CHAPTER 5

ENGINEERING
CALCULATIONS

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ENGINEERING CALCULATIONS

5-1 INTRODUCTION

This section provides guidelines for performing various engineering calculations associated with the design of stormwater management facilities such as extended-detention and retention basins and multi-stage outlet structures. The prerequisite information for using these calculations is the determination of the hydrologic characteristics of the contributing watershed in the form of the peak discharge (in *cfs*), or a runoff hydrograph, depending on the hydrologic and hydraulic routing methods used. (Refer to **Section 4-4** in **Chapter 4** for hydrologic methods.)

5-2 GENERAL INFORMATION: DETENTION, EXTENDED-DETENTION AND RETENTION BASIN DESIGN CALCULATIONS

Based on Virginia's Stormwater Management Regulations, a stormwater management basin may be designed to *control water quantity* (for flood control and channel erosion control) and to *enhance* (or treat) *water quality*. The type of basin selected (extended-detention, retention, infiltration, etc.) and the relationship between its design components *(design inflow, storage volume* and *outflow)* will dictate the size of the basin and serve as the basis for its hydraulic design. Some design component parameters such as *design storm return frequency*, *allowable discharge rates*, etc., may be specified by the local regulatory authority, based upon the specific needs of certain watersheds or stream channels within that locality. Occasionally, as in stream channel erosion control, it may be up to the engineer to document and analyze the specific needs of the downstream channel and establish the design parameters.

The ***design inflow*** is either the *peak flow* or the *runoff hydrograph* from the developed watershed. This *inflow* becomes the input data for the basin sizing calculations, often called ***routings***. Various routing methods are available. Note that the format of the hydrologic input data will usually be dictated by whatever routing method is chosen. (The methods discussed in this handbook require the use of a *peak discharge* or an actual *runoff hydrograph*.) Generally, larger and more complex projects will require a detailed analysis, which includes a runoff hydrograph. Preliminary studies and small projects may be designed using simpler, shortcut techniques that only require a peak discharge. For all projects, the designer must document the hydrologic conditions to support the inflow portion of the hydraulic relationship.

Achieving adequate ***storage volume*** within a basin can usually be accomplished by manipulation of the site grades and strategic placement of the permanent features such as buildings and parking lots. Sometimes, the location of a stormwater facility will be dictated by the site topography and available outfall location. (Refer to **Chapter 3** for basin planning considerations and design criteria.) Storage volume calculations will be discussed in detail later in this chapter.

5-3 ALLOWABLE RELEASE RATES

The allowable release rates for a stormwater facility are dependent on the proposed function(s) of that facility, such as ***flood control***, ***channel erosion control***, or ***water quality enhancement***. For example, a basin used for *water quality enhancement* is designed to detain the ***water quality volume*** and slowly release it over a specified amount of time. This water quality volume is the ***first flush*** of runoff, which is considered to contain the largest concentration of pollutants (Schueler 1987). (Refer to **Section 5-6** for water quality volume calculations.) In contrast, a basin used for *flood* or *channel erosion control* is designed to detain and release runoff from a given storm event at a *predetermined maximum release rate*. This *release rate* may vary from one watershed to another based on predeveloped conditions.

Localities, through stormwater management and erosion control ordinances, have traditionally set the allowable release rates for given frequency storm events to equal the watershed's pre-developed rates. This technique has become a convenient and consistent mechanism to establish the design parameters for a stormwater management facility, particularly as it relates to flood control or stream channel erosion control.

Chapter 4 discusses the impact of development on the hydrologic cycle and the difficulty in re-establishing the pre-developed runoff characteristics. Although it is popular to set a stormwater basin's allowable release rate to the watershed's pre-developed rate, this technique rarely duplicates existing conditions, particularly as it relates to storm frequencies and duration.

In Virginia, the allowable release rate for controlling stream channel erosion or flooding may be established by ordinance using the state's minimum criteria, or by analyzing specific downstream topographic, geographic or geologic conditions to select alternate criteria. **The engineer should be aware of what the local requirements are <u>before</u> designing**.

The design examples and calculations in this handbook use the state minimum requirements for illustrative purposes. **Example 1,** which considers a single homogeneous watershed, is summarized here to show the allowable release rates calculated for the basin. These release rates, as required by the state stormwater regulations, are the pre-developed runoff rates for the 2- and 10-year design storms. **Table 5-1** provides a summary of the hydrologic analysis for **Example 1**. (The complete solution to **Example 1** is provided in **Chapter 6**.)

TABLE 5 - 1
Hydrologic Summary*, *Example 1, SCS Methods

TR-55 GRAPHICAL PEAK DISCHARGE					
CONDITION	**DA**	**RCN**	t_c	Q_2	Q_{10}
PRE-DEV	25 *ac.*	64	0.87 *hr.*	8.5 *cfs**	26.8 *cfs**
POST-DEV	25 *ac.*	75	0.35 *hr.*	29.9 *cfs*	70.6 *cfs*
TR-20 COMPUTER RUN					
PRE-DEV	25 *ac.*	64	0.87 *hr.*	8.0 *cfs**	25.5 *cfs**
POST-DEV	25 *ac.*	75	0.35 *hr.*	25.9 *cfs*	61.1 *cfs*

***Allowable release rate**

5-4 STORAGE VOLUME REQUIREMENT ESTIMATES

Stormwater management facilities are designed using a trial and error process. The designer does many iterative routings to select a minimum facility size with the proper outlet controls. Each iterative routing requires that the facility size (*stage-storage relationship*) and the outlet configuration (*stage-discharge relationship*) be evaluated for performance against the watershed requirements. A graphical evaluation of the ***inflow hydrograph*** versus an approximation of the ***outflow rating curve*** provides the designer with an estimate of the required ***storage volume***. Starting with this **assumed** required volume, the number of iterations is reduced.

The ***graphical hydrograph analysis*** requires that the evaluation of the watershed's hydrology produce a runoff hydrograph for the appropriate design storms. The state stormwater management regulations allow the use of SCS methods or the modified rational method (critical storm duration approach) for analysis. Many techniques are available to generate the resulting runoff hydrographs based on these methods. It is the designer's responsibility to be familiar with the limitations and assumptions of the methods as they apply to generating hydrographs (refer to **Chapter 4, Hydrologic Methods**).

Graphical procedures can be time consuming, especially when dealing with multiple storms, and are therefore not practical when designing a detention facility for a small site development. Shortcut procedures have been developed to allow the engineer to approximate the storage volume requirements. Such methods include **TR-55: Storage Volume for Detention Basins**, **Section 5-4.2**, and **Critical Storm Duration-Modified Rational Method-Direct Solution**, **Section 5-4.4**, which can be used as planning tools. Final design should be refined using a more accurate hydrograph routing procedure. Sometimes, these shortcut methods may be used for final design, but they must be used with caution since they only approximate the required storage volume (refer to the assumptions and limitations for each method).

It should be noted that the TR-55 tabular hydrograph method does not produce a full hydrograph. The tabular method generates only the portion of the hydrograph that contains the peak discharge and some of the time steps just before and just after the peak. The missing values must be extrapolated, thus potentially reducing the accuracy of the hydrograph analysis. It is recommended that if SCS methods are to be used, a full hydrograph be generated using one of the available computer programs. The accuracy of the analysis can only be as accurate as the hydrograph used.

5-4.1 Graphical Hydrograph Analysis - SCS Methods

The following procedure represents a graphical hydrograph analysis that results in the approximation of the required storage volume for a proposed stormwater management basin. **Example 1** is presented here to illustrate this technique. See **Table 5-1** for a summary of the hydrology. The TR-20 computer-generated inflow hydrograph is used for this example. The allowable discharge from the proposed basin has been established by ordinance (based on pre-developed watershed discharge).

Information Needed:

The pre- and post-developed hydrology, which includes the pre-developed peak rate of runoff ***(allowable release rate)*** and the post-developed runoff hydrograph (***inflow hydrograph***) is required for hydrograph analysis (see **Table 5-1**).

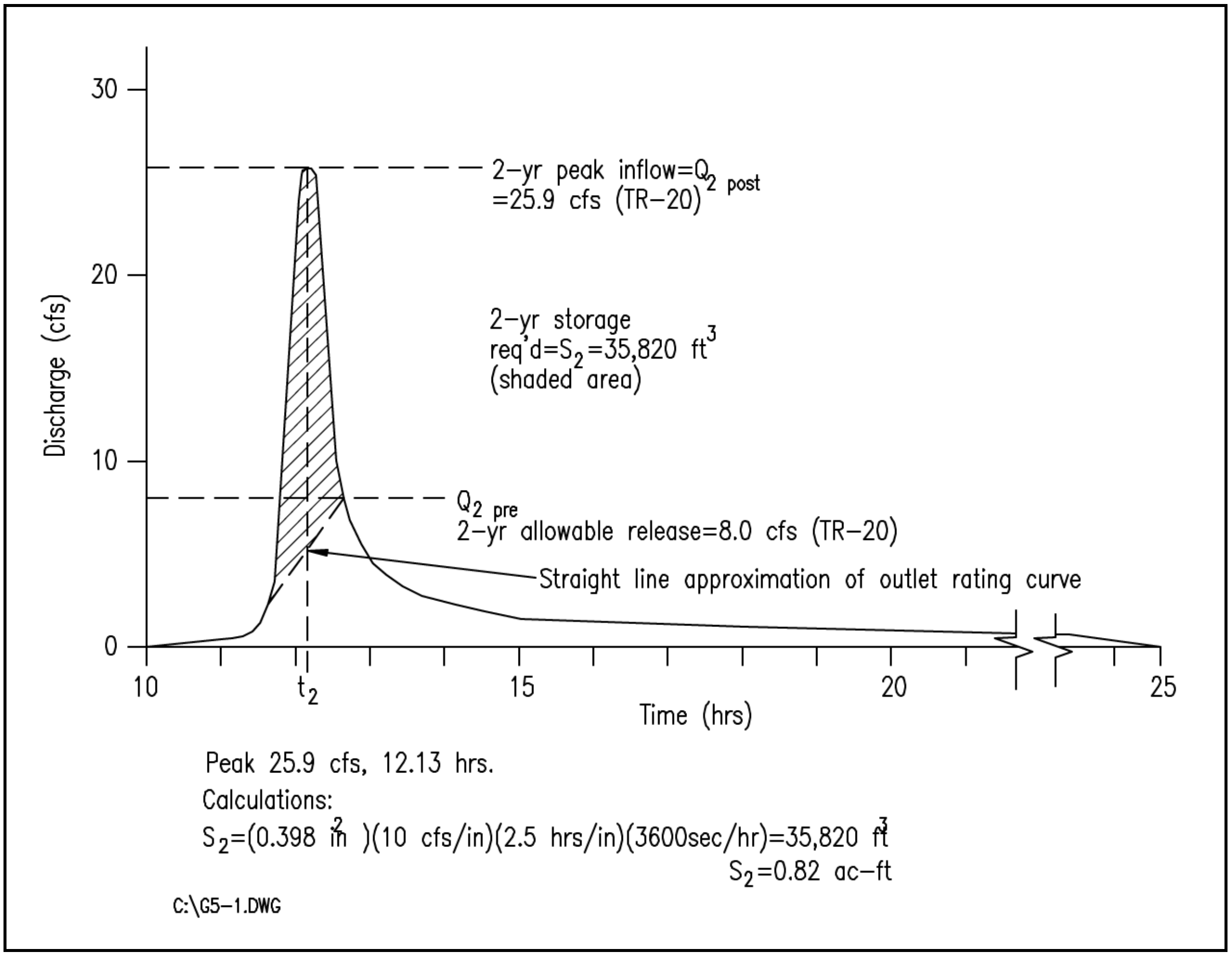
Procedure

(Refer to **Figure 5-1** for the 2-year developed inflow hydrograph and **Figure 5-2** for the 10-year developed inflow hydrograph):

1. Commencing with the plot of the 2-year developed inflow hydrograph (Discharge vs. Time), the 2-year allowable release rate, $Q_2 = 8\ cfs$, is plotted as a horizontal line starting at time $t = 0$ and continuing to the point where it intersects the falling limb of the hydrograph.

