter pipes, openings should be 6 inches or less. Collapsible racks have been used in some places if clogging becomes excessive or a person becomes pinned to the rack. Alternately debris for culvert openings can be caught upstream from the opening by using pipes placed in the ground or a chain safety net (USBR, 1978, UDFCD, 1991). Racks can be hinged on top to allow for easy opening and cleaning.

The control for the outlet should not shift to the grate. Nor should the grate cause the headwater to rise above planned levels. Therefore, headlosses through the grate should be calculated. A number of empirical loss equations exist though many have difficult to estimate variables. Two will be given to allow for comparison.

Metcalf & Eddy (1972) give the following equation (based on German experiments) for losses. Grate openings should be calculated assuming a certain percentage blockage as a worst case to determine losses and upstream head. Often 40 to 50 percent is chosen as a working assumption.

$$\mathbf{H_g = K_{g_1} (w/x)^{4/3} (V_u^2/2g) \sin \theta_g} \qquad \textbf{(6.17)}$$

Where: H_g = head loss through grate, ft

K_{g_1}= bar shape factor:

- 2.42 - sharp edged rectangular
- 1.83 - rectangular bars with semicircular upstream faces
- 1.79 - circular bars
- 1.67 - rectangular bars with semicircular up- and downstream faces

w = maximum cross sectional bar width facing the flow, in.

x = minimum clear spacing between bars, in.

V_u = approach velocity, ft/s

θ = angle of the grate with respect to the horizontal, degrees

The Corps of Engineers (HDC, 1988) has developed curves for trash racks based on similar and additional tests. These curves are for vertical racks but presumably they can be adjusted, in a manner similar to the previous equation, through multiplication by the sine of the angle of the grate with respect to the horizontal.

$$\mathbf{H_g = K_{g_2} V_u^2/2g} \qquad \textbf{(6.18)}$$

Where K_{g_2} is defined from a series of fit curves as:

- sharp edged rectangular (length/thickness = 10)
  $K_{g_2} = 0.00158 - 0.03217\ A_r + 7.1786\ A_r^2$

- sharp edged rectangular (length/thickness = 5)
  $K_{g_2} = -0.00731 + 0.69453\ A_r + 7.0856\ A_r^2$
- round edged rectangular (length/thickness = 10.9)
  $K_{g_2} = -0.00101 + 0.02520\ A_r + 6.0000\ A_r^2$
- circular cross section
  $K_{g_2} = 0.00866 + 0.13589\ A_r + 6.0357\ A_r^2$

and A_r is the ratio of the area of the bars to the area of the grate section.

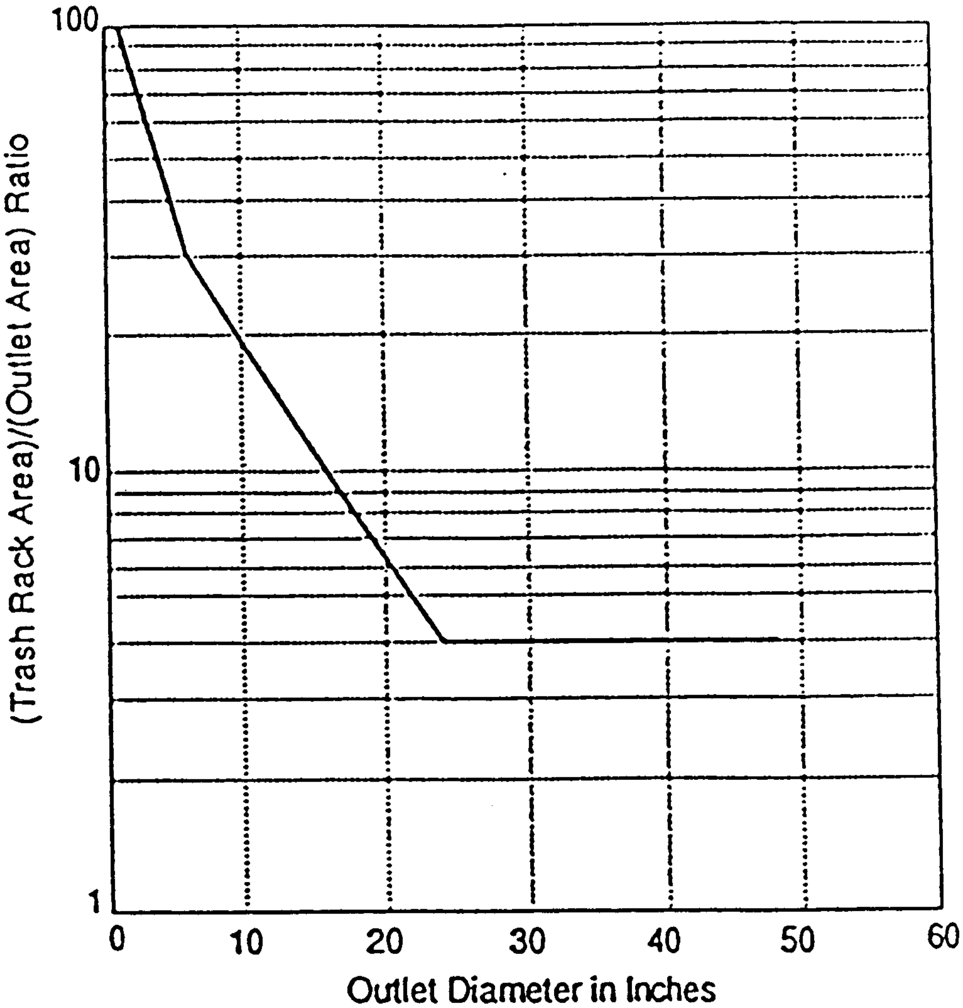


Figure 6-8 Minimum Rack Size vs. Outlet Diameter (UDCFD, 1992)

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CHAPTER

WATER QUALITY BEST MANAGEMENT PRACTICES 7

Chapter Table of Contents

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7.1.1 Introduction

Best management practices (BMPs) are the basic mitigation measures used in the stormwater quality management plans to control pollutants within Horry County. Section 7.2 of this chapter presents the details of structural best management practices and their use within the County drainage system. The other major category of BMPs include the many non-structural or source control practices that can be used for pollution prevention and control of pollutants. In most cases, it is much easier and less costly to prevent the pollutants from entering the drainage system than trying to control pollutants with structural BMPs. Thus, within the "treatment train" concept, the non-structural BMPs should be the first line of defense in protecting the receiving streams. If used properly, the non-structural BMPs can be very effective in controlling pollutants and greatly reduce the need for structural BMPs. In addition, non-structural BMPs tend to be less costly, easier to design and implement and easier to maintain than structural BMPs. Non-structural BMPs normally do not have technical or engineering designs associated with them but are measures that Horry County or other agencies or groups might require or implement to assist in the management of water quality and the control of pollutants within the County. Following is a brief discussion of some non-structural BMPs that can be used within a stormwater quality management plan for different portions of the Horry County drainage system.

7.1.2 Public Education/Participation

Public education/participation is not so much a best management practice as it is a method by which to implement BMPs. Public education/participation are vital components of many of the individual source control BMPs. A public education and participation plan provides the County with a strategy for educating its employees, the public, and businesses about the importance of protecting stormwater from improper use, storage, and disposal of pollutants. County employees must be trained, especially those that work in departments not directly related to stormwater but whose actions affect stormwater. Residents must become aware that a variety of hazardous products are used in the home and that their improper use and disposal can pollute stormwater and groundwater supplies. Businesses, particularly smaller ones that may not be regulated by Federal, State, or local regulations, must be informed of ways to reduce their potential to pollute stormwater.

7.1.3 Land Use Planning/Management

This BMP presents an important opportunity to reduce the pollutants in stormwater runoff by using a comprehensive planning process to control or prevent certain land use activities in areas where water quality is sensitive to development. It is applicable to all types of land use and represents one of the most effective pollution prevention practices. Subdivision regulations, zoning ordinances, preliminary plan reviews and detailed plan reviews, are tools that may be used to mitigate stormwater contamination in newly developing areas. Also, master planning, cluster development, terracing and buffers are ways to use land use planning as a BMP in the normal design for subdivisions and other urban developments. Limiting impervious areas is one of the most effective land use management tools, since nationwide research has consistently documented increases in pollution loads with increases in impervious cover. In addition to controlling impervious area cover, directly connected impervious areas should be kept to a minimum. This is especially important for large impervious areas such as parking lots and highways and it can also be effective for small impervious areas such as roof drainage.

7.1.4 Material Use Controls

There are three major BMPs included in this category:

1. Housekeeping Practices

2. Safer Alternative Products
3. Pesticide/Fertilizer Use

In housekeeping practices, the goal is to promote efficient and safe practices such as storage, use, cleanup, and disposal, when handling potentially harmful materials such as fertilizers, pesticides, cleaning solutions, paint products, automotive products, and swimming pool chemicals. In addition, the use of less harmful products can be promoted. Alternatives exist for most product classes including fertilizers, pesticides, cleaning solutions, and automotive and paint products.

Pesticides and fertilizers have become an important component of land use and maintenance for municipalities, commercial land uses and residential landowners. Any usage of pesticides and fertilizers increases the potential for stormwater pollution. BMPs for pesticides and fertilizers include education in their use, control runoff from affected areas, control times when they are used, provide proper disposal areas, etc.

7.1.5 Material Exposure Controls

There are two major BMPs included in this category:

1. Material Storage Control
2. Vehicle Use Reduction

Material storage control is used to prevent or reduce the discharge of pollutants to stormwater from material delivery and storage by minimizing the storage of hazardous materials onsite, storing materials in a designated area, installing secondary containment, conducting regular inspections, and training employees and subcontractors.

Vehicle use reduction is used to reduce the discharge of pollutants to stormwater from vehicle use by highlighting the stormwater impacts, promoting the benefits to stormwater of alternative transportation, and integrating initiatives with existing or emerging regulations and programs.

7.1.6 Material Disposal And Recycling

There are three major BMPs included in this category:

1. Storm Drain System Signs
2. Household Hazardous Waste Collection
3. Used Oil Collection

Stenciling of the storm drain system (inlets, catch basins, channels, and creeks) with prohibitive language/graphic icons discourages the illegal dumping of unwanted materials. Storm drain system signs act as highly visible source controls that are typically stenciled directly adjacent to storm drain inlets.

Household hazardous wastes are defined as waste materials which are typically found in homes or similar sources, which exhibit characteristics such as: corrosivity, ignitability, reactivity, and/or toxicity, or are listed as hazardous materials by the EPA. Household hazardous waste collection programs are a preventative rather than curative measure and may reduce the need for more elaborate treatment controls. Programs can be a combination of permanent collection centers, mobile collection centers, curbside collection, recycling, reuse, and source reduction.

Used oil recycling is a responsible alternative to improper disposal practices such as dumping oil in the sanitary sewer or storm drain system, applying oil to roads for dust control, placing used oil and filters in the trash for disposal to landfill, or simply pouring used oil on the ground.

Commonly used oil collection alternatives are a temporary "drop off" site on designated collection days or the use of private collectors such as automobile service stations, quick oil change centers and auto parts stores.

7.1.7 Spill Prevention And Cleanup

There are two major BMPs included in this category:

1. Vehicle Spill Control
2. Aboveground Tank Spill Control

The purpose of a vehicle spill control program is to prevent or reduce the discharge of pollutants to stormwater from vehicle leaks and spills by reducing the chance for spills by preventive maintenance, stopping the source of spills, containing and cleaning up spills, properly disposing of spill materials, and training employees. It is also very important to respond to spills quickly and effectively.

Aboveground tank spill control programs prevent or reduce the discharge of pollutants to stormwater by installing safeguards against accidental releases, installing secondary containment, conducting regular inspections, and training employees in standard operating procedures and spill cleanup techniques.

7.1.8 Dumping Controls

This BMP addresses the implementation of measures to detect, correct, and enforce against illegal dumping of pollutants on streets and into the storm drain system, streams, and creeks. Substances illegally dumped on streets and into the storm drain system and creeks include paints, used oil and other automotive fluids, construction debris, chemicals, fresh concrete, leaves, grass clippings, and pet wastes.

7.1.9 Connection Controls

There are three major BMPs included in this category:

1. Illicit Connection Prevention
2. Illicit Connection Detection and Removal
3. Leaking Sanitary Sewer Control

Illicit connection protection tries to prevent unwarranted physical connections to the storm drain system from sanitary sewers, floor drains, etc., through regulation, regular inspection, testing, and education. In addition, programs include implementation control procedures for detection and removal of illegal connections from the storm drain conveyance system. Procedures include field screening, follow-up testing, and complaint investigation.

Leaking sanitary sewer control includes implementing control procedures for identifying, repairing, and remediating infiltration, inflow, and wet weather overflows from sanitary sewers into the storm drain conveyance system. Procedures include field screening, testing, and complaint investigation.

7.1.10 Street/Storm Drain Maintenance

There are seven major BMPs included in this category:

1. Roadway Cleaning

2. Catch Basin Cleaning
3. Vegetation Controls
4. Storm Drain Flushing
5. Roadway/Bridge Maintenance
6. Detention/Infiltration Device Maintenance
7. Drainage Channel/Creek Maintenance

Roadway cleaning may help reduce the discharge of pollutants to stormwater from street surfaces by conducting cleaning on a regular basis. However, cleaning often removes the larger sizes of pollutants but not the smaller sizes. Most pollutants are deposited within three feet of the curb which is where the roadway cleaning should be concentrated. Catch basin cleaning on a regular basis also helps reduce pollutants in the storm drain system, reduces high pollutant concentrations during the first flush of storms, prevents clogging of the downstream conveyance system and restores the catch basins' sediment trapping capacity.

Vegetation control typically involves a combination of chemical (herbicide) application and mechanical methods. Mechanical vegetation control includes leaving existing vegetation, cutting less frequently, hand-cutting, planting low maintenance vegetation, mulching, collecting and properly disposing of clippings and cuttings, and educating employees.

Storm drains can be "flushed" with water to suspend and remove deposited materials. Flushing is particularly beneficial for storm drain pipes with grades too flat to be self-cleansing. Flushing helps ensure pipes convey design flow and removes pollutants from the storm drain system. However, flushing will only push the pollutants into downstream receiving waters unless the discharge from the flushing is captured and removed from the drainage system.

Roadway/bridge maintenance is used to prevent or reduce the discharge of pollutants to stormwater by paving as little are as possible, designing bridges to collect and convey stormwater to proper locations, using measures to prevent runoff from entering the drainage system, properly disposing of maintenance wastes, and training employees.

Proper maintenance and silt removal is required on both a routine and corrective basis to promote effective stormwater pollutant removal efficiency for wet and dry detention ponds and infiltration devices. Also, regularly removing illegally dumped items and material from storm drainage channels and creeks will reduce pollutant levels.

7.1.11 Permanent Erosion Control

There are three major BMPs included in this category:

1. Erosion Control - Permanent Vegetation
2. Erosion Control - Flow Control
3. Erosion Control - Channel Stabilization

Vegetation is a highly effective method for providing long term, cost effective erosion protection for a wide variety of conditions. It is primarily used to protect the soil surface from the impact of rain and the energy of the wind. Vegetation is also effective in reducing the velocity and sediment load in runoff sheet flow.

Channel stabilization addresses the problem of erosion due to concentrated flows. Concentrated flows occur in channels, swales, creeks, rivers and other watercourses in which a substantial drainage area drains into a central point. Overland sheet flow begins to collect and concentrate in the form of rills and gullies after overland flow of as little as 100 feet. Erosion due to concentrated flow is typically extensive, causing large soil loss, undermining foundations and decreasing the flow capacity of watercourses.

Proper selection of ground cover is dependent on the type of soil, the time of year of planting, and the anticipated conditions that the ground cover will be subjected. In addition, mulching is a form of erosion protection that is commonly used in conjunction with establishment of vegetation. It typically improves infiltration of water, reduces runoff, holds seed, fertilizer and lime in place, retains soil moisture, helps maintain temperatures, aids in germination, retards erosion and helps establish plants in disturbed areas.

Once flow is allowed to concentrate, it is more difficult to control erosion problems. Thus, every effort should be made to maintain sheet flow conditions for runoff. Where concentrated flows are unavoidable, the following techniques can be used to control erosion and resulting water quality problems.

- Riprap
- Level Spreaders
- Gabions
- Armor Protection
- Check Dams
- Diversions

For more information on erosion control consult the publication, Erosion and Sediment Control Practices For Developing Areas, South Carolina Land Resources Conservation Commission, Erosion and Sediment Control Division.

7.2 Structural BMP Specifications

7.2.1 Introduction

To provide some guidance in the design and use of different structural BMPs, this section gives specifications and performance standards for several BMPs that could find application within Horry County.

Following are the required specifications, recommended specifications, operation and maintenance requirements, and performance standards for nine different structural BMPs. For the design of extended dry detention basins and wet detention ponds refer to Chapter 6 - Storage Facilities for examples of storage design. For grassed swales, refer to Chapter 5 - Open Channel Design for examples of channel design. For example designs of the several infiltration facilities included as BMPs in the chapter, refer to Appendix A at the end of this chapter.

- Sediment Forebay
- Extended Dry Detention Basins
- Wet Detention Ponds
- Sand Filters
- Constructed Wetlands
- Infiltration Trenches
- Filter Strips and Flow Spreaders
- Grassed/Biofiltration Swales
- Oil/Grit Separators

For each structural BMP, performance standards are included to give a general idea of the pollution removal rates of different structural BMPs. The general design criteria for Horry County is to design for the water quality volumes as specified by the South Carolina Office of Ocean and Coastal Resource Management (OCRM). However, the literature related to performance standards often gives removal rates for different design criteria (e.g., 24-hour detention time, 0.5 inch per impervious area). Thus, these values, which may differ from OCRM criteria, should only be used for comparison of different structural BMPs and not for specific designs or to state accurate removal rates for different BMPs. Note that with the addition of appropriate vegetation, extended dry detention basins and wet detention ponds can function as bioretention facilities.

7.2.2 Sediment Forebay

For many of the BMPs included in this chapter, especially ponds and infiltration facilities, sediment forebays or equivalent upstream pretreatment should be included. Following are the general criteria to be used for sediment forebay design:

- The forebay should consist of a separate cell, formed by an acceptable barrier.
- The forebay should be sized to contain 0.1 inches of runoff per impervious acre of contributing drainage. The forebay storage volume counts toward the total water quality storage requirements.
- Exit velocities from the forebay should be non-erosive.
- Direct maintenance access for appropriate equipment should be provided to the forebay.
- The bottom of the forebay may be hardened (e.g., using concrete, paver blocks, etc.) to make sediment removal easier.
- A fixed vertical sediment depth maker should be installed in the forebay to measure sediment deposition over time.
- Sediment removal in forebay should occur when 50% of the total capacity has been lost.

