

### 3. Stormwater Calculations

#### 3.1. Stormwater Management Objectives

The objective of BMPs is to minimize the adverse effects of development by mimicking, as closely as possible, the runoff characteristics of the site in its undeveloped state. These characteristics include:

- Moderation of runoff peak flows and volumes to minimize downstream erosion and damage to in-stream aquatic habitat.
- Removal of pollutants such as sediment, nutrients, pathological bacteria and heavy metals.
- Infiltration of rainfall to replenish the water table and provide stable base flow to streams.

The preferred stormwater management approach is to preserve the natural storage, infiltration, and pollutant-treatment functions of each drainage area where practical, and where not practical to construct BMPs that mimic those natural functions as closely as possible.

Stormwater calculations are required to analyze a proposed new development for its impacts on peak flows and volumes. Stormwater programs in North Carolina typically include provisions to control and treat a certain volume of stormwater runoff and/or provisions to control the peak stormwater discharge rate. Additional calculations are required to design BMPs with appropriate treatment capacity and correctly sized outlet structures. Table 3-1 summarizes the stormwater calculations and allowable methods that will be presented in this chapter.

**Table 3-1**  
Summary of Stormwater Calculations

Calculation of:	Section	Allowable Methods
Peak Flow	3.2	Rational Method
Runoff Volume	3.3	Simple Method Discrete SCS Curve Number Method
Storage Volume	3.4	Stage-Storage Table
Hydraulic Performance of the Outlet Device	3.5	Weir Equations Orifice Equation
Stage-Storage-Discharge	3.6	Chainsaw Routing Others: HEC-HMS, WinTR-55, SWIMM
Channel Geometry	3.7	Manning Equation
Nutrient Loading	3.8	DWQ Neuse TN Export Worksheet DWQ Tar-Pamlico Nutrient Export Worksheet
Pollutant Removal of BMPs	3.9	Stand-alone BMPs Multiple Drainage Areas BMPs in Parallel BMPs in Series

**Note:** Designers may adopt different calculation methods, but the method chosen must provide equivalent or greater protection than the methods presented here.

### 3.2. Peak Flow Calculations

Some of the state’s stormwater programs require providing attenuation of peak runoff; for example, that the post-development flow rate for the one-year, 24-hour storm may not exceed the pre-development flow rate (Neuse and Tar-Pamlico NSW Programs). In addition, it is also important to compute flow rates from the watershed when designing BMPs such as grassed swales, filter strips, and restored riparian buffers.

The primary method that is used to determine peak runoff rate for North Carolina’s stormwater programs is the Rational Method. The Rational equation is given as:

$$Q = C * I * A$$

- Where:
- Q = Estimated design discharge (cfs)
  - C = Composite runoff coefficient (unitless) for the watershed
  - I = Rainfall intensity (in/hr) for the designated design storm in the geographic region of interest
  - A = Watershed area (ac)

The composite runoff coefficient reflects the surface characteristics of the contributing watershed. The range of runoff coefficient values varies from 0 - 1.0, with higher values corresponding to greater runoff rate potential. The runoff coefficient is determined by estimating the area of different land uses within each drainage area. Table 3-2 presents values of runoff coefficients for various pervious and impervious surfaces. The Division believes that the Rational Method is most applicable to drainage areas approximately 20 acres or less.

**Table 3-2**

Rational runoff coefficients (ASCE, 1975; Viessman, et al., 1996; and Malcom, 1999)

Description of Surface	Rational Runoff Coefficients, C
Unimproved Areas	0.35
Asphalt	0.95
Concrete	0.95
Brick	0.85
Roofs, inclined	1.00
Roofs, flat	0.90
Lawns, sandy soil, flat (<2%)	0.10
Lawns, sandy soil, average (2-7%)	0.15
Lawns, sandy soil, steep (>7%)	0.20
Lawns, heavy soil, flat (<2%)	0.15
Lawns, heavy soil, average (2-5%)	0.20
Lawns, heavy soil, steep (>7%)	0.30
Wooded areas	0.15

The appropriate value for *I*, precipitation intensity in inches per hour, can be obtained from the NOAA web site at: <http://hdsc.nws.noaa.gov/hdsc/pfds/>. This web site

allows the user to select from one of NOAA’s numerous data stations throughout the state. Then, the user can ask for precipitation intensity and view a table that displays precipitation intensity estimates for various annual return intervals (ARIs) (1 year through 1000 years) and various storm durations (5 minutes through 60 days). The requirements of the applicable stormwater program will determine the appropriate values for ARI and storm duration. If the design is for a level spreader that is receiving runoff directly from the drainage area, then the value for *I* should simply be one inch per hour (more information on level spreader design in Chapter 8).

### 3.3. Runoff Volume

Many stormwater programs have a volume control requirement; that is, capturing the first 1 or 1.5 inches of stormwater and retaining it for 2 to 5 days. There are two primary methods that can be used to determine the volume of runoff from a given design storm: the Simple Method (Schueler, 1987) and the discrete SCS Curve Number Method (NRCS, 1986). Both of these methods are intended for use at the scale of a single drainage area. Stormwater BMPs shall be designed to treat a volume that is at least as large as the volume calculated using the Simple Method. If the SCS Method yields a greater volume, then it can also be used.

#### 3.3.1. Simple Method

The Simple Method uses a minimal amount of information such as watershed drainage area, impervious area, and design storm depth to estimate the volume of runoff. The Simple Method was developed by measuring the runoff from many watersheds with known impervious areas and curve-fitting a relationship between percent imperviousness and the fraction of rainfall converted to runoff (the runoff coefficient). This relationship is presented below:

$$R_V = 0.05 + 0.9 * I_A$$

Where:  $R_V$  = Runoff coefficient [storm runoff (in)/storm rainfall (in)], unitless  
 $I_A$  = Impervious fraction [impervious portion of drainage area (ac)/drainage area (ac)], unitless.