2. A diagonal line is then drawn from the beginning of the inflow hydrograph to the intersection point described above. This line represents the ***hypothetical rating curve*** of the control structure and approximates the rising limb of the outflow hydrograph for the 2-year storm.

FIGURE 5 - 1
SCS Runoff Hydrograph, Example 1, 2-Year Post-Developed



3. The ***storage volume*** is then approximated by calculating the area under the inflow hydrograph, less the area under the rising limb of the outflow hydrograph. This is shown as the shaded area in **Figure 5-1**. The storage volume required for the 2-year storm, S_2, can be approximated by measuring the shaded area with a planimeter.

The vertical scale of a hydrograph is in cubic feet per second (*cfs*) and the horizontal scale is in hours (hrs). Therefore, the area, as measured in square inches (in^2), is multiplied by scale conversion factors of *cfs* per inch, hours per inch, and 3600 seconds per hour, to yield an area in cubic feet (ft^3). The conversion is as follows:

$$\begin{aligned} S_2 &= (0.398\ in^2)(10\ cfs/in.)(2.5\ hrs./in.)(3{,}600\ sec./hr.) \\ &= 35{,}820\ ft^3 \\ &= 0.82\ ac.ft. \end{aligned}$$

4. On a plot of the 10-year inflow hydrograph, the 10-year allowable release rate, Q_{10} , is plotted as a horizontal line extending from time zero to the point where it intersects the falling limb of the hydrograph.

5. By trial and error, the time t_2 , at which the S_2 volume occurs while maintaining the 2-year release, is determined by planimeter. This is represented by the shaded area to the left of t_2 on **Figure 5-2**. From the intersection point of t_2 and the 2-year allowable release rate, Q_2, a line is drawn to connect to the intersection point of the 10-year allowable release rate and the falling limb of the hydrograph. This intersection point is t_{10} , and the connecting line is a straight line approximation of the *outlet rating curve*.

6. The area under the inflow hydrograph from time t_2 to time t_{10} , less the area under the rising limb of the hypothetical rating curve, represents the additional volume (shaded area to the right of t_2 on **Figure 5-2**) needed to meet the 10-year storm storage requirements.

7. The total storage volume, S_{10} , required, can be computed by adding this additional storage volume to S_2 . This is represented by the total shaded area under the hydrograph.

$$\begin{aligned} S_{10} &= (0.89\ in^2)(10\ cfs/in.)(2.5\ hrs./in.)(3{,}600\ sec./hr.) \\ &= 80{,}100\ ft^3 \\ &= 1.84\ ac.ft. \end{aligned}$$

These steps may be repeated if storage of the 100-year storm, or any other design frequency storm, is required by ordinance or downstream conditions.

In summary, the total volume of storage required is the area <u>under</u> the runoff hydrograph curve and <u>above</u> the basin outflow curve. It should be noted that the outflow rating curve is approximated as a straight line. The actual shape of the outflow rating curve will depend on the type of outlet device used. **Figure 5-3** shows the typical shapes of outlet rating curves for orifice and weir outlet structures. The straight line approximation is reasonable for an orifice outlet structure. However, this approximation will likely **underestimate** the storage volume required when a weir outlet structure is used. Depending on the complexity of the design and the need for an exact engineered solution, the use of a more rigorous sizing technique, such as a storage indication routing, may be necessary.

FIGURE 5 - 2
SCS Runoff Hydrograph, Example 1, 10-Year Post-Developed

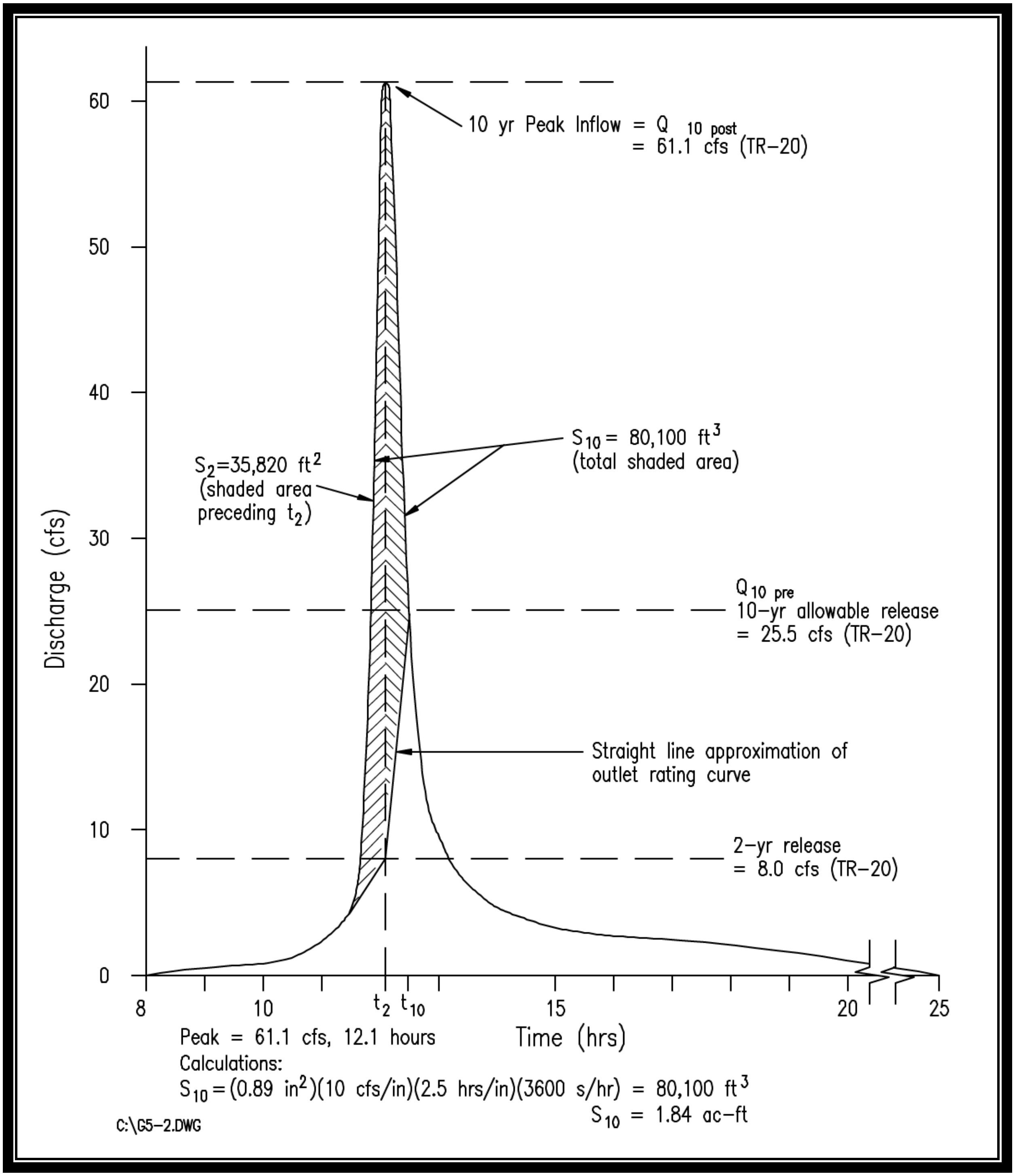
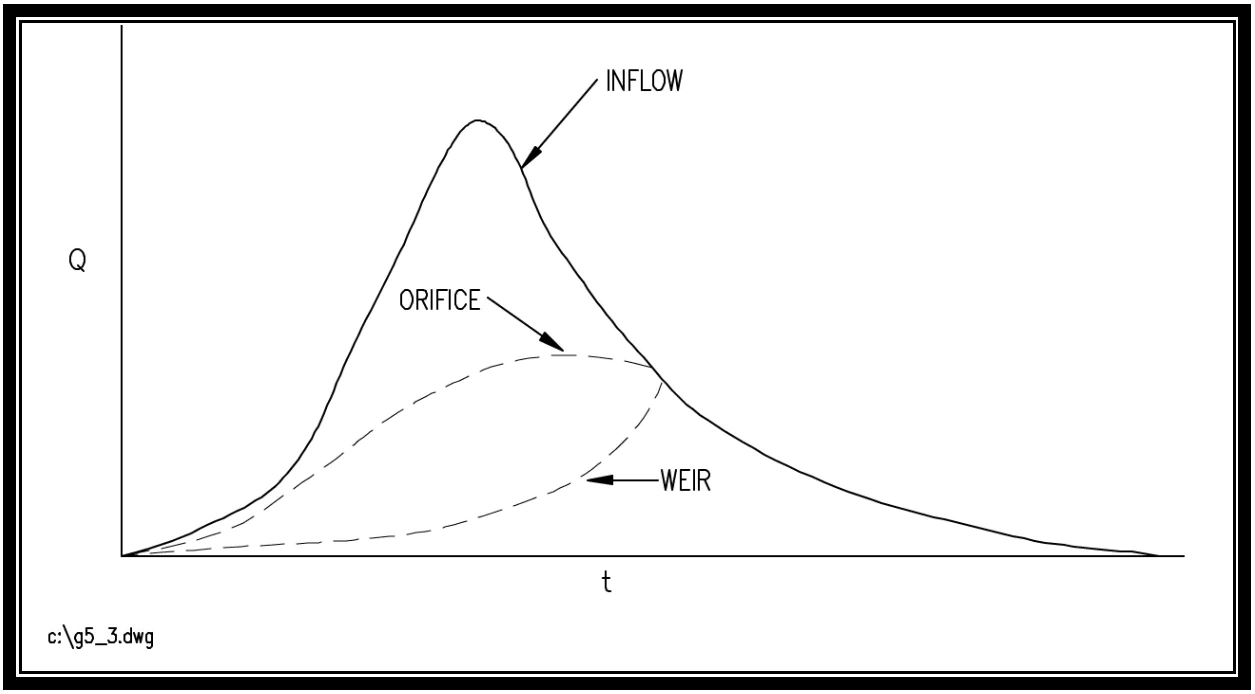


FIGURE 5 - 3

Typical Outlet Rating Curves for Orifice and Weir Outlet Devices

5-4.2 TR-55: Storage Volume for Detention Basins (Shortcut Method)



The TR-55 Storage Volume for Detention Basins, or TR-55 shortcut procedure, provides similar results to the graphical analysis described in **Section 5-4.1**. This method is based on average storage and routing effects for many structures. TR-55 can be used for single-stage or multi-stage outflow devices. The only constraints are that 1) *each stage requires a design storm and a computation of the storage required for it*, and 2) *the discharge of the upper stage(s) includes the discharge of the lower stage(s)*. Refer to TR-55 for more detailed discussions and limitations.