7.2.3 Extended Dry Detention Basins

<u>Standard Specifications For Extended Dry Detention Basins</u>

<u>Required Specifications</u>

Extended dry detention basins should be designed with a detention time of 48 hours. If the extended dry detention basin is to be designed for only water quality purposes, then the basin should be designed to capture the water quality volumes as specified by OCRM.

- Pilot channel of paved or concrete material for erosion control (alternately use turf if there is little low flow). Size such that any event runoff will overflow the low flow channel onto the basin floor.
- The floor of the basin should have a minimum slope of 2 percent toward the low flow channel.
- Flowpaths from inflow points to outlets should be maximized. Flowpaths of 1.5:1 (length relative to width) and irregular shapes are recommended.
- Side slopes should be no greater than 3:1 if mowed.
- Inlet and outlet located to maximize flow length.
- Design for full development upstream of control.
- Riprap protection (or other suitable erosion control means) should be provided for the outlet and all inlet structures into the basin.
- One (1) foot minimum freeboard above peak stage for top of embankment.
- Emergency spillway designed to pass the 100-year storm event (must be paved in fill areas).

- Each basin should have a drain pipe that can drain the pond within 24 hours.
- Maintenance access minimum of 25 feet wide.
- Trash racks, filters or other debris protection provided on outlet.
- Anti-vortex plates should be used on outlet.
- Insure no outlet leakage and use anti-seep collars.
- Provide benchmark for sediment removal.

Recommended Specifications

- Two stage design (Top stage – intended to be dry except for stormwater runoff from larger, infrequent storm events. Bottom stage – sized to store up to 50% of the water quality volume.)
- Top stage should have slopes between 2% and 5% and a depth of 2 to 5 feet.
- Bottom stage should have depths from 1.5 to 3 feet deep with a shallow wetland or pool (6 to 12 in.).
- Manage buffer and basin as meadow.
- Minimum 25-foot wide buffer around pool.
- Provide on-site disposal areas for two sediment removal cycles.
- Anti-seep collars or filter diaphragms should be used on barrel of principal spillway.
- Design as off-line basin to bypass larger flows.
- Design as sediment settling basin for pretreatment of the larger particles.

Operation And Maintenance Recommendations

A stormwater management easement and maintenance agreement should be required for each facility.

- Extended dry detention basins are used where lack of water or other multi-use considerations preclude the use of wet detention ponds or constructed wetlands.
- Operation and maintenance is the same as for wet detention ponds.
- Maintenance activities include keeping the outlets unclogged, control of vegetation, removal of sediment deposits, and keep aesthetics of area acceptable.

Performance Standards

- Soluble pollutant removal rates are low for extended dry detention basins but can be enhanced either with greatly increased detention time, through the use of shallow marshes to increase biological uptake, or through using an infiltration device downstream from the outlet orifice.
- Average annual pollutant removal capability of extended dry detention basins are as follows:

Pollutant	**1 Inch Rain Detained 24 hours**	**Same as Previous w/ Shallow Marsh**
Sediment	80-100%	80-100%
Total Phosphorus	40-60%	60-80%
Total Nitrogen	20-40%	40-60%
BOD	40-60%	40-60%
Metals	60-80%	60-80%

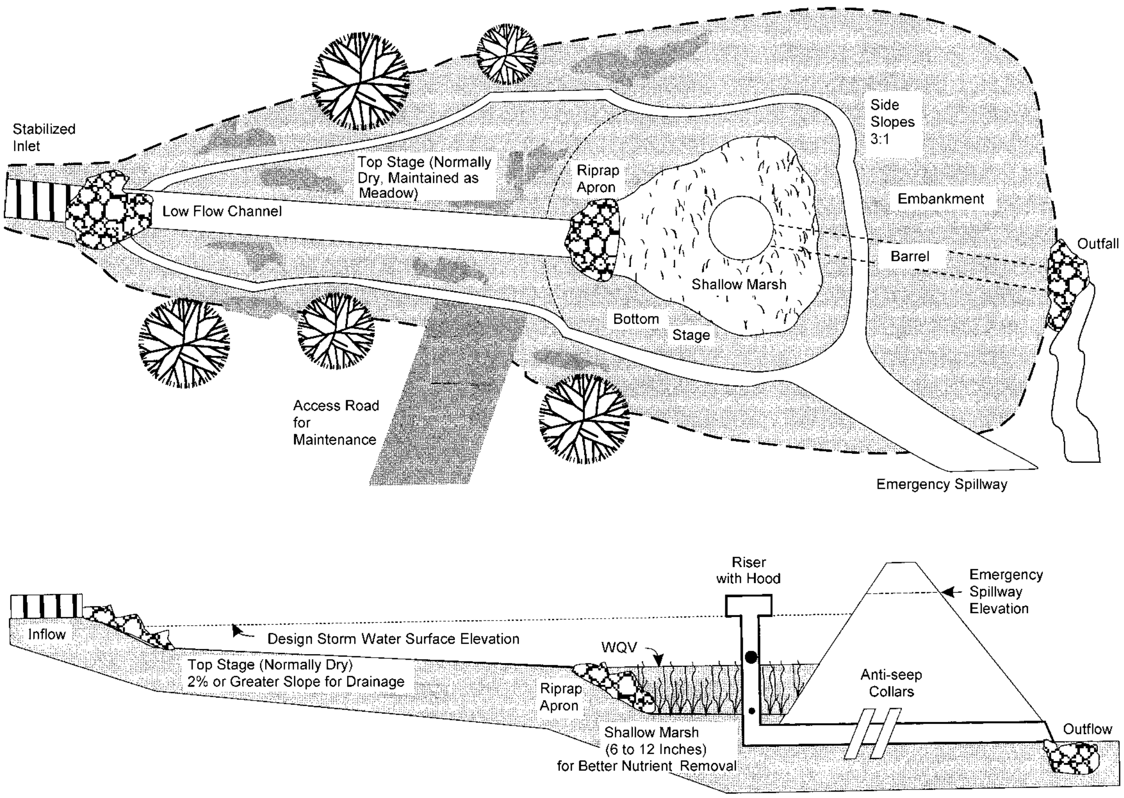

Figure 7-1 Extended Dry Detention Basin

Source: Controlling Urban Runoff

7.2.4 Wet Detention Ponds

Standard Specifications For Wet Detention Ponds

Required Specifications

Wet detention ponds should be designed with a minimum detention time of 48 hours. If the wet detention pond is to be designed for only water quality purposes, then the pond should be designed to capture the water quality volumes as specified by OCRM.

- Minimum length to width ratio of 3:1 (preferably expanding outward toward the outlet). Irregular shorelines for larger ponds also provide visual variety.
- Inlet and outlet located to maximize flow length. Use baffles if short circuiting cannot be prevented with inlet-outlet placement. Long flowpaths and irregular shapes are recommended.
- Minimum depth of permanent pool 2 to 3 feet, maximum depth of 6 to 8 feet. Average depth should be 3 to 7 feet.
- Design for full development upstream of control.

- Side slopes should be no greater than 3:1 if mowed.
- Riprap protection should be provided (or other suitable erosion control means) for the outlet and all inlet structures into the pond. Individual boulders or baffle plates can work for this.
- Minimum drainage area of 10 acres.
- Anti-seep collars or filter diaphragms should be provided on barrel of principal spillway.
- If reinforced concrete pipe is used for the principal spillway, O-ring gaskets (ASTM C361) should be used to create watertight joints.
- One (1) foot minimum freeboard above peak stage for top of embankment.
- Emergency drain; i.e. sluice gate, drawdown pipe; capable of draining within 24 hours should be installed.
- Emergency spillway designed to pass the 100-year storm event.
- Bypass greater than the design storms.
- Trash racks, filters, hoods or other debris control provided on riser.
- Principal spillway/riser should incorporate anti-floatation, anti-vortex, and trash-rack designs.
- Maintenance access minimum of 25 feet wide.
- Provide benchmark for sediment removal.

<u>Recommended Specifications</u>

- Multi-objective use such as amenities or flood control.
- Landscaping management of buffer as meadow.
- Minimum length to width ratio of 3:1 to 4:1 (preferably wedge shaped).
- Use reinforced concrete instead of corrugated metal for pipes.
- Sediment forebay for larger ponds (often designed for 5 to 15 percent of total volume). Forebay should have separate drain for de-watering. Grass biofilters for smaller ponds.
- Consider artificial mixing for small sheltered ponds.
- Provision should be made for vehicle access at a 4:1 slope.
- Impervious soil boundary to prevent drawdown may be needed.
- Shallow marsh area around fringe 25 to 50 percent of area (including aquatic vegetation) should be established.
- A safety bench with a minimum width of 10 feet should be provided around the permanent pool.
- The perimeter of all deep permanent pool areas (four feet or greater in depth) should be surrounded by two benches with a combined minimum width of 15 feet:
  - A safety bench that extends outward from the normal water edge to the toe of the pond side slope. The maximum slope of the safety bench should be 6%.
  - An aquatic bench that extends inward from the normal shoreline and has a maximum depth of eighteen inches below the normal pool water surface elevation. An aquatic bench is not required in forebays.
- Minimum 25 foot wide buffer around pool.
- On-site disposal areas, for two sediment removal cycles, should be provided and protected from runoff.
- An oil and grease skimmer may be needed for sites with high production of pollutants.

<u>Operation And Maintenance Recommendations</u>

- Sediment to be removed when 20% of storage volume of the facility is filled (design storage volume must account for volume lost to sediment storage).
- Sediment traps should be cleaned out when filled.
- No woody vegetation should be allowed on the embankment without special designs.

- Vegetation over 18 inches high should be cut unless it is part of planned landscaping.
- Debris should be removed from blocking inlet and outlet structures and from areas of potential clogging.
- The outlet control should be kept structurally sound, free from erosion, and functioning as designed.
- Periodic removal of dead vegetation should be accomplished.
- Inspection requirements should be outlined in the maintenance agreement.
- The site should be inspected and debris removed after every major storm.
- All special maintenance responsibilities should be listed in the maintenance agreement.
- Mow embankment and side slopes at least twice a year.
- Consider chemical treatment by alum if algal blooms are a problem.

Performance Standards

- Wet ponds are very effective in removal of both the soluble and particulate fractions of pollution.
- Average annual pollutant removal capability of wet detention ponds are as follows:

Pollutant	**0.5 Inch Per Impervious Acre**
Sediment	60-80%
Total Phosphorus	40-60%
Total Nitrogen	20-40%
BOD	20-40%
Metals	20-40%

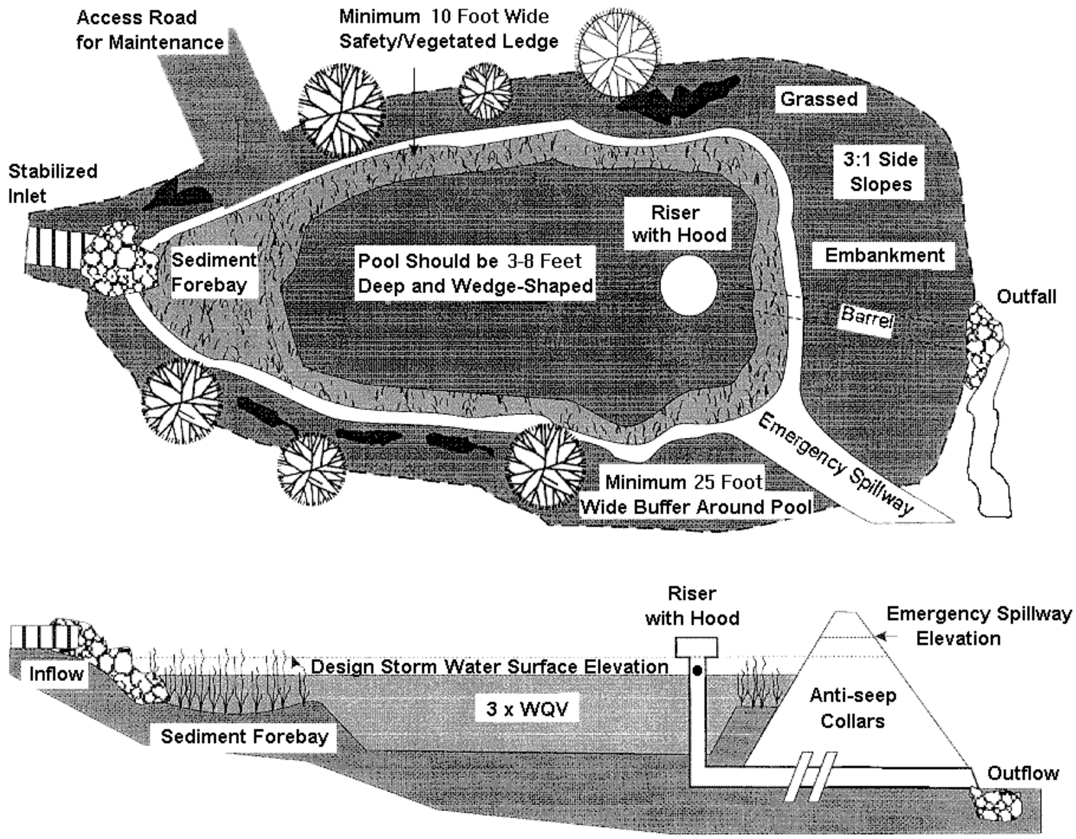

Figure 7-2 Wet Detention Pond

Source: Controlling Urban Runoff

7.2.5 Sand Filters

<u>Standard Specifications For Sand Filters</u>

<u>Required Specifications</u>

- Maximum contributing drainage area to an individual stormwater filtering system is usually less than 10 acres.
- Design volume as per OCRM requirements.
- Adequate pretreatment (e.g., filter strips) is required to prevent sediment from overloading the filters.
- Most stormwater filters normally require one to six feet of head.
- Designed to completely empty in 36 hours.
- Inlet structure should be designed to spread the flow uniformly across the surface of the filter media.
- Stone riprap or other dissipation devices should be installed to prevent gouging of the sand media and to promote uniform flow.
- Final sand bed depth should be at least 18 inches.

- Underdrain pipes should consist of main collector pipes and perforated lateral branch pipes.
- The underdrain piping should be reinforced to withstand the weight of the overburden.
- Internal diameters of lateral branch pipes should be 4 inches or greater (6 inches preferred) and perforations should be 3/8 inch.
- Maximum spacing between rows of perforations should not exceed 6 inches.
- All piping should be schedule 40 polyvinyl chloride or greater strength.
- Minimum grade of piping should be 1/8 inch per foot (1% slope).
- Access for cleaning all underdrain piping should be provided.
- Surface filters may have a grass cover to aid in pollution adsorption.
- Vegetation should be established over the contributing drainage areas before runoff can be accepted into the facility.

<u>Recommended Specifications</u>

- Two sand bed configurations are recommended for use:

1) Sand Bed with Gravel Layer;
   - Top layer of sand should be a minimum of 18 inches of 0.02 - 0.04 inch diameter sand (smaller sand size is acceptable).
   - A layer of one-half to 2-inch diameter gravel under the sand should be provided for a minimum of 2 inches of cover over the top of the underdrain lateral pipes.
   - No gravel is required under the lateral pipes.
   - The sand and gravel should be separated by a layer of geotextile fabric (permeable filter fabric).

2) Sand Bed with Trench Design;
   - Top layer of sand is to be 12-18 inches of 0.02 - 0.04 inch diameter sand (smaller size is acceptable).
   - Laterals to be placed in trenches with a covering of one-half to 2-inch gravel and geotextile fabric.
   - The lateral pipes are to be underlain by a layer of drainage matting.
   - A presettling basin and/or biofiltration swale is recommended to pretreat runoff discharging to the sand filter.
   - A maximum spacing of 10 feet between lateral underdrain pipes is recommended.

<u>Operation And Maintenance Recommendations</u>

- A stormwater management easement and maintenance agreement should be required for each facility. The maintenance covenant should require the owner of the sand filter to periodically clean the structure.
- Scrape off sediment layer buildup during dry periods with steel rakes or other devices.
- Replace some or all of the sand when permeability of the filter media is reduced to unacceptable levels, which should be specified in the design of the facility. A minimum infiltration rate of 0.5 inches per hour should be used for all infiltration designs.

<u>Performance Standards</u>

- Sand Filtration Basins
  - All runoff up to design volume is filtered through sand bed.

- The storage volume is based on runoff volume of the 1 inch rainfall event.
- Estimated long-term pollutant removal rates as follows:

	Pollutant	Removal Rate
Primary Pollutants	Total Phosphorus	65%
	Lead	50-70%
	BOD	60%
Other Pollutants	Sediment	85%
	Total Nitrogen	50%
	Zinc	60-80%
	COD	80%
	Bacteria	50-60%

- Filtration System Performance Enhancement
  - Sand/peat beds have higher removal effectiveness due to adsorptive properties of peat.
  - Designs incorporating vegetative cover on the filter bed increase nutrient removal.
- Pretreatment (sedimentation or oil and grease removal) will enhance the performance of the filter and will decrease the maintenance frequency required to maintain effective performance.

