CHAPTER

3

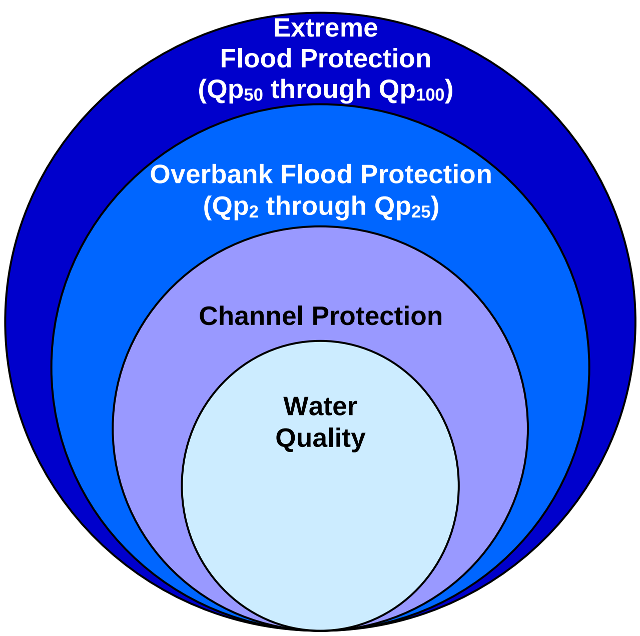
WATER QUALITY STANDARDS

3.1 Water Quality Protection Approach

This chapter represents policies, criteria and calculation methods for the design of the water quality best management practices (BMPs) presented in Chapter 4 of this manual. The design criteria presented herein communicate the regional approach to address the key adverse impacts of stormwater runoff from a development site presented in Chapter 1. The purpose of the design criteria is to provide a framework for design of the site's stormwater management system in order to remove stormwater runoff pollutants, improve water quality, and prevent downstream streambank and channel erosion. This chapter does not provide criteria and calculation guidance for stormwater quantity (e.g., hydraulic drainage design, detention/retention) design; please refer to the ordinances and other regulatory code of the local jurisdiction for stormwater quantity regulations.

While this manual does not address local stormwater quantity design requirements, site designers should note that design criteria for water quality, channel protection and stormwater quantity can often be blended together. This enables the sizing and design of structural stormwater controls in conjunction with each other to address the overall stormwater impacts from a development site. When stormwater design criteria are considered as a set, the site designer can control the range of design events, from the smallest amounts of runoff that are treated for water quality, to events requiring extreme flood protection, such as the 100-year storm. Figure 3-1 graphically illustrates the relative volume requirements of the various stormwater controls and demonstrates that, in some cases, the controls can be "nested" within one another (i.e., the extreme flood protection volume requirement also contains the overbank flood protection volume, the channel protection volume and the water quality treatment volume).

Figure 3-1. Integration of Stormwater Controls



3.2 General Policies

The following general policies shall apply to all water quality management and channel protection design calculations.

1. Design computations shall be performed in accordance with the calculation guidance provided in this manual, or other criteria that the local jurisdiction establishes based on scientific and engineering information.
2. Stormwater runoff resulting from post-development conditions must be routed at appropriately small time intervals through water quality BMPs, as appropriate, using either hand calculations or computer models that are widely accepted among engineering professionals.
3. All design computations utilized in the design of water quality BMPs must be prepared by a registered engineer or landscape architect proficient in the field of hydrology and hydraulics and licensed to practice in the State of Tennessee.

3.3 Water Quality Management

3.3.1 Minimum Standard and General Policies

Local ordinances require that stormwater runoff discharging from new development or redevelopment sites be treated to remove pollutants prior to discharge from the site. This requirement shall be implemented in accordance with the **Water Quality Minimum Treatment Standard** and associated policies presented in items 1 through 5 below. Policies that are specific to individual design calculations and/or BMPs are included later in this chapter.

1. Water quality BMPs shall be designed to remove, at a minimum, 80% of the average annual post-development total suspended solids (TSS) load from the stormwater volume required for water quality treatment, called the water quality treatment volume (WQv). This standard is also referred to in this manual as “the 80% TSS removal standard”.
2. WQv and % TSS removal shall be calculated for the development or redevelopment in accordance with the policies and calculation guidance provided in this chapter. In order to comply with the 80% TSS removal standard, the result of the % TSS removal calculations for the development or redevelopment must be equal to, or greater than, 80%.
3. It is presumed that a stormwater management system complies with the Water Quality Minimum Treatment Standard if structural BMPs are selected, designed, constructed and maintained in accordance with the design criteria specified in this manual. Only those BMPs that are published in Chapter 4 of this manual are permitted for use as a water quality BMPs. Other BMPs are prohibited, unless approved by the local jurisdiction. The structural BMPs deemed acceptable for use to attain the Water Quality Minimum Treatment Standard are listed in Table 3-1.
4. Table 3-1 also presents the % TSS removal value assigned to each BMP. This value shall be used to calculate the total weighted % TSS removal for the development site.
5. The local jurisdiction may require additional water quality treatment criteria or controls to conform to State and/or Federal regulatory requirements, and/or additional watershed or site-specific water quality requirements that are defined by the State or Federal officials, or the local jurisdiction. For example, additional treatment criteria may be required if, in the opinion of the local jurisdiction, the new development or redevelopment is considered a pollutant “hotspot”, where the land use or activities may generate highly contaminated runoff, with concentrations of pollutants in excess of those typically found in storm water. Examples of hot spot land uses might include operations producing concrete or asphalt, auto repair shops, auto supply shops, large commercial parking areas, restaurants.

Table 3-1. TSS Removal % for Structural BMPs

Structural BMP	TSS Removal %
General Application BMPs	
Wet Basin	80
Wet Extended Detention	80
Micropool Extended Detention Basin	80
Multiple Basin System	80
Dry Extended Detention Basin	60
Conventional Dry Detention Basins	10
Shallow Wetland	80
Extended Detention Shallow Wetland	80
Basin/Wetland System	80
Pocket Wetland	80
Bioretention Area	85
Sand Filters (Surface and Perimeter)	80
Infiltration Trench	90
WQ Dry Swales	90
Wet Swales	75
Filter Strip	50
Grass Channel[1]	30
Gravity (oil-grit) Separator	30
Modular Porous Paver Systems[2]	*
Porous Pavement/Concrete[2]	*
Limited Application BMPs	
Organic Filter	80
Underground Sand Filter	80
Submerged Gravel Wetland	75
Alum Treatment System	90
Manufactured BMPs	10[3]
Underground Detention	10

1 – Refers to open channel practice not designed for water quality.
2 – These practices are not treatment BMPs but are source control BMPs, so they are not assigned a pollutant removal.
3 – Provisional % TSS Removal value pending third party information. See Section 4.4.5 in Chapter 4 for policies for manufactured BMPs.

3.3.2 Calculation of % TSS Removal

The % TSS removal for the BMPs proposed for a new development or redevelopment must be calculated using the equations presented in this section.

3.3.2.1 Multiple BMPs

Equation 3-1 is an area-weighted TSS reduction equation that accounts for the TSS reduction that is contributed from each water quality BMP that is utilized on the site. This equation is applicable to those developments or redevelopments where multiple BMPs are used to treat the WQv. If only one BMP is utilized for WQv treatment, then the % TSS removal value is simply that assigned to the BMP (see Table 3-1). Equation 3-1 is applicable in situations where a site has multiple subwatersheds that flow to different BMPs, and none of the BMPs are placed downstream of another BMP.

Equation 3-1

$$\%TSS = \frac{\sum_{1}^{n}(TSS_1 A_1 + TSS_2 A_2 + \ldots + TSS_n A_n)}{\sum_{1}^{n}(A_1 + A_2 + \ldots + A_n)}$$

where:

TSS_n = TSS removal percentage for each structural BMP located on-site (%);
A_n = the area draining to each BMP (acres).

3.3.2.2 BMPs in Series

It will often be the case that the site designer will want to use two or more BMPs (structural and/or non-structural) in series, where stormwater treated in one BMP is discharged into another BMP for further treatment. Such BMP combinations are also called treatment trains. How and why BMPs might be used in treatment trains is discussed in Chapter 4 of this manual. This section presents the calculation of the total % TSS removal for treatment trains.

Equation 3-2 is used to calculate the total % TSS removal for a treatment train comprised of two or more structural BMPs.

Equation 3-2

$$TSS_{train} = TSS_A + TSS_B - \frac{(TSS_A \times TSS_B)}{100}$$

where:

TSS_{train} = total TSS removal for treatment train (%);
TSS_A = % TSS removal of the first (upstream) BMP, from Table 3-2 (%)
TSS_B = % TSS removal of the second (downstream) BMP, from Table 3-2 (%).

For development sites where the treatment train provides the only water quality treatment on the site, TSS_{train} must be greater than or equal to 80%. For development sites that have other structural BMPs for water quality treatment that are not included in the treatment train, TSS_{train} must be included in Equation 3-1 in the calculation of the overall % TSS removal for the site. An example application of the latter situation is presented below.

<u>Example 3-1. Calculation of %TSS for BMPs in Series</u>

A water quality management system located on a 30 acre development site consists of a dry extended detention basin, a water quality dry swale, and a shallow wetland. The extended detention basin and dry swale are located in series, with the basin as the upstream control. The treatment train treats stormwater runoff from 20 acres of the site. The shallow wetland treats 10 acres. All three facilities are designed in accordance with this manual. What is the % TSS removal rate for the site?
The % TSS removal value for each BMP located on the site is determined from Table 3-1, as follows:

Control A (dry extended detention basin) = 60% TSS removal

Control B (water quality dry swale) = 90% TSS removal
Control C (shallow wetland) = 80% TSS removal

Step 1: Calculate TSS_{train}:

TSS_{train} = A + B – (A x B)/100 = 60 + 90 - (60 X 90)/100 = 96% removal

Step 2: Calculate % TSS removal for the site:

%TSS = ((TSS_{train} x 20 acres) + (%$TSS_{wetland}$ x 10 acres)) ÷ 30 acres
%TSS = ((96% x 20 acres) + (80% x 10 acres)) ÷ 30 acres = 91%

Therefore, the % TSS removal for the site is 91%, which exceeds the minimum standard of 80% TSS removal. No other BMPs need to be constructed at the site.