Information Needed:

To calculate the required storage volume using TR-55, the pre- and post-developed hydrology per SCS methods is needed (refer to **Chapter 4**). This includes the watershed's *pre-developed peak rate of discharge,* or *allowable release rate*, Q_o , the watershed's *post-developed peak rate of discharge*, or *inflow*, Q_i , for the appropriate design storms, and the watershed's *post-developed runoff*, Q, in inches. (Note that this method does **not** require a hydrograph.)

Once the above parameters are known, the TR-55 Manual can be used to approximate the storage volume required for each design storm. The following procedure summarizes the TR-55 shortcut method using the 25-acre watershed presented in **Example 1**.

Procedure:

1. Determine the peak developed inflow, Q_i, and the allowable release rate, Q_o, from the hydrology for the appropriate design storm. The 2-year storm flow rates from Example 1 (TR-55 Graphical peak discharge) are used here:

$$Q_{o_2} = 8.5 \ cfs \ ; \quad Q_{i_2} = 29.9 \ cfs$$

   Using the ratio of the allowable release rate, Q_o , to the peak developed inflow, Q_i , or Q_o/Q_i , for the appropriate design storm, use **Figure 5-4** (or Figure 6-1 in TR-55) to obtain the ratio of storage volume, V_s, to runoff volume, V_r , or V_s/V_r .

   From **Example 1**:

$$Q_{o_2}/Q_{i_2} = 8.5 \,/\, 29.9 = 0.28$$

   From **Figure 5-4** or TR-55 Figure 6.1:

$$V_{s_2} \ / \ V_{r_2} ; 39$$

2. Determine the runoff volume, V_r , in *ac.ft.*, from the TR-55 worksheets for the appropriate design storm.

$$V_r = Q \ A_m \ 53.33$$

where:

Q	=	*runoff, in inches, from TR-55 Worksheet 2*
A_m	=	*drainage area, in square miles*
53.33	=	*conversion factor to acre-feet*

From **Example 1**:

$$Q_2 = 1.30 \text{ in.}$$
$$A_m = 25 \text{ ac.} \,/\, 640 \text{ ac./mi}^2 = 0.039 \text{ mi}^2$$

$$V_{r_2} = 1.30(.039)\ 53.33$$
$$= 2.70 \text{ ac.ft.}$$

3. Multiply the V_s/V_r ratio from Step 1 by the runoff volume, V_r, from Step 2, to determine the volume of storage required, V_s, in acre-feet.

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

From **Example 1**:

$$(.39)(2.70 \text{ ac.ft.}) = 1.05 \text{ ac.ft.}$$

4. Repeat these steps for each additional design storm as required to determine the approximate storage requirements. The 10-year storm storage requirements from **Example 1** are presented here:

   a. $Q_o = 26.8$ *cfs*
   $Q_i = 70.6$ *cfs*

   $Q_o/Q_i = 26.8/70.6 = 0.38$; From **Figure 5-4** or TR-55 Figure 6-1: $V_s/V_r = .33$

   b. $V_r = QA_m\ 53.33 = 2.85$ *in.(.039 sq.mi.)(53.33) = 5.93 ac.ft.*

   c. $V_s = (V_s/V_r)V_r = (.33)$ *5.93 ac.ft. = 1.96 ac.ft.*

This volume represents the total storage required for the 2-year storm and the 10-year storm.

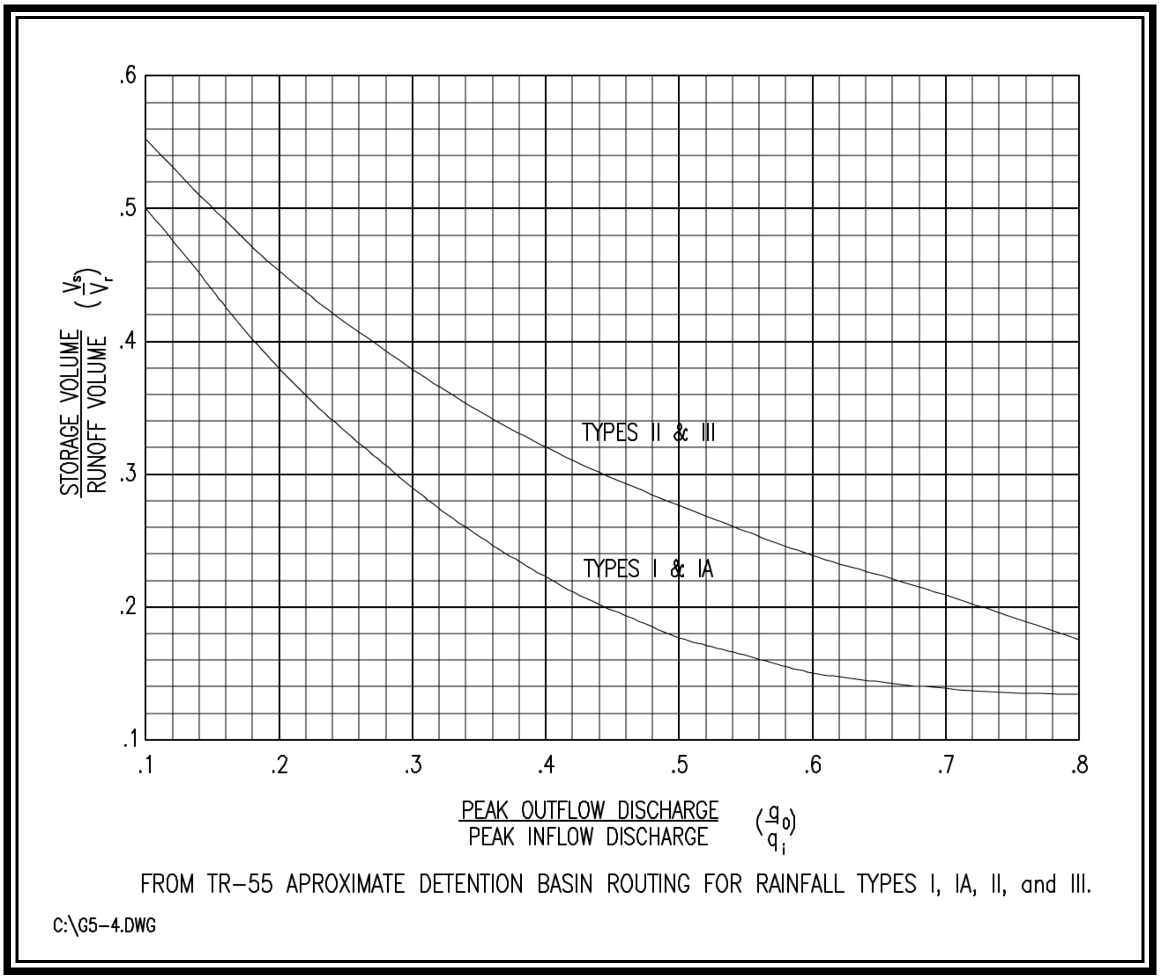
5. NOTE: The volume from **#4** above may need to be increased if additional storage is required for water quality purposes or channel erosion control. Refer to **Section 5-6** or **Section 5-10**, respectively.

The design procedure presented above should be used with TR-55 Worksheet 6a, as shown in **Example 1** of **Chapter 6**. The worksheet includes an area to plot the *stage-storage curve*, from which actual elevations corresponding to the required storage volumes can be derived. **Table 5-2** provides a summary of the required storage volumes using the graphical SCS hydrograph analysis and the TR-55 shortcut method.

TABLE 5 - 2
Storage Volume Requirements, Example 1

Method	2-*yr*. Storage Required	10-*yr*. Storage Required
Graphical Hydrograph Analysis	0.82 *ac.ft.*	1.84 *ac.ft.*
TR-55 Shortcut Method	1.05 *ac.ft.*	1.96 *ac.ft.*

FIGURE 5 - 4
Approximate Detention Basin Routing for Rainfall Types I, IA, II and III



Source: SCS TR-55 Urban Hydrology for Small Watersheds: Figure 6-1

5-4.3 Graphical Hydrograph Analysis, Modified Rational Method - Critical Storm Duration

The Modified Rational Method uses the ***critical storm duration*** to calculate the ***maximum storage volume*** for a detention facility. This *critical storm duration* is the storm duration that generates the greatest volume of runoff and, therefore, requires the most storage. In contrast, the Rational Method produces a triangular runoff hydrograph that gives the peak inflow at time = t_c and falls to zero flow at time = $2.5t_c$. In theory, this hydrograph represents a storm whose duration equals the time of concentration, t_c, resulting in the greatest peak discharge for the given return frequency storm. The volume of runoff, however, is of greater consequence in sizing a detention facility. A storm whose duration is longer than the t_c may not produce as large a peak rate of runoff, but it may generate a greater **volume** of runoff. By using the Modified Rational Method, the designer can evaluate several different storm durations to verify which one requires the greatest volume of storage with respect to the allowable release rate. It is this *maximum storage volume* that the basin must be designed to detain.