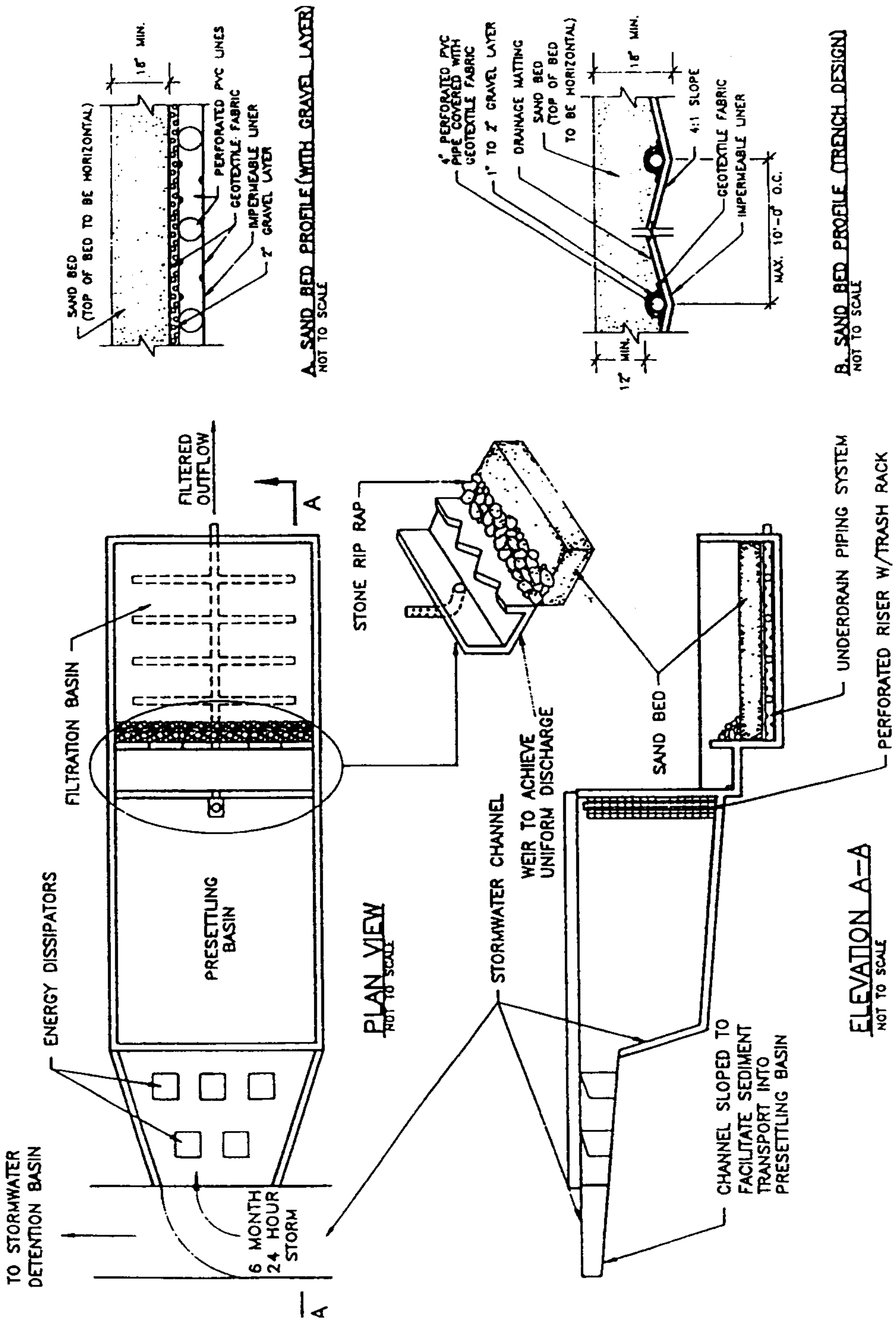

Figure 7-3 Sand Filtration Basin

Source: Stormwater Management Manual For The Puget Sound Basin

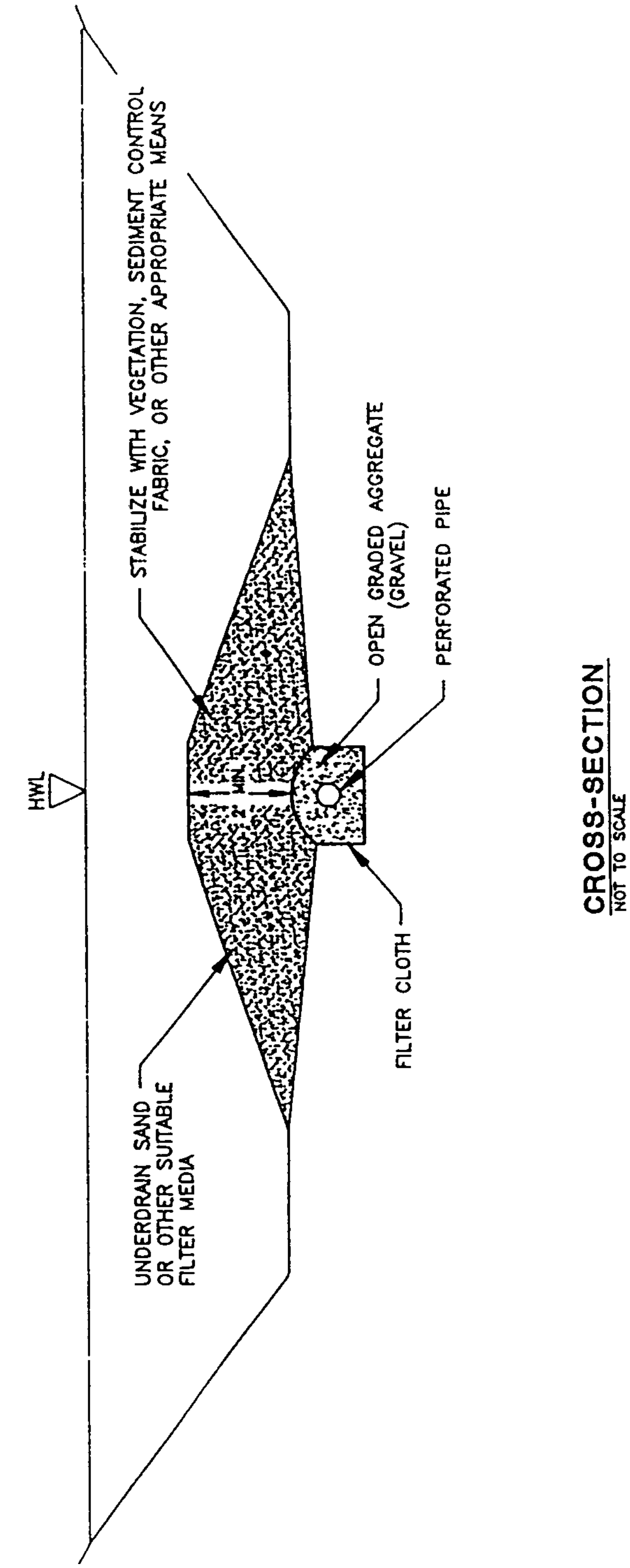

Figure 7-4 Cross-Section Of Elevated Sand Filter

Source: Florida Erosion And Sediment Control Handbook

7.2.6 Constructed Wetlands

Standard Specifications For Constructed Wetlands

Required Specifications

- Inflow of water must be greater than that leaving the basin by infiltration or exfiltration.
- A water balance should be performed to demonstrate that a stormwater wetland can withstand a thirty day drought at summer evaporation rates without completely drawing down.
- Designed for an extended detention time of 48 hours for the design volume specified by OCRM.
- The orifices used for extended detention will be vulnerable to blockage from plant material or other debris that will enter the basin with stormwater runoff. Therefore, some form of protection against blockage should be installed (such as some type of non-corrodible wire mesh).
- Surface area of the wetland should account for a minimum of 1 percent of the area of the watershed draining into it (1.5 percent for a shallow marsh design).
- The length to width ratio should be at least 2 to 1.
- A soil depth of at least 4 inches should be used for shallow wetland basins.
- A minimum of 35 percent of the total surface area should have a depth of six inches or less and at least 65 percent of the total surface area should be shallower than 18 inches.
- The deeper area of the wetland should include the outlet structure so outflow from the basin is not interfered with by sediment buildup.
- A forebay should be established at the pond inflow points to capture larger sediments and be 4 to 6 feet deep. Direct maintenance access to the forebay should be provided with access 25 feet wide minimum and 5:1 slope maximum. Sediment depth markers should be provided.
- If high water velocity is a potential problem, some type of energy dissipation device should be installed.
- The designer should maximize use of existing- and post-grading pondscaping design to create both horizontal and vertical diversity and habitat.
- A minimum of 2 aggressive wetland species (primary species - Figure 7-5) of vegetation should be established in quantity on the wetland.
- Three additional wetland species (secondary species- Figure 7-5) of vegetation should be planted on the wetland, although in far less numbers than the two primary species.
- 30 to 50 percent of the shallow (12 inches or less) area of the basin should be planted with wetland vegetation. The optimal depth requirements for several common species of emergent wetland plants are often six inches of water or less.
- Approximately 50 individuals of each secondary species should be planted per acre; set out in 10 clumps of approximately 5 individuals and planted within 6 feet of the edge of the pond in the shallow area leading up to the ponds edge; spaced as far apart as possible, but no need to segregate species to different areas of the wetland.
- Wetland mulch, if used, should be spread over the high marsh area and adjacent wet zones (-6 to +6 inches of depth) to depths of 3 to 6 inches.
- A minimum 25 foot buffer, for all but pocket wetlands, should be established and planted with riparian and upland vegetation (50 foot buffer if wildlife habitat value required in design). An additional 15 feet setback to structures should be included.
- Surrounding slopes should be stabilized by planting in order to trap sediments and some pollutants and prevent them from entering the wetland.

- A maintenance plan should be provided and adequate provision made for ongoing inspection and maintenance, with more intense activity for the first three years after construction.
- The wetland should be maintained to prevent loss of area of ponded water available for emergent vegetation due to sedimentation and/or accumulation of plant material.
- Local assistance should be obtained for information concerning plants to be used, planting schedule, soil requirements, mulch requirements, etc.

<u>Recommended Specifications</u>

- It is recommended that the frequently flooded zone surrounding the wetland be located within approximately 10 to 20 feet from the edge of the permanent pool.
- Soil types conducive to wetland vegetation should be used during construction.
- The wetland should be designed to allow slow percolation of the runoff through the substrate (add a layer of clay for porous substrates).
- The depth of the forebay should be in excess of 3 feet and contain approximately 10 percent of the total volume of the normal pool.
- As much vegetation as possible and as much distance as possible should separate the basin inlet from the outlet.
- Of the 75 percent of the wetland that should be 12 inches deep or less, it is recommended that approximately 25 percent range from 6 inches deep to 12 inches deep, and that the remaining 50 percent be 6 inches or less in depth.
- The water should gradually get shallower about 10 feet from the edge of the pond.
- The planted areas should be made as square as possible within the overall design of the wetland, rather than long and narrow.
- The only site preparation that is necessary for the actual planting (besides flooding the basin) is to ensure that the substrate is soft enough to permit relatively easy insertion of the plants.

<u>Operation And Maintenance Recommendations</u>

- A stormwater management easement and maintenance agreement should be required for each facility. The maintenance covenant should require the owner of the wetland to periodically clean the structure. The maintenance agreement should provide for ongoing inspection and maintenance, with more intense activity for the first three years after construction.
- The wetland should be maintained to prevent loss of area of ponded water available for emergent vegetation due to sedimentation and/or accumulation of plant material.
- Sediment forebays should be cleaned every 2 to 5 years except for pocket wetlands without forebays, which are cleaned after a six inch accumulation of sediment.
- The ponded water area may be maintained by raising the elevation of the water level in the permanent pond, by raising the height of the orifice in the outlet structure, or by removing accumulated solids by excavation.
- Water levels may need to be supplemented or drained periodically until vegetation is fully established.
- It may be desirable to remove contaminated sediment bottoms or to harvest above ground biomas and remove it from the site in order to permanently remove pollutants from the wetland.

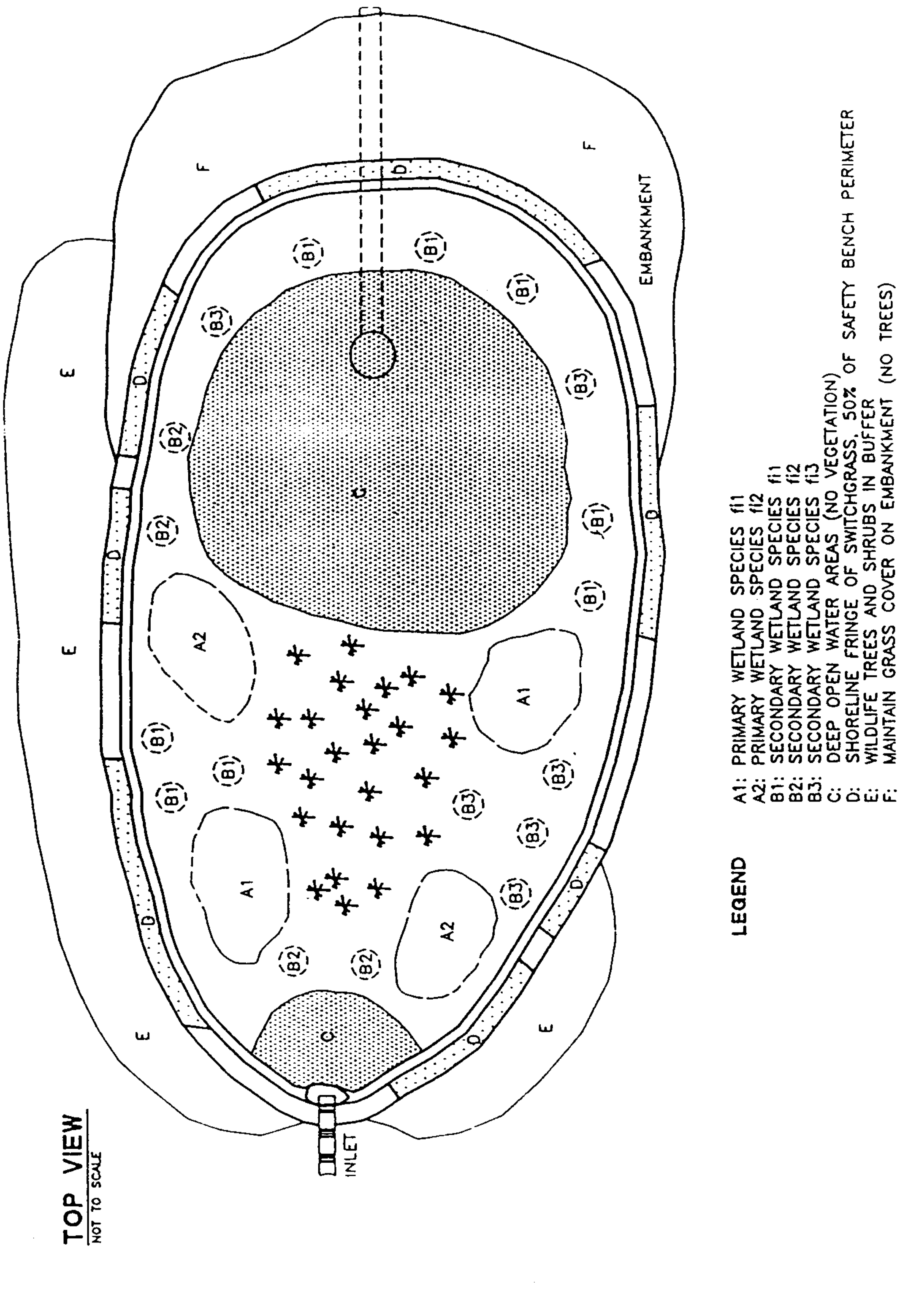

Figure 7-5 Shallow Marsh Planting Strategies for Constructed Wetlands

Source: Controlling Urban Runoff

<u>Performance Standards</u>

- Performance depends on appropriate plantings for the soils, climate, and types of pollutants or land use (oil and grease, high sediment loads, high nutrient loads) in the drainage area.
- Design performance depends on protecting marsh-type plantings.
- Performance enhancement can be obtained by increasing the size of the marsh area, by incorporating multiple pools into marsh area, or by incorporating a network of shallow channels in the marshy area.
- Estimated long-term pollutant removal rates as follows:

	Pollutant	**Removal Rate**
Primary Pollutants	Total Phosphorus	50-60%
	Lead	75-85%
	BOD	50-60%
Other Pollutants	Sediment	90-99%
	Total Nitrogen	40-50%
	Zinc	75-85%
	COD	55-65%

7.2.7 Infiltration Trenches

Standard Specifications For Infiltration Trenches

Required Specifications

- Used in small drainage areas less than 5 acres.
- Designed to drain the design water volume in 48 hours.
- A minimum of one soils boring is required for every 50 feet of trench length, and no less than 2 soils logs for each proposed trench location.
- Each soils boring should extend a minimum of 3 feet below the bottom of the trench, describe the NRCS series of the soil, the textural class of the soil horizon(s) through the depth of the log, and note any evidence of high ground water level, such as mottling. In addition, the location of impermeable soil layers or dissimilar soil layers should be determined.
- For runoff treatment, the soil infiltration rate should be between 0.5 and 2.4 inches per hour.
- Soil textures with minimum infiltration rates of 0.5 inches per hour or less are not suitable for infiltration trenches.
- Soils should have a clay content of less than 15 percent and a silt/clay content of less than 40 percent.
- Soils that have a 30 percent or greater clay content are not suitable for infiltration trenches.
- Soils that are suitable for infiltration systems are silt loam, loam, sandy loam, loamy sand, and sand.
- The use of infiltration systems on fill is not allowed due to the possibility of creating an unstable subgrade.
- A minimum of 3 feet difference is required between the bottom of the infiltration trench and the groundwater table and to bedrock.
- Site slope must be less than 20 percent, and the trench must be horizontal.
- The proximity of building foundations should be at least 25 feet horizontally.
- A minimum distance of 100 feet from water supply wells should be maintained when the

runoff is from industrial or commercial areas.

- The design infiltration rate should be equal to one-half the infiltration rate found from the soil textural analysis.
- Water quality infiltration trenches must be preceded by a pretreatment BMP.
- If the trench is preceded by a presettling basin, then the combination of both BMPs must be designed to drain the design water volume within 48 hours.
- Stone aggregate backfill material for the trench should have a maximum diameter of 3 inches and a minimum diameter of 1.5 inches. For design purposes, void space for these aggregates may be assumed to be in the range of 30 percent to 40 percent. Void ratio of 0.40 should be used to design stone reservoirs for infiltration practices.
- The aggregate should be completely surrounded with an engineering filter fabric. If the trench has an aggregate surface, filter fabric should surround all aggregate fill material except for the top one foot.
- Runoff must infiltrate through at least 18 inches of soil.
- An observation well should be installed for every 50 feet of trench length.
- The observation well should consist of perforated PVC pipe, 6 inches in diameter, located in the center of the structure, and be constructed flush with the ground elevation of the trench.
- The top of the observation well should be capped to discourage vandalism and tampering.
- Bypass larger flows.