3.3.2.3 Calculation of % TSS Removal for Flow-through Situations

BMPs within a treatment train may sometimes be separated by a contributing drainage area. In this case, equation 3-2 cannot be used, since some of the flow entering the downstream BMP has not been treated by the upstream BMP. This section presents the calculation of the total % TSS removal for flow-through situations.

To calculate the total % TSS removal for a treatment train separated by a contributing drainage area, Equation 3-3 shall be used.

Equation 3-3

$$TSS_{train} = \frac{TSS_A A_A + TSS_B A_B + \dfrac{TSS_B A_A (100 - TSS_A)}{100}}{A_A + A_B}$$

where:

TSS_{train} = total TSS removal for treatment train (%);
TSS_A = % TSS removal of the first (upstream) BMP, from Table 3-2 (%)
TSS_B = % TSS removal of the second (downstream) BMP, from Table 3-2 (%)
A_A = Area draining to BMP A
A_B = Area draining to BMP B.

For development sites where the treatment train provides the only stormwater treatment on the site, TSS_{train} must be greater than or equal to 80%. An example application of Equation 3-3 is shown below.

Example 3-2. Calculation of %TSS in a Flow-through Situation

A stormwater management system located on a 9 acre development site consists of a dry extended detention pond, and a bioretention cell. Five acres drain to the bioretention cell, which then drains to a pipe system. The pipe system also drains an additional 4 acres that have not been treated for water quality. The pipe system leads to a dry extended detention pond, that is used for final treatment. Both facilities are designed in accordance with the guidance in this manual. What is the % TSS removal rate for the site?

The % TSS removal value for each BMP located on the site is determined from Table 2-2, as follows:

Control A (bioretention cell) = 85% TSS removal
Control B (dry extended detention pond) = 60% TSS removal

Step 1: Calculate TSS_{train}:

$$TSS_{train} = \frac{TSS_A A_A + TSS_B A_B + \dfrac{TSS_B A_A (100 - TSS_A)}{100}}{A_A + A_B}$$

$$TSS_{train} = \frac{85x5 + 60x4 + \dfrac{60*5(100-85)}{100}}{5+4}$$

$$TSS_{train} = 78.9\%$$

The % TSS removal for the site is 78.9%, which is below the minimum standard of 80% TSS removal. The conversion of the stormwater pipe system to a grass swale would add additional pollutant removal and help the site meet the 80% criteria.

3.3.3 Calculation of the Water Quality Volume (WQv)

The calculation of % TSS removal tells the designer how well the water is treated. Next, the designer must consider how *much* water must be treated. The volume of water that must be treated to the 80% TSS removal standard is called the water quality volume (WQv). Compliance with the 80% TSS removal standard requires the calculation of the WQv for the entire development site. To obtain the lowest WQv for the site, this calculation should be performed ***after*** better site design practices that may be envisioned for the site have been considered and are included in design plans.

The WQv shall be calculated using Equation 3-4, as follows:

Equation 3-4

$$WQv = \frac{PRvA}{12}$$

where:

WQv = water quality volume of the site (acre-feet);
P = rainfall depth for the 85% storm event (1.04 inches);
Rv = runoff coefficient; and,
A = site area (acres).

The runoff coefficient (Rv) shall be calculated using Equation 3-5.

Equation 3-5

$$Rv = 0.015 + 0.0092I$$

where:

I = percent impervious area of the site (see Equation 3-6 below).

3.3.4 The Determination of Percent Imperviousness

Impervious areas are defined as impermeable surfaces which prevent the percolation of water into the soil. Impervious surfaces include, but are not limited to, paved surfaces such as walkways, sidewalks, patios, parking areas and driveways, packed gravel or soil, and structure rooftops. Other examples of impervious areas are paved recreation areas including pool houses and pool decks intended for use as a private (multi-family) or public recreation area, paved athletic courts (e.g., basketball, tennis), and storage buildings.

The percent impervious area (I) that is used to determine WQv is calculated using Equation 3-6.

Equation 3-6

$$I = \frac{I_A}{A} X 100\%$$

where:

I_A = cumulative area of all impervious surfaces on the site (acres);
A = site area (acres).

The determination of the impervious area (I_A) in order to calculate WQv shall be performed in the following manner:

1. For residential subdivisions that will be served by one or more water quality BMPs, I_A shall be determined using percent (%) impervious values that were developed by the Soil Conservation Service (SCS)[1]. Where the average lot size of a subdivision or a drainage area within the subdivision falls between the lot size categories shown in Table 3-2, the site designer may interpolate the % impervious value based on Table 3-2.

Table 3-2. % Impervious Area Values for Subdivisions

Residential Lot Size Range[1]	% Impervious
Less than ¼ acre	65
¼ acre	38
⅓ acre	30
½ acre	25
¾ acre	22.5[2]
1 acre	20
2 acres and greater	15

1 – Includes lots and streets. Common areas must be measured separately.
2 – The % impervious value is linearly interpolated from SCS data.

The values shown in Table 3-2 shall be utilized only for the portion of the subdivision that is covered by individual residential lots and streets. Other areas, such as common areas for recreation or meeting facilities, shall be added separately in the calculation of I_A. The calculation of the % impervious value for a residential subdivision having a common area is presented in Example 3-3.

If lot sizes within a single subdivision fall into more than one of the lot size ranges listed in Table 3-2, the site designer shall consider the total amount of imperviousness in each lot range separately in the determination of the percent impervious value. Example 3-3 includes the calculation of the % impervious value for a residential subdivision having variable lot sizes.

[1] The Soil Conservation Service is now known as the Natural Resource Conservation Service (NRCS).

2. For planned unit developments where the building and paving footprints are known, as well as all nonresidential developments, I_A shall be determined from the measured impervious footprints for all impervious areas as defined above. It is required that the footprint for all impervious surfaces in the proposed development and the calculation of I_A be shown in the stormwater management plan.

After the development and/or redevelopment of the property is complete, property improvement activities that do not require the submittal of a water quality management plan will not require recalculation of the impervious percentage and WQv.

Example 3-3. Calculation of Percent Impervious Area (I)

A site design engineer is preparing a water quality management plan for a proposed residential development. The subdivision has a total area of 31 acres, and will include 52 residential lots ranging in area from approximately ¼ acre to no greater than 1 acre (as shown in the table below). Three (3) acres will be preserved as an undisturbed forested vegetated buffer located along a stream that crosses the property, and therefore, there is no impervious coverage within these three acres. Another three (3) acres will be utilized for a recreational common area which includes a community pool, tennis courts and an associated parking lot. Due to local topography on the site, the subdivision drains to two separate water quality management facilities, herein called Facility A and Facility B, both of which provide water quality treatment. Twelve acres, including the 3 acre vegetated buffer and 3 acre common area, drain to Facility A. The other 19 acres drain to Facility B. The following table provides lot size, area and impervious data for the proposed subdivision. What is the % impervious area for the site?

A Lot Size	B Number of Lots in Size Range	C Sub-total Area of Lots in Size Range	D % Impervious (from Table 3-2)
DRAINAGE AREA A (AREA DRAINING TO FACILITY A)			
approx. ⅓ acre	0	0 acres	30
approx. ½ acre	0	0 acres	25
approx. ¾ acre	2	1.3 acres	22.5
approx. 1 acre	5	4.7 acres	20
Area A Totals	7 lots	6.0 acres	--
DRAINAGE AREA B (AREA DRAINING TO FACILITY B)			
approx. ⅓ acre	21	6.6 acres	30
approx. ½ acre	16	7.3 acres	25
approx. ¾ acre	7	4.3 acres	22.5
approx. 1 acre	1	0.8 acres	20
Area B Totals	45 lots	19.0 acres	--

Since the site will be served by two separate detention facilities, it is best to determine the impervious area for each drainage area, rather than the overall impervious area for the site. For ease in calculation, the site design engineer decided not to interpolate impervious area values, preferring to group lots into approximate lot sizes that correspond to Table 3-2.

Step 1: Determine the total impervious area for the portion of each drainage area that is covered by residential lots and associated subdivision roads ($I_{\text{residential areas}}$):

This is calculated by multiplying the sub-total area of each lot size range (column C from the above table) by the corresponding % impervious in that lot size range (column D from the above table). Results of this calculation are shown in the table below.

A Lot Size	B Sub-total Area of Lots in Size Range	C % Impervious (from Table 3-2)	D Sub-total Impervious Area
DRAINAGE AREA A (AREA DRAINING TO FACILITY A)			
approx. ⅓ acre	0 acres	30	0 x 0.30 = 0 ac
approx. ½ acre	0 acres	25	0 x 0.25 = 0 ac
approx. ¾ acre	1.3 acres	22.5	1.3 x 0.225 = 0.29 ac
approx. 1 acre	4.7 acres	20	4.7 x 0.20 = 0.94 ac
Area A Totals	6.0 acres	--	1.23 acres

DRAINAGE AREA B (AREA DRAINING TO FACILITY B)			
approx. ⅓ acre	6.6 acres	30	6.6 x 0.30 = 1.98 ac
approx. ½ acre	7.3 acres	25	7.5 x 0.25 = 1.88 ac
approx. ¾ acre	4.3 acres	22.5	4.3 x 0.225 = 0.97 ac
approx. 1 acre	0.8 acres	20	0.8 x 0.20 = 0.16 ac
Area B Totals	19.0 acres	--	4.99 acres

Thus, the portions of the site where residential lots are located are covered by impervious surfaces as follows:
$I_{A\ \text{residential areas}}$ = 1.23 acres
$I_{B\ \text{residential areas}}$ = 4.99 acres

<u>Step 2:</u> Measure the area of impervious footprints in the common areas that are located in Area A ($I_{A\ \text{common areas}}$):

The following table presents the measurements of the impervious areas located in the common area.