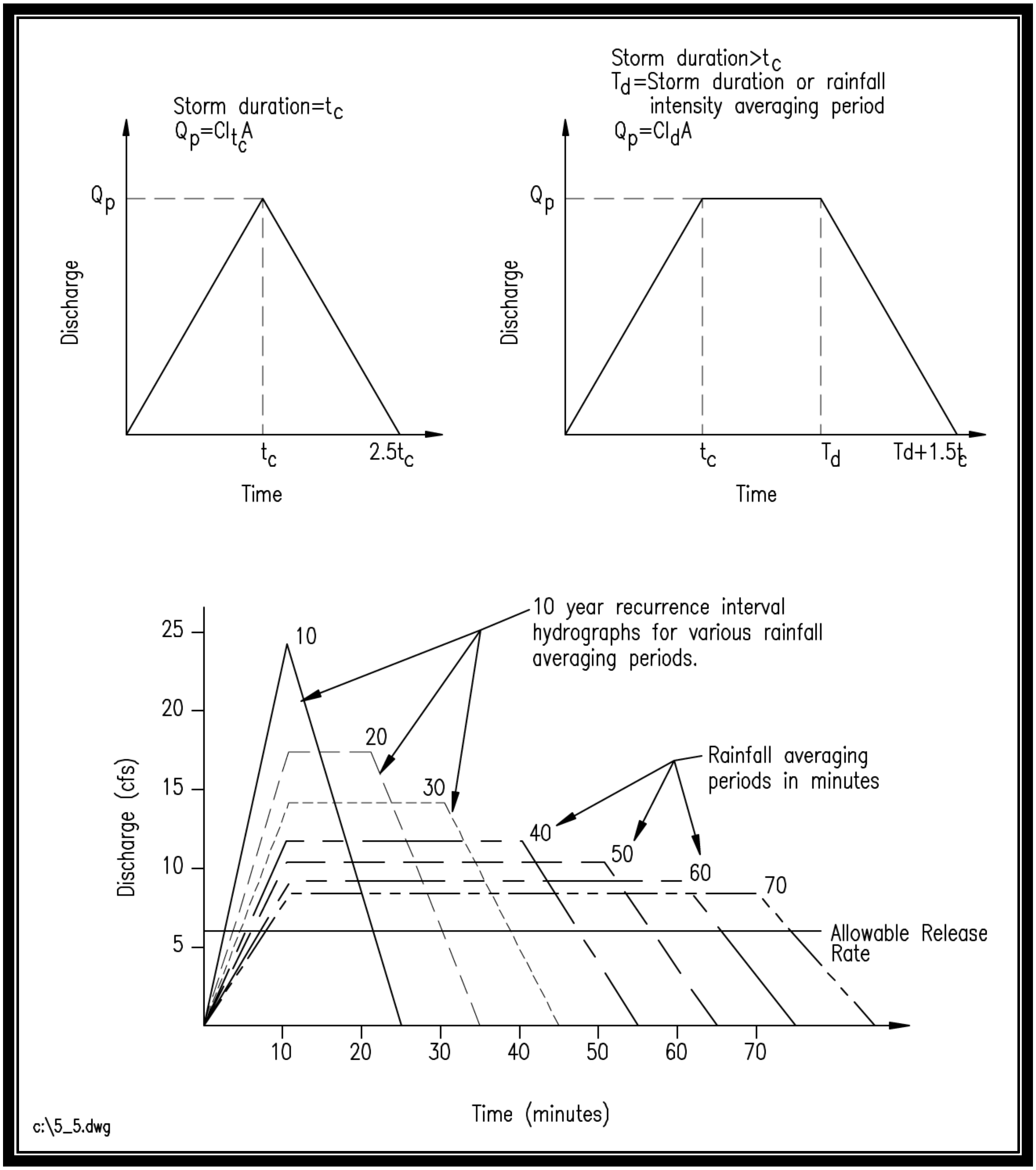
The first step in determining the critical storm duration is to use the post-developed time of concentration, t_c, to generate a post-developed runoff hydrograph. Rainfall intensity averaging periods, T_d, representing time periods incrementally longer than the t_c, are then used to generate a "family" of runoff hydrographs for the same drainage area. These hydrographs will be trapezoidal with the peak discharges, Q_i, based upon the intensity, I, of the averaging period, T_d. **Figure 5-5** shows the construction of a typical triangular and trapezoidal hydrograph using the modified rational method, and a family of trapezoidal hydrographs representing storms of different durations.

Note that the duration of the receding limb of the trapezoidal hydrograph, in **Figure 5-5**, is set to equal 1.5 times the time of concentration, t_c. Also, the total hydrograph duration is $2.5t_c$ versus $2t_c$ as discussed in **Chapter 4**. This longer duration is considered more representative of actual storm and runoff dynamics. It is also more analogous to the SCS unit hydrograph where the receding limb extends longer than the rising limb.

The Modified Rational Method assumes that the rainfall intensity averaging period is equal to the actual storm duration. This means that the rainfall and runoff that occur before and after the rainfall averaging period are not accounted for. **Therefore, the Modified Rational Method may underestimate the required storage volume for any given storm event**.

The rainfall intensity averaging periods are chosen arbitrarily. However, the designer should select periods for which the corresponding intensity-duration-frequency (I-D-F) curves are available, i.e., 10 *min*., 20 *min*., 30 *min*., etc. The shortest period selected should be the time of concentration, t_c. A straight line starting at $Q = 0$ and $t = 0$ and intercepting the inflow hydrograph on the receding limb at the allowable release rate, Q_o, represents the outflow rating curve. The time averaging period hydrograph that represents the greatest storage volume required is the one with the largest area between the inflow hydrograph and outflow rating curve. This determination is made by a graphical analysis of the hydrographs.

FIGURE 5 - 5
Modified Rational Method Hydrographs



The following procedure represents a graphical analysis very similar to the one described in **Section 5-4.1**. **Example 1** from **Chapter 6** will be used again. Note that **the rational and modified rational methods should normally be used in homogeneous drainage areas of less than 20 acres, with a t_c of less than 20 minutes**. Although the watershed in **Example 1** has a drainage area of 25 acres and a t_c of greater than 20 minutes, it will be used here for illustrative purposes. Note that the pre- and post-developed peak discharges are much greater than those calculated using the SCS method applied to the same watershed. This difference may be the result of the large acreage and t_c values.

A summary of the hydrology is found in **Table 5-3**. Note that the t_c calculations were performed using the more rigorous SCS TR-55 method.

TABLE 5 - 3
Hydrologic Summary, Example 1, Rational Method

Rational Method					
CONDITION	**D.A.**	***C***	***T_c***	***Q_2***	***Q_{10}***
Pre-developed	*25 ac.*	*.38*	*.87 hr* *52 min.*	*17 cfs*	*24 cfs*
Post-developed	*25 ac.*	*.59*	*.35 hr.* *21 min.*	*49 cfs*	*65 cfs*

Information Needed:

The Modified Rational Method-Critical Storm Duration Approach is very similar to SCS methods since it requires pre- and post-developed hydrology in the form of a pre-developed peak rate of runoff (***allowable release rate)*** and a post-developed runoff hydrograph (***inflow hydrograph***), as developed using the Rational Method.

Procedure: (Refer to **Figures 5-6** and **5-7**.)

1. Plot the 2-year developed condition inflow hydrograph (triangular) based on the developed condition, t_c.

2. Plot a family of hydrographs, with the time averaging period, T_d, of each hydrograph increasing incrementally from 21 minutes (developed condition t_c) to 60 minutes, as shown in **Figure 5-6**. Note that the first hydrograph is a Type 1 Modified Rational Method triangular hydrograph, as shown in **Figure 4-7** in **Chapter 4,** where the storm duration, *d*, or T_d, is equal

to the time of concentration, t_c. The remaining hydrographs are trapezoidal, or Type 2 hydrographs. The peak discharge for each hydrograph is calculated using the rational equation, $Q = CIA$, where the intensity, I, from the I-D-F curve is determined using the rainfall intensity averaging period as the storm duration.

3. Superimpose the outflow rating curve on each inflow hydrograph. The area between the two curves then represents the storage volume required, as shown in **Figure 5-6**. Similar cautions, as described in the SCS Methods, **Section 5-4.1**, regarding the straight line approximation of the outlet discharge curve apply here as well. The actual shape of the outflow curve depends on the type of outlet device.

4. Compute and tabulate the required storage volume for each of the selected rainfall durations or time averaging periods, T_d, using the procedures described in **Section 5-4.1**.

   The storm duration that requires the maximum storage is the *critical storm* and is used for the sizing of the basin. (A storm duration equal to the t_c produces the largest storage volume required for the 2-year storm presented here.)

5. Repeat Steps 1 through 4 above for the analysis of the 10-year storage requirements. (**Figure 5-7 represents this procedure repeated for the 10-year design storm**.)

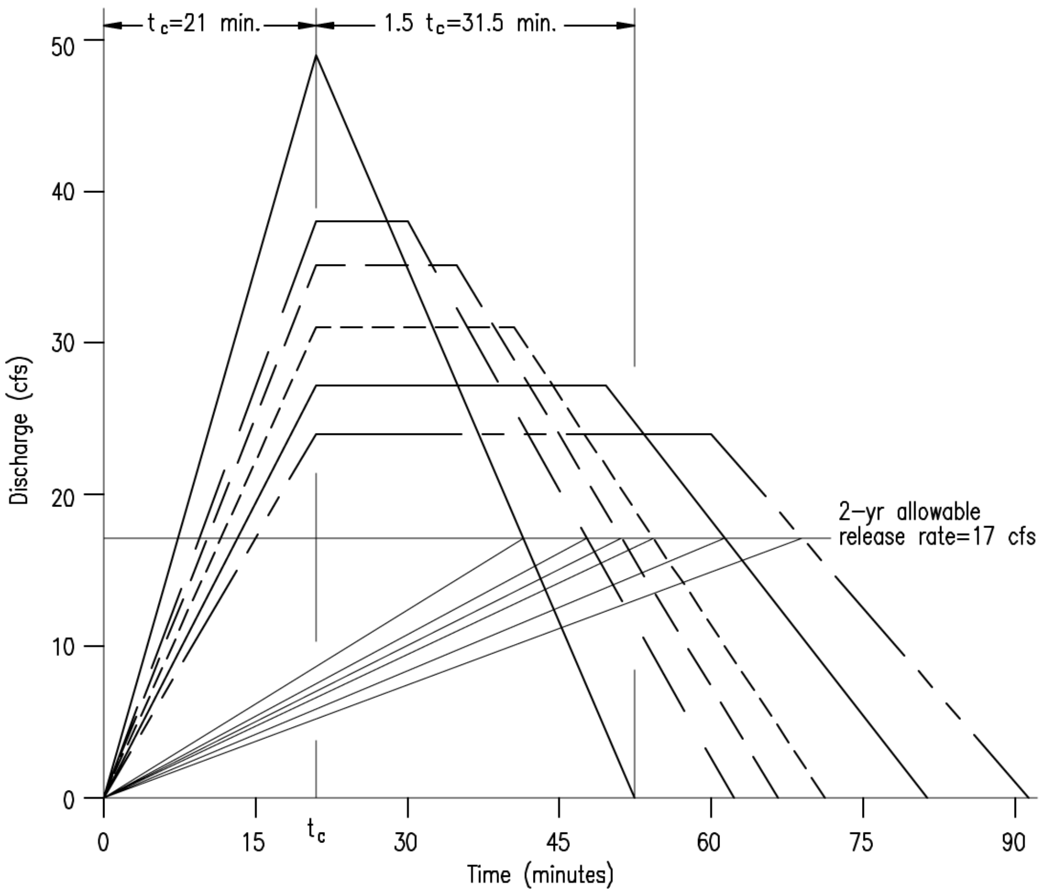
Conveyance systems should still be designed using the Rational Method, as opposed to the Modified Rational Method, to ensure their design for the peak rate of runoff.