Once the runoff coefficient is determined, the volume of runoff that must be controlled is given by the equation below:

$$V = 3630 * R_D * R_v * A$$

Where:  $V$  = Volume of runoff that must be controlled for the design storm (ft<sup>3</sup>)  
 $R_D$  = Design storm rainfall depth (in) (*Typically, 1.0" or 1.5"*)  
 $A$  = Watershed area (ac)

**Example Runoff Volume Calculations using The Simple Method: Projects draining to SA or ORW waters in a Coastal County:**

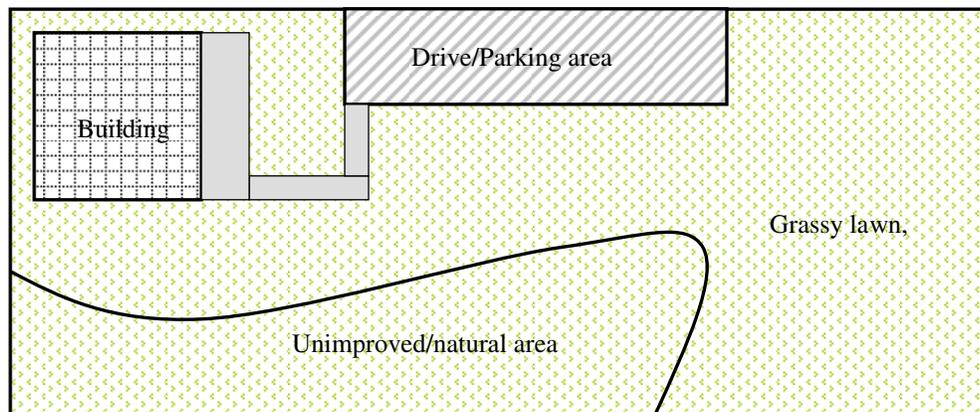
Effective October 1, 2008, the revised Coastal Stormwater Rules (Session Law 2008-211) stipulate new volume control requirements for **high density projects draining to shellfishing waters (SA) or Outstanding Resource Waters (ORW)** in the Coastal Counties. The designer must compare different scenarios: runoff from first 1.5" of rainfall the pre- /post-development for the 1-year, 24-hour storm. These criteria provide a useful demonstration of runoff volume calculations used in different North Carolina stormwater programs.

The following example uses the Simple Method (Schueler, 1987) to estimate runoff volume in a hypothetical situation:

*Example:* What is the appropriate runoff volume for choosing and designing a BMP to control stormwater runoff from a **high density development project within one-half mile and draining to SA waters?**

*Description:* 4-acre site within a half-mile and draining to SA waters will be developed with a 15,000 sq. foot building (0.34 ac), 0.5 acre of asphalt road, parking, and concrete walkway areas (see Figure 3-1), and 1.86 acres of grassy lawn. The rest of the property (1.3 ac) will remain unimproved.

**Figure 3-1**  
Schematic of Property for Example Runoff Volume Calculation



The BUA density will be  $(0.34 + 0.5)/4 = 21.5\%$ . This will be considered a high-density project as it is above the 12% threshold. Because of this, the engineer must determine which runoff volume is greater in the following design conditions. Note that the area where BMPs will eventually be installed is assumed to be grass areas in these calculations.

First, runoff from the first 1.5" of rainfall:

Calculate the Simple Method Runoff Coefficient with  $R_V = 0.05 + 0.9 * I_A$  :

$$R_V = 0.05 + 0.9 * (0.84 \text{ ac} / 4 \text{ ac})$$

$$= \mathbf{0.239}$$

Calculate Runoff Volume with  $V = 3630 * R_D * R_v * A$  :

$$V_{1.5"} = 3630 * 1.5 * 0.239 * 4$$

$$= \mathbf{5,205 \text{ cu. ft}}$$

Compare the volume difference between the pre- and post-development runoff from the 1-year, 24-hour rainfall event. For this example, we will assume the project is in the vicinity of Wilmington, NC.

Pre-development:  $R_V = 0.05 + 0.9 * (0 \text{ ac} / 4 \text{ ac})$

$$= \mathbf{0.05}$$

Post-development:  $R_V = 0.05 + 0.9 * (0.84 \text{ ac} / 4 \text{ ac})$

$$= \mathbf{0.239}$$

Data from the National Oceanic and Atmospheric Administration (NOAA) indicate the depth of the 1-year, 24-hour rain event in Wilmington, NC is approximately **3.83 inches**. (We looked up the **WILMINGTON WSO AIRPORT, NORTH CAROLINA** location for this example). Therefore:

$$V_{\Delta(\text{post-predevelopment})} = 3630 * 3.83 * (0.239 - 0.05) * 4$$

$$= \mathbf{10,511 \text{ cu. ft}}$$

The greater of the two is the difference between the pre- and post-development. To comply with the Coastal Stormwater requirements, the BMP should be designed for 10,511 cubic feet of runoff.

### 3.3.2. Discrete SCS Curve Number Method

The SCS method (SCS, 1985; NRCS, 1986) is an alternative method for calculating the volume of stormwater runoff that is generated from a given amount of rainfall. **It may only be used when the site design is a Low Impact Development (LID).**

The SCS runoff equation is given below:

$$3.3.1. \quad \left( \frac{P - 0.2S}{P + 0.8S} \right)^2$$

Where:  $Q^*$  = Runoff depth (in)  
 $P$  = Rainfall depth (in)  
 $S$  = Potential maximum retention after rainfall begins (in)

$S$  is related to the soil and surface characteristics of the drainage area through the curve number (CN) by the following equation:

$$S = \frac{1000}{CN} - 10$$

Where: CN is the curve number, unitless.

The curve number, CN, describes the characteristics of the drainage area that determine the amount of runoff generated by a given storm: hydrologic soil group and ground cover. Soils are classified into four hydrologic soil groups (A, B, C, and D) based on their minimum infiltration rate, with A having the highest infiltration potential and D having the lowest. The four soil groups are summarized in Table 3-3.

The required treatment volume is determined by multiplying the runoff depth ( $Q^*$ ) by the drainage area.

**Table 3-3**  
**Four Hydrologic Soil Groups as Defined by the SCS (1986)**

Group A	A soils have low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sand or gravel and have a high rate of water transmission (greater than 0.30 in/hr). The textures of these soils are typically sand, loamy sand, or sandy loam.
Group B	B soils have moderate infiltration rates when thoroughly wetted and consist chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission (0.15-0.30 in/hr). The textures of these soils are typically silt loam or loam.
Group C	C soils have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture. These soils have a low rate of water transmission (0.05-0.15 in/hr). The texture of these soils is typically sandy clay loam.
Group D	D soils have high runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very low rate of water transmission (0-0.05 in/hr). The textures of these soils are typically clay loam, silty clay loam, sandy clay, silty clay, or clay.

Table 3-4 lists the hydrologic soil grouping for most soil series in North Carolina. Some soils may reside in two groups depending on the presence of a high water table that limits infiltration. If these soils are effectively drained, they are placed in the group with lower runoff potential. For example, Cape Fear soil is classified as B/D, which indicates that it is in group B if drained and in group D if undrained. If a soil at a given site is not listed in Table 3-4, the surface layer soil texture may be used to determine the hydrologic soil group. The texture may be determined by soil analysis or from the local soil survey.