Area Description	Impervious Area
Community pool (includes pool, surrounding deck, maintenance building and sidewalk from parking lot)	0.8 acres
Tennis court (includes two courts, surrounding paved areas, and sidewalk from parking lot)	1.2 acres
Common area driveway and parking lot	0.7 acres
Total impervious areas	2.7 acres

Thus, 2.7 acres of the 3 acre common area, located in Area A, is covered by impervious surfaces. $I_{A\ \text{common areas}}$ = 2.7 acres

<u>Step 3:</u> Calculate the % impervious area (I) for each drainage area of the site using Equation 3-6. Because the vegetated buffer is entirely undisturbed, and therefore entirely pervious, it is not considered in the calculation.

For Area A:

$I_A = ((I_{A\ \text{residential areas}} + I_{A\ \text{common areas}}) \div 12 \text{ acres}) \times 100\%$
$I_A = ((1.23 \text{ acres} + 2.7 \text{ acres}) \div 12 \text{ acres}) \times 100\%$
$I_A = (3.9 \text{ acres} \div 12 \text{ acres}) \times 100\%$
$I_A = 32.8\%$

For Area B:

$I_B = (I_{B\ \text{residential areas}} \div 19 \text{ acres}) \times 100\%$
$I_B = (4.99 \text{ acres} \div 19 \text{ acres}) \times 100\%$
$I_B = 26.3\%$

Therefore, the % impervious area for Area A (I_A) for the site is 32.8%. The % impervious area for Area B (I_B) is 26.3%.

3.3.5 Reducing the WQv

It is important to remember that the WQv is proportional to impervious area, such that the amount of stormwater runoff requiring treatment increases as impervious area increases. In other words, the more you pave, the more you treat. Therefore, to reduce the amount of stormwater runoff that must be treated, the developer must find ways to reduce site imperviousness. Reductions in imperviousness are beneficial from a water quality management standpoint. Decreases in impervious area equate to less runoff, lower post-development peak discharges, and typically lower pollutant discharges. This can result in lower water quality management costs, as structural BMPs, channel protection, and flooding protection controls can be smaller in size.

In order to reduce the WQv for a development site, site designers are encouraged to use better site design practices. Better site design can be defined as a combination of non-structural design approaches intended to reduce the impacts of stormwater runoff from development through the conservation of natural areas, reduction of impervious areas, and integration of non-structural water quality BMPs. Such practices are often collectively referred to as “non-structural practices or non-structural BMPs”. By implementing a combination of these non-structural approaches, it is possible to reduce the amount of runoff and pollutants that are generated from a site and provide for some non-structural on-site treatment and control of runoff.

The use of better site design practices on a development or redevelopment site to attain the 80% TSS removal standard is not required. A strong incentive for the use of such practices is provided via the WQv method (since it is proportional to impervious area) and through prescribed WQv reductions for the use of specific better site design practices. The WQv reductions are listed in Table 3-3 on the following page. Check with the local jurisdiction to determine which of these reductions are available for use in that jurisdiction. Detailed policies and design requirements for reductions and better site design practices are presented in Chapter 5 of this manual.

3.3.6 The Design of Outlets Used for Extended Detention

Once the WQv has been determined, the volume must be treated to the 80% TSS removal standard through the use of the BMPs found in Chapter 4. Several of the BMPs achieve TSS removal through extended detention (ED). Therefore, ED orifice sizing is required for these BMPs. For a structural control facility that will provide both WQv extended detention and channel protection volume control (to be discussed in section 3.4) (wet ED pond, micropool ED pond, and shallow ED wetland), there will be a need to design <u>two</u> outlet orifices. The water quality control outlet will be sized using drawdown time principles described below. The minimum standard for the channel protection and the sizing of the channel protection outlet is discussed in detail in section 3.4.

Table 3-3. Summary of WQv Reductions for Better Site Design

WQv Reduction	Description
Reduction 1: Natural area preservation	Undisturbed natural areas are conserved, thereby retaining the pre-development hydrologic and water quality characteristics.
Reduction 2: Managed area preservation	Managed areas of open space are preserved, reducing total site runoff and retaining near pre-development hydrologic and water quality characteristics.
Reduction 3: Stream and vegetated buffers	Stormwater runoff is treated by directing sheet flow runoff through a naturally vegetated or forested buffer as overland flow.
Reduction 4: Vegetated channels	Vegetated channels are used to provide water quality treatment.
Reduction 5: Impervious area disconnection	Overland flow filtration/infiltration zones are incorporated into the site design to receive runoff from rooftops and other small impervious areas.
Reduction 6: Environmentally sensitive large lot neighborhood	A group of site design techniques are applied to low and very low density residential development.

(The following procedures are based on the water quality outlet design procedures included in the Virginia Stormwater Management Handbook, 1999)

In an extended detention facility for water quality treatment, the storage volume is detained and released over a specified amount of time (e.g., no less than 24-hours). The release period is a brim drawdown time, with the assumption that the entire WQv is present in the basin at the beginning of drawdown. The entire calculated volume drains out of the basin over no less than 24 hours. In reality, however, water is flowing out of the basin prior to the full or brim volume being reached. Therefore, the extended detention outlet can be sized using either of the following two methods:

1. Use the maximum hydraulic head associated with the storage volume and maximum flow, and approximate the orifice size needed to achieve the required drawdown time. This procedure is outlined in Example 3-5.
2. Use a drawdown analysis to determine the drawdown time.

This is a accurate method for determining orifice sizes. Example 3-5 illustrates this method.

Example 3-4. ED Outlet Design Method 1: Maximum Hydraulic Head

A wet ED pond sized for the required water quality volume will be used here to illustrate the sizing procedure for an extended-detention orifice. Given the following information, calculate the required orifice size for water quality design.

- Water Quality Volume (WQv) = 0.76 ac-ft = 33,106 ft^3
- Maximum Hydraulic Head (H_{max}) = 5.0 ft (from stage vs. storage data)

Step 1. Determine the maximum discharge resulting from the 24-hour drawdown requirement. It is calculated by dividing the WQv by the required time to find the average discharge, and then multiplying by two to obtain the maximum discharge.

Q_{avg} = 33,106ft^3/(24hr)(3,600sec/hr) = 0.38 cfs
Q_{max} = 2Q_{avg} = 0.76 cfs

Step 2. Determine the required orifice diameter by using the standard orifice equation and Q_{max} and H_{max}:
Q = CA(2gH)$^{0.5}$, or A = Q/C(2gh)$^{0.5}$
A = 0.76/0.6[(2)(32.2)(5.0)]$^{0.5}$ = 0.071 ft^2

Step 3. Determine pipe diameter
A = 3.14d^2/4, then d = (4A/3.14)$^{0.5}$
D = [4(0.071)/3.14]$^{0.5}$ = 0.30 ft = 3.61 inches

Therefore, use a 3.6-inch diameter water quality orifice.

Example 3-5. ED Outlet Design Method 2: Drawdown Analysis

Using the data from the previous example (Example 3-4) use Method 2 to calculate the size of the outlet orifice. Use of a spreadsheet is highly recommended.

- Water Quality Volume (WQv) = 0.76 ac-ft = 33,106 ft^3
- Maximum Hydraulic Head (H_{max}) = 5.0 ft (from stage vs storage data)

Step 1. Determine the pond stage-storage curve at increments of 0.1' or less.

Step 2. Choose pond water elevation (first increment at H_{max}, others at end elevation of previous increment).

Step 3. Assume an orifice size:
Orifice diameter = 1"
Orifice area = (π/4)*(Diam/12)^2
Orifice area = (3.14/4)*(1/12)^2 = 0.00545 ft^2

Step 4. Calculate flowrate at water surface elevation using orifice equation:
Q = CA(2gH)$^{0.5}$
Q = 0.6*0.00545*(2*32.2*5)$^{0.5}$
Q = 0.0587 cfs

Step 5. Calculate time to drain pond volume increment (keeping track of elapsed time):
Time = Volume/Flowrate (Volume of increment from stage-storage curve)
Time = 200/.0587 = 3407 seconds = 56.8 minutes

Step 6. Repeat steps 1 through 5 for each elevation from WQv elevation to orifice center (keeping track of elapsed time).

Step 7. Check whether total drawdown time is greater than 24-hours:

3.3.7 Calculating the Water Quality Peak Discharge

The peak rate of discharge for the water quality design storm (Q_{wq}), also called the water quality peak discharge, is needed to size water quality BMPs that are located off-line, such as sand filters and infiltration trenches. See Chapter 4 of this manual for more information on off-line (versus on-line) BMPs.

This method is utilized for the sizing of water quality treatment controls. More traditional peak discharge calculation methods are not appropriate for this application for a variety of reasons. First, the use of more traditional methods, such as the Rational Method would require the choosing of an arbitrary design storm event that will differ from the 85th percentile storm event that must be treated for water quality. Further, conventional SCS methods have been found to underestimate the volume and rate of runoff for rainfall events of less than two inches. This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff bypasses the structural control due to an inadequately sized diversion structure and leads to the design of undersized bypass channels.

The method employed to calculate the water quality peak discharge uses the runoff coefficient to find the depth of runoff for the water quality storm of 1.04 inches. The SCS method is then used to find a unit peak discharge that is combined with the runoff depth to find a peak runoff rate.

The following procedure can be used to calculate Q_{wq}. This procedure relies on the Rv and the simplified peak discharge calculation:

1. Utilize Equation 3-7 to calculate D_{wq}.

Equation 3-7

$$D_{wq} = 1.04Rv$$

where:
D_{wq} = water quality runoff depth, in inches
Rv = runoff coefficient (see Equation 3-5)

2. A runoff curve number (CN) can be estimated using the standard SCS Runoff Curve Number estimation technique, or can be computed utilizing Equation 3-8 (Pitt, 1994).