TABLE 5 - 4
Storage Volume Requirements - Example 1

Method	2-*yr*. Storage Required	10-*yr*. Storage Required
Graphical Hydrograph Analysis	*0.82 ac.ft.*	*1.84 ac.ft.*
TR-55 Shortcut Method	*1.05 ac.ft.*	*1.96 ac.ft.*
Modified Rational Method - Critical Storm Duration	*1.16 ac.ft.* T_d *= 21 min.*	*1.56 ac.ft.* T_d *= 40 min.*

FIGURE 5 - 6

Modified Rational Method Runoff Hydrograph, Example 1, 2-Year Post-Developed Condition



Conversion – in^2 to ft^3= 10 cfs/in x 0.25 hrs/in x 3600 sec/hr=9000 ft^3/in^2

Rainfall Avg. Period (T)	Intensity I(in/hr)	Q Peak (cfs)	Area (in^2)	Storage Volume (ft^3)
21 min	3.3	49	5.62	50,580*
30 min	2.6	38	5.51	49,590
35 min	2.4	35	5.53	49,770
40 min	2.1	31	5.29	47,610
50 min	1.8	27	5.25	47,250
60 min	1.6	24	5.24	47,160

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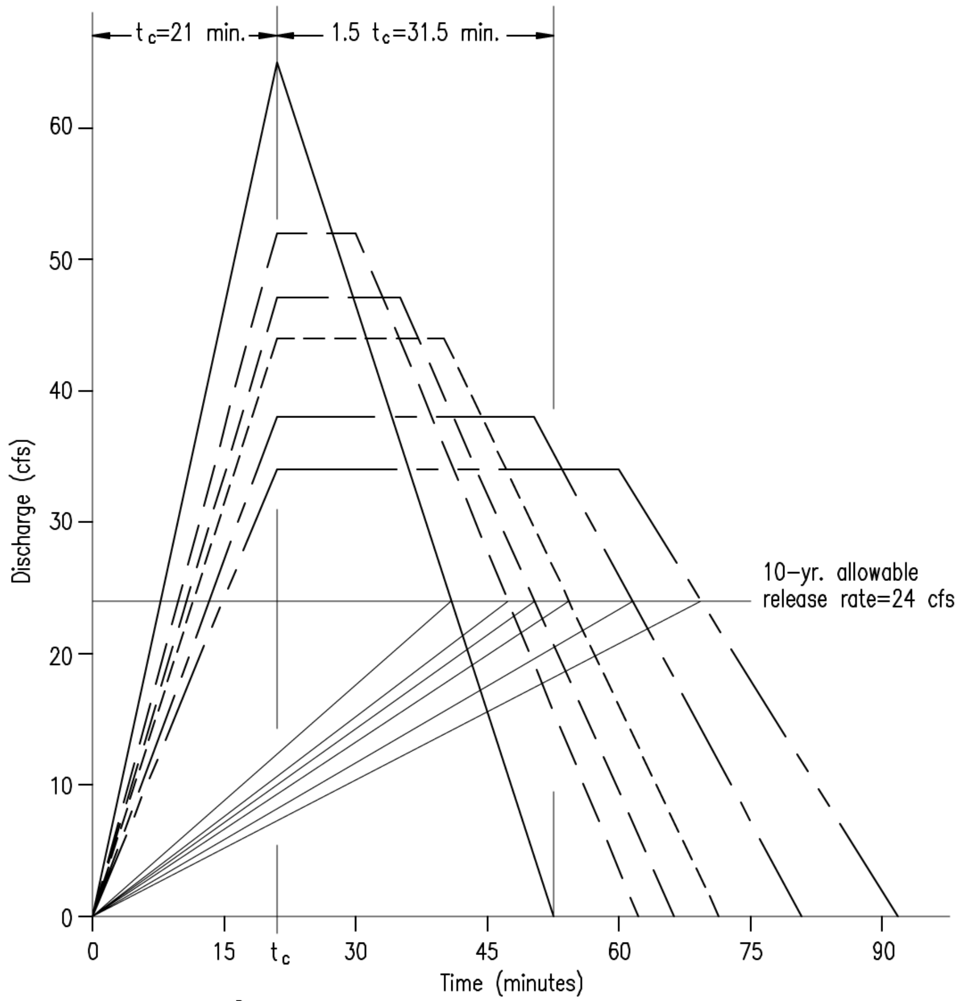
FIGURE 5 - 7

Modified Rational Method Runoff Hydrograph, Example 1, 10-Year Post-Developed Condition



Conversion – in² to ft³ = 9000 ft³/in²

Rainfall Avg. Period (T)	Intensity I(in/hr)	Q Peak (cfs)	Area (in²)	Storage Volume (ft³)
21 min	4.4	65	7.07	63,630
30 min	3.5	52	7.33	65,970
35 min	3.2	47	7.31	65,790
40 min	3.0	44	7.53	67,770*
50 min	2.6	38	7.45	67,050
60 min	2.3	34	7.48	67,320

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5-4.4 Modified Rational Method, Critical Storm Duration - Direct Solution

A direct solution to the Modified Rational Method, Critical Storm Duration has been developed to eliminate the time intensive, iterative process of generating multiple hydrographs. This direct solution takes into account the storm duration and allows the designer to solve for the time at which the storage volume curve has a slope equal to zero, which corresponds to maximum storage. The basic derivation of this method is provided below, followed by the procedure as applied to **Example 1**.

Storage Volume

The runoff hydrograph developed with the Modified Rational Method, Critical Storm Duration will be either triangular or trapezoidal in shape. The outflow hydrograph of the basin is approximated by a straight line starting at *0 cfs* at the time $t=0$ and intercepting the receding leg of the runoff hydrograph at the allowable discharge, q_o.

The straight line representation of the outflow hydrograph is a conservative approximation of the shape of the outflow hydrograph for an orifice control release structure. This method should be used with caution when designing a weir control release structure.

The required storage volume is represented by the area between the inflow hydrograph and the outflow hydrograph in **Figure 5-8**. This area can be approximated using the following equation:

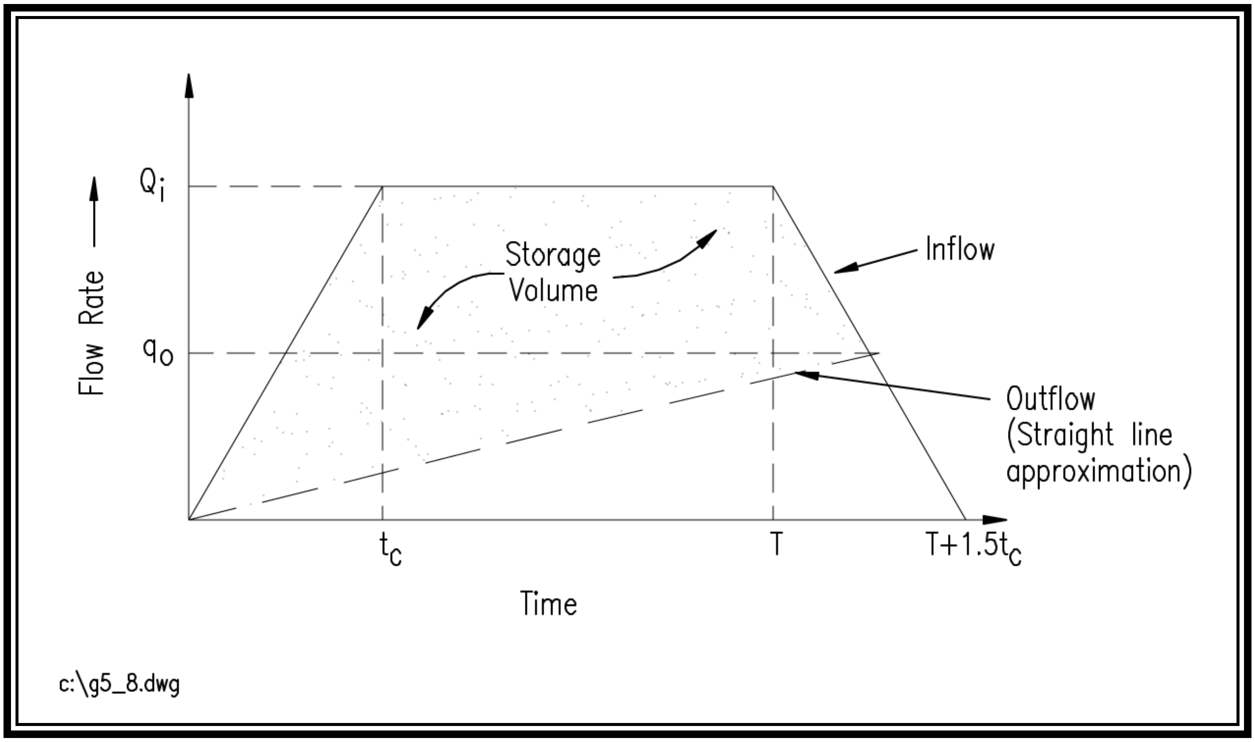
$$V = \left[Q_i T_d + \frac{Q_i t_c}{4} - \frac{q_o T_d}{2} - \frac{3 q_o t_c}{4} \right] 60$$

Equation 5-1
Trapezoidal Hydrograph Storage Volume Equation

Where:
V = *required storage volume, ft*^{3}
Q_i = *inflow peak discharge, cfs, for the critical storm duration,* T_d
t_c = *post-developed time of concentration, min.*
q_o = *allowable peak outflow, cfs*
T_d = *critical storm duration, min.*

The allowable peak outflow is established by ordinance or downstream conditions. The ***critical storm duration***, T_d, is an unknown and must be determined to solve for the intensity, I, and to ultimately calculate the peak inflow, Q_i. Therefore, a relationship between rainfall intensity, I, and the critical storm duration, T_d, must be established.