**Table 3-4**  
 Hydrologic soil groups for soil types found in North Carolina (Malcom, 1989)

Alaga	A	Dragston	D/C	Louisa	B	Ridgeland	C
Alamance	B	Dunbar	D/B	Louisburg	B	Rimini	C
Albany	C/A	Duplin	C/B	Lucy	A	Roanoke	D
Altavista	C/B	Durham	B	Lumbee	D/C	Rosman	B
Americus	A	Dykes	B	Lynchburg	C/B	Rumford	B
Appling	B	Edneyville	B	Lynn Haven	D/C	Ruston	B
Ashe	B	Elbert	D	Madison	B	Ruttege	D/B
Augusta	C	Elioak	B	Magnolia	B	Saluda	C/B
Avery	B	Elsinboro	B	Mantachie	C/B	Scranton	D/B
Aycock	B	Enon	C	Manteo	D	Seneca	C/B
Barclay	C	Eustis	A	Marlboro	B	Starr	B
Barth	C	Exum	C/B	Masada	B	State	B
Bayboro	D/C	Faceville	B	Maxton	B	Suncook	A
Bertie	C/B	Fannin	B	Mayodan	B	Surry	B
Bibb	D/B	Fletcher	B	McColl	D/C	Talladega	C
Bladen	D/C	Fuquay	B	Mecklenburg	C	Tallepoosa	C
Blaney	B	Georgeville	B	Meggett	D/C	Tate	B
Blanton	A	Gilead	C	Molena	A	Taturn	B
Bowie	B	Goldsboro	C/B	Musella	B	Thurmont	B
Braddock	B	Goldston	C	Myatt	D/C	Toccoa	B
Bradley	B	Granville	B	Nahunta	C/B	Toisnot	C/B
Brandywine	B	Grover	B	Nason	C	Torhuna	C/A
Brevard	B	Guin	A	Nixonton	B	Toxaway	D
Bucks	B	Gwinnett	B	Norfolk	B	Transylvania	B
Buncombe	A	Hartsells	B	Ochlockonee	B	Troup	A
Burton	B	Hatboro	D/C	Ocilla	C/B	Tuckerman	D/C
Byars	D	Hayesville	B	Olustee	D/C	Tusquitee	B
Cahaba	B	Haywood	B	Onslow	B	Unison	B
Cape Fear	D/B	Helena	C	Orange	D	Vance	C
Caroline	C	Herndon	B	Orangeburg	B	Varina	C
Cartecay	C	Hiwassee	B	Osier	D	Vaocluse	C
Cataula	C	Hoffman	C	Pacolet	B	Wadesboro	B
Cecil	B	Hulett	B	Pactolus	C/A	Wagram	A
Chandler	B	Hyde	D/C	Pamlico	D/C	Wahee	D/C
Chastain	D	Invershiel	C	Pantego	D/C	Wake	D
Chester	B	Iredell	D	Pasquotank	D/B	Watauga	B
Chesterfield	B	Iuka	C	Pelham	D/C	Wedowee	B
Chewacla	C	Izagora	C	Pender	D		
Chipley	C/A	Johnston	D/B	Penn	C/B		
Clifton	B	Johus	C/B	Pinkston	C		
Codurus	C	Kalmia	B	Plummer	D/B		
Colfax	C	Kenansville	A	Pocalla	A		
Comus	B	Kershaw	A	Pocomoke	D/B		
Congaree	B	Kinston	D/C	Pomello	C/A		
Cowarts	C	Lakeland	A	Ponzer	D/C		
Coxville	D/C	Leaf	D/C	Porters	B		
Craven	C	Lenoir	D/B	Portsmouth	D/C		
Davidson	B	Leon	C/B	Rabun	B		
Delanco	C	Liddell	D/C	Rains	D/B		
Dorovan	D	Lloyd	B	Ramsey	D		
Dothan	B	Lockhart	B	Ranger	C		

The type of ground cover at a given site greatly affects the volume of runoff. Undisturbed natural areas, such as woods and brush, have high infiltration potentials whereas impervious surfaces, such as parking lots and roofs, will not infiltrate runoff at all. The ground surface can vary extensively, particularly in urban areas, and Table 3-5 lists appropriate curve numbers for most urban land use types according to hydrologic soil group. Land use maps, site plans, and field reconnaissance are all effective methods for determining the ground cover.

**Table 3-5**  
Runoff curve numbers in urban areas for the SCS method (SCS, 1986)

Cover Description	Curve Numbers for Hydrologic Soil Group			
	A	B	C	D
<i>Fully developed urban areas</i>				
Open Space (lawns, parks, golf courses, etc.)				
Poor condition (< 50% grass cover)	68	79	86	89
Fair condition (50% to 75% grass cover)	49	69	79	84
Good condition (> 75% grass cover)	39	61	74	80
Impervious areas:				
Paved parking lots, roofs, driveways, etc.	98	98	98	98
Streets and roads:				
Paved; curbs and storm sewers	98	98	98	98
Paved; open ditches	83	89	98	98
Gravel	76	85	89	91
Dirt	72	82	85	88
<i>Developing urban areas</i>				
Newly graded areas	77	86	91	94
Pasture (< 50% ground cover or heavily grazed)	68	79	86	89
Pasture (50% to 75% ground cover or not heavily grazed)	49	69	79	84
Pasture (>75% ground cover or lightly grazed)	39	61	74	80
Meadow - continuous grass, protected from grazing and generally mowed for hay	30	58	71	78
Brush (< 50% ground cover)	48	67	77	83
Brush (50% to 75% ground cover)	35	56	70	77
Brush (>75% ground cover)	30	48	65	73
Woods (Forest litter, small trees, and brush destroyed by heavy grazing or regular burning)	45	66	77	83
Woods (Woods are grazed but not burned, and some forest litter covers the soil)	36	60	73	79
Woods (Woods are protected from grazing, and litter and brush adequately cover the soil)	30	55	70	77

Most drainage areas include a combination of land uses. The SCS Curve Number Model should be applied separately: once for areas where impervious cover is directly connected to surface water via a swale or pipe and a second time for the remainder of the site. The runoff volumes computed from each of these computations should be added to determine the runoff volume for the entire site.

For the portion of the site that is NOT directly connected impervious surface, a composite curve number can be determined to apply in the SCS Curve Number Model. The composite curve number must be area-weighted based on the distribution of land uses at the site. Runoff from impervious areas that is allowed to flow over pervious

areas has the potential to infiltrate into the soil (for example, where roof downspouts are diffused over a lawn). Disconnected impervious areas produce less runoff than impervious areas that are directly connected to a storm drainage system.