Equation 3-8

$$CN = \frac{1000}{10 + 5P + 10D_{wq} - 10\left(D_{wq}^{\ 2} + 1.25 D_{wq} P\right)^{1/2}}$$

where:
CN = runoff curve number
P = the 85th percentile rainfall, in inches (use 1.04 inches)
D_{wq} = water quality runoff depth, in inches (see Equation 3-7)

3. Determine the initial abstraction (I_a) from Table 3-4, and the ratio I_a/P is then computed (P = 1.04 inches).

4. Compute the drainage area time of concentration (t_c) for the post-development land use with standard SCS methods.

5. The time of concentration is used with the ratio I_a/P to obtain the unit peak discharge, q_u, from Figure 3-2 for the Type II rainfall distribution. If the ratio I_a/P lies outside the range shown in the figure, use the limiting values.

6. The water quality peak discharge (Q_{wq}) is computed using Equation 3-9.

Equation 3-9 $$Q_{wq} = q_u A D_{wq}$$

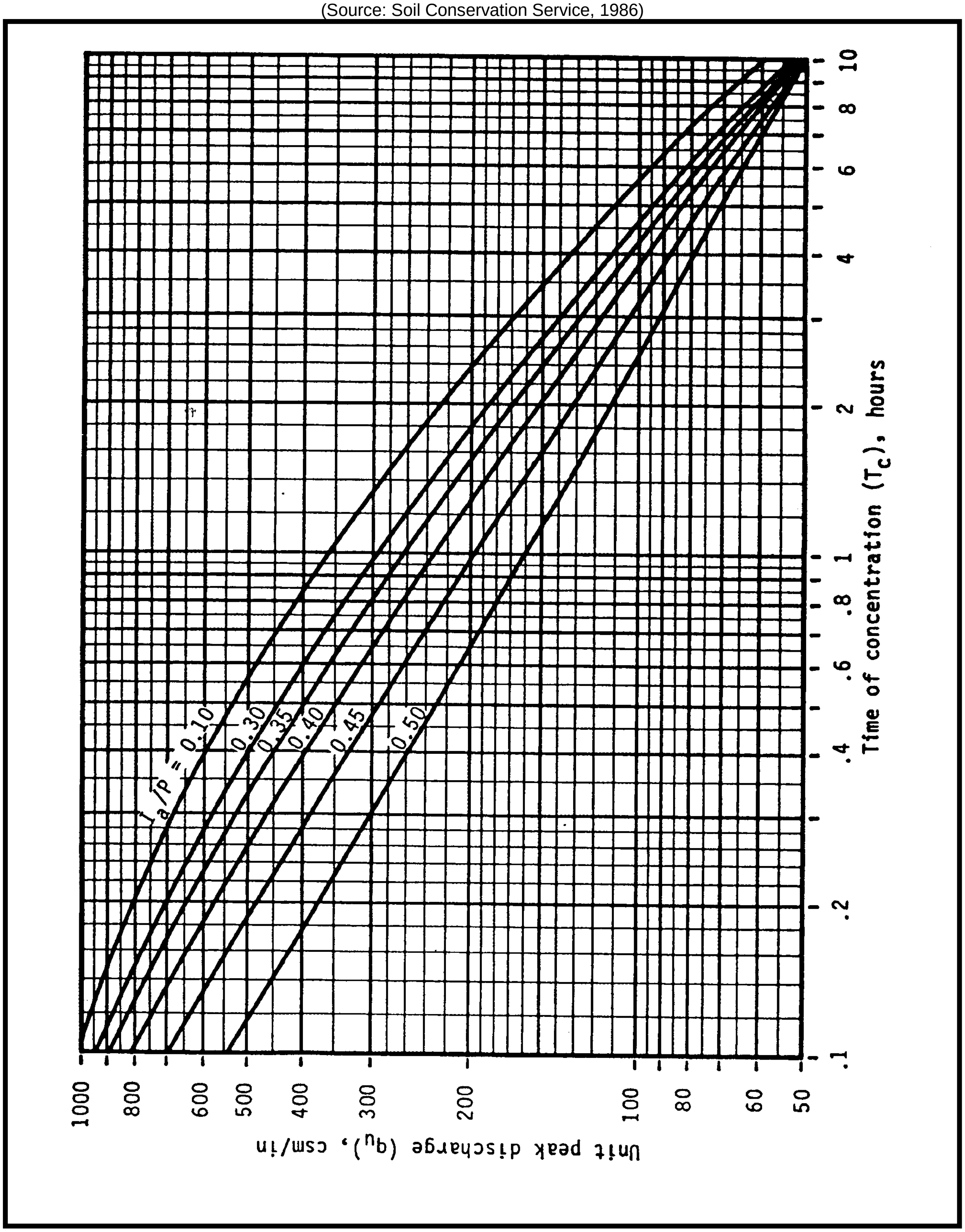
where:

Q_{wq} = the water quality peak discharge (cfs)
q_u = the unit peak discharge (cfs/mi²/inch)
A = drainage area (mi^2)
D_{wq} = water quality runoff depth, in inches (see Equation 3-7)

Table 3-4. Initial Abstraction (I_a) for Runoff Curve Numbers
(Source: SCS, TR-55, Second Edition, June 1986)

Curve Number	I_a (in)	Curve Number	I_a (in)
40	3.000	70	0.857
41	2.878	71	0.817
42	2.762	72	0.778
43	2.651	73	0.740
44	2.545	74	0.703
45	2.444	75	0.667
46	2.348	76	0.632
47	2.255	77	0.597
48	2.167	78	0.564
49	2.082	79	0.532
50	2.000	80	0.500
51	1.922	81	0.469
52	1.846	82	0.439
53	1.774	83	0.410
54	1.704	84	0.381
55	1.636	85	0.353
56	1.571	86	0.326
57	1.509	87	0.299
58	1.448	88	0.273
59	1.390	89	0.247
60	1.333	90	0.222
61	1.279	91	0.198
62	1.226	92	0.174
63	1.175	93	0.151
64	1.125	94	0.128
65	1.077	95	0.105
66	1.030	96	0.083
67	0.985	97	0.062
68	0.941	98	0.041
69	0.899	-	

Figure 3-2. SCS Type II Unit Peak Discharge Graph
(Source: Soil Conservation Service, 1986)



An example illustrating calculation of the water quality peak flow is given below.

Example 3-6. Calculation of Water Quality Peak Flow

For a 50 acre site, with 18 impervious acres.

Step 1: Compute volumetric runoff coefficient, Rv using Equation 3-5:

$Rv = 0.015+(0.0092)(I) = 0.015+(0.00920(18/50)(100) = 0.35$

Step 2. Compute depth of runoff that must be treated for water quality, D_{wq} using equation 3-7:

$D_{wq} = 1.04Rv = 1.04(0.35) = 0.36$ inches

Step 3: Compute the synthetic curve number (CN) using Equation 3-8:

$CN = 1000/[10 + 5(1.04) + 10(0.36) – 10[(0.36)^2 +1.25(0.36)(1.04)]^{0.5}= 90$

Step 4: Find Ia from CN with Table 3-4:

$Ia = 0.22$ inches
$Ia/P = 0.22/1.04 = 0.21$

Step 5: Compute time of concentration, Tc: using SCS standard methods

Tc computed as 0.35 hours.

Step 6: Find q_u, using $Tc = 0.35$ and $I_a/P = 0.21$ using Figure 3-2:

$q_u = 580$ cfs/mi²/in

Step 7: Compute water quality peak flow rate using Equation 3-9.

$Q_{wq} = 580(50/640)(0.36)(1) = 16.3$ cfs

3.3.8 Water Balance Calculations

Water balance calculations can help to determine if a drainage area is large enough or has the right characteristics to support a permanent pool of water during average or extreme conditions. When in doubt, a water balance calculation may be advisable for retention pond and wetland design.

The details of a rigorous water balance are beyond the scope of this manual. However, a simplified procedure is described herein that will provide an estimate of pool viability and point to the need for more rigorous analysis. Water balance can also be used to help establish planting zones in a wetland design.

3.3.8.1 Basic Equations

Water balance is defined as the change in volume of the permanent pool resulting from the total inflow minus the total outflow (actual or potential). Equation 3-10 presents this calculation.

Equation 3-10 $$\Delta V = \sum I - \sum O$$

where:

- Δ = delta or "change in"
- V = basin volume (ac-ft)
- Σ = "the sum of"
- I = Inflows (ac-ft)
- O = Outflows (ac-ft)

The inflows consist of rainfall, runoff and baseflow into the basin. The outflows consist of infiltration, evaporation, evapotranspiration, and surface overflow out of the basin or wetland. Equation 3-10 can be expanded to reflect these factors, as shown in Equation 3-11. Key variables in Equation 3-11 are discussed in detail below the equation.

Equation 3-11 $$\Delta V = PA + R_o + Bf - ID - EA - EtA - Of$$

where:

- P = precipitation (ft)
- A = area of basin (ac)
- R_o = runoff (ac-ft)
- B_f = baseflow (ac-ft)
- I = infiltration (ac-ftt)
- E = evaporation (ft)
- Et = evapotranspiration (ft)
- Of = overflow (ac-ft)
- D = number of days in a given month

Rainfall (P) – Monthly rainfall values can be obtained from the National Weather Service climatology at *http://www.srh.noaa.gov/mrx/climat.htm.* Monthly values are commonly used for calculations of values over a season. Rainfall is then the direct amount that falls on the basin surface for the period in question. When multiplied by the basin surface area (in acres) it becomes acre-feet of volume. Table 3-5 presents average monthly rainfall values for northeast Tennessee based on a 30-year period of record.

Table 3-5. Average Rainfall Values in Feet for the Tri-Cities

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
P (feet)	0.29	0.28	0.33	0.27	0.36	0.32	0.35	0.25	0.26	0.19	0.26	0.28
Annual Precipitation 3.44												

Source: www.ncdc.noaa.gov/oa/climate/online/ccd/nrmpcp.txt

Runoff (R_o) – Runoff is equivalent to the rainfall for the period times the "efficiency" of the watershed, which is equal to the ratio of runoff to rainfall (Q/P). In lieu of gage information, Q/P can be estimated one of several ways. The best method would be to perform long-term simulation modeling using rainfall records and a watershed model.