FIGURE 5 - 8
Trapezoidal Hydrograph Storage Volume Estimate



Rainfall Intensity

The rainfall intensity as taken from the I-D-F curves is dependent on the time of concentration, t_c, of a given watershed. Setting the storm duration, T_d, equal to the time of concentration, t_c, will provide the maximum peak discharge. As stated previously, however, it does not necessarily generate the maximum ***volume*** of discharge. Since this maximum volume of runoff is of interest, and the storm duration is unknown, the rainfall intensity, *I*, must be represented as a function of *time*, *frequency*, and *location*. The relationship is expressed as follows:

$$I = \frac{a}{b + T_d}$$

Equation 5-2
Modified Rational Method Intensity, (*I*), Equation

where:
- I = *rainfall intensity, in./hr.*
- T_d = *rainfall duration or rainfall intensity averaging period, min.*
- a & b = *rainfall constants developed for storms of various recurrence intervals and various geographic locations, as shown in* ***Table 5-5***

TABLE 5 - 5
Rainfall Constants for Virginia*

Duration - 5 minutes to 2 hours			
Station	**Rainfall Frequency**	**Constants**	
Wytheville	2 5 10	117.7 168.6 197.8	19.1 23.8 25.2
Lynchburg	2 5 10	118.8 158.9 189.8	17.2 20.6 22.6
Richmond	2 5 10	130.3 166.9 189.2	18.5 20.9 22.1
Norfolk	2 5 10	126.3 173.8 201.0	17.2 22.7 23.9
Cape Henry	2 5 10	143.2 173.9 203.9	21.0 22.7 24.8
The above constants are based on linear regression analyses of the frequency intensity-duration curves contained in the VDOT Drainage Manual. (Adapted from DCR Course "C" Training Notebook.)			

*****For a more comprehensive list, see Appendix 5A***.

The rainfall constants, ***a*** and ***b***, were developed from linear regression analyses of the I-D-F curves and can be generated for any area where such curves are available. The limitations associated with the I-D-F curves, such as duration, return frequency, etc., will also limit development of the constants. **Table 5-5** provides rainfall constants for various regions in Virginia. Substituting **Equation 5-2** into the rational equation results in the following:

$$Q = C\left(\frac{a}{b+T_d}\right)A$$

Equation 5-3
Rearranged Rational Equation

where: Q = *peak rate of discharge, cfs*
a & b = *rainfall constants developed for storms of various recurrence intervals and various geographic locations, as shown in* ***Table 5-5***
T_d = *critical storm duration, min.*
C = *runoff coefficient*
A = *drainage area, acres*

Substituting this relationship for Q_i , **Equation 5-1** then becomes:

$$V = \left[\left[C \left(\frac{a}{b+T_d} \right) A \right] T_d + \frac{\left[C \left(\frac{a}{b+T_d} \right) A \right] t_c}{4} - \frac{q_o T_d}{2} - \frac{3 q_o t_c}{4} \right] 60$$

Equation 5-4
Substitute *Equation 5-3* into *Equation 5-1*

Maximum Storage Volume

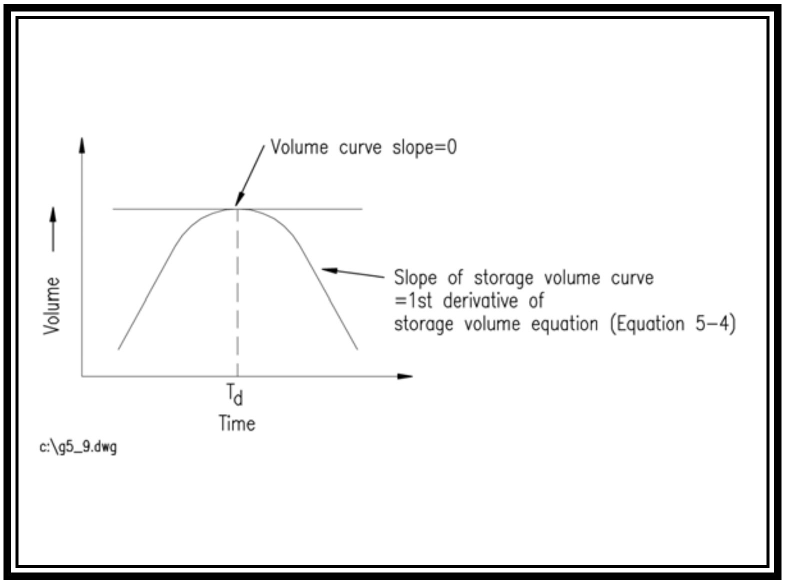
The first derivative of this storage volume equation, **Equation 5-4,** with respect to time is an equation that represents the slope of the storage volume curve plotted versus time. When this equation is set to equal zero, and solved for T_d, it represents the time, T_d, at which the slope of the storage volume curve is zero, or at a maximum, as shown in **Figure 5-9**. **Equation 5-5** represents the first derivative of the storage volume equation with respect to time and can be solved for T_d.

$$T_d = \sqrt{\frac{2CAa(b - t_c/4)}{q_o}} - b$$

Equation 5-5
Critical Storm Duration, T_d

where: T_d = *critical storm duration, min.*
C = *runoff coefficient*
A = *drainage area, acres*
a & b = *rainfall constants developed for storms of various recurrence intervals and various geographic locations, as shown in* ***Table 5-5***
t_c = *time of concentration, min.*
q_o = *allowable peak outflow, cfs*

FIGURE 5 - 9
Storage Volume vs. Time Curve



Equation 5-5 is solved for T_d . Then, T_d is substituted into **Equation 5-3** to solve for Q_i , and Q_i is substituted into **Equation 5-1** to solve for the required storage volume. Once the storage volume is known, the outlet structure and the stormwater facility can be sized. This method provides a direct solution to the graphical analysis of the family of hydrographs described in **Section 5-4.3** and is quicker to use. The following procedure illustrates this method using **Example 1**:

<u>Information Needed:</u>

The Modified Rational Method-Direct Solution is similar to the previous methods since it requires determination of the pre- and post-developed hydrology, as described in **Section 4-3.1**, resulting in a pre-developed peak rate of runoff (***allowable release rate***) and a post-developed ***runoff hydrograph***. **Table 5-3** provides a summary of the hydrology from **Example 1**. The rainfall constants ***a*** and ***b*** for the watershed are determined from **Table 5-5**.

Procedure:

1. Determine the 2-year critical storm duration by solving **Equation 5-5**:

$$T_{d_2} = \sqrt{\frac{2CA\,a\,(b - t_c/4)}{q_{o_2}}} - b$$

Where, from **Example 1**:

T_{d_2} = *2-year critical storm duration, min.*
C = *developed condition runoff coefficient = .59*
A = *drainage area = 25.0 acres*
t_c = *post-developed time of concentration = 21 min.*
q_{o_2} = *allowable peak outflow = 17 cfs (pre-developed peak rate of discharge)*
a_2 = *2-year rainfall constant = 130.3*
b_2 = *2-year rainfall constant = 18.5*

$$T_{d_2} = \sqrt{\frac{2(.59)(25.0)(130.3)(18.5 - 21/4)}{17}} - 18.5$$

$$= \sqrt{2995.9} - 18.5$$

$$T_{d_2} = 36.2 \text{ min.}$$

2. Solve for the 2-year critical storm duration intensity, I_2, using **Equation 5-2** and the 2-year critical storm duration T_{d_2}:

$$I_2 = \frac{a}{b + T_{d_2}}$$

where: T_{d_2} = *critical storm duration = 36.2 min.*
a = *2-year rainfall constant = 130.3*
b = *2-year rainfall constant = 18.5*

$$I_2 = \frac{130.3}{18.5 + 36.2} = 2.38 \text{ in./hr.}$$

3. Determine the 2-year peak inflow, Q_{i_2}, using the **Rational Equation** and the critical storm duration intensity I_2:

$$Q_{i_2} = CI_2A$$

where: Q_{i_2} = *2-year peak inflow, cfs*
C = *developed condition runoff coefficient = .59*
I_2 = *critical storm intensity = 2.38 in./hr.*
A = *drainage area = 25 acres*

$$Q_{i_2} = (0.59)(2.38)(25)$$

$$Q_{i_2} = 35.1 \text{ cfs}$$

4. Determine the 2-year required storage volume for the 2-year critical storm duration, T_{d_2}, using **Equation 5-1**:

$$V_2 = \left[Q_{i_2}T_{d_2} + \frac{Q_{i_2}t_c}{4} - \frac{q_{o_2}T_{d_2}}{2} - \frac{3q_{o_2}t_c}{4}\right]60$$

where: V_2 = *2-year required storage, ft^3*
Q_{i_2} = *2-year peak inflow for critical storm = 35.1 cfs*
C = *developed runoff coefficient = .59*
A = *area = 25.0 acres*
T_{d_2} = *critical storm duration = 36.2 min.*
t_c = *developed condition time of concentration = 21 min.*
q_{o_2} = *2-year allowable peak outflow = 17 cfs*

$$V_2 = \left|(35.1)(36.2) + \left(\frac{(35.1)(21)}{4}\right) - \left(\frac{(17)(36.2)}{2}\right) - \frac{3(17)(21)}{4}\right| 60$$

$$V_2 = 52{,}764 \text{ ft}^3 = 1.21 \text{ ac.ft.}$$

Repeat Steps 2 through 4 for the 10-year storm, as follows:

5. Determine the 10-year critical storm duration $T_{d_{10}}$, using **Equation 5-5** as follows:

$$T_{d_{10}} = \sqrt{\frac{2(.59)(25.0)(189.2)(22.1-21/4)}{24}} - 22.1$$

$$T_{d_{10}} = \sqrt{3918.6} - 22.1$$

$$T_{d_{10}} = 40.5 \ min.$$

Where:
$T_{d_{10}}$ = *10-year critical storm duration, min.*
C = *developed condition runoff coefficient = .59*
A = *drainage area = 25 acres*
t_c = *post-developed time of concentration = 21 min.*
$q_{o_{10}}$ = *24 cfs*
a_{10} = *189.2*
b_{10} = *22.1*

6. Solve for the 10-year critical storm duration intensity, I_{10}, using **Equation 5-2**, and the 10-year critical storm duration, $T_{d_{10}}$.

$$I_{10} = \frac{a}{b+T_{d_{10}}}$$

$$I_{10} = \frac{189.2}{22.1 + 40.5} = 3.02 \ in./hr.$$

7. Determine the 10-year peak inflow, $Q_{i_{10}}$, using the **Rational Equation** and the critical storm duration intensity I_{10}:

$$Q_{i_{10}} = C \, I_{10} \, A$$

Where: $Q_{i_{10}}$ = *10-year peak inflow*
C = *developed condition runoff coefficient = .59*
I_{10} = *critical storm intensity = 3.02 in./hr.*
A = *drainage area = 25.0 ac.*

$$Q_{i_{10}} = (.59)(3.02)(25.0)$$

$$Q_{i_{10}} = 44.5 \text{ cfs}$$

8. Determine the required 10-year storage volume for the 10-year critical storm duration, $T_{d_{10}}$, using **Equation 5-1**:

$$V_{10} = \left[Q_{i_{10}} T_{d_{10}} + \frac{Q_{i_{10}} t_c}{4} - \frac{q_{o_{10}} T_{d_{10}}}{2} - \frac{3 q_{o_{10}} t_c}{4} \right] 60$$

Where: V_{10} = *required storage, ft*^{3}
$Q_{i_{10}}$ = *44.5 cfs*
C = *.59*
A = *25 ac.*
$T_{d_{10}}$ = *40.5 min.*
t_c = *21 min.*
$q_{o_{10}}$ = *24 cfs*

$$V_{10} = \left[(44.5)(40.5) + \frac{(44.5)(21)}{4} - \frac{(24)(40.5)}{2} - \frac{3(24)(21)}{4} \right] 60$$

$$V_{10} = 70{,}308 \text{ ft}^3 = 1.61 \text{ ac.ft.}$$

V_2 and V_{10} represent the total storage volume required for the 2-year and 10-year storms, respectively. **Table 5-6** provides a summary of the four different sizing procedures used in this chapter, as applied to **Example 1**. The engineer should choose one of these methods based on the complexity and size of the watershed and the chosen hydrologic method. Using the stage-storage curve, a multi-stage riser structure can then be designed to control the appropriate storms and, if required, the water quality volume.