**Table 3-6**  
How to apply the SCS Curve Number Method

Step 1.	Divide the drainage area into land uses and assign an appropriate CN to each one (see Table 3.5).
Step 2.	Compute $Q^*$ for any impervious surfaces that are directly linked to surface waters via a swale or pipe. Find the runoff volume from the directly connected impervious surfaces by multiplying $Q^*$ times the area of the directly connected impervious surfaces.
Step 3.	Composite a curve number for the remainder of the site by using a weighted average. If the composite CN is equal to or below 64, assume that there is no runoff resulting from either the 1 or 1½ inch storm. If the composite CN is above 64, compute $Q^*$ for this area. Find the runoff volume from the remainder of the site by multiplying $Q^*$ times the area of the remainder of the site.
Step 4.	Find the runoff volume from the whole site by adding the results of Step 2 and Step 3.

### 3.4. Storage Volume

Volume control is typically provided through detention structures with volume above the water operating level and below the required freeboard. Some BMPs do not have the capability to provide this volume control due to their design, and others can include storage volume within the media of the BMP. Each individual BMP chapter discusses the specific calculations for meeting the volume control requirements. However, since many of the BMPs use storage volume in a detention structure, this section will discuss an acceptable method of calculating that volume.

Storage volume within a detention structure shall be calculated using a stage-storage method. A table shall be provided showing incremental elevations of the BMP with square footage values at the listed elevations. The elevation increments shall be no more than 1 foot. Columns can then be produced showing the incremental volume and cumulative volume of storage provided. See Table 3-7 below for an example of a storage volume calculation. This method can be used for basin shapes as simple as a rectangle or as intricate as a curved, landscape designed wetland feature. It can also be used to calculate sediment storage volume and operating volume within BMPs.

**Table 3-7**  
Stage-Storage Volume Calculation Table Example

Elevation	Surface Area (sf)	Incremental Volume (cf)	Cumulative Volume (cf)
less than 725	operating volume	0	0
725	10,000	0	0
726	13,000	11,500	11,500
727	16,500	14,750	26,250
728	21,500	19,000	45,250
729	26,000	23,750	69,000
over 729	freeboard	0	69,000

### 3.5. Hydraulic Performance of the Outlet Device

In order to successfully design a stormwater treatment system, it is crucial to analyze the way in which the outlet devices release stormwater outflow. Typically, these devices can be considered as either weirs or orifices. A weir is a dam placed horizontally along a stream or channel to raise its level or divert its flow. Some uses for weirs are in the design of stormwater BMPs are:

- Check dams in channels,
- Flow splitter devices,
- Flow into a pipe before it is completely submerged, and
- Level spreaders.

An orifice is simply a hole. In the design of stormwater BMPs, orifices are used to drain a BMP that is detaining stormwater for volume control and pollutant removal. It is important to determine the size an orifice correctly so that the appropriate drawdown rate can be provided.

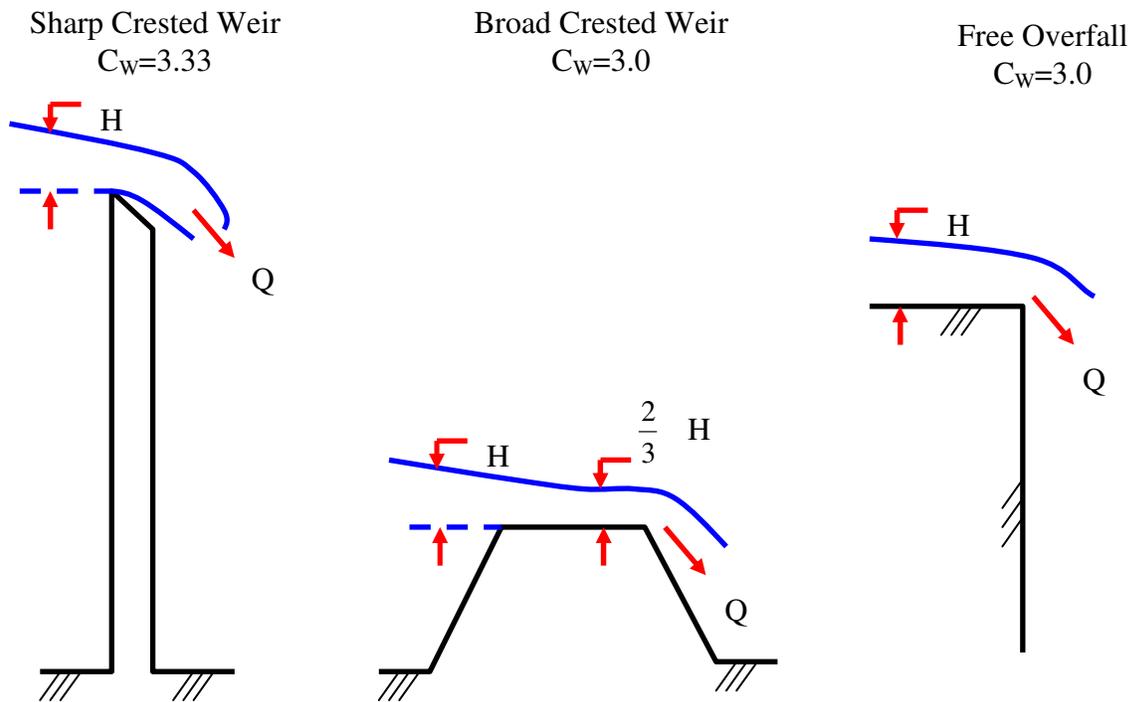
#### 3.5.1. Weir Equations

Three kinds of weirs are typically used: sharp-crested, broad-crested and v-notch. For sharp-crested and broad-crested weirs, the basic equation is:

$$Q = C_w L H^{1.5}$$

- Where:
- $Q$  = Discharge (cfs)
  - $C_w$  = Coefficient of discharge (dimensionless) – see below
  - $L$  = Length of weir (ft), measured along the crest
  - $H$  = Driving head (ft), measured vertically from the crest of the weir to the water surface at a point far enough upstream to be essentially level