Equation 3-12 gives a ratio of runoff to rainfall volume for a particular storm. If it can be assumed that the average storm that produces runoff has a similar ratio, then the Rv value can serve as the ratio of rainfall to runoff. Not all storms produce runoff in an urban setting. Typical initial losses (often called "initial abstractions") are normally taken between 0.1 and 0.2 inches. When compared to the rainfall records in northeast Tennessee, this is equivalent to about a 10% runoff volume loss. Thus, in a water balance calculation, a factor of 0.9 should be applied to the calculated Rv value to account for storms that produce no runoff. Equation 3-13 reflects this approach. Total runoff volume is then simply the product of runoff depth (Q) times the drainage area (A) to the basin, as shown in equation 3-12.

Equation 3-12 $$R_o = QxA$$

where:

R_o = total runoff volume
Q = runoff depth (ft)
A = basin area (ft^2)

Equation 3-13 $$Q = 0.9PRv$$

where:

Q = runoff depth (ft)
P = precipitation (ft)
Rv = volumetric runoff coefficient (Equation 3-5)

<u>Baseflow (B_f)</u> – Most water quality basins and wetlands have little, if any, baseflow, as they are rarely placed across perennial streams. If so placed, baseflow must be estimated from observation or through theoretical estimates. Methods of estimation and baseflow separation can be found in most hydrology textbooks.

<u>Infiltration (I)</u> – Infiltration is a very complex subject and cannot be covered in detail here. The amount of infiltration depends on soils, water table depth, rock layers, surface disturbance, the presence or absence of a liner in the basin, and other factors. The infiltration rate is governed by the Darcy equation, shown in Equation 3-14.

Equation 3-14 $$I = Ak_hG_h$$

where:

I = infiltration (ac-ft/day)
A = cross sectional area through which the water infiltrates (ac)
k_h = saturated hydraulic conductivity or infiltration rate (ft/day)
G_h = hydraulic gradient = pressure head/distance

G_h can be set equal to 1.0 for basin bottoms and 0.5 for basin sides steeper than about 4:1. Infiltration rate can be established through testing, though not always accurately. Table 3-6 can be used for initial estimation of the saturated hydraulic conductivity.

Table 3-6. Saturated Hydraulic Conductivity
(Source: Ferguson and Debo, 1990)

Material	Hydraulic Conductivity Kh	
	in/hr	ft/day
ASTM Crushed Stone No. 3	50,000	100,000
ASTM Crushed Stone No. 4	40,000	80,000
ASTM Crushed Stone No. 5	25,000	50,000
ASTM Crushed Stone No. 6	15,000	30,000
Sand	8.27	16.54
Loamy sand	2.41	4.82
Sandy loam	1.02	2.04
Loam	0.52	1.04
Silt loam	0.27	0.54
Sandy clay loam	0.17	0.34
Clay loam	0.09	0.18
Silty clay loam	0.06	0.12
Sandy clay	0.05	0.10
Silty clay	0.04	0.08

Material	Hydraulic Conductivity Kh	
	in/hr	ft/day
Clay	0.02	0.04

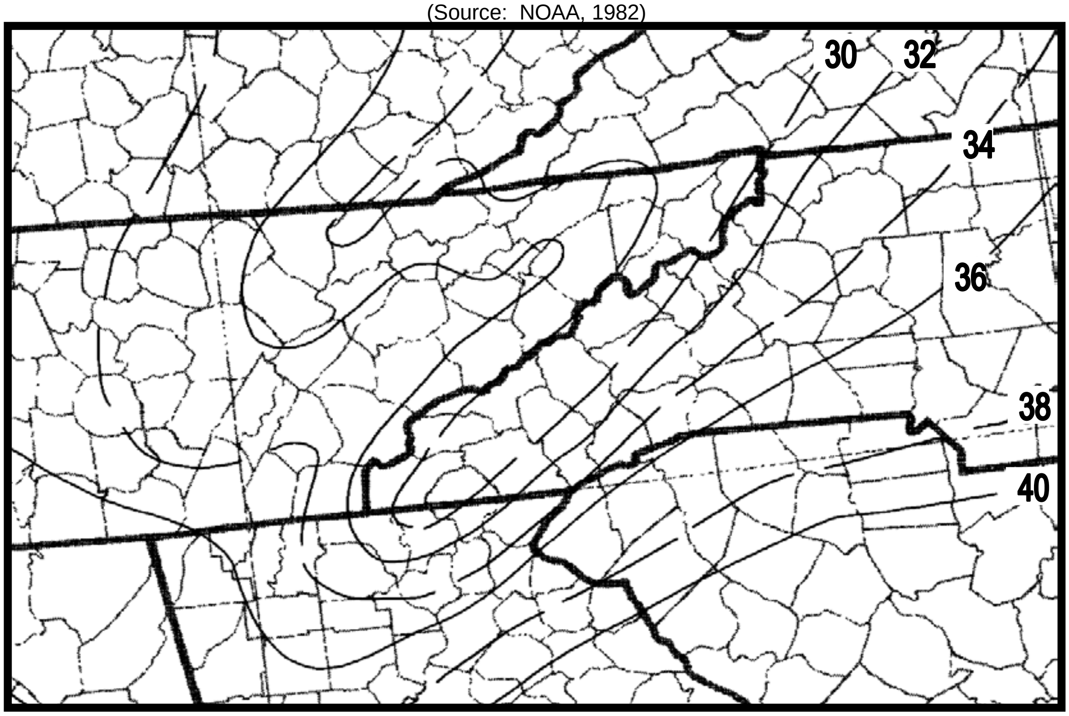
Evaporation (E) – Evaporation is from an open lake water surface. Evaporation rates are dependent on differences in vapor pressure, which, in turn, depend on temperature, wind, atmospheric pressure, water purity, and shape and depth of the basin. It is estimated or measured in a number of ways, which can be found in most hydrology textbooks. Pan evaporation methods are also used.

Table 3-7 presents pan evaporation rate distributions for a typical 12-month period based on pan evaporation information from one station in Bristol, TN. Figure 3-3 depicts a map of annual free water surface (FWS) evaporation averages for Tennessee based on a National Oceanic and Atmospheric Administration (NOAA) assessment done in 1982. FWS evaporation differs from lake evaporation for larger and deeper lakes, but can be used as an estimate of it for the type of structural water quality basins and wetlands being designed in northeast Tennessee. Total annual values can be estimated from this map and distributed in accordance with the percentages presented in Table 3-7.

Table 3-7. Pan Evaporation Rates - Monthly Distribution

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
3.1%	4.0%	7.1%	10.0%	11.9%	12.8%	12.7%	12.0%	10.4%	8.1%	4.6%	3.2%

Figure 3-3. Average Annual Free Water Surface Evaporation (in inches)
(Source: NOAA, 1982)



Evapotranspiration (E_t). Evapotranspiration consists of the combination of evaporation and transpiration by plants. The estimation of E_t for crops is well documented and has become standard practice. However, the estimating methods for wetlands are not documented, nor are there consistent studies to assist the designer in estimating the wetland plant demand on water volumes. Literature values for various places in the United States vary around the free water surface lake evaporation values. Estimating E_t only becomes important when wetlands are being designed and emergent vegetation covers a significant portion of the basin surface. In these cases conservative estimates of lake evaporation should be compared to crop-based E_t estimates and a decision made. Crop-based E_t estimates can be obtained from typical hydrology textbooks or from the web sites mentioned above. A value of zero shall be assumed for E_t unless the wetland design dictates otherwise.

Overflow (O_f) – Overflow is considered as excess runoff, and in water balance design is either not considered since the concern is for average precipitation values, or is considered lost for all volumes above the maximum basin storage. Obviously, for long-term simulations of rainfall-runoff, large storms would play an important part in basin design.

Example 3-7. Water Balance Calculation for Basin

Bristol Farms, a 26-acre site in Bristol, is being developed along with an estimated 0.5-acre surface area basin. There is no baseflow. The desired basin volume to the overflow point is 2 acre-feet. Will the site be able to support the basin volume? From the basic site data we find that the site is 75% impervious with sandy clay loam soil.

Step 1: From Equation 3-5, Rv = 0.015 + 0.0092(75) = 0.71. With the correction factor of 0.9 the watershed efficiency is 0.64.
The annual lake evaporation from Figure 3-3 is about 30 inches.
For a sandy clay loam the infiltration rate is Kh = 0.34 ft/day (Table 3-6).
From a grading plan, it is known that 10% of the total basin area is sloped greater than 4:1.
Monthly rainfall for the local area was found from the website provided above.

Step 2: The table below shows summary calculations for this site for each month of the year.

	Value	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	Days per Month	31	28	31	30	31	30	31	31	30	31	30	31
2	Precip. (in)	3.52	3.4	3.91	3.23	4.32	3.89	4.21	3	3.08	2.3	3.08	3.39
3	Evap. Dist. (%)	3.1	4	7.1	10	11.9	12.8	12.7	12	10.4	8.1	4.6	3.2
4	R_0 (ac-ft)	4.88	4.71	5.42	4.48	5.99	5.39	5.84	4.16	4.27	3.19	4.27	4.70
5	P (ac-ft)	0.15	0.14	0.16	0.13	0.18	0.16	0.18	0.13	0.13	0.10	0.13	0.14
6	E (ac-ft)	0.04	0.05	0.09	0.13	0.15	0.16	0.16	0.15	0.13	0.10	0.06	0.04
7	I (ac-ft)	5.01	4.52	5.01	4.85	5.01	4.85	5.01	5.01	4.85	5.01	4.85	5.01
8	Bal. (ac-ft)	-0.02	0.28	0.48	-0.37	1.01	0.54	0.85	-0.87	-0.58	-1.82	-0.51	-0.21
9	Run. Bal. (ac-ft)	0.00	0.28	0.76	0.39	1.40	1.94	2.00	1.13	0.55	0.00	0.00	0.00

Explanation of Table:

1. Days per month
2. Monthly precipitation from website is shown in Table 3-5.

3. Distribution of evaporation by month from Table 3-7.
4. Watershed efficiency of 0.64 x rainfall multiplied x site area and converted to ac-ft.
5. Precipitation volume directly into basin equals precipitation depth times basin surface area Pv=PA.
6. Evaporation volume equals percent evaporation by month (line 3) times 2.5 feet (Figure 3-3 converted to feet) multiplied by pond area (AC).
7. Infiltration volume equals the hydraulic conductivity (Table 36) times the pond area multiplied by the composite hydraulic gradient for the pond times the number of days in the month. I_v = I (days per month).
8. Balance is Lines (4 + 5) minus lines (6 + 7).
9. Running Balance is accumulated total from line 8 keeping in mind that all volume above 2 acre-feet overflows and is lost in the trial design.