TABLE 5 - 6
Summary of Results: Storage Volume Requirement Estimates, Example 1

Method	2-*yr*. Storage Required	10-*yr*. Storage Required
Graphical Hydrograph Analysis	*0.82 ac.ft.*	*1.84 ac.ft.*
TR-55 Shortcut Method	*1.05 ac.ft.*	*1.96 ac.ft.*
Modified Rational Method - Critical Storm Duration - Graphical Solution	*1.16 ac.ft.*	*1.56 ac.ft.*
Modified Rational Method - Critical Storm Duration - Direct Solution	*1.21 ac.ft.* T_d = *36.2 min.*	*1.61 ac.ft.* T_d = *40.5 min.*

5-5 STAGE-STORAGE CURVE

By using one of the above methods for determining the storage volume requirements, the engineer now has sufficient information to place and grade the proposed stormwater facility. Remember, **this is a preliminary sizing which needs to be refined during the actual design**. By trial and error, the approximate required volume can be achieved by designing the basin to fit the site geometry and topography. The storage volume can be computed by planimetering the contours and creating a ***stage-storage curve***.

5-5.1 Storage Volume Calculations

For retention/detention basins with vertical sides, such as tanks and vaults, the storage volume is simply the bottom surface area times the height. For basins with graded (2H:1V, 3H:1V, etc.) side slopes or an irregular shape, the stored volume can be computed by the following procedure. **Figure 5-10** represents the stage-storage computation worksheet completed for **Example 1**. A blank worksheet can be found at the end of this chapter (see **Figure 5-27**). (Note that other methods for computing basin volumes are available, such as the Conic Method for Reservoir Volumes, but they are not presented here.)

Procedure:

1. Planimeter or otherwise compute the area enclosed by each contour and enter the measured value into Columns 1 and 2 of **Figure 5-10**. The invert of the lowest control orifice represents zero storage. This will correspond to the bottom of the facility for extended-detention or detention facilities, or the permanent pool elevation for retention basins.

2. Convert the planimetered area (often in square inches) to units of square feet in Column 3 of **Figure 5-10**.

3. Calculate the average area between each contour.

   The average area between two contours is computed by adding the area planimetered for the first elevation, column 3, to the area planimetered for the second elevation, also Column 3, and then dividing their sum by 2. This average is then written in Column 4 of **Figure 5-10**.

   From **Figure 5-10**:

   Average area, elevation *81-82:* $\frac{0 + 1800}{2} = 900\ ft^2$.

   Average area, elevation *82-84*: $\frac{1800 + 3240}{2} = 2{,}520\ ft^2$.

   Average area, elevation *84-86:* $\frac{3240 + 5175}{2} = 4{,}207\ ft^2$.

   This procedure is repeated to calculate the average area found between any two consecutive contours.

4. Calculate the *volume* between each contour by multiplying the average area from step 3 (Column 4) by the contour interval and placing this product in Column 6. From **Figure 5-10**:

   Contour interval between *81* and $82 = 1\ ft. \times 900\ ft^2 = 900\ ft^3$
   Contour interval between *82* and $84 = 2\ ft. \times 2{,}520\ ft^2 = 5{,}040\ ft^3$

   This procedure is repeated for each measured contour interval.

FIGURE 5 - 10
Stage-Storage Computation Worksheet, Example 1

PROJECT: **EXAMPLE 1** SHEET ____ OF ____

COUNTY: ________ COMPUTED BY: ________ DATE: ________

DESCRIPTION: ________

ATTACH COPY OF TOPO: SCALE - 1" = **30** *ft.*

1	2	3	4	5	6	7	8
ELEV.	AREA *(in^2)*	AREA *(ft^2)*	AVG. AREA *(ft^2)*	INTERVAL	VOL. *(ft^3)*	TOTAL VOLUME *(ft^3)*	TOTAL VOLUME *(ac.ft.)*
81	0	0				0	0
			900	1	900		
82	2.0	1800				900	.02
			2520	2	5040		
84	3.6	3240				5940	.14
			4207	2	8414		
86	5.75	5175				14354	.33
			7614	2	15228		
88	11.17	10053				29582	.68
			12991	2	25982		
90	17.7	15930				55564	1.28
			20700	2	41400		
92	28.3	25470				96964	2.23
			31102	1	31102		
93	40.8	36734				128066	2.94
			38105	1	38105		
94	43.9	39476				166171	3.81

5. Sum the volume for each contour interval in Column 7. Using **Figure 5-10,** this is simply the sum of the volumes computed in the previous step:

   Contour *81*, *Volume = 0*

   Contour *82, Volume = 0 + 900 = 900* ft^3

   Contour *84, Volume = 900 + 5,040 = 5,940* ft^3

   Contour *86*, *Volume = 5,940 + 8,414 = 14,354* ft^3

   Column 8 allows for the volume to be tabulated in units of acre-feet: $ft^3 \div 43{,}560\ ft^2/ac.$

   This procedure is then repeated for each measured contour interval.

6. Plot the stage-storage curve with *stage* on the y-axis versus *storage* on the x-axis. **Figure 5-11** represents the stage-storage curve for **Example 1** in units of feet (stage) versus acre-feet (storage).

The stage-storage curve allows the designer to estimate the *design high water elevation* for each of the design storms if the required storage volume has been determined. This allows for a preliminary design of the riser orifice sizes and their configuration.

5-6 WATER QUALITY AND CHANNEL EROSION CONTROL VOLUME CALCULATIONS

Virginia's Stormwater Management Regulations require that the first flush of runoff, or the water quality volume, be treated to enhance water quality. The ***water quality volume*** (V_{wq}) is the first 0.5 inches of runoff from the impervious area of development. The water quality volume must be treated
using one or a combination of BMP's depending on the total size of the contributing watershed, amount of impervious area, and site conditions. (Refer to **Chapters 2** and **3** for **BMP Selection Criteria** and **BMP Minimum Standards and Specifications**, respectively.)

The water quality volume is calculated as follows:

$$V_{wq}\ (ft^3) = \text{Impervious area}\ (ft^2) \times (\tfrac{1}{2}\ in.) / (12\ in./ft.)$$

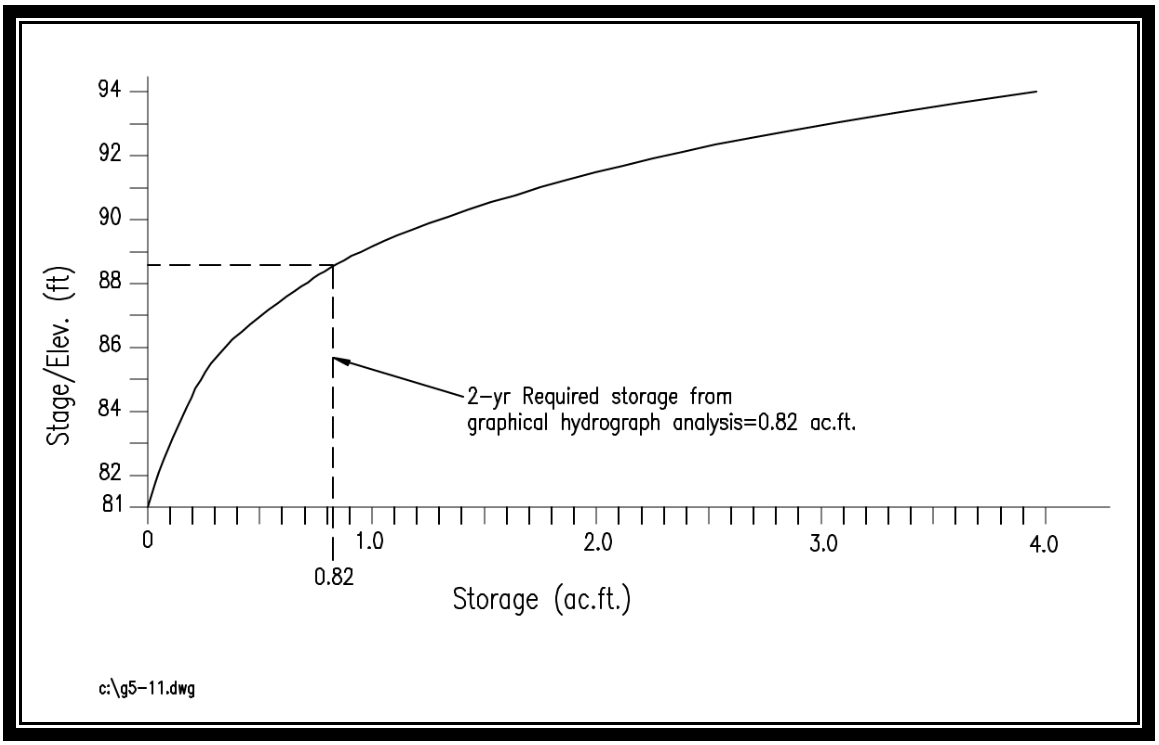
$$V_{wq}\ (ac.ft.) = V_{wq}\ (ft^3) / 43{,}560\ ft^2/ac.$$

The water quality volume for a wet BMP may be dependent on the specific design criteria for that BMP based on the watershed's imperviousness or the desired pollutant removal efficiency (using performance-based or technology-based criteria, respectively). The design criteria for each of the

BMPs, including extended-detention and retention basins, infiltration devices, constructed wetlands, marshes, etc., are presented in **Chapter 3**. This discussion is focused on the calculations associated with the control of the water quality volume in extended-detention and retention basins.