Recommended Specifications

- Infiltration trenches work well for residential lots, commercial areas, parking lots, and open space areas.
- Can be installed under a swale to increase the storage of the infiltration system.
- Infiltration systems should not be constructed until all construction areas draining to them are fully stabilized.
- An analysis should be made to determine any possible adverse effects of seepage zones when there are nearby building foundations, basements, roads, parking lots, or sloping sites.

Operation And Maintenance Recommendations

- A stormwater management easement and maintenance agreement should be required for each facility. The maintenance covenant should require the owner of the infiltration trench to periodically clean the structure.
- The trench should be monitored after every large storm (rainfall greater than 1 inch in 24 hours) for the first year after completion of construction and be monitored quarterly thereafter.
- Sediment buildup in the top foot of stone aggregate or the surface inlet should be monitored on the same schedule as the observation well.

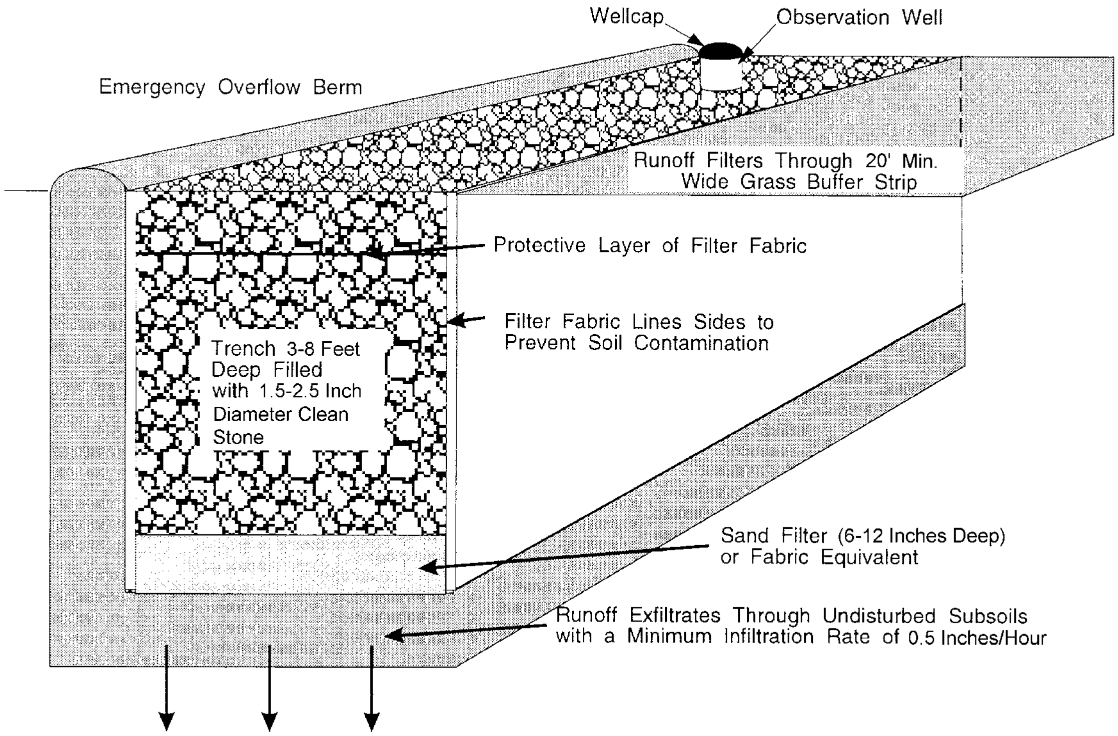

Figure 7-6 Infiltration Trench

Source: Stormwater Management Manual For The Puget Sound Basin

Performance Standards

- Full exfiltration trench
  - Runoff can only exit the trench by exfiltrating through the stone into the underlying soils.
  - The storage volume is based on runoff volume of the 1-inch rainfall storm.
  - Estimated long-term pollutant removal rates as follows:

	Pollutant	Removal Rate
Primary Pollutants	Total Phosphorus	65-75%
	Lead	95-99%
	BOD	90%
Other Pollutants	Sediment	90-99%
	Total Nitrogen	60-70%
	Bacteria	98%

- Water quality trench
  - The storage volume is based on first flush volume of 1 inch of runoff from the contributing area.

- Estimated long-term pollutant removal rates as follows:

	Pollutant	Removal Rate
Primary Pollutants	Total Phosphorus	60-70%
	Lead	85-90%
	BOD	80%
Other Pollutants	Sediment	90%
	Total Nitrogen	55-60%
	Bacteria	90%

- Partial exfiltration system
  - The trench is not designed to rely completely on exfiltration to dispose of the captured runoff volume, a perforated pipe is used to drain part of the volume, being placed either beneath or near the top of the trench.
  - The system is not as effective as a full exfiltration system.
  - Addition of a layer of very sandy soil over the gravel trench results in removal rates as high as 60 percent for the suspended sediment and trace metal loads, 50 percent for oxygen demand, and 40 percent for nutrient loads.

- Pretreatment
  - Pretreatment minimizes trench maintenance requirements.
  - Suspended sediment loads, which will clog the trench, can be reduced by requiring that the stormwater runoff pass through a 20-foot grassed filter strip prior to entering the trench.
  - Hydrocarbon loadings (oil and grease) that will clog the filter fabric and sand filter underlaying the trench can be reduced by the use of oil and grit chambers (when receiving large parking lot and roadway runoff).

7.2.8 Filter Strips And Flow Spreaders

Standard Specifications For Filter Strips And Flow Spreaders

Required Specifications

- The use of filter strips and flow spreaders should be limited to drainage areas of 10 acres or less with the optimal size being less than 5 acres.
- Capacity of the spreader and/or filter strip length (perpendicular to flow) should be determined by estimating the volume of flow that is diverted to the spreader for water quality control.
- Drainage area into spreader should be restricted so that maximum flow will not exceed 30 cfs.
- Channel grade for the last 20 feet of the dike or diversion entering the level spreader should be less than or equal to 1% and designed to provide a smooth transition into spreader.
- Grade of a level spreader should be 0%.
- Depth of a level spreader as measured from the lip should be at least 6 inches.

- Appropriate length, width, and depth of flow spreader should be selected from the following table.

Design Flow (cfs)	Entrance Width (ft)	Depth (ft)	End Width (ft)	Length (ft)

0 - 10	10	0.5	3	10
10 - 20	16	0.6	3	20
20 - 30	24	0.7	3	30

- The level spreader lip should be constructed on undisturbed soil (not fill material) to uniform height and zero grade over length of the spreader.
- The released runoff to the outlet should be on undisturbed stabilized areas in sheet flow and not allowed to reconcentrate below the structure.
- Slope of the filter strip from a level spreader should not exceed 10 percent.
- All disturbed areas should be vegetated immediately after construction.
- Filter strip width to be a minimum of 20 feet.

Recommended Specifications

- Top edge of filter strip should directly abut the contributing impervious area and follow the same elevation contour line.
- Runoff water containing high sediment loads to be treated in a sediment trapping device before release in a flow spreader.
- Spreader lip to be protected with erosion resistant material, such as fiberglass matting or a rigid non-erodible material for higher flows, to prevent erosion and allow vegetation to be established.
- Wooded filter strips are preferred to gravel strips.

Operation And Maintenance Recommendations

- A stormwater management easement and maintenance agreement should be required for each facility. The maintenance covenant should require the owner of the filter strip/flow spreader to periodically clean the structure.
- Flow spreader should be inspected after every rainfall until vegetation is established, and needed repairs made promptly.
- After area is stabilized, inspections should be made quarterly.
- Vegetation should be kept in a healthy, vigorous condition.
- Filter strip and flow spreader should be maintained in a manner to achieve sheet flow.

Performance Standards

- General Performance Information
    - Filter strips must accept stormwater runoff as overland sheet flow in order to effectively filter suspended materials out of the overland flow.
    - In order to function properly, the strip should be at least as wide as the flow path entering the filter, and flow entering a filter strip must be spread relatively uniformly over the width of the strip.
    - The removal of soluble pollutants is low because the degree of infiltration provided is generally very small.
    - Removals of nutrients and oxygen demand decrease as the amount of clay in the soil increases.

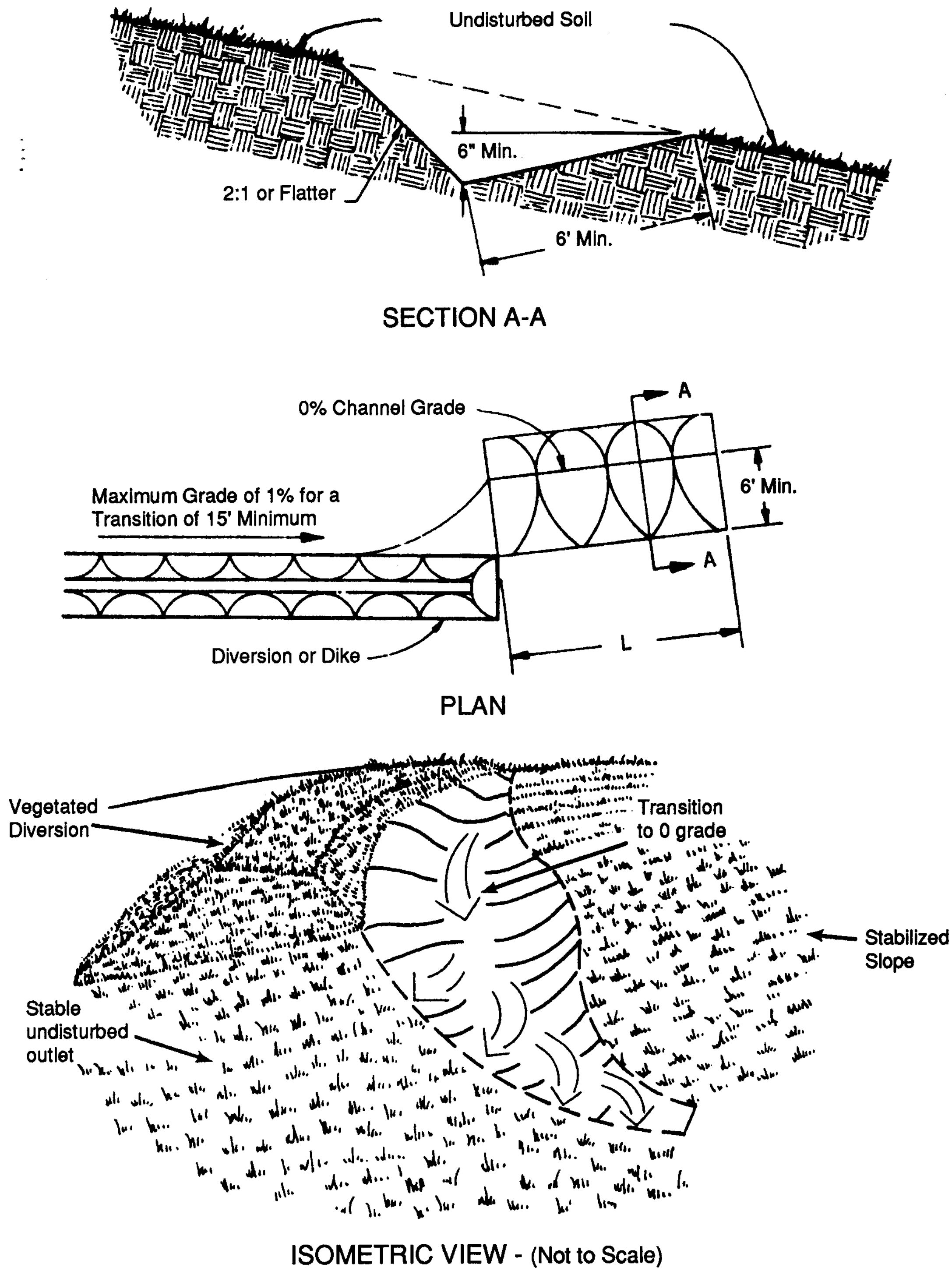

Figure 7-7 Flow Spreader

Source: North Carolina Erosion And Sediment Control Planning And Design Manual, 1988

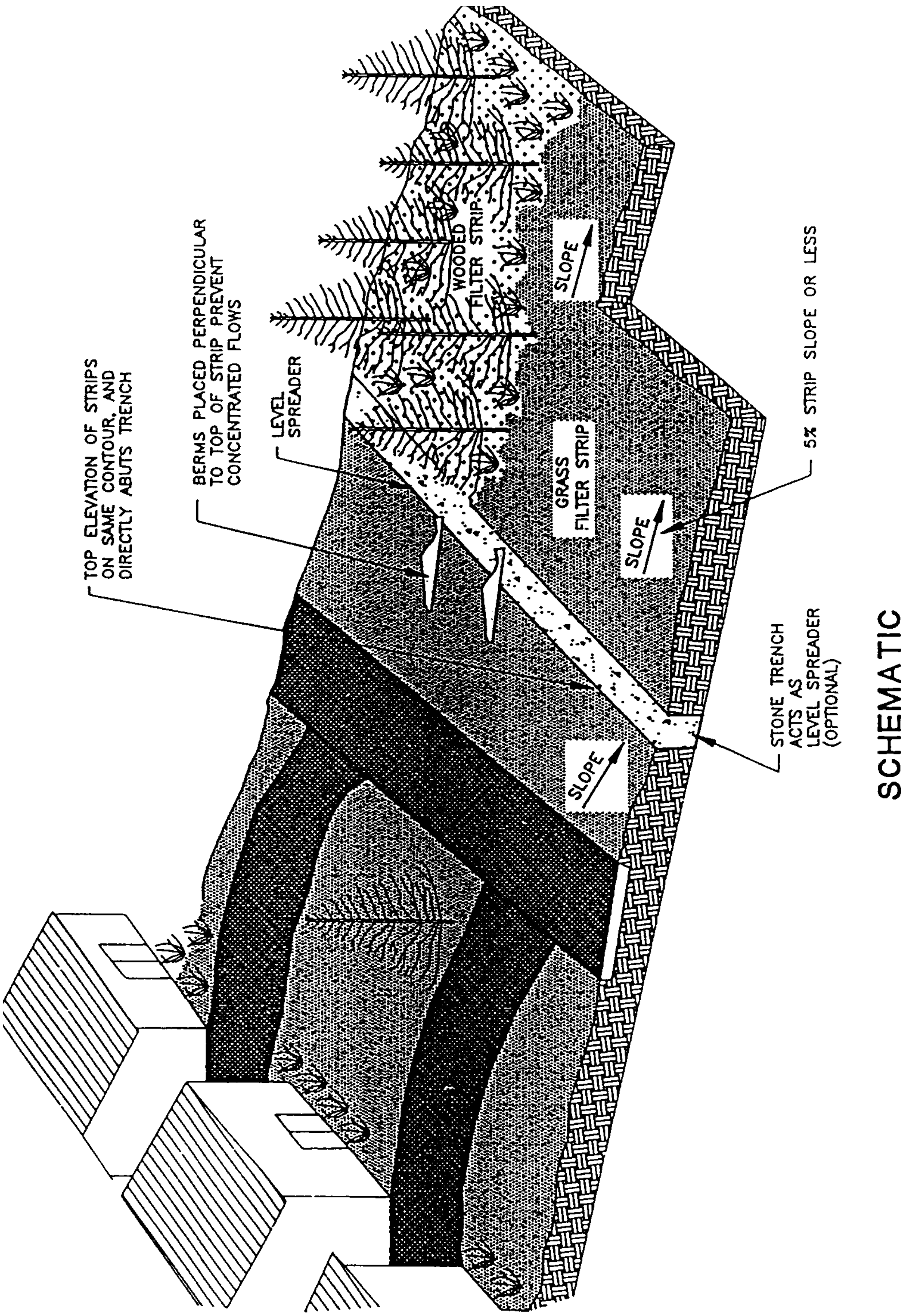

Figure 7-8 Schematic Of A Filter Strip

Source: Controlling Urban Runoff

- The use of filter strips to treat parking lot runoff or street runoff should incorporate a level spreading device such as a shallow stone filled trench or slotted parking blocks.

- 20-Foot Wide Grassed Filter Strip
  - Minimal pollutant removal. This design primarily removes the coarser suspended particles in runoff by the lowering of runoff velocities.
  - Pollutant removal enhanced by mild slopes, minimal mowing/ maintenance, sustaining natural cover if possible.
  - Long term estimated removal of pollutants is as follows:

	Pollutant	**Removal Rate**
Primary Pollutants	Total Phosphorus	10%
	Lead	30%
	BOD	10%
Other Pollutants	Sediment	30%
	Total Nitrogen	10%
	COD	10%
	Copper	30%
	Zinc	30%

- 100-Foot Wide Grassed Filter Strip
  - Maximal natural pollutant removal. This design removes both fine and coarse suspended particles in runoff by lowering runoff velocities over a significant length of flow path.
  - Pollutant removals can be enhanced by mild slopes, minimal mowing/maintenance, sustaining natural cover if possible.
  - Long term estimated removal of pollutants is as follows:

	Pollutant	**Removal Rate**
Primary Pollutants	Total Phosphorus	50%
	Lead	90%
	BOD	70%
Other Pollutants	Sediment	90%
	Total Nitrogen	50%
	COD	70%
	Copper	90%
	Zinc	90%

7.2.9 Grassed/Biofiltration Swales

Standard Specifications For Grassed Swales

Grassed swales are also described as biofiltration swales with the major difference being that grassed swales often have check dams where biofiltration swales do not.

Required Specifications

- Grassed swales should only convey standing or flowing water following a storm.

- As a water quality BMP, grass swales should be designed for the water quality volumes specified by OCRM. If the entire channel design storm is to be accommodated in the swale (e.g., 25-year), then the swale should be designed for this event.
- Limited to peak discharges generally less than 5 to 10 cfs.
- Limited to runoff velocities less than 2.5 ft/s.
- Maximum design flow depth to be 1 foot.
- Swale slopes should be graded as close to zero as drainage will permit.
- Swale slope should not exceed 4 percent (2 percent is preferred).
- Swale cross-section should have side slopes of 3:1 (h:v) or flatter.
- Underlying soils should have a high permeability (fc > 0.5 inches per hour).
- Swale area should be tilled before grass cover is established.
- Dense cover of a water tolerant, erosion resistant grass should be established.
- To obtain credit as a water quality BMP, grassed swales must have a minimum length of 100 feet.

Recommended Specifications

- As a BMP, grassed swales are limited to residential or institutional areas where percentage of impervious area is relatively small.
- Seasonally high water table to be greater than 3 feet below the bottom of the swale.
- Check dams can be installed in swales to promote additional infiltration. Recommended method is to sink a railroad tie halfway into the swale. Riprap stone should be placed on the downstream side to prevent erosion.
- Maximum ponding time behind check dam to be less than 48 hours. Minimum ponding time of 30 minutes is recommended to meet water quality goals.