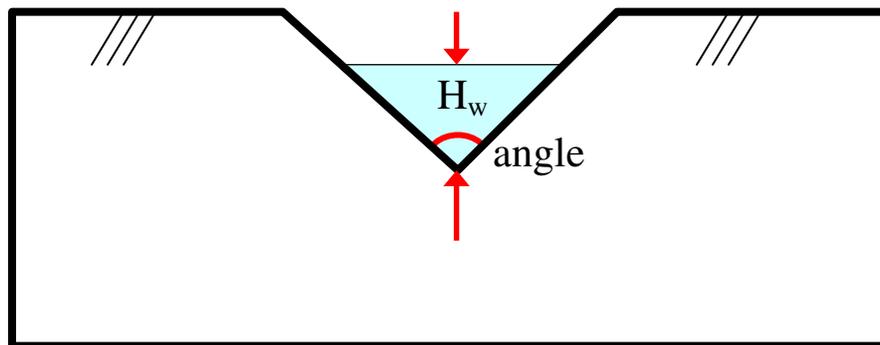
**Figure 3-2**  
Schematic sections through weirs (Malcom 1989)



For v-notch weirs, the basic equation is:

$$Q = C_v H_w^{5/2}$$

- Where:
- $Q$  = Discharge (cfs)
  - $C_v$  = Weir flow coefficient for V-notch weirs
    - 2.50 for 90 degrees
    - 1.44 for 60 degrees
    - 1.03 for 45 degrees
  - $H_w$  = Difference between pool elevation and notch (ft)



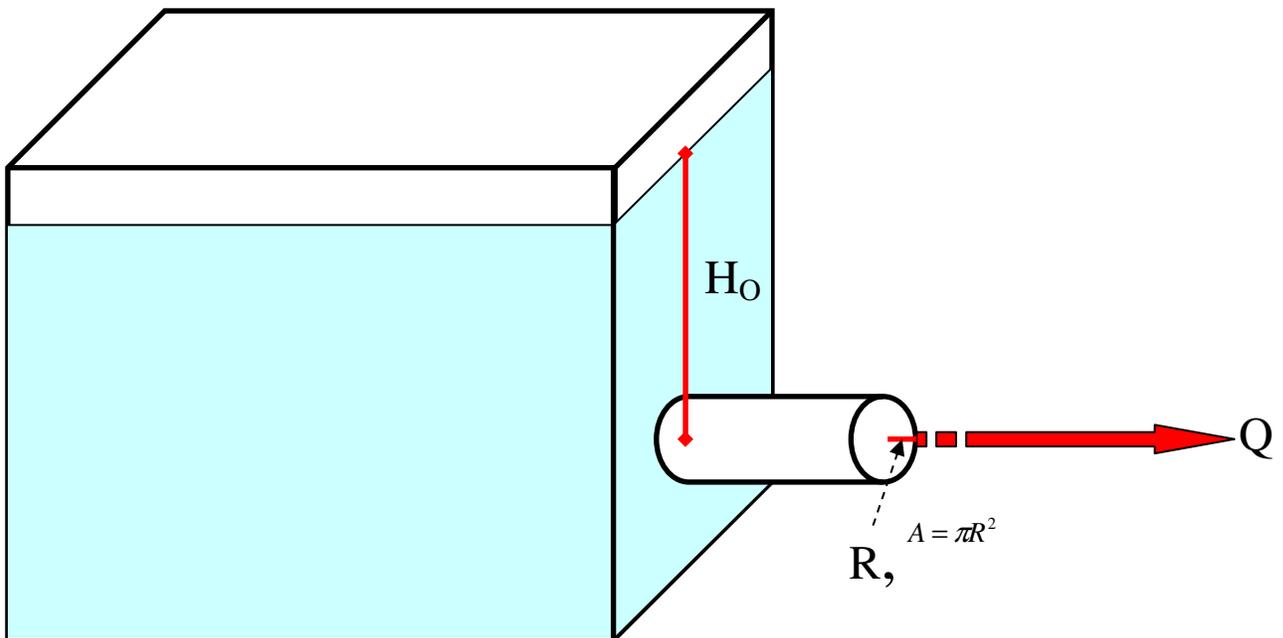
### 3.5.2. Orifice Equation

The basic equation for orifices is:

$$Q = C_D A \sqrt{2gH_o}$$

- Where:
- $Q$  = Discharge (cfs)
  - $C_D$  = Coefficient of discharge (dimensionless) – see Table 3-8
  - $A$  = Cross-sectional area of flow at the orifice entrance (sq ft)
  - $g$  = Acceleration of gravity (32.2 ft/sec<sup>2</sup>)
  - $H_o$  = Driving head (ft), measured from the centroid of the orifice area to the water surface – **Note: Usually use  $H_o/3$  to compute drawdown through an orifice to reflect the fact that head is decreasing as the drawdown occurs.**

**Figure 3-3**  
Schematic section through an orifice



**Table 3-8**  
Values of Coefficient of Discharge,  $C_D$  (Malcom, 1989)

Entrance Condition	$C_D$
Typical default value	0.60
Square-edged entrance	0.59
Concrete pipe, grooved end	0.65
Corrugated metal pipe, mitred to slope	0.52
Corrugated metal pipe, projecting from fill	1.00

## 3.6. Stage-Storage-Discharge Model

Creating a stage-storage-discharge model is crucial for stormwater BMPs that involve detention of stormwater, particularly stormwater wetlands and wet detention basins. These BMPs provide volume control for the specified storm (for example, the 1 or 1½ inch storm) in a temporary pool that is above the permanent pool.

(Please note that some BMPs do not have the capability to provide this volume control due to their design, and others can include storage volume within the media of the BMP. Each BMP Section will discuss the specific calculations for meeting the volume control requirements.)

### 3.6.1. Chainsaw Routing

The Chainsaw Routing method is appropriate for the routine design of small systems. Three sets of source data are needed to apply the Chainsaw Routing method:

- The inflow hydrograph,
- The size and shape of the storage basin, and
- The hydraulics of the outlet device.

The application of the Chainsaw Routing method is described in detail in Elements of Urban Stormwater Design (Dr. H. Rooney Malcom, P.E. 1989). Please refer to Section III, Stormwater Impoundments for a detailed explanation and examples of its use. This reference is available in Appendix B.