It can be seen that for this example the basin has potential to go dry in late fall. This can be remedied in a number of ways including compaction of the basin bottom, placement of a clay or geosynthetic liner, and modification of the basin geometry to decrease the surface area.

3.4 Channel Protection

3.4.1 Minimum Standard

Local ordinances require adherence to the channel protection standard for applicable new development or redevelopments prior to discharge from the site. This requirement shall be implemented in accordance with the **Channel Protection Standard** and associated policies presented in items 1 and 2 below.

1. The runoff volume from the 1-year frequency, 24-hour storm, herein called the Channel Protection Volume (CPv), shall be captured and discharged over no less than a 24-hour period utilizing the design criteria and guidance provided in this manual. In the design of the channel protection control, the 24-hour release period shall be measured from the approximate center-of-mass of inflow to the approximate center-of-mass of outflow.
2. The local jurisdiction may approve downstream channel protection provided by an alternative approach than that stated above if sufficient hydrologic and hydraulic analysis shows that the alternative approach will offer adequate channel protection from erosion.

3.4.2 Estimation of the Channel Protection Volume

The Simplified SCS Peak Runoff Rate Calculation approach can be used for estimation of the channel protection volume (CPv) prior to storage facility design. For the calculation of CPv, this approach must be modified to determine the volume for a 1-year frequency, 24-hour duration design storm event. The calculation procedure is as follows.

Step 1. The 1-year, 24-hour rainfall depth (P, in inches) is determined for the selected location. Consult your local jurisdiction to determine the amount of rainfall to utilize for this calculation.

Step 2. A runoff curve number (CN) is then estimated using standard SCS Runoff Curve Number estimation techniques.

Step 3. The CN value is used to determine the initial abstraction (I_a) from Table 3-4, and the ratio I_a/P is computed.

Step 4. The accumulated runoff (Q_d, inches) can then be calculated using the SCS method.

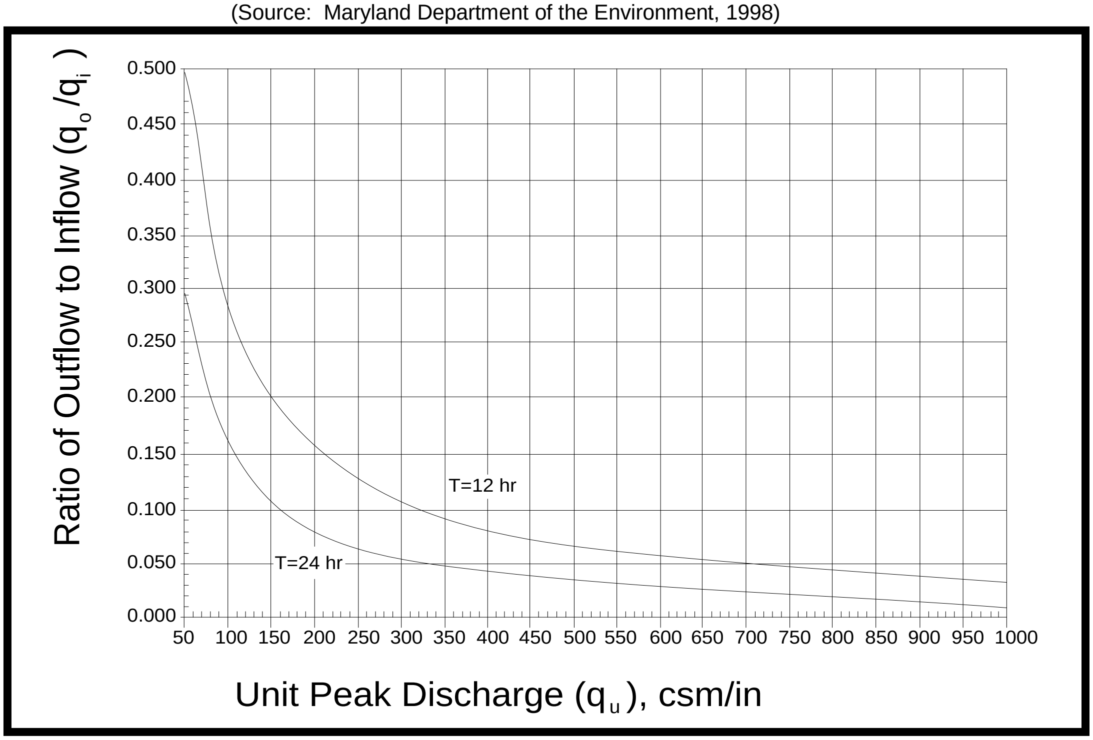
$$Q_d = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad I_a = 0.2S \qquad S = \frac{100}{CN} - 10$$

Step 5. Compute the drainage area time of concentration (t_c) for the post-development land use using standard SCS methods.

Step 6. Use t_c with the ratio I_a/P to obtain the unit peak discharge, q_u, from Figure 3-2 for the Type II rainfall distribution. If the ratio I_a/P lies outside the range shown in the figure, either use the limiting values or use another peak discharge method.

Step 7. Knowing q_u and T (extended detention time, minimum of 24 hours and maximum of 72 hours); the q_o/q_i ratio (peak outflow discharge/peak inflow discharge) can be estimated from Figure 3-4.

Figure 3-4. Detention Time vs. Discharge Ratios
(Source: Maryland Department of the Environment, 1998)



Step 8. V_s/V_r is then determined using the SCS detention basin routing formula of Equation 3-14 or using Figure 3-5. Equation 3-15 is suspect when the expression q_o/q_i approaches the limits of 0.1 and 0.8.

Equation 3-15

$$\frac{V_s}{V_r} = 0.682 - 1.43\left(\frac{q_o}{q_i}\right) + 1.64\left(\frac{q_o}{q_i}\right)^2 - 0.804\left(\frac{q_o}{q_i}\right)^3$$

where:

V_s = required storage volume (acre-feet)
V_r = runoff volume (acre-feet)
q_o = peak outflow discharge (cfs)
q_i = peak inflow discharge (cfs)

Step 9. The required storage volume (CPv in this case) can then be calculated using Equation 3-16. To check the CPv estimate, the volume must be incorporated into a BMP design and the 1-year 24-hour storm routed through the BMP. The CPv is adequate when the 1-year 24-hour design storm is detained for 24 hours, measured from the centroid of the inflow hydrograph to the centroid of the outflow hydrograph.

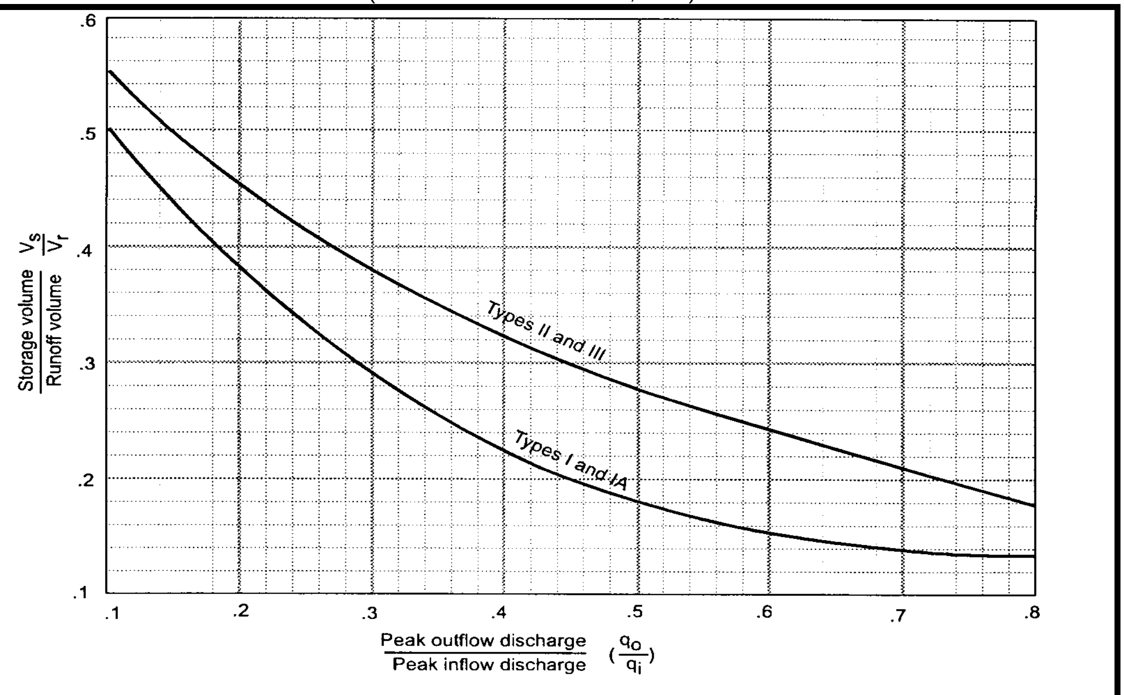
Equation 3-16

$$V_s = \frac{\left(\frac{V_s}{V_r}\right) Q_d A}{12}$$

where:

Vs and Vr are defined above
Qd = the developed runoff depth for the design storm (inches)
A = total drainage area (acres)

Figure 3-5. Approximate Detention Basin Routing for Rainfall Types I, IA, II, and III
(Source: USDA SCS TR-55, 1986)



<u>**Example 3-8. Estimation of CPv**</u>

Estimate the CPv necessary for a 50-acre wooded watershed, which will be developed as follows:

Forest land - good cover (hydrologic soil group B) = 10 ac
Forest land - good cover (hydrologic soil group C) = 10 ac
Residential with 1/3 acre lots (hydrologic soil group B) = 20 ac
Industrial development (hydrological soil group C) = 10 ac

Other data include the following:
Total impervious area = 18 acres
% of pond and swamp area = 0

Step 1 Determine the rainfall depth (P) for the 1-year 24-hour design storm for the local jurisdiction.
The 1-year, 24 hour rainfall = 2.5 inches = P

Step 2 Determine the weighted runoff coefficient as in the table below.