Operation And Maintenance Recommendations

- A stormwater management easement and maintenance agreement should be required for each facility. The maintenance covenant should require the owner of the grassed swale to periodically clean the structure.
- Grass swales should be maintained to keep grass cover dense and vigorous.
- Maintenance should include periodic mowing, occasional spot reseeding, and weed control.
- Swale grasses should never be mowed close to the ground. Grass heights in the 4 to 6 inch range are recommended.
- Fertilization of grass swales should be done when needed to maintain the health of the grass, with care not to over-apply the fertilizer.

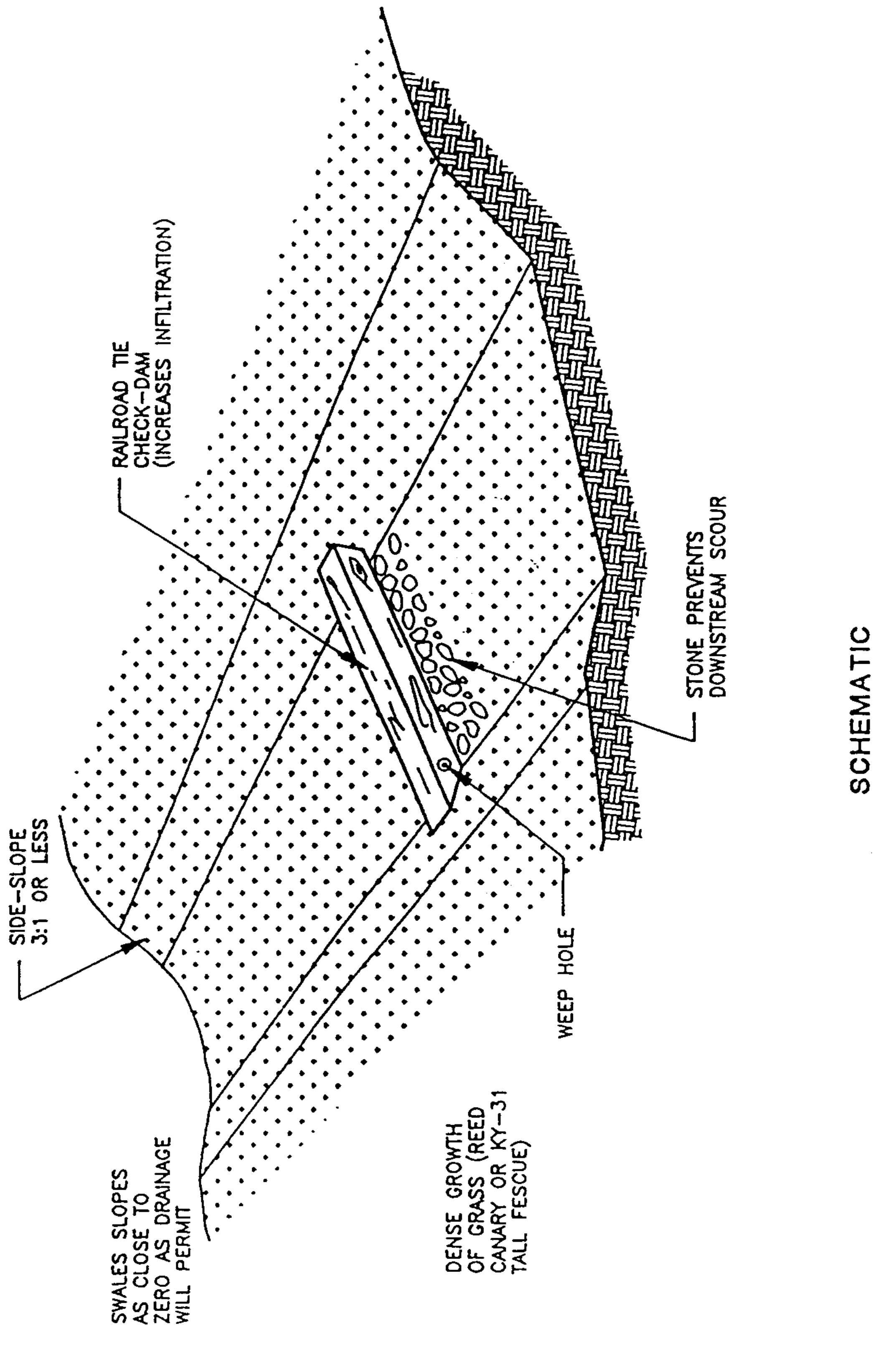

Figure 7-9 Schematic Of A Grass Swale

Source: Controlling Urban Runoff

Performance Standards

- General Performance Information
  - Grassed swales provide a water quality benefit by filtering suspended material out of the overland flow. They have little or no value at removing soluble pollutants because the degree of infiltration provided is generally small.
  - In order to function optimally, a grassed swale must be in an area where its longitudinal slope is very slight (2% or less). The table below shows the low removal rates for grassed swales on a 5 percent slope. If discharges or velocities are greater than those recommended (greater than 10 cfs or 2.5 ft/s, respectively), the ability of the swale to perform as a water quality BMP is severely impaired.
  - The use of check dams in the swale helps to lower the discharge velocity and can, in some cases, allow their beneficial use in situations where the swale slope is greater than recommended.
  - Rainfall events of less than 0.25 inches may show increased removals due to the slower velocities in the swales.

- Grassed Swales on a 5% Slope
  - Long term estimated removal of pollutants is as follows:

	Pollutant	**Removal Rate**
Primary Pollutants	Total Phosphorus	10%
	Lead	10%
	BOD	10%
Other Pollutants	Sediment	10%
	Total Nitrogen	10%
	COD	10%
	Copper	10%
	Zinc	10%

- Grassed Swales on a Slope Less Than 5% With Check Dams
  - Long term estimated removal of pollutants is as follows:

	Pollutant	**Removal Rate**
Primary Pollutants	Total Phosphorus	30%
	Lead	10%
	BOD	30%
Other Pollutants	Sediment	30%
	Total Nitrogen	30%
	COD	30%
	Copper	10%
	Zinc	10%

7.2.10 Oil/Grit Separators

Standard Specifications For Oil/Grit Separators

Required Specifications

- Separators should be sized for the design water volumes specified by OCRM.
- Separator should be structurally sound and designed for acceptable traffic loadings where subject to traffic loadings.
- Separator should be designed to be watertight.
- Volume of separator should be at least 400 cubic feet per acre tributary to the facility (first two chambers).
- Forebay or first chamber should be designed to collect floatables and larger settleable solids. Its surface area should not be less than 20 square feet per 10,000 square feet of drainage area.
- Oil absorbent pads, oil skimmers, or other approved methods for removing accumulated oil should be provided.
- Separator pool should be at least 4 feet deep.
- Weirs, openings, and pipes should be sized to pass as a minimum a 25-year storm.
- Manholes should be provided to each chamber to provide access for cleaning.

Recommended Specifications

- Separator to be located close to the source before pollutants are conveyed to storm sewers or other BMPs.
- Use only on sites of less than one acre.
- Provide perforated covers as trash racks on orifices leading from first to second chamber.
- Use three chambers for treatment similar to Figure 7-10.
- Center chamber may contain a coalescing medium to enhance the gravity separating process.
- Storm drain inlet in third chamber to be located above floor to permit additional settling.
- Stormwater from rooftops and other impervious areas not likely to be polluted with oil should not discharge to the separator.
- Design to bypass flows above 400 cubic feet per acre.

Operation And Maintenance Recommendations

- A stormwater management easement and maintenance agreement should be required for each facility. The maintenance covenant should require the owner of the separator to periodically clean the structure.
- Cleaning quarterly should be a minimum schedule with more intense land uses such as gas stations requiring cleaning as often as monthly.
- Cleaning should include pumping out waste water and grit and having the water processed to remove oils and metals.

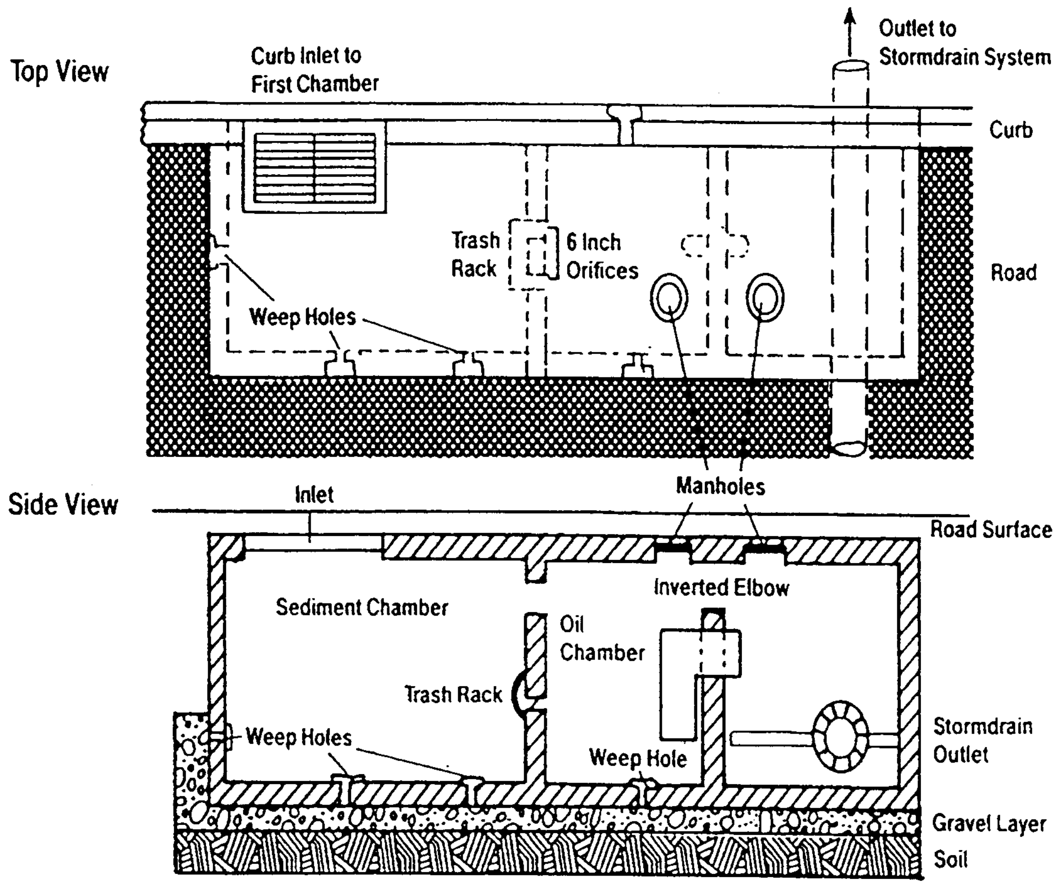

Figure 7-10 Oil/Grit Separator

Source: City of Rockville, MD

<u>Performance Standards</u>

- Oil and Grease Separator Performance
  - These devices are for the most part ineffective as a stand alone treatment of stormwater runoff quality, unless hydrocarbons are the only pollutant of concern.
  - Hydrocarbons in urban runoff can effectively clog the infiltration capacity of underlying soils because they tend to attach themselves to particles in the water column and settle to the bottom of the BMP.
  - The removal of hydrocarbons will extend the maintenance interval required for downstream BMPs by removing these substances which impair their effectiveness.
- Primary Pollutants - Phosphorus, Lead, BOD
  - Performance standards as related to phosphorus, lead, and BOD are not relevant.

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Appendix A - Example Design Applications

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A.1 Example Design - Infiltration Trenches

<u>Site Layout</u>

The site for an infiltration trench consists of two areas:

1. the portion of the watershed that contributes direct runoff to the infiltration trench, which is denoted as A_u; and

2. the portion of the watershed allocated to the basin (does not contribute runoff to the trench), which is denoted as A_b. The subscripts u and b are used to indicate the upland and basin drainage areas, respectively.

<u>Design Procedure</u>

Step 1 Using the hydrologic procedures described in Chapter 2 - Hydrology, determine the volume of runoff that would be produced by the first 1-inch of rainfall over the upstream drainage area (upland area is A_u), ΔQ_u.

Step 2 Compute the maximum allowable trench depth (d_{max}) from the feasibility equation 7.A.1. Select the trench design depth (d_t) based on the depth that is at least two feet above the seasonal high groundwater table, or a depth less than or equal to d_{max}, whichever results in the smaller depth.

$$\mathbf{d_{max} = ft_s/V_r} \qquad \mathbf{(7.A.1)}$$

Where: f = minimum infiltration rate, in/hr
t_s = storage time, hr
V_r = void ratio in soil or rock

Step 3 Compute the trench surface area (A_t) from Equation 7.A.2:

$$\mathbf{A_t = (\Delta Q_u A_u)/(V_r d_t - P + ft_s)} \qquad \mathbf{(7.A.2)}$$

Where: A_t = surface area of trench, ft^2
P = rainfall depth, ft
Other variables previously defined

Step 4 Compute the trench length from equation 7.A.3.

$$\mathbf{L_t = (\Delta Q_u A_u)/[(V_r d_t - P + ft_s)W_t]} \qquad \mathbf{(7.A.3)}$$

Where: L_t = length of trench, ft
W_t = width of trench, ft
Other variables previously defined

In the event that the side walls of the trench must be sloped for stability during construction, the surface dimensions of the trench area should be based on equation 7.A.4:

$$\mathbf{A_t = (L_t - Zd_t)\,(W_t - Zd_t)} \qquad \mathbf{(7.A.4)}$$

Where: Z = trench side slope ratio

The design procedure would begin by selecting a top width (W_t) that is greater than $2Zd_t$, for a specified side ratio (Z). The length (L_t) is then determined as:

$$\mathbf{L_t = Zd_t + (A_t)/(W_t - Zd_t)} \qquad \textbf{(7.A.5)}$$

Example Application

An infiltration trench with surface inlets will be used to control the first inch of rainfall from a 4 acre commercial site. Following are the calculations for the surface area of the trench.

Design Data: $f = 1.02$ in/hr
$V_r = 0.4$
Depth to groundwater = 5 feet
Depth to bedrock = 8 feet

Step 1 Compute the upland increase in runoff volume (ΔQ_u) from hydrologic procedures described in Chapter 2 - Hydrology, for the first inch of rainfall.

$\Delta Q_u = 4.17$ in.

Step 2 Compute the maximum allowable trench depth (d_{max}) by the feasibility formula, Equation 7.A.1:

$d_{max} = ft_s/V_r$

$d_{max} = (1.02)(48)/0.4 = 122.4$ in $= 10.2$ ft

For this example, the depth to the groundwater table is 5 feet and the depth to bedrock is 8 feet.

Step 3 Select a trench design depth less than d_{max} and at least two feet above the groundwater table (subtract one foot for the overlying soil cover with surface inlets).

For B soils, $d_t = 4.0$ ft

Step 4 Compute the trench surface area (A_t) by Equation 7.A.2:

$A_t = (\Delta Q_u A_u)/(V_r d_t - P + ft_s)$

Where:

$\Delta Q_u = 4.17$ in $= 0.35$ ft
$A_u = 4$ acres x $43{,}560$ ft^2/acre $= 174{,}240$ ft^2
$V_r = 0.40$
$d_t = 4.0$ ft
$P = 1$ in $= 0.08$ ft
$f = 1.02$ in/hr. $= 0.085$ ft/hr
$t_s = 48$ hours

Substituting:

A_t = [(0.35)(174,240)]/[(0.40)(4.0) - 0.08 + (0.085 x 48)]
A_t = 10,890 ft^2

A.2 Example Design - Vegetated Swales

Site Layout

The site layout will consist of the portion of the watershed that contributes direct runoff to the swale area or the upland area, which is denoted as A_u; and the portion of the watershed allocated for swale storage, which is denoted as A_s. It is important to note that the upland area (A_u) does not include the area allotted to the swale surface (A_s). Swale locations are usually either on the side or back of the property line or along the side of roadways. Installation of berms or check dams at certain intervals along the length of the swale will result in storage.

Design Procedure

Step 1 Using the hydrologic procedures described in Chapter 2 - Hydrology, determine the volume of runoff that would be produced by the first 1-inch of rainfall over the upstream drainage area (upland area is A_u), ΔQ_u.

Step 2 Compute the maximum allowable swale depth (d_{max}) from the feasibility equation 7.A.6. Select the swale design depth (d_t) based on the depth that is at least two feet above the seasonal high groundwater table, or a depth less than or equal to d_{max}, whichever results in the smaller depth.

$$\mathbf{d_{max} = ft_s/V_r} \qquad \mathbf{(7.A.6)}$$

Where: f = minimum infiltration rate, in/hr
t_s = storage time, hr
V_r = void ratio in soil or rock

Step 3 The swale surface area dimensions can be determined from Equations 7.A.7 and 7.A.8. The bottom width (W_b) is selected along with the side slope ratio (Z), and depth of check dam (d_s). The swale top width (W) and total hydraulic length (L_T) may be computed as:

$$\mathbf{W = W_b + 2d_sZ} \qquad \mathbf{(7.A.7)}$$
$$\mathbf{L_T = [\Delta Q_u A_u]/[(d_s/4)(W+W_b) + W(ft_s-P)]} \qquad \mathbf{(7.A.8)}$$

Where: P = rainfall depth, ft
Other variables previously defined

Step 4 The number of check dams needed to impound and store the increased runoff volume is determined as:

$$\mathbf{N_s = L_T/L} \qquad \mathbf{(7.A.9)}$$

Where: L = length of swale behind each check dam, ft

Step 5 The maximum required spacing between check dams is computed as:

$$\mathbf{L = d_s/S_s} \qquad \textbf{(7.A.10)}$$

Where: S_s = bottom slope of swale, ft/ft

Step 6 If L_t is restricted by the site layout, the level of control provided by the swales is determined by:

$$\mathbf{Q_s = V_w/A_u} \qquad \textbf{(7.A.11)}$$

Where: $V_w = [d_s (W + W_b)L] [N_s]/4$

<u>Example Application</u>

Medium density residential lots of 1/4 acres are to be developed. A total of 12 lots will be created. The site will be designed to be managed with vegetated swales with check dams located along the back of the lots. The total area of the development is 3.0 acres with 38% impervious area and 62% pervious area. Following are the calculations for the swale design:

Design Data: f = 2.41 in/hr
$V_r = 0.4$
Depth to groundwater = 6 feet
Depth to bedrock = 9 feet

Step 1 Compute the upland increase in runoff volume (ΔQ_u) from hydrologic procedures described in Chapter 2 - Hydrology, for the first inch of rainfall.