### 3.6.2. Other Models

Other models may be used to assist in determining stage-storage-discharge through a detention BMP. These models include:

- HEC-HMS, developed by the U.S. Army Corps of Engineers, provides a variety of options for simulating precipitation-runoff processes. This model can simulate unit hydrograph and hydrologic routing options. The latest version also has capabilities for continuous soil moisture accounting and reservoir routing operations.  
<http://www.hec.usace.army.mil/software/hec-hms/download.html>
- WinTR-55, developed by the NRCS, can be used to analyze the hydrology of small watersheds. A final version (including programs, sample data, and documentation) is now complete.  
<http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-models-wintr55.html>.
- SWIMM, developed by the EPA, can be used to analyze stormwater quantity and quality associated with runoff from urban areas. Both single-event and continuous simulation can be performed on catchments having storm sewers, or combined sewers and natural drainage, for prediction of flows, stages and pollutant concentrations.  
<http://www.epa.gov/ceampubl/swmm.htm>

### 3.7. Channel Geometry

The Manning Equation is the model of choice for determining the cross-section for a trapezoidal stormwater channel. It is applicable where (Malcom 1989):

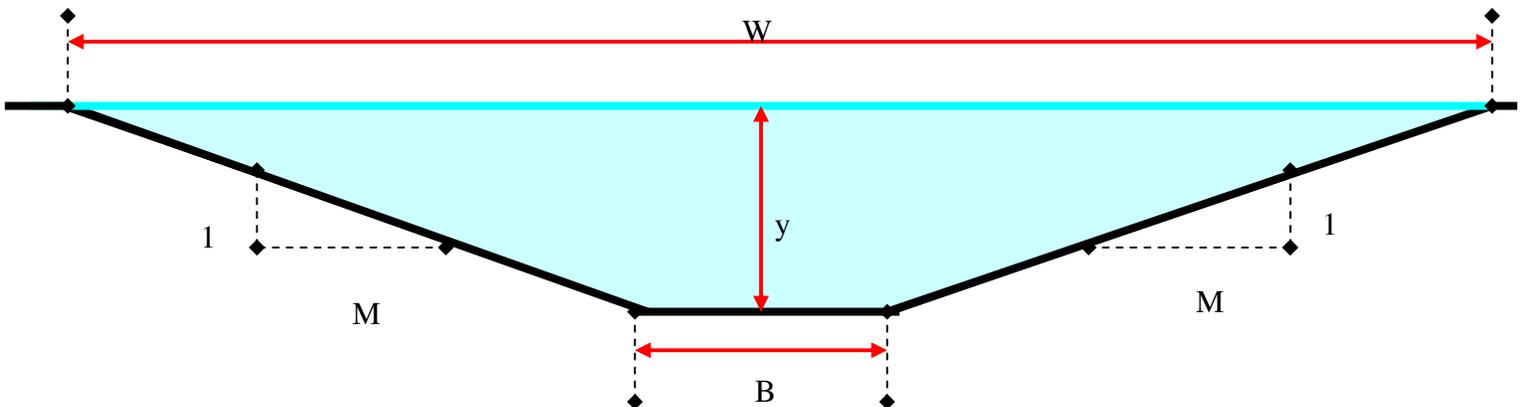
- Stormwater is flowing under the influences of gravity, and
- Flow is steady - it does not vary with time (Although discharge does vary during the passage of a flood wave, it is essentially steady during the time around the peak, the time of interest in channel design.)

The Manning Equation can be stated as:

$$Q = \frac{1.489}{n} A R^{0.667} S^{0.5}$$

- Where:
- $Q$  = Peak discharge to the channel (cfs)
  - $n$  = Manning roughness coefficient (dimensionless)
  - $A$  = Cross-sectional area of flow (sq ft), the area through which flow takes place (*see below*)
  - $R$  = Hydraulic radius (ft), found by dividing cross-sectional area,  $A$  (sq ft), by wetted perimeter,  $P$  (ft) (*see below*)
  - $S$  = Longitudinal slope of the invert of the channel (ft fall/ft run).

**Figure 3-4**  
Diagram of a trapezoidal channel\*



\* M is governed by channel side slope requirements, which are typically 3:1 (M=3) unless otherwise specified in this manual.

The Manning roughness coefficient is an experimentally determined value that is a function of the nature of the channel lining.

**Table 3-9**

Rational runoff coefficients (adopted from Munson, et al., 1990 and Chow et al., 1988)

<b>Channel lining</b>	<b>Manning roughness coefficient, n</b>
Asphalt	0.016
Concrete, finished	0.012
Concrete, unfinished	0.014
Grass	0.035
Gravel bottom with riprap sides	0.033
Weeds	0.040

The cross-sectional area of flow, A, can be determined by the following equation:

$$A = By + My^2$$

The wetted perimeter, P, is the distance along the cross section against which the water is flowing. It does not include the free water surface. P can be determined by the following equation:

$$P = B + 2y (1 + M^2)^{0.5}$$

The hydraulic radius, R, can be determined by the following equation:

$$R = \frac{A}{P}$$

For the three equations above, the variables have the following meanings (also refer to Figure 3-4):

- A = Cross-sectional area of flow (sq ft)
- B = Bottom width of the channel (ft)
- M = Side slope ratio (ft horizontal/ft vertical) *(determined by channel side slope requirements)*
- P = Wetted perimeter (ft)
- R = Hydraulic radius (ft)
- y = Depth of flow (ft)

### 3.8. Nutrient Loading

Nutrient control requirements currently apply to new development and redevelopment projects occurring in the jurisdictions of the largest local governments in the Neuse and Tar-Pamlico River Basins.

In the affected jurisdictions in the Neuse and Tar-Pamlico River Basins, new developments are required to meet specified nutrient export rates. These export rates can be met on site through one or more of the following measures:

- Site planning that reduces land uses that contribute high nutrient loadings,
- Implementation of BMPs that remove nutrients, and
- Offset payments to the Wetland Restoration Fund.

For the Neuse River Basin, the nutrient of concern is total nitrogen (TN). The TN export limit is 3.6 pounds per acre per year (lb/ac/yr). Nitrogen load from new developments that exceeds this performance standard may be offset by payment of a fee to the Wetlands Restoration Fund provided. However, no new residential development may exceed 6.0 lb/ac/yr and no new nonresidential development may exceed 10.0 lb/ac/yr.

For the Tar-Pamlico River Basin, the nutrients of concern are TN and total phosphorus (TP). The TN export limit is 4.0 lb/ac/yr and the TP export limit is 0.40 lb/ac/yr. Just like in the Neuse River Basin, nitrogen load from new developments that exceeds this performance standard may be offset by payment of a fee to the Wetlands Restoration Fund provided. However, no new residential development may exceed 6.0 lb/ac/yr and no new nonresidential development may exceed 10.0 lb/ac/yr.

The above nutrient export limits were determined based on analyzing the nutrient loading coming from undeveloped land within each river basin, which consists mainly of forested land and agricultural land. The nutrient loading limit on new development then represents a 30 percent reduction from the overall average nutrient loading rate of the undeveloped land. The rate for TN is slightly higher in the Tar-Pamlico basin than in the Neuse basin due to the higher ratio between agricultural lands (high nutrient loading rate) and forested lands (low nutrient loading rate).