Dev. #	Area (ac)	% Total	CN	Composite CN[1]
1	10	20	55	11
2	10	20	70	14
3	20	40	72	28.8
4	10	20	91	18.2
Total	50	100	-	72

1 – Composite CN = % Total * CN./100

Step 3 Calculate I_a/P for CN= 72,
I_a = 0.778 (Table 3-4)
I_a/P = (0.778/2.5) = 0.31

Step 4 Calculate Qd for 1-year 24-hour storm using SCS equation
Qd = (2.5-0.778)2/(2.5-0.778+5*0.778) = 0.53 inches

Step 5 Calculate Tc.
Utilizing standard methods for overland, shallow concentrated and channel flow:
Tc = 0.35 hours (assumed)

Step 6 Calculate unit discharge from Figure 3-2 using Tc and Ia/P from previous steps
Unit discharge from Figure 3-2 = q_u (1-year) = 540 csm/in

Step 7 Estimate channel protection volume (CPv = V_S)
Knowing q_u (1-year) = 540 csm/in from Step 6 and T (extended detention time of 24 hours), find q_o/q_i from Figure 3-4.
q_o/q_i = 0.035

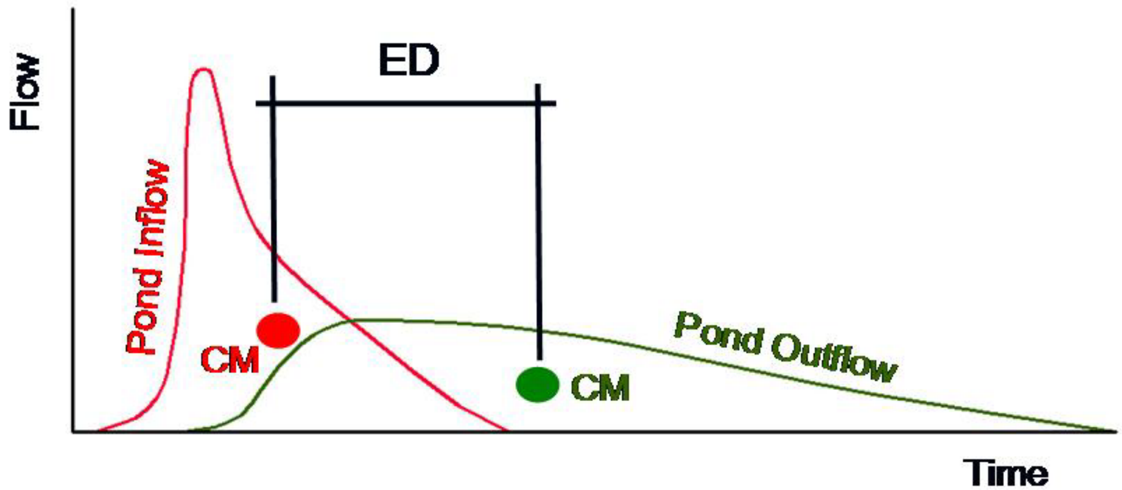
Step 8 Estimate storage/runoff using Equation 3-15,
$V_s/V_r = 0.682 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 – 0.804(q_o/q_i)^3$
$V_s/V_r = 0.682 - 1.43(0.035) + 1.64(0.035)^2 – 0.804(0.035)^3 = 0.63$

Step 9 The necessary detention volume is then calculated using Equation 3-16
CPv = V_s ≈ (Vs/Vr)*Qd*A/12 = (0.63)(0.53)(50)/12 ≈ 1.39 ac-ft

3.4.3 The Design of Channel Protection Outlets

The previous example provides an estimate of the volume required for channel protection storage. In order for the downstream channel to be protected, an orifice must next be sized to accomplish the detention criteria. The purpose of channel protection outlets is to prevent the erosive channel-forming flows that occur during the 1 to 2 year storm. This purpose is accomplished by extending the detention of the 1-year 24-hour design storm to 24 hours. The detention time is measured from the centroid of the inflow hydrograph to the centroid of the outflow hydrograph as shown in Figure 3-6.

Figure 3-6. Illustration of the Channel Protection Standard



3.4.3.1 Channel Protection Outlet Sizing

Channel protection outlets, then, must be sized using hydrograph routing techniques. The channel protection volume estimated in Section 3.4.2 will have a channel protection outlet placed at the bottom of it. The size of the outlet can only be estimated initially. Routing the 1-year 24-hour inflow hydrograph through the pond will provide an outflow hydrograph. If the centroid to centroid detention time is less than 24 hours, the channel protection orifice must be made smaller. The water quality orifice may preclude reaching the CPv 24 hour detention time, in which case, the water quality orifice must be made smaller. The water quality and channel protection orifices can be combined so long as both water quality and channel protection criteria are met.

3.5 Downstream Impact Analysis

3.5.1 Background

Local jurisdiction's stormwater design criteria may require the design to control peak discharges at the outlet of a site, such that the post-development peak discharge does not exceed the pre-development peak discharge. Typically, this peak discharge control is achieved through construction of one or more on-site detention facilities. Peak discharge control does not always provide effective water quantity control from the site, and may actually exacerbate flooding problems downstream of the site. Moreover, master plans have shown that a development site's location within a watershed may preclude the requirement for overbank flood control from a particular site.

A major reason for negative impacts due to stormwater detention facilities involves the timing of the peak discharge from the site in relation to the peak discharges in the receiving stream and/or its tributaries. If detention structures are indiscriminately placed in a watershed without consideration of the relative timing of downstream peak discharges, the structural control may actually increase the peak discharge downstream. An example of this situation is presented in Figure 3-7, which shows a comparison of the total downstream flow on a receiving stream (after development) with and without detention controls. In Figure 3-7, the smaller dashed-dot and solid lines denote the runoff hydrograph for a development site with and without detention, respectively. These runoff hydrographs will combine with a larger runoff hydrograph of the receiving stream (not shown). The combined discharges from the site and receiving stream are shown in the larger solid and dashed lines.

Figure 3-7. Potential Effect of On-Site Detention on Receiving Streams

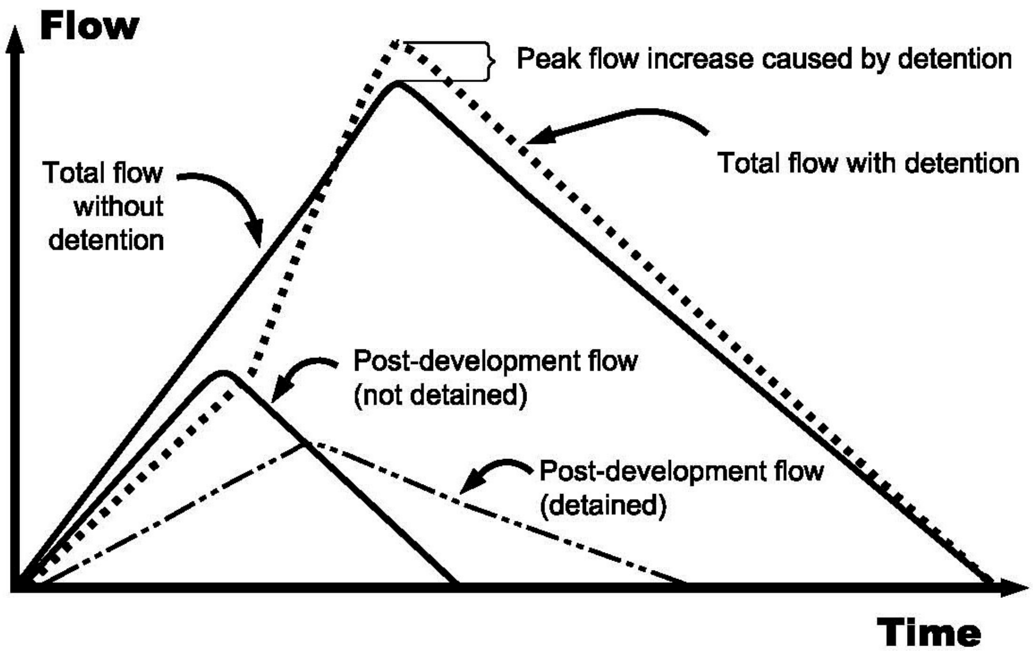
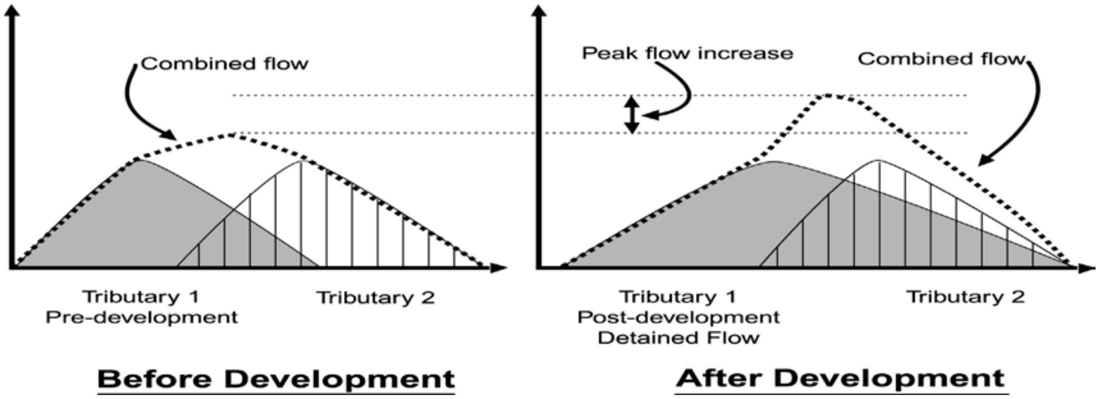


Figure 3-7 conveys a possible consequence of detention. The post-development flow from the site is reduced as required by flood protection design criteria to result in the detained flow (the smaller dashed-dot hydrograph). However, the timing of the peak discharge for the detained post-development flow, while reduced in magnitude, corresponds more closely with the timing of the peak discharge of the receiving stream (not shown) than the peak discharge of the post-development flow that was not detained. Therefore, the combination of the detained flow with the flow in the receiving stream is actually higher than would occur if no detention were required, as shown in the larger dashed hydrograph. Hence, there is a peak flow increase that is caused by detention.