$\Delta Q_u = 1.37$ in.

Step 2 Compute the maximum allowable swale depth (d_{max}) from the feasibility formula, Equation 7.A.6:

$d_{max} = ft_s/V_r$

$d_{max} = 2.41(48)/0.4 = 289.2$ in. $= 24.1$ ft

Step 4 Select the swale check dam design depth (d_s). In this case, use:
$d_s = 2.5$ ft

Step 5 Select the swale bottom width (W_b) and the swale side slope ratio (Z), assuming the same side slopes. Determine the top width (W) of the swale check dam from:

$W = W_b + 2d_sZ$

Where:

Z = 5 (5h/1v)
$W_b = 14$ ft

Substituting:

$W = 14 + 2(2.5)(5) = 39$ ft

Step 6 The site layout allows for a swale length of 480 feet along the back of all the lots. The total swale length (L_T) is fixed so that the volume of swale storage (V_w) may be determined from the dimensions given:

$V_w = [d_s (W + W_b)L_T]/4$

$V_W = [2.5 (39 + 14)480]/4 = 15{,}900$ ft^3

Step 7 The number of swale check dams (N_s) that need to be constructed to achieve the volume of storage over the total swale length will vary with the depth of each check dam (d_s), given as:

$N_s = L_T/L$

where L is the length of swale behind each check dam, given as:

$L = d_s/S_s$

Note: S_s = bottom slope of swale = 0.03 ft/ft

$L = 2.5/0.03 = 83.3$ ft

$N_s = 480/83.3 = 5.8$ (use 6 with an adjusted L of 80.0 feet*)

* The adjusted length is determined as $L = L_T/N_s$

Thus, the design storage is 15,900 ft^3, which is greater than the required storage of 14,920 ft^3 (1.37 in x 1ft/12in x 3 acres x 43,560 ft^2/acre) so that sufficient storage is provided by the design.

For further information on the design of infiltration measures, the following references are recommended:

Stormwater Infiltration Structure Design, National Stone Association, 1415 Elliot Place, NW, Washington, D.C., 1994. Available from McTrans, University of Florida, Gainesville, Florida, 32611, (352) 392-0378. Includes a computer model for infiltration design.

Maryland Department of Natural Resources, Water Resources Administration, Stormwater Management Division, Standards and Specifications for Infiltration Practices, 1984.

CHAPTER

EXAMPLE STORMWATER PLAN 8

Chapter Table of Contents

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8.1 Introduction

8.1.1 Purpose

This chapter on an Example Stormwater Plan has been included to provide the design professional who is responsible for preparing and submitting for approval stormwater management plans in Horry County, South Carolina with some guidance on what a typical plan submittal may include. The example as set forth in this chapter is by no means intended to establish a standard format by which all plans are to be submitted. However, the design procedures as outlined in the other chapters along with this example should contribute to more uniform plan submittals and in turn an efficient plan review process.

8.2 Pre-Development Site Conditions

8.2.1 Site Description

This 47.08 acre site in its pre-development state, consists of upland agricultural fields with three (3) small areas of upland wooded forest. The agricultural fields have been actively farmed in the past and are currently covered with moderate to heavy low growing vegetation. The small wooded areas are heavily forested with moderate to heavy underbrush. This site is located on Kayla Lane off of S.C. Highway 9 approximately 1.5 miles west of the Longs crossroads. (See Figure 8-1 for Location Map)

8.2.2 Soil Types and Drainage Basins

Per the Soil Conservation Service-"Soil Survey for Horry County", this site consists of Yauhannah (HSG "B"), Yemassee (HSG "C"), Ogeechee (HSG "B/D") and Bladen (HSG "D") soils. The "Ogeechee" soil group is in a drained condition due to the existing ditch system, installed over the years this site has been farmed, and has been considered HSG "B" for runoff calculations. The predominant hydrologic soil group "HSG" on this site is HSG "B". This site is included in the "Big Cedar Branch" watershed as depicted on the United States Geological Survey Quadrangle (See Figure 8-2). Per references to the U.S.G.S. Quadrangle, 1993 Horry County Aerial Photography and on-site reconnaissance, this watershed appears to contain approximately 365 acres of land including this site. Per the various references mentioned above and on-site topographic information this watershed has been subdivided into seven (7) off-site hydrographs and two (2) on-site hydrographs (See Figure 8-3). The total off-site drainage area has been subdivided into the several small areas to more accurately route the off-site runoff to this site. This was necessary due to the presence of numerous existing ditches around this site which intercept the off-site runoff at various points. The off-site and on-site hydrograph boundaries have been delineated and the area, composite CN# and Tc for each of the hydrographs has been computed. The off-site hydrographs are denominated HOS1, HOS1-A, HOS2, HOS3, HOS3-A, HOS4 and HOS5. The on-site hydrographs are denominated H1 and H2.

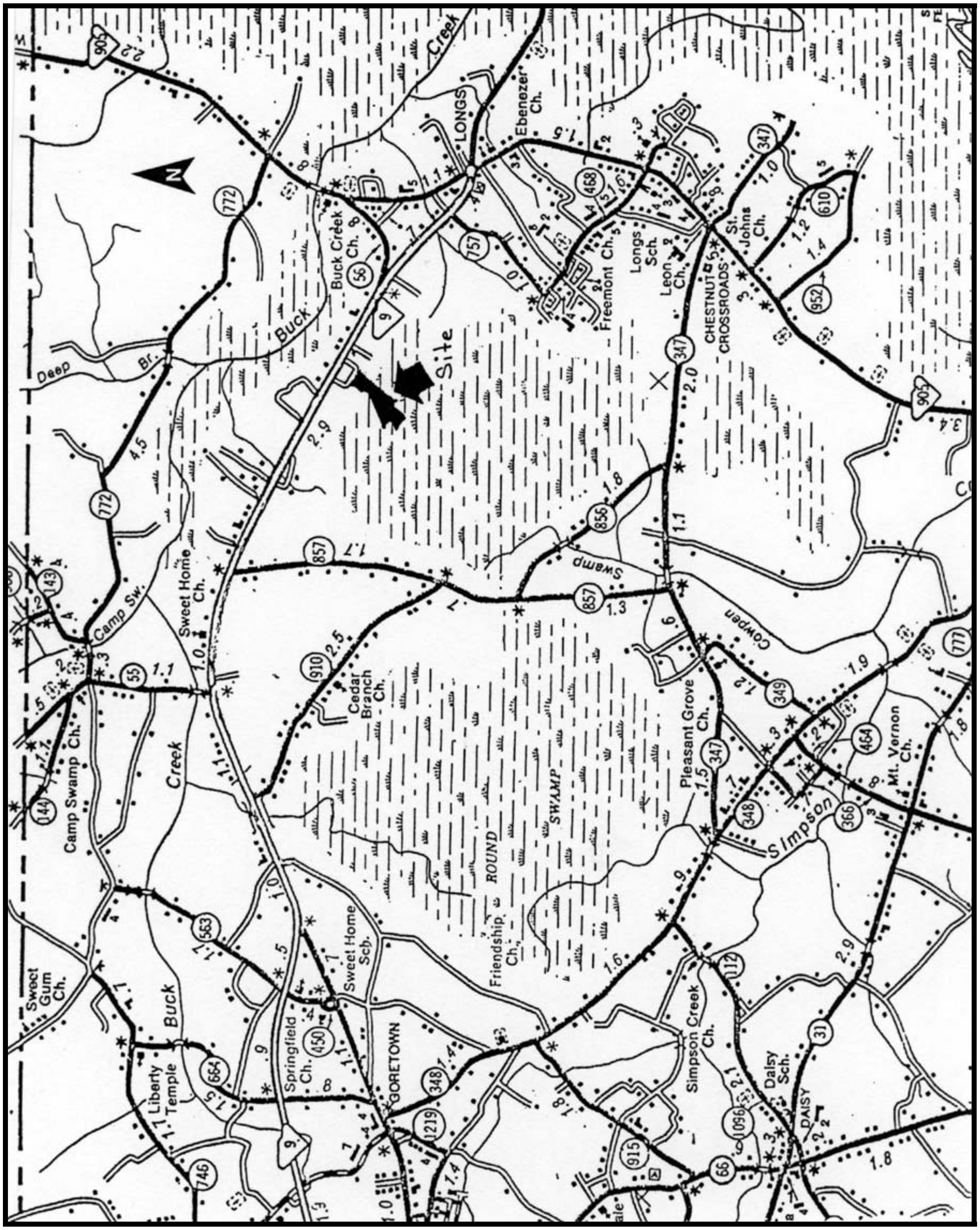

Figure 8-1 Site Location Map

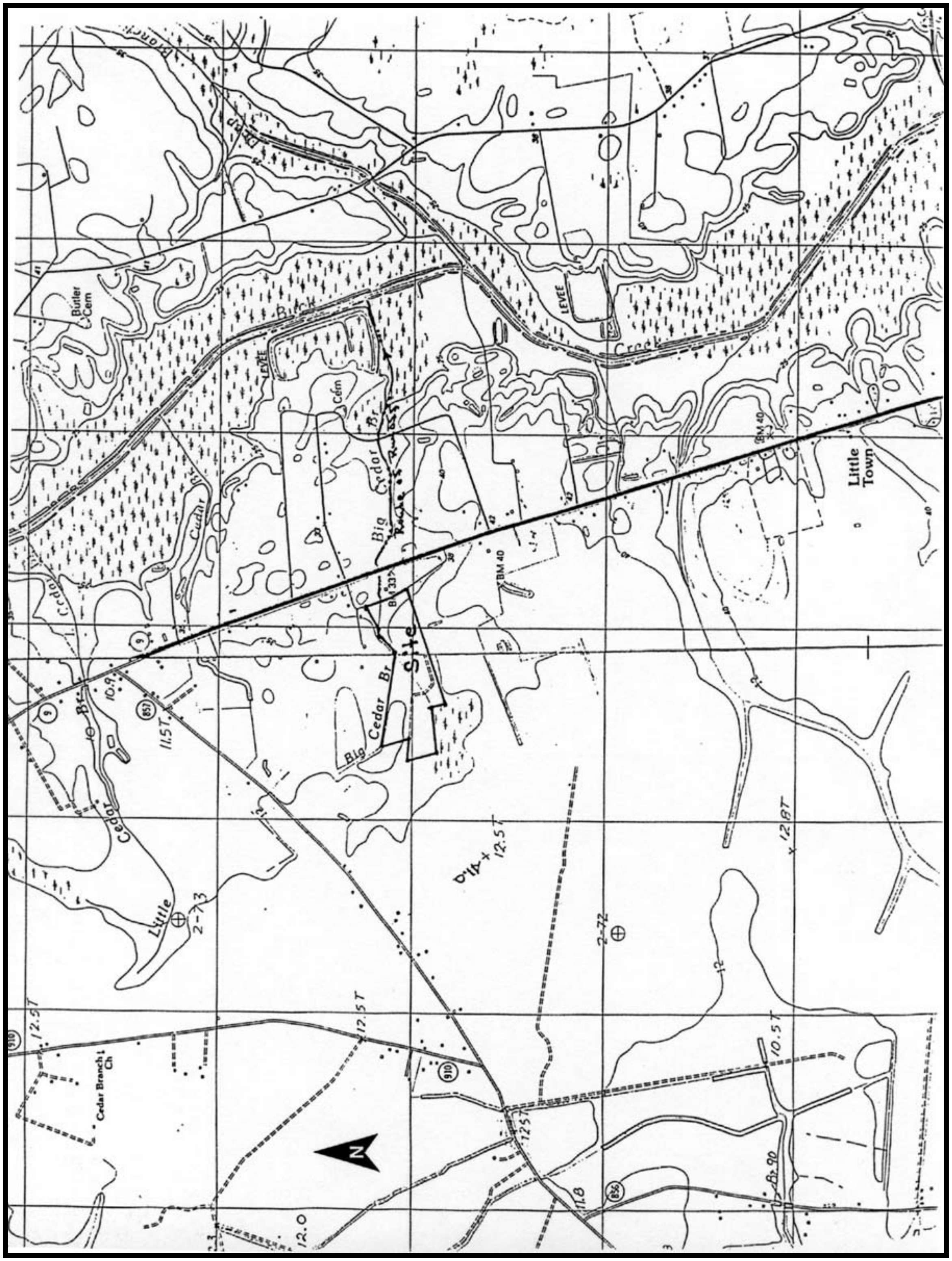

Figure 8-2 Site Location Map
United States Geological Survey Quadrangle
Showing "Big Cedar Branch" Watershed

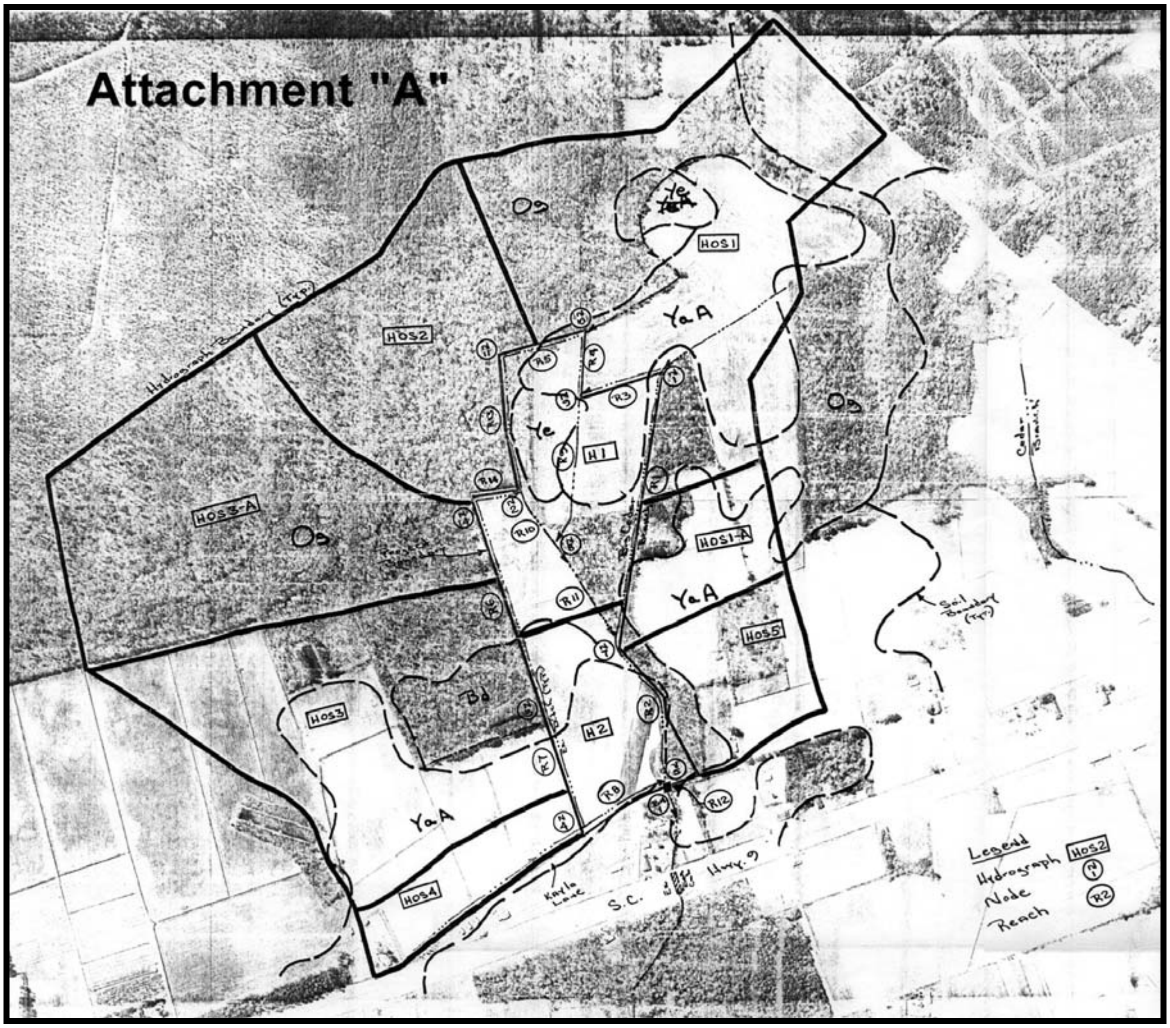

Figure 8-3 Attachment "A" On-site and Off-site Drainage Areas

8.2.3 Outfalls and Receiving Waterbodies

As noted above, the off-site and on-site runoff discharges to an existing ditch system which is interconnected with "Big Cedar Branch". This site and the adjoining area, contributing off-site runoff through this site, are located at the upper reach of the "Big Cedar Branch" watershed. "Big Cedar Branch" is located along the northern property boundary and extends upstream and off-site at the north western property corner. This existing outfall drains toward the east and crosses Kayla Lane by way of two (2) existing 48" R.C.P. culverts. "Big Cedar Branch" continues east from Kayla Lane to S.C. Highway 9 and crosses S.C. Highway 9 by way of two (2) existing 5' x 5' box culverts. The outfall then continues east from S.C. Highway 9 through an existing forested "branch" until reaching "Buck Creek Canal". "Buck Creek Canal" then runs southeast to the "Waccamaw River" approximately 5.3 miles downstream of the "Big Cedar Branch" and "Buck Creek Canal" intersection.

The existing dual 48" culverts which cross Kayla Lane appear to be the restricting hydraulic element within the watershed upstream of Kayla Lane. These parallel culverts control the discharge for the upper reach of "Big Cedar Branch". Local knowledge and observations indicated that during storm events which approximate a 10-year frequency rainfall the existing ditches upstream of the 48" culverts flow approximately full within one (1) foot of the top of ditch banks. Additionally, the 48" culverts appear to be at "full flow" when the runoff peaks at the Kayla Lane culvert crossing. This information has been used as a "bench mark" to analyze the pre-development stormwater model.

The receiving waterbody for this site is "Buck Creek Canal" which is located approximately 4,700 feet downstream of Kayla Lane. (See Figure 8-2)

8.2.4 Calculations

Because the 48" culverts crossing Kayla Lane are the controlling factor in the release of stormwater upstream, this hydraulic condition results in the detention / storage of on-site and off-site runoff within the ditch systems located upstream of these two (2) culverts. The pre-development versus post-development analysis will therefore center on the discharge from this structure in both the pre-development and post-development storm events.