In addition to the requirements for nutrient loading rates, new developments subject to this program must show that there is no net increase in peak flow leaving the site from the predevelopment conditions for the 1-year, 24-hour storm. The intent of this requirement is to protect the stream channel and adjacent riparian buffer, which help to prevent additional nutrients from entering streams.

For both the Neuse and Tar-Pamlico River Basins, the affected local governments are responsible for reviewing new development plans to see that they comply with the nutrient export limits and peak flow control requirements. The affected jurisdictions are listed in Table 3-10 below. The EMC may also designate additional local governments in the Neuse and Tar-Pamlico basins to comply with the NSW stormwater rule in the future.

**Table 3-10**

Local governments affected by the Neuse and Tar-Pamlico NSW Stormwater Programs

Neuse River Basin	Tar-Pamlico River Basin
Cary	Greenville
Durham	Henderson
Garner	Oxford
Goldsboro	Rocky Mount
Havelock	Tarboro
Kinston	Washington
New Bern	Beaufort County *
Raleigh	Edgecombe County *
Smithfield	Franklin County *
Wilson	Nash County *
Durham County *	Pitt County *
Johnston County *	
Orange County *	
Wake County *	
Wayne County *	

\* Applicable areas are those under the direct jurisdiction of the respective county.

**3.8.1. DWQ Neuse TN Export Worksheets**

In the Neuse River Basin, there is a Model Stormwater Program for Nutrient Control that was developed as part of a joint effort between the DWQ and the affected local governments.

Two model methodologies that may be used to calculate the TN export rate from a new development are summarized below. However, local governments may propose alternative approaches where it can be demonstrated to be equivalent.

- Method 1 is intended for residential developments where lots are shown but the actual footprint of buildings are not shown on site plans. This method estimates the impervious surface resulting from building footprints on individual lots based on typical impervious areas associated with given lot sizes.
- Method 2 is for residential, commercial and industrial developments when the entire footprint of the roads, parking lots, buildings and any other built-upon area is shown on the site plans. This method is simpler and more accurate since it does not require estimating the impervious surface based on lot size like Method 1.

Worksheets for Methods 1 and 2 are presented on pages 10 through 12 of *The Neuse River Basin: Model Stormwater Program for Nitrogen Control*. This model program is available at: [http://h2o.enr.state.nc.us/su/PDF\\_Files/Neuse/FinalModel\\_Plan.pdf](http://h2o.enr.state.nc.us/su/PDF_Files/Neuse/FinalModel_Plan.pdf)

### 3.8.2. DWQ Tar-Pamlico Nutrient Export Worksheets

In order to meet the requirements of the Tar-Pamlico NSW Stormwater Program, each of the affected local governments has developed its own program for Nutrient Control. These programs can be accessed at the DWQ web site at:

[http://h2o.enr.state.nc.us/nps/Tar-Pamlico\\_Nutrient\\_Trading\\_Program\\_files/Tar-PamlicoLocalStormwaterPrograms00.htm](http://h2o.enr.state.nc.us/nps/Tar-Pamlico_Nutrient_Trading_Program_files/Tar-PamlicoLocalStormwaterPrograms00.htm)

There are a number of differences between the Neuse and Tar-Pamlico Nutrient Export Worksheets that are summarized as follows:

- The Tar-Pamlico model computes the export of both TN and TP, whereas the Neuse model is only computes TN.
- The Tar-Pamlico model has a separate version for the Piedmont versus the Coastal Plain; the Neuse model does not.
- The Tar-Pamlico breaks the urban land uses into a greater number of categories than the Neuse model.
- The Tar-Pamlico model combines the Methods 1 and 2 used in the Neuse model into a single method.
- Most importantly, the Tar-Pamlico model is presented as an interactive spreadsheet where the designer can input data about the site of interest and the model will compute the appropriate TN and TP export rates in lb/ac/yr.
- The Tar-Pamlico model also allows the designer to input information about the BMPs used to treat each drainage area and the spreadsheet will compute the new loading rate resulting from the use of the BMP.

The interactive Tar-Pamlico nutrient export worksheets are available on the DWQ's web site at:

- Coastal Plain: <http://h2o.enr.state.nc.us/nps/documents/N-PCalcsheetCoastProtected10-04.xls>
- Piedmont: <http://h2o.enr.state.nc.us/nps/documents/N-PCalcsheetPiedProtected10-04.xls>

## 3.9. Pollutant Removal of BMPs

### 3.9.1. Stand-alone BMPs

Throughout this manual, each BMP is assigned a removal rate for TSS, TN and TP. In the case of TSS, the calculation of pollutant removal for a single BMP is not needed because the designer will be required simply to select a BMP that removes 85% of TSS. However, a pollutant removal calculation will be necessary in order to determine whether the nutrient removal requirements of the Neuse and Tar-Pamlico NSW Stormwater Programs have been met.

For a single BMP treating stormwater in an affected area of the Neuse or Tar-Pamlico River Basins, the removal of the a pollutant by a single BMP is shown below:

$$PL_e = PL_i * RE$$

Where:  $PL_e$  = Pollutant Loading in the Effluent (lb/ac/yr)  
 $PL_i$  = Pollutant Loading in the Influent (lb/ac/yr)  
 $RE$  = Listed Removal Efficiency of the BMP

**3.9.2. Multiple Drainage Areas**

When a site contains multiple drainage areas, the overall site must still meet the quantity and quality requirements of the applicable stormwater program before leaving the site. In addition, if a drainage area extends upstream from a site, the stormwater volume and pollutant transport calculations must be performed for that upstream section of the drainage area and included in the total volume and loading values for the site for treatment by the appropriate BMP. Calculations for BMPs that are located in separate drainage areas must be performed as presented above for single BMPs; however, nutrient loadings from each drainage area (or “catchment”) may be weighted to meet the overall site removal requirement.

**3.9.3. BMPs in Parallel**

If designed properly, a parallel placement of multiple BMPs could additively combine the storage volume of each individual BMP to provide a total volume control function that would meet the stormwater program requirements. However, a parallel placement of multiple BMPs would not increase pollutant removal efficiencies but would just provide a flow-weighted removal proportional to the individual removal rate and the fraction of total flow passing through each particular BMP.