Poor peak discharge timing can have an even greater impact when one considers all the developments located in a watershed and the cumulative effects of increases in runoff volume and the duration of high volume runoff in the channel, as well as peak discharge timing. Even if peak discharges are handled effectively at the site level and immediately downstream, the longer duration of higher flows due to the increased volume from many developments located on or near a stream may combine with downstream tributaries and receiving streams to dramatically increase the downstream peak flows.

Figure 3-8 illustrates this concept. The figure shows the pre- and post-development hydrographs at the confluence of two tributaries. Development occurs, meets the local flood protection criteria (i.e., the post-development peak flow is equal to the pre-development peak flow at the outlet from the site), and discharges to Tributary 1. When the post-development detained flow from Tributary 1 combines with the first downstream tributary (Tributary 2), it causes a peak flow increase when compared to the pre-development combined flow. This is due to the increased volume and timing of runoff from Tributary 1, relative to the peak flow and timing in Tributary 2. In this case, the detention volumes on Tributary 1 would have to have been increased to account for the downstream timing of the combined hydrographs to mitigate the impact of the increased runoff volume.

Figure 3-8. Potential Effect of Cumulative Detention Basins



Potential problems such as those described above are quite common, but can be avoided through the use of a stormwater master plan and/or downstream analysis of the effects of a planned development. Studies have shown that if a developer is required to assess the impacts of a development downstream to the point where the developed property is 10% of the total drainage area, and there are no adverse impacts (i.e., stream peak discharge increases), then there is assurance that there will not be significant increases in flooding problems further downstream. For example, for a 10-acre site, the assessment would have to take place down to a point where the total accumulated drainage area is 100 acres.

While this assessment does require some additional labor on the part of the design engineer, it allows smart stormwater management within a watershed. The assessment provides the developer, the local jurisdiction and downstream property owners with a better understanding (and corresponding documentation) of the potential downstream impacts of development. In turn, this information identifies those developments for which waivers or reductions in the flood protection requirements may prove beneficial.

3.5.2 Minimum Standard

Policies pertaining to the downstream impact analysis, if required by the local jurisdiction, are listed below.

1. Downstream impact analysis shall be required for all developments and redevelopments for which a water quality management plan is required. The analysis shall determine if the proposed development or redevelopment causes an increase in peak discharge as compared to pre-development runoff rates for the same site, or has the potential to cause downstream channel and streambank erosion. This analysis must be done for all storm events that are required for peak flow control by the local jurisdiction. Peak flows must be analyzed at the outfall(s) of the site, and at each downstream tributary junction and each public or major private downstream stormwater conveyance structure to the point(s) in the stormwater system where the area of the portion of the site draining into the system is less than or equal to 10% of the total drainage area above that point.

2. If the downstream impact analysis shows that the development or redevelopment causes an increase in peak discharges, downstream flood protection shall be provided such that the calculated peak discharges for the locally specified storm events after development of the site are not greater than that which would result from the same duration storms in the same downstream analysis area prior to development or redevelopment. These criteria must be applied throughout the 10% downstream analysis area.

3. Downstream flood protection can be provided by downstream conveyance improvements and/or purchase of flow easements in lieu of peak discharge controls subject to prior approval

by the local jurisdiction and satisfaction of the following requirements:

(1) Sufficient hydrologic and hydraulic analysis must be presented that shows that the alternative approach will offer adequate protection from downstream flooding for all potentially affected downstream property owners.

(2) The applicant is responsible for submittal and approval of any necessary CLOMR prior to construction, and a LOMR upon completion of construction.

(3) The applicant is responsible for all State and Federal permits that may be applicable to the site including TDEC NPDES and ARAP permits, US Army Corps of Engineers Section 404 permits, and TVA Section 26A permits.

4. Developments and redevelopments that do not cause an increase in peak discharges are not exempt from conformance with the minimum standards for water quality treatment (WQv) and channel protection (CPv), presented earlier in this chapter.

5. The downstream analysis should be performed after any WQv reductions for better site design practices have been taken into consideration in the calculation of peak discharges leaving the site. While there are no reductions for flood protection criteria, the use of better site design practices will inherently reduce runoff volumes and potentially reduce post-development peak discharges, both on-site and downstream of the site.

6. The data and results of the downstream analysis must be presented to the local jurisdiction as part of the water quality management plan.

Typical steps in the application of the ten-percent rule are:

1. Using a topographic map determine the lower limit of the “zone of influence” (i.e., the 10% point), and determine all 10% rule comparison points (at the outlet of the site and at all downstream tributary junctions or other points of interest).
2. Using a hydrologic model determine the pre-development peak discharges for the storms specified by the local jurisdiction and the timing of those peaks at each tributary junction beginning at the pond outlet and ending at the next tributary junction beyond the 10% point.
3. Change the site land use to post-development conditions and determine the post-development peak discharges and timing for the same storms. Design the structural control facility such that the post-development peak discharges from the site <u>for all storm events</u> do not increase the pre-development peak discharges at the outlet of the site and at each downstream tributary junction and each public or major private downstream stormwater conveyance structure located within the zone of influence.
4. If post-development conditions do increase the peak flow within the zone of influence, the structural control facility must be redesigned or conveyance improvements/flow easements may be allowed by the local jurisdiction (see item 3 in the previous section).

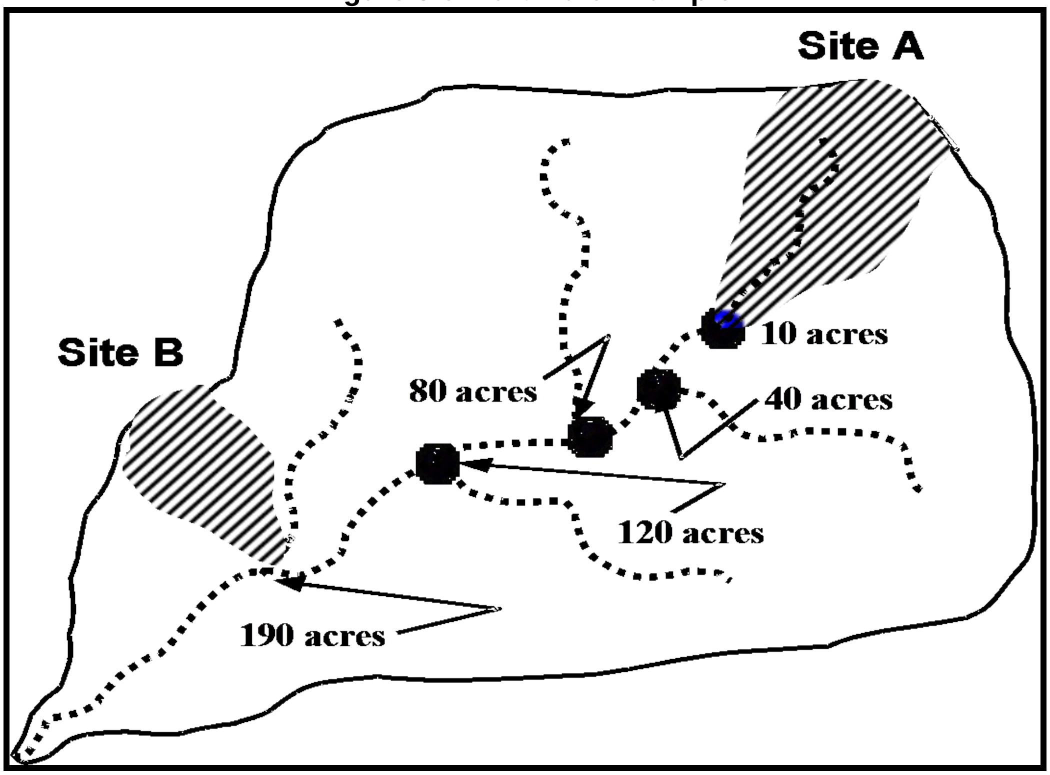
<u>Example 3-9. Ten Percent Rule Example</u>

Figure 3-9 illustrates the concept of the ten-percent rule for two sites in a watershed.

Site A is a development of 10 acres, all draining to a wet ED stormwater pond. Looking downstream at each tributary in turn, it is determined that the analysis should end at the tributary marked “120 acres.” The 100-acre (10%) point is in between the 80-acre and 120-acre tributary junction points.

The designer constructs a simple HEC-1 (HEC-HMS) model of the 120-acre areas using single existing condition sub-watersheds for each tributary. Key detention structures existing in other tributaries must be modeled. An approximate curve number is used since the *actual* peak flow is not the key for initial analysis; only the increase or decrease is important. The accuracy in curve number determination is not as significant as an accurate estimate of the time of concentration. Since flooding is an issue downstream, the pond is designed (through several iterations) until the peak flow does not increase at junction points downstream to the 120-acre point.

Figure 3-9. 10% Rule Example



Site B is located downstream at the point where the total drainage area is 190 acres. The site itself is only 6 acres. The first tributary junction downstream from the 10% point is the junction of the site outlet with the stream. The total 190 acres is modeled as one basin with care taken to estimate the time of concentration for input into the hydrologic model of the watershed. The model shows that a detention facility, in this case, will actually <u>increase</u> the peak flow in the stream.

3.6 References

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