See Table 8-2 for a comparison summary of the Pre-development versus Post-development runoff, as analyzed at the two (2) 48" culverts crossing Kayla Lane, during the ten (10) and twenty-five (25) year storm events.

8.3 Post-Development Site Conditions

8.3.1 Site Description

This 47.08-acre site in its post-development state will contain 176 single-family homes with associated infrastructure and a "wet pond" stormwater management system. The roadways will be constructed of asphalt 22 feet wide with 18 inch concrete curb and gutter. The roadways will be drained by way of catch basins and culverts to the detention pond system. A portion of the lots will drain toward the roadway catch basins and the remainder will drain via sheet flow over grassed surfaces to the detention pond system and existing perimeter ditches. The total built upon area equals 15.4 acres.

8.3.2 Inlet and Pipe Calculations

Inlet A

<u>Determine design flow to inlet A</u>

Total area draining to inlet A = 0.58 acres

Overland Flow
- Slope = 0.50% = 0.0050 ft/ft
- Flow length = 50 ft
- From Table 2-4 in Chapter 2, (for Lawns, sandy soil, flat, 2%) C = 0.10
  - Using Figure 2-1 in Chapter 2, overland flow time = 15 min.

Flow in Curb and Gutter
- Slope = 0.50% = 0.0050 ft/ft
- Flow length = 250 ft

From Table 2-4 in Chapter 2, (for Streets, asphaltic and concrete)
use C = 0.95
Using Figure 2-1 in Chapter 2, curb and gutter flow time = 6 min.

Total Flow Time (time of concentration, t_c) = 15 min. + 6 min. = 21 min.

Rainfall Intensity (I) for 25-year storm with t_c = 21 min.

From Table 2-3 in Chapter 2:

t_c = 15 min., I = 6.24 in/hr

t_c = 21 min., I = ? in/hr

t_c = 30 min., I = 4.99 in/hr

Interpolating the above values, I = 5.74 in/hr

Design flow can be computed using the Rational Method (equation 2.1 in Chapter 2)

$Q = CIA$

From Table 2-4 in Chapter 2, for suburban use C = 0.40

I = 5.74 in/hr

A = 0.58 acres

$Q = (0.4)(5.74)(0.58)$

Q = 1.33 cfs

<u>Determine flow capacity of grated inlet in sag of vertical curve</u>

Frame and grate selected is the Vulcan Foundry model V-4510-1 with 310 square inches of opening and dimensions of W = 23.5 in. and L = 36 in. To determine the capacity of this inlet, we need to first consider the maximum allowable spread on this type of road. The spread limits in Chapter 3 indicate that the maximum spread is 10 feet. The typical road cross-section requires one-quarter inch fall per foot which equates to 2.08% slope. Thus, the maximum depth (d) at the inlet will be,

$d = TS_x$

where T = spread and S_x = cross slope

d = (10 ft)(0.0208 ft/ft)

d = 0.21 ft

In Chapter 3, grate inlets in a sag operate as a weir up to about 0.4 feet of depth. Thus, this grate will operate as a weir and its capacity can be computed by equation 3.10,

$Q_i = CPd^{1.5}$

If one assumes that this grate is 25% clogged (grate inlet in a residential subdivision), the width should be reduced by 25% in determining the perimeter (P). Therefore, the effective perimeter is P = 1.5 + 3 + 1.5 = 6 feet. Thus, the inlet capacity is,

$Q_i = (3.0)(6\text{ ft})(0.21)^{1.5}$

Q_i = 1.73 cfs

Since the inlet capacity (1.73 cfs) exceeds the design flow (1.33 cfs), the inlet adequately handles the flow.

<u>Determine flow capacity of drainage pipe between Inlets A and B</u>

Knowing the design flow, the next step is to determine a culvert size and slope that will carry this flow. In this case, the pipe is flowing under Inlet Control and the pipe capacity is determined using the Manning's Equation for full flow, equation 3.16,

$Q = [0.463\ D^{8/3}S^{1/2}]/n$

$Q = 1.33$ cfs

D = pipe diameter (ft)

S = pipe slope = 0.55% = 0.0055 ft/ft

n = 0.013 for Reinforced Concrete Pipe (RCP)

$1.33 \text{ cfs} = [0.463\ D^{8/3}(0.0055)^{1/2}]/(0.013)$

Minimum D = 0.773 ft = 9.28 in.

Therefore, a 15" RCP is selected at 0.55% slope.

We now move downstream to the next drainage structure and perform similar design calculations.

Inlet B

Determine design flow to inlet B

Total area draining to inlet B = 0.89 acres

Overland Flow

Slope = 0.50% = 0.0050 ft/ft

Flow length = 140 ft

From Table 2-4 in Chapter 2, (for Lawns, sandy soil, flat, 2%) C = 0.10

Using Figure 2-1 in Chapter 2, overland flow time = 26 min.

Flow in Curb and Gutter

Slope = 0.50% = 0.0050 ft/ft

Flow length = 193 ft

From Table 2-4 in Chapter 2, (for Streets, asphaltic and concrete) use C = 0.95

Using Figure 2-1 in Chapter 2, overland flow time = 5 min.

Total Flow Time (time of concentration, t_c) = 26 min. + 5 min. = 31 min.

Rainfall Intensity (I) for 25-year storm with t_c = 31 min.

From Table 2-3 in Chapter 2:

t_c = 30 min., I = 4.99 in/hr

t_c = 31 min., I = ? in/hr

t_c = 60 min., I = 3.47 in/hr

Interpolating the above values, I = 4.94 in/hr

Design flow can be computed using the Rational Method (equation 2.1 in Chapter 2)

$Q = CIA$

From Table 2-4 in Chapter 2, for suburban use C = 0.40

I = 4.94 in/hr

A = 0.89 acres

$Q = (0.4)(4.94)(0.89)$

Q = 1.76 cfs

Determine flow capacity of grated inlet in sag of vertical curve

In our previous calculations, the design capacity for the Vulcan Foundry model V-4510-1 was determined to be 1.73 cfs. Needing to accommodate a flow of 1.76 cfs from this drainage area, that grate inlet would not be adequate. We would either need to redesign a portion of this development to reduce the design flow or select another grate inlet with a higher capacity under these operating conditions. In this case, we will opt to select another grate inlet to meet our design flow needs by placing two of the V-4510-1 grate inlets in series. This will provide us with an additional 3 feet in the perimeter.

Again, assuming that these grates are 25% clogged (grate inlet in a residential subdivision), the width should be reduced by 25% in determining the perimeter (P). Therefore, the effective perimeter is P = 1.5 + 6 + 1.5 = 9 feet. Thus, the capacity of the inlets in series is,

$Q_i = (3.0)(9 \text{ ft})(0.21)^{1.5}$

$Q_i = 2.60$ cfs

With the inlet capacity (2.60 cfs) now exceeding the design flow (1.76 cfs), the inlets in series will adequately handle the flow.

Determine flow capacity of drainage pipe between Inlet B and Outlet

The outlet pipe draining to the detention pond must handle both the flow from the grate inlet as well as the flow from the pipe connecting Inlet B to Inlet A. One can not simply add these flows together to obtain the new total flow. Instead the controlling time of concentration needs to be determined along with totaling the drainage area and the overall runoff coefficient.

The time of concentration to Inlet A is 21 minutes. Next, we must consider the pipe travel time from Inlet A to Inlet B. To do this, we first need to compute the flow velocity in this 15" pipe assuming that the pipe is flowing full,

$V = Q/A$
$V = (1.33 \text{ cfs})/(1.23 \text{ sf})$
$V = 1.08$ ft/s

Using this velocity and the pipe length of 22 feet, we compute the pipe travel time as,

$T_t = L/V$
$T_t = (22 \text{ ft})/(1.08 \text{ ft/s})$
$T_t = 20.4 \text{ sec} = 0.3 \text{ min.}$

Therefore, the total time of concentration for this route is t_c = 21 min. + 0.3 min. = 21.3 min.

This time of concentration is compared to the time of concentration directly to Inlet B of 31 minutes. The greater of these two times of concentration is now the controlling one. In this case, it will be 31 minutes.

This pipe's total drainage area is that of both Inlet A and B or 1.47 acres (0.58 ac. + 0.89 ac.). Also, the overall runoff coefficient for this total area will be 0.40. We now compute the new flow that this pipe must handle by using the Rational Method,

$Q = CIA$
$Q = (0.40)(4.94 \text{ in/hr})(1.47 \text{ ac.})$
$Q = 2.90$ cfs

The next step is to now determine a culvert size and slope that will carry this flow. In this case, the pipe is flowing under Inlet Control and the pipe capacity is determined using the Manning's Equation for full flow, equation 3.16,

$Q = [0.463\ D^{8/3} S^{1/2}]/n$

$Q = 2.90$ cfs

D = pipe diameter (ft)

S = pipe slope = 2.80% = 0.028 ft/ft

$n = 0.024$ for Corrugated Plastic Pipe (CPP)

$2.90 \text{ cfs} = [0.463\ D^{8/3}(0.028)^{1/2}]/(0.024)$

Minimum $D = 0.961$ ft = 11.5 in.

Therefore, an 18" CPP is selected at 2.80% slope.

The same procedure as outlined above is used to size all the remaining inlets and pipes in the development. Table 8-1 shows an example of a Drainage Calculations table that can be used to summarize these pipe calculations. Table 8-1 contains the design calculations and construction data for the two pipes discussed above. Also, Figure 4-4 in Chapter 4 provides another convenient form to organize culvert design calculations.

Figures 8-4 and 8-5 show the completed Drainage / Soil and Erosion Control Plans for this development. These plans show the location of all the proposed storm drainage structures, pipes, and detention ponds. These plans also call out the rim and invert elevations of the drainage structures as well as the length, size, material, and slope of the drainage pipes. The location and size of proposed drainage easements are also shown. In regards to soil and erosion control, the plans call out those measures to be implemented during construction to minimize soil loss. Notes to the contractor are also included to help clarify some of these instructions.

Table 8-1
DRAINAGE CALCULATIONS

LINE	AREA (ACRES)	ACCUM. AREA	COEFF. "C"	"Tc" (MIN.)	"I" (IN./HR.)	Q=CIA (REQD.)	SLOPE (FT./FT.)	MANNING'S "n"	DIA. (IN.)	Q (AVAIL.)	LENGTH (FT.)	INV. ELEV. UPPER	INV. ELEV. LOWER
A to B	0.58	0.58	0.4	21	5.74	1.33	0.0055	0.013	15	4.79	22	32.27	32.15
B to Outlet	0.89	1.47	0.4	31	4.94	1.76	0.028	0.024	18	9.52	110	28.82	31.90

Table 8-1 Drainage Calculations

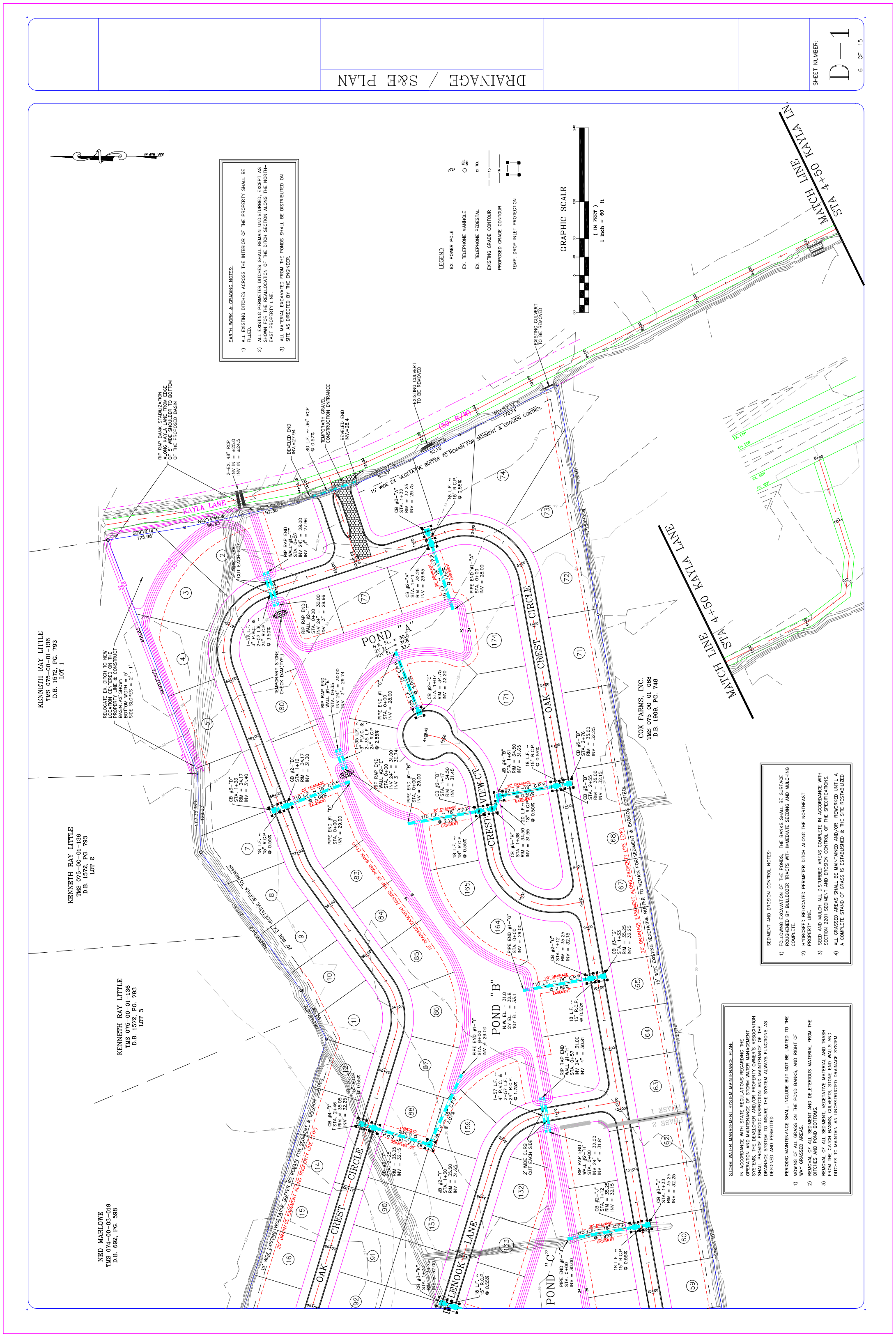

Figure 8-4 Sheet D-1

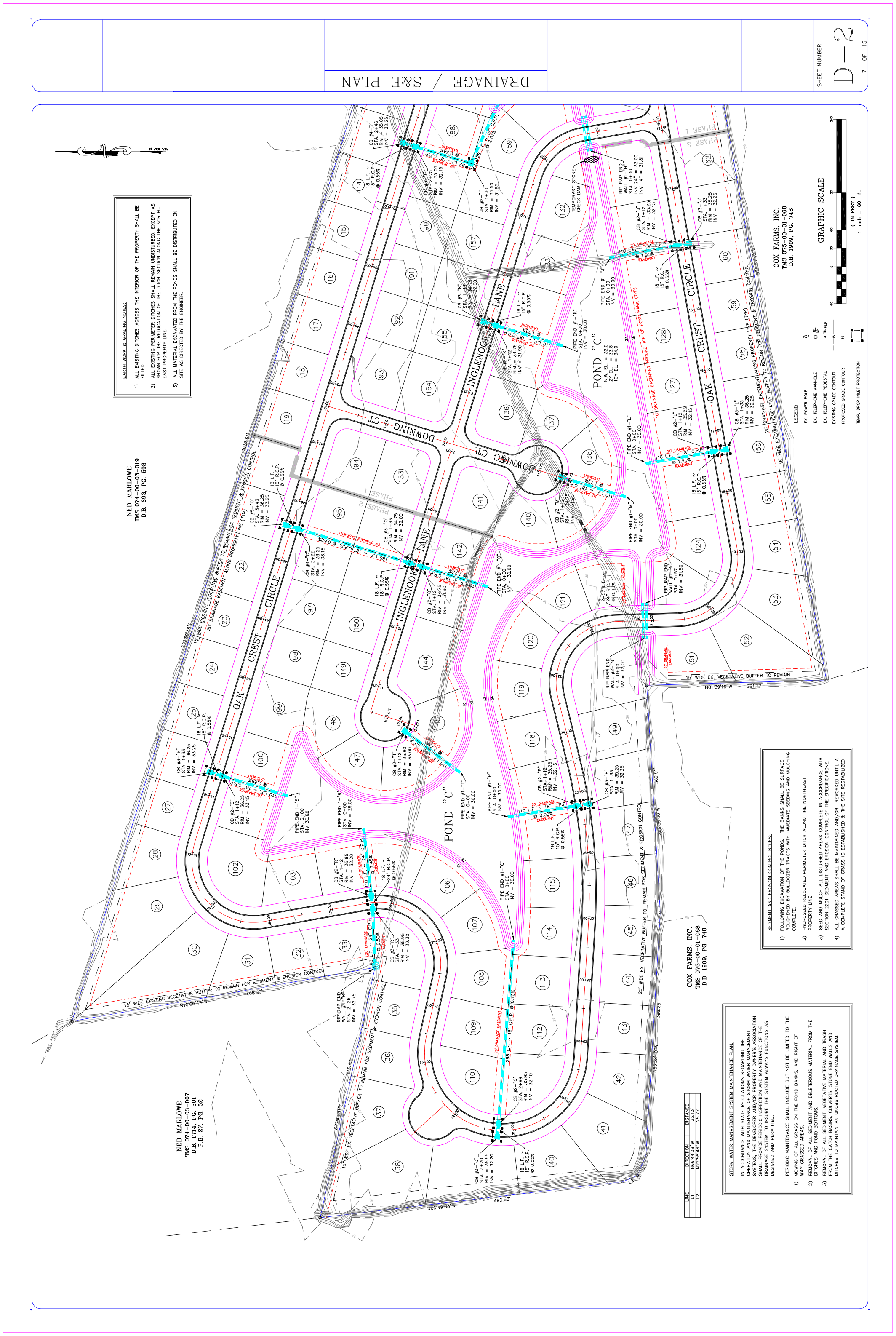

Figure 8-5 Sheet D-2

8.3.3 Soil Types and Drainage Basins

The soil types encountered on this site include moderately drained fine sandy loam and loamy fine sand on the higher elevations and moderately drained loamy fine sand on the lower elevations. Land slopes are generally from 0 - 2 percent on the higher elevations and 0 - 0.5 percent on the lower elevations. Typical soil erosion potential will be "slight" for the soils encountered on this site. Sediment and erosion control measures to be utilized on this project will include maintaining an existing vegetative buffer adjacent to the existing ditches along the property boundary, detention ponds functioning as sediment traps with stone check dams constructed at each pond outfall, outlet protection devices on each catch basin inlet, surface roughening with seeding and mulching of the pond banks and riprap stabilization of portions of the existing ditch system.