**3.9.4. BMPs in Series**

Multiple BMPs may be placed in series within the same drainage area to combine treatment capabilities. If multiple BMPs are placed in series, they can utilize the combined volume control capabilities and increase combined removal efficiency. The volume control capabilities are additive, however, the pollutant removal rates are not. The overall efficiency ( $E$ ) for a given pollutant (TSS, TN or TP) of multiple BMPs in series is computed as follows (Division of Watershed Management, 2004):

$$E = A + B - \left[ \frac{(AxB)}{100} \right]$$

Where:  $E$  = Total pollutant removal efficiency (%)  
 $A$  = Efficiency of the First or Upstream BMP  
 $B$  = Efficiency of the Second or Downstream BMP

For more than two BMPs in series, the equation can be applied iteratively from upstream to downstream using the calculated total efficiency as the upstream efficiency in each successive iteration.

It has been found that pollutant removal effectiveness does not continue to increase when using the same removal mechanism over and over. For any set of multiple BMPs placed in series, the combined removal efficiency equation can use pollutant removal efficiency values from a maximum of two BMPs with the same removal mechanism. Additional BMPs with the same removal mechanism will not increase the removal efficiency, but they can contribute to volume control capabilities. A categorization of the BMP removal mechanisms is provided in Table 3-11.

**Table 3-11**  
BMPs categorized by removal mechanism

<b>Removal Mechanism</b>	<b>BMPs</b>
Detention/Retention	Dry Extended Detention Basin Wet Detention Basin Stormwater Wetlands
Filtration	Sand Filters Bioretention
Infiltration	Infiltration Devices Porous Pavement
Natural Conveyance	Filter Strip Grassed Swale Restored Riparian Buffer

This discussion of multiple BMPs placed in series in this section is assuming the BMPs are directly successive. If there is additional drainage area between them, the flow volume and pollutant load from that additional drainage area must be added into the calculations between the discharge of the first BMP and the influent of the second BMP.

### 3.10. Wetlands Area Exclusion: Policy for Projects Under Old Coastal Rules

As of October 1, 2008, no CAMA-jurisdictional wetlands areas may be included in the calculations to determine the built upon area percentage. All other wetlands can be included in the calculations. For projects with vested rights, reviewed for redevelopment, or reviewed as a permit modification under the 1995 Coastal Stormwater Rules, the Division of Water Quality implemented the following policy for calculating project area:

#### 3.10.1. Wetlands Area Exclusion Policy (1995 Coastal Rules Only)

As development continues, the available land for development continues to degrade. Most of the available development tracts along the coast have large expanses of wetlands. This policy outlines a method whereby some credit is given for the wetlands, but which prevents a pocket of high density from being formed on the uplands.

The amount of wetlands that can be included in the site area is dictated by whichever of the following two calculation methods yields the least amount of site area:

- Method 1: Calculate the acreage of the wetlands. Divide the wetland acreage by the total site acreage. If the result is 25% or less, then the total project area is the total site acreage. If the result is more than 25%, then calculate the amount of wetland to include such that no more than 25% of the site area acreage consists of wetlands in this manner: subtract the entire wetlands area from the site area to get the uplands area, then divide the uplands area by 0.75.
- Method 2: Calculate the area of a 100 foot wide strip of wetlands measured from the uplands area into the wetland. Subtract the total wetland area from the total project area, and add back the 100' wide strip of wetland area.

Compare the two numbers, and choose the one that yields the smallest project area, as shown in the following three examples. Please note that in each of these examples, the 100' wetland strip is measured from the 401/404 line waterward (into the wetlands) of the project for a maximum distance of 100', or to the mean high water (MHW) line or to the property line, whichever comes first. No credit for wetlands located outside of the property line or beyond the MHW line can be given. By definition, the project area includes only that area above the MHW Line.

#### EXAMPLE A

Total site area = 55 acres  
 Total wetland area = 15 acres  
 100' wide strip of wetland area = 5 acres

Method 1:     15 acres wetlands / 55 acres site = 27.3% (> 25%)  
                   55 acres site - 15 acres wetlands = 40 acres uplands

Project area for calculating density =  $40 / .75 = 53.33$  acres

Method 2:  $55 - 15 = 40$

Project area for calculating density =  $40 + 5 = 45$  acres

Since  $45 < 53.33$ , use area derived via Method 2 to calculate the allowable density.

### **EXAMPLE B**

Total site area = 55 acres

Total wetland area = 10 acres

100' wide strip of wetland area = 5 acres

Method 1:  $10 / 55 = 18.2\%$  (<25%)

Project area for calculating density = 55 acres

Method 2:  $55 - 10 = 45$  acres

Project area for calculating density =  $45 + 5 = 50$  acres

Since  $50 < 55$ , use area derived via Method 2 to calculate the allowable density.

### **EXAMPLE C**

Total Site Area = 55 acres

Total Wetland Area = 40 acres

100' strip of wetland area = 15 acres

Method 1:  $40 / 55 = 73\%$  (>25%)

$55 - 40 = 15$  acres

Project area for calculating density =  $15 / .75 = 20$  acres

Method 2:  $55 - 40 = 15$

Project area for calculating density =  $15 + 15 = 30$  acres

Since  $20 < 30$ , use area derived via Method 1 for calculating the allowable density.

September 28, 2007 Changes:

1. 3.2: Deleted a sentence (below the Rational Method) that led to confusion in unit conversions.
2. 3.2: Clarified guidance on using the Rational Method on drainage areas over 20 acres.
3. 3.3: Clarified that the calculated treatment volume shall be at least as large as the volume calculated using the Simple Method.
4. 3.3.1: Clarified that the variable, “Design storm depth” is typically 1.0” or 1.5”.
5. 3.3.2: Clarified that for Low Impact Development (LID) designs using the Discrete SCS Curve Number Method, the treatment volume is equal to the runoff depth multiplied by the drainage area.
6. 3.5.2: Changed the average drawdown head from  $H_o/2$  to  $H_o/3$  in, “Note: Usually use  $H_o/3$  to compute drawdown through an orifice to reflect the fact that head is decreasing as the drawdown occurs.”
7. 3.5.2: Corrected a typo in the orifice equation. “ $C_o$ ” now reads “ $C_D$ ”. (*Note: This change is not visible in Track Changes.*)
8. 3.7: Corrected a statement indicated that all channel side slopes were to be 3:1. Side slope requirements are typically 3:1, unless otherwise specified in the manual.

June 2009 Changes:

1. Added example calculations for projects in areas draining to SA waters, per 2008 Coastal SW Rules
2. Section 3.9.2 revised to clarify that nutrient loadings from multiple drainage areas may be weighted to meet an overall site nutrient removal loading requirement.
3. Wetlands Exclusion Policy included here