Per Figure 8-6 the post-developed site has been subdivided into three (3) major runoff areas. These areas have been further subdivided into smaller areas for the purpose of routing the hydrographs to their intercepting element within the hydraulic stormwater model.

Off-Site Areas- As addressed in the pre-development site conditions, the off-site hydrographs have been routed through the adjacent existing ditch system and through this project. These hydrographs are identical in both the pre-development and post-development models.

On-Site Area One (1)- This total area is comprised of the post-developed property which will contribute runoff directly to the proposed detention ponds. This area has been subdivided into three (3) smaller areas to route the runoff to its respective interception point for conveyance to each pond and to size the individual culverts which convey the runoff from the roadways to the ponds. These areas are denominated hydrographs "H2O, H25 and H30". Hydrograph "H30" is routed through Pond "A". Hydrograph "H25" is routed through pond "B". Hydrograph "H20" is routed through Pond "C". Ponds "A, B and C" are interconnected in series by way of two (2) parallel 24" culverts at each road crossing or easement between the ponds.

On-Site Area Two (2)- This total area is comprised of the post-developed property which contributes direct runoff to the existing ditch system around the property boundary. This area has been subdivided into nine (9) smaller areas to route the runoff to its respective interception point along the perimeter ditch and conveyance through the existing ditch system. Although the runoff from these areas is directed toward the property boundary, detention of this runoff is still provided by the on-site ponds and existing ditch system upstream of the two (2) 48" culverts crossing Kayla Lane. The existing ditch system is interconnected to the proposed stormwater management system at three (3) separate points along the property boundary. These connection points include, "Node 9" where a proposed 24" culvert is routed from the existing ditch to "Pond C", "Node 2" where two (2) proposed 24" culverts are routed from the existing ditch to "Pond C" and at "Node 5" where the headwater at the existing 48" culverts under Kayla Lane creates a tailwater conditions at "Pond A".

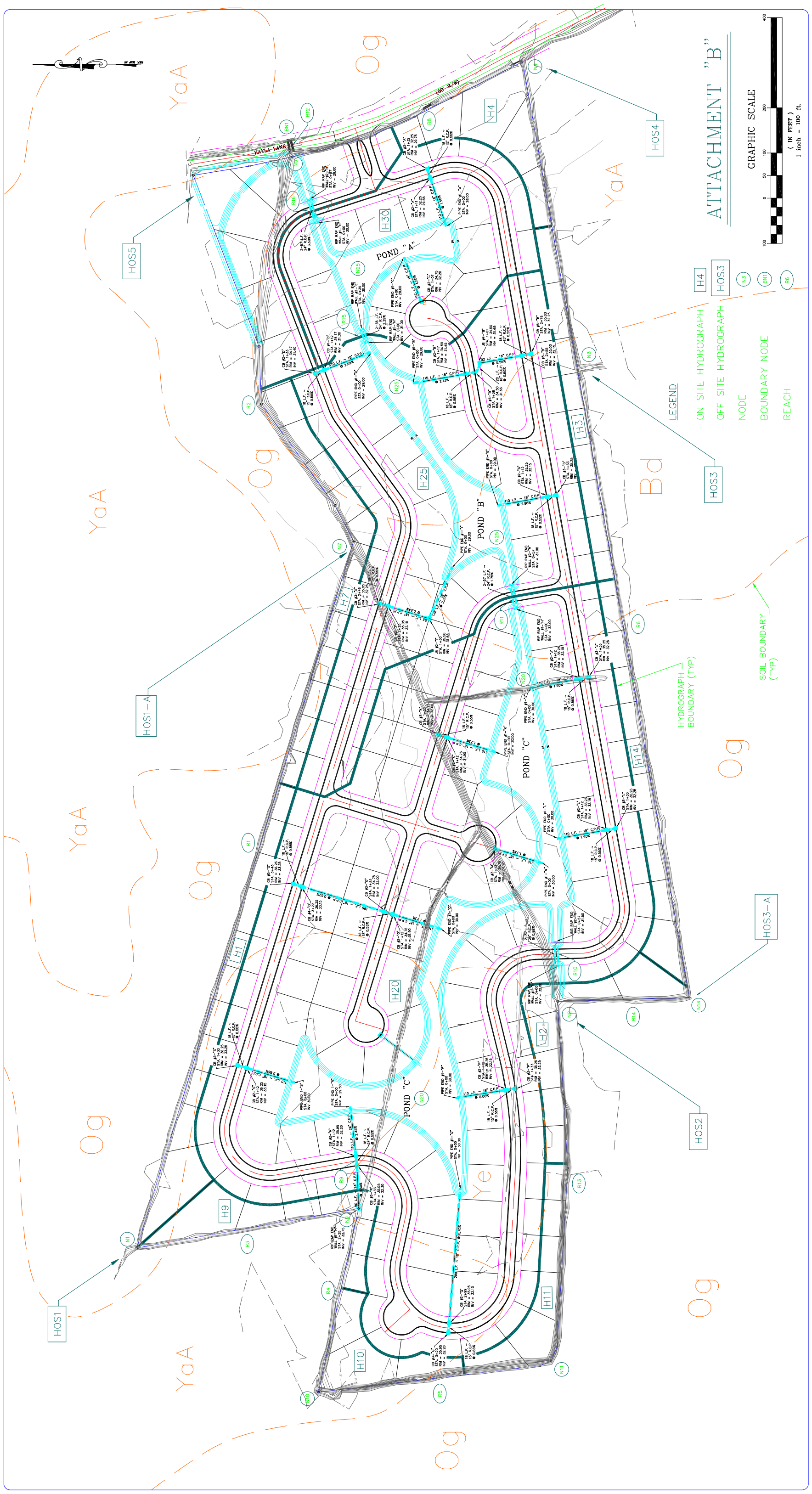

Figure 8-6 Attachment "B"

8.3.4 Pre-development verses Post-development

The post-development stormwater analysis of this project and watershed, as compared to the pre-development conditions, indicates that greater hydraulic efficiency is achieved by routing a significant portion of the off-site runoff through the proposed pond system. Due to the roughness coefficients of the pre-developed existing ditch system, these existing structures tend to develop higher headwater to produce a comparable flow as provided in the proposed drainage system. The proposed drainage system is therefore capable of providing the required detention measures for the post-development runoff of this project and enhancing the hydraulic performance of the existing watershed.

See Table 8-2 for a comparison summary of the Pre-development versus Post-development runoff, as analyzed at the two (2) 48" culverts crossing Kayla Lane, during the ten (10) and twenty-five (25) year storm events.

Table 8-2 Pre- versus Post-Development Comparison Summary

Storm Event (Year)	Pre-development Peak Discharge (cfs)	Pre-development Peak Velocity (fps)	Post-development Peak Discharge (cfs)	Post-development Peak Velocity (fps)
10	98.12	3.9	98.46	3.9
25	120.00	4.9	102.66	4.1

Also, Figure 8-7 shows a comparison of the Pre-development and Post-development hydrographs, as analyzed at the two (2) 48" culverts crossing Kayla Lane, for the twenty-five (25) year storm.

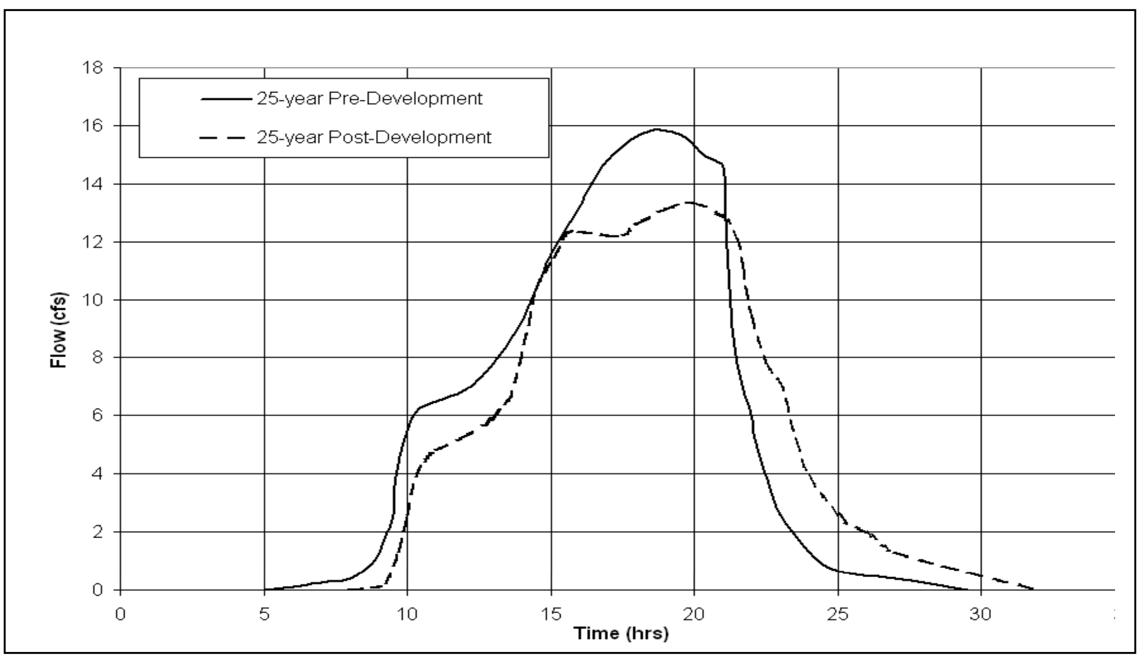

Figure 8-7 Pre- and Post-Development 25-year Storm Hydrographs

8.3.5 Ten Percent Downstream Analysis

For all storage facilities, channel routing calculations shall proceed downstream to a location where the proposed site development land area is less than ten percent (10%) of the total drainage area to that point. At this location, the effect of the hydrograph routed through the proposed storage facility on the downstream hydrograph shall be assessed and shown not to have detrimental effects on downstream areas.

For this particular development of 47.08 acres, we will need to proceed downstream to a location where the total drainage area is approximately 500 acres. As previously discussed, this development will discharge into "Big Cedar Creek". This creek then discharges into "Buck Creek Canal" approximately 2,850 feet downstream as shown in Figure 8-2. At this point, the total drainage area exceeds the 500 acres 10% requirement.

In comparing the Pre- and Post-Development peak discharges and hydrographs in the previous section, the existing two (2) 48" culverts crossing Kayla Lane were used as our point of interest. These culverts not only drain this 47.08 acre development but they also handle approximately 320 acres of off-site drainage area. Therefore, to determine the downstream effects of just this development, we first had to generate discharge hydrographs from just the 47.08 acre development to be able to compare these to the larger watershed hydrograph.

Similar to generating the hydrographs for this development, it was necessary to determine the input data for the larger watershed. This was done by determining the land uses from existing aerial photographs and the soil types from the existing soil maps. The topography of the larger watershed could be determined from the USGS Quadrangle sheet but in this case the topography was determined from the recent Horry County digital terrain model. Once all the watershed site characteristics were all gathered, a discharge hydrograph at the 10% location was generated. This hydrograph is shown in Figure 8-8.

In assessing the downstream effects, the hydrographs from the site development are moved downstream using channel routing calculations and compared to the larger watershed hydrograph. The Pre- and Post-Development 25-year hydrographs at the 10% location are also shown in Figure 8-8. In order to determine if the storage facilities at this development will causes detrimental effects at this 10% location, we compare the hydrographs and in particular the times to peak. From Figure 8-8, the time to peak for the Post-Development hydrograph is 20 hours while the time to peak for the larger watershed is 22.5 hours. Since the time to peaks are offset, the storage facilities at this site development should not have an adverse impact on downstream areas.

8.3.6 Wetlands

There are no freshwater wetlands located on this site as determined by the U.S. Army Corps of Engineers. See Figure 8-7 for the wetlands determination and certification letter provided by the U.S. Army Corps of Engineers.

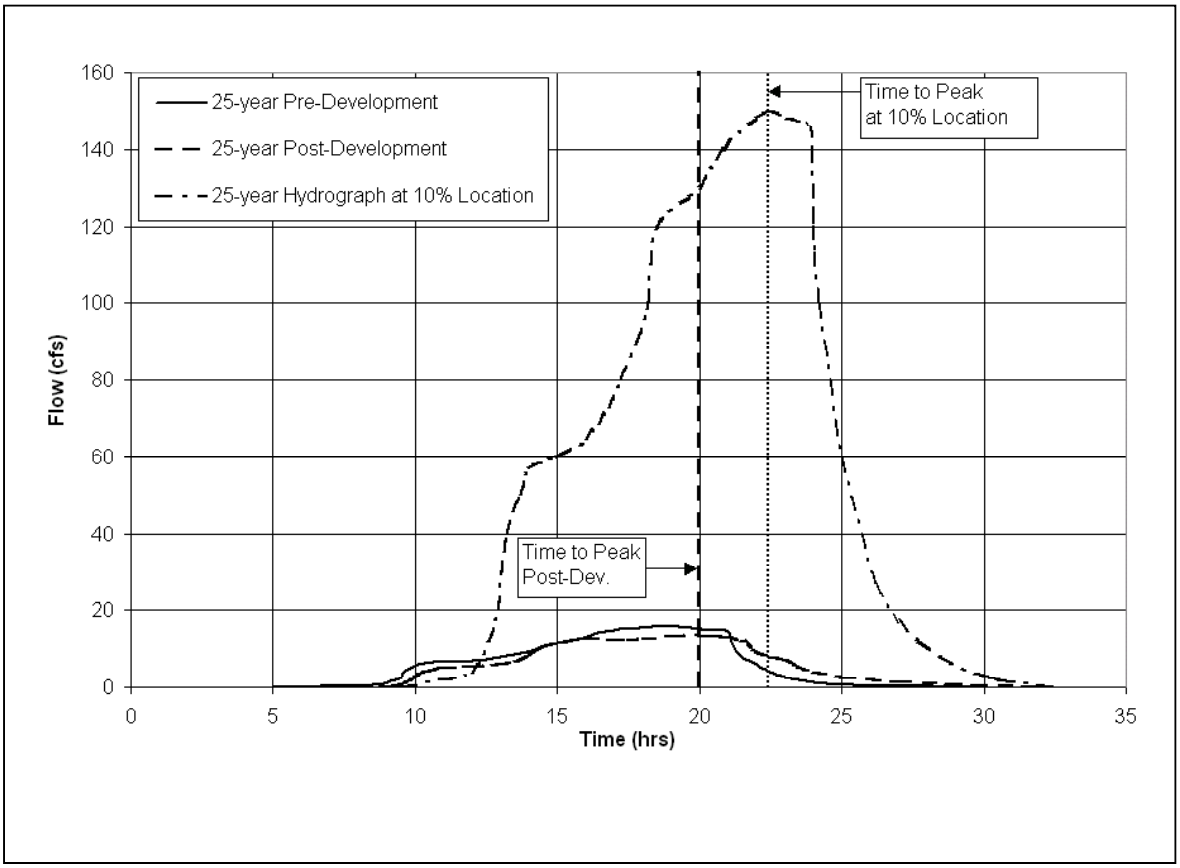

Figure 8-8 Hydrograph Comparison at the 10% Downstream Location

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00256/99-060

NOTIFICATION OF JURISDICTIONAL DETERMINATION
U.S. Army Corps of Engineers
Charleston District

Action ID: 84-99-0201(Y) County Horry

Property Owner/Authorized Agent________________________________

Address________________________________

________________________________ Telephone Number________________

Size and Location of Property 47.08 acres off of S.C. Hwy 9 as indicated on the plat by Affiliated Surveyors, dated Nov. 3, 1998 and titled "Boundary Survey For Mary Lord Johnson Newton Lacey, Horry County South Carolina Simpson Creek Twp.

Indicate Which of the Following Apply to the Property:

Jurisdictional Determination ("JD") Needed

_____ There are areas within the jurisdiction of the Corps which we strongly suggest should be delineated and surveyed. The surveyed boundaries must be verified by our staff before the Corps will make a final determination.

Large or Commercial Tracts

_____ Because of the size of your property and our present workload, our identification and delineation of the jurisdictional areas cannot be accomplished in a timely manner. You may wish to employ a consultant to obtain a more timely delineation. Once your consultant has flagged the jurisdictional areas, Corps staff will review it, and if it is accurate, we strongly recommend that you have the line surveyed for final approval by the Corps. The Corps will not make a final JD without an approved survey.

JD Finished

_____ The jurisdictional areas have been delineated, and the limits of Corps jurisdiction have been explained to you. Unless there is a change in the law or our published regulations, this JD may be relied upon for a period not to exceed five years from the date of this notification.

No Wetlands

__✓__ There are no areas which are subject to the permit requirements of Section 404 of the Clean Water Act (33 USC 1344). Unless there is a change in the law or our published regulations, this JD may be relied upon for a period not to exceed five years from the date of this notification.

Coastal Zone

__✓__ The property is located in the South Carolina Coastal Zone. In addition to any above requirements, you should contact the S. C. DHEC Office of Ocean and Coastal Resource Management at 1-843-744-5838 for their requirements.

Placement of dredged material or fill material in waters of the U.S. without a Department of the Army permit or exemption may result in injunctive relief (restoration) and substantial civil penalties under Section 309 of the Clean Water Act (33 USC 1319). A permit is not required for work restricted entirely to existing high ground. If you have any questions regarding the limits of our jurisdiction, what constitutes a regulated activity, or our regulatory program in general, please contact us at 1-800-208-2054 or 1-843-727-4330.

Property owner/Authorized Agent Signature________________________________

Project Manager Signature ________________________________

Date 5-11-99

Survey Plat or Field Sketch of described property and the JD must be attached to the Yellow (File) copy of this form

SAC Form 304
August 1998

Figure 8-9 Wetlands Determination and Certification Letter