Virginia's Stormwater Management Regulations allow for the control of downstream channel erosion by detaining a specified volume of runoff for a period of time. Specifically, 24-hour extended detention of the runoff from the 1-year frequency storm is proposed as an alternate criteria to the 2-year peak rate reduction specified in MS-19 of the Virginia Erosion and Sediment Control Regulations, and the channel erosion component of the Virginia Stormwater Management Regulations. The channel erosion control volume (V_{ce}) is calculated by first determining the depth of runoff (in inches) based on the fraction of rainfall to runoff (runoff curve number) and then multiplying the runoff depth by the drainage area to be controlled. This procedure will be discussed in **5-6.3**.

FIGURE 5 - 11
Stage vs. Storage Curve, Example 1



5-6.1 Retention Basins - Water Quality Volume

The permanent pool feature of a retention basin allows for settling of particulate pollutants, such as sediment and other pollutants that attach adsorb to these particulates. Therefore, it is essential that the volume of the pool be both large enough and properly configured to prevent ***short-circuiting***. (Short-circuiting results when runoff enters the pool and exits without sufficient time for the settling process to occur.)

The permanent pool, or "dead" storage volume, of a retention facility is a function of the water quality volume. For example, a permanent pool sized to contain four times the water quality volume provides greater pollutant removal capacity than a permanent pool sized to contain two times the water quality volume. **Chapter 3** provides the pollutant removal efficiencies for various permanent pool sizes and criteria for permanent pool geometry.

Example 1 analyzes a 25-acre watershed. The water quality volume and permanent pool volume calculations for a retention basin serving this watershed are as follows:

Procedure

1. Calculate the water quality volume, V_{wq} , for the given watershed.

   From **Example 1**, the commercial/industrial development disturbs *11.9* acres, with *9.28* acres (*404,236 ft^2.*) of impervious cover after development.

$$V_{wq} = 404{,}236\ ft^2 \times ½\ in. / 12\ in./ft.$$

$$= 16{,}843\ ft^3$$

$$= 16{,}843\ ft^3/43{,}560\ ft^2/ac.$$

$$V_{wq} = 0.38\ ac.ft.$$

2. Size the permanent pool based on the desired *pollutant removal efficiency* or the drainage area *impervious cover*.

   The pool volume will be sized based upon the desired pollutant removal efficiency. Referring to **Table 3.06-1**, the permanent pool must be sized for 4 x V_{wq} for a pollutant removal efficiency of 65%.

$$\textit{Permanent Pool Volume} = V_{wq} \times 4.0$$

$$\textit{Permanent Pool Volume} = 0.38\ ac.ft. \times 4.0 = 1.52\ ac.ft.$$

5-6.2 Extended-Detention Basins - Water Quality Volume and Orifice Design

A ***water quality extended-detention basin*** treats the water quality volume by detaining it and releasing it over a specified amount of time. In theory, extended-detention of the water quality volume will allow the particulate pollutants to settle out of the *first flush* of runoff, functioning similarly to a permanent pool. Virginia's Stormwater Management Regulations pertaining to water quality specify a 30-hour draw down time for the water quality volume. This is a *brim draw down* time, beginning at the time of peak storage of the water quality volume. Brim-draw down time means the time required for the entire calculated volume to drain out of the basin. This assumes that the brim volume is present in the basin prior to any discharge. In reality, however, water is flowing out of the basin prior to the full or brim volume being reached. Therefore, the extended detention orifice can be sized using either of the following methods:

1. Use the *maximum hydraulic head* associated with the water quality volume (V_{wq}) and calculate the orifice size needed to achieve the required draw down time, and route the the water quality volume through the basin to verify the actual storage volume used and the drawdown time.

2. Approximate the orifice size using the *average hydraulic head* associated with the water quality volume (V_{wq}) and the required draw down time.

The two methods for calculating the required size of the extended detention orifice allow for a quick and conservative design (Method 2 above) and a similarly quick estimation with a routing to verify the performance of the design (Method 1).

Method 1, which uses the *maximum hydraulic head* and maximum discharge in the calculation, results in a slightly larger orifice than the same procedure using the *average hydraulic head* (Method 2). The routing allows the designer to verify the performance of the calculated orifice size. As a result of the routing effect however, the actual basin storage volume used to achieve the draw down time will be less than the computed brim draw down volume. It should be noted that the routing of the extended detention of the runoff from storms larger than the water quality storm (such as the 1-year frequency storm for channel erosion control) will result in proportionately larger reduction in the actual storage volume needed to achieve the required extended detention. (Refer to **Section 5-6.3** for the extended detention design procedures for channel erosion protection.)

The procedure used to size an extended detention orifice includes the first steps of the design of a multi-stage riser for a basin controlling water quality and/or channel erosion, and peak discharge. These steps are repeated for sizing the 2-year and 10-year release openings. Other design storms may be used as required by ordinance or downstream conditions.

Method 1: Water quality orifice design using maximum hydraulic head and routing of the water quality volume.

A water quality extended-detention basin sized for two times the water quality volume will be used here to illustrate the sizing procedure for an extended-detention orifice.

Procedure:

1. Calculate the water quality volume, V_{wq}, required for treatment.

   From **Example 1**:

   $V_{wq} = 404{,}236\ ft^2 \times ½\ in/\ 12\ in/ft = 16{,}843\ ft^3$

   $V_{wq} = 16{,}843\ ft^3/\ 43{,}560\ ft^2/ac = 0.38\ ac.ft.$

   For extended-detention basins, $2 \times V_{wq} = 2(0.38\ ac.ft.) = 0.76\ ac.ft. = 33{,}106\ ft^3$.

2. Determine the maximum hydraulic head, h_{max}, corresponding to the required water quality volume.

   From the **Example 1** stage vs. storage curve (**Figure 5-11**):

   0.76 *ac.ft.* occurs at elevation 88 *ft*. (approximate). Therefore, $h_{max} = 88 - 81 = 7.0\ ft.$

3. Determine the maximum discharge, Q_{max}, resulting from the 30-hour drawdown requirement.

   The *maximum discharge* is calculated by dividing the required volume, in ft^3, by the required time, in seconds, to find the average discharge, and then multiplying by 2, to determine the maximum discharge.

   From **Example 1**:

$$Q_{avg} = \frac{33{,}106\ ft^3}{(30\,hr.)\,(3{,}600\,sec./hr.)} = 0.30\ cfs$$

$$Q_{max} = 2 \times 0.30\ cfs = 0.60\ cfs$$

4. Determine the required orifice diameter by rearranging the **Orifice Equation, Equation 5-6** to solve for the orifice area, in ft^2, and then diameter, in *ft.*

   Insert the values for Q_{max} and h_{max} into the **Rearranged Orifice Equation, Equation 5-7** to solve for the orifice area, and then solve for the orifice diameter.

$$Q = Ca\sqrt{2gh}$$

Equation 5-6
Orifice Equation

$$a = \frac{Q}{C\sqrt{2gh}}$$

Equation 5-7
Rearranged Orifice Equation

where: Q = *discharge, cfs*
C = *dimensionless coefficient = 0.6*
a = *area of the orifice, ft^2*
g = *gravitational acceleration, 32.2 ft/sec^2*
h = *head, ft.*

From **Example 1**:

For orifice area:

$$a = \frac{0.6}{0.6\sqrt{(2)(32.2)(7.0)}}$$

For orifice diameter:

$$a = 0.047\ ft^2 = \pi r^2 = \pi d^2/4$$

$$d = \sqrt{\frac{4a}{\pi}} = \sqrt{\frac{4(0.047\ ft^2)}{\pi}}$$

d = orifice diameter = 0.245 ft = 2.94"

Use a 3-inch diameter water quality orifice.

Routing the water quality volume (V_{wq}) of 0.76 *ac.ft.,* occurring at elevation 88 feet, through a 3-inch water quality orifice will allow the designer to verify the draw down time, as well as the maximum elevation of 88 feet.

<u>Route the water quality volume</u>.

This calculation will give the engineer the *inflow-storage-outflow relationship* in order to verify the actual storage volume needed for the extended detention of the water quality volume. The routing procedure takes into account the discharge that occurs before maximum or *brim* storage of the water quality volume, as opposed to the brim drawdown described in Method 2. The routing procedure is simply a more accurate analysis of the storage volume used while water is flowing into and out of the basin. Therefore, the actual volume of the basin used will be less than the volume as defined by the regulation. This procedure will come in handy if the site to be developed is tight and the area needed for the stormwater basin must be "squeezed" as much as possible.

The routing effect of water entering and discharging from the basin simultaneously will also result in the actual drawdown time being less than the calculated 30 hours. Judgement should be used to determine whether the orifice size should be reduced to achieve the required 30 hours or if the actual time achieved will provide adequate pollutant removal.

NOTE: The designer will notice a significant reduction in the actual storage volume used when routing the extended detention of the runoff from the 1-year frequency storm (channel erosion control). Please refer to **Chapter 5-6.3** and **Chapter 5-11** for the appropriate design procedures when extended detention is provided for channel erosion control.

Routing the water quality volume depends on the ability to work backwards from the design runoff volume of 0.5 inches to find the rainfall amount. Using SCS methods, the rainfall needed to generate 0.5 inches of runoff from an impervious surface (*RCN=98*) is 0.7 inches. The SCS design storm is the Type II, 24-hour storm. Therefore, the *water quality storm* using SCS methods is defined as the SCS Type II, 24-hour storm, with a rainfall depth = 0.7 inches.

The rational method does not provide a design storm derived from a specified rainfall depth. Its rainfall depth depends on the storm duration (watershed t_c) and the storm return frequency. **Since the water quality storm varies with runoff amount, not the design storm return frequency, an input runoff hydrograph representing the water quality volume cannot be generated using rational method parameters**. Therefore Method 1, routing of the water quality volume, must use SCS methods. See **Chapter 4** for details on SCS methods.

Continuing with **Example 1**, the procedure is as follows:

Procedure (contd.)**:**

5. Calculate a stage-discharge relationship using the **Orifice Equation, Equation 5-6** and the orifice size determined in Step 4.

From **Example 1**, using the 3-inch diameter orifice, the calculation is as follows:

$$Q = Ca\sqrt{2gh}$$

Orifice Equation 5-6

$$Q = 0.6(.047)\sqrt{(2)(32.2)(h)}$$

$$Q = 0.22\sqrt{h}$$

where: h = *water surface elevation minus the orifice's centerline elevation*, in ft.*

Note: If the orifice size is small relative to the anticipated head, h, values of h may be defined as the water surface elevation minus the invert of the orifice elevation.

7. Complete a stage-discharge table for the range of elevations in the basin, as shown in **Table 5-7**:

TABLE 5 - 7
Stage-Discharge Table: Water Quality Orifice Design

Elevation	*h* (*ft*)	*Q* (*cfs*)
81	0	0
82	1	0.2
83	2	0.3
84	3	0.4
85	4	0.4
86	5	0.5
87	6	0.5
88	7	0.6

8. Determine the time of concentration as defined in **Chapter 4** for the impervious area.

   From **Example 1**, the developed time of concentration, $t_c = 0.46$ *hours*. The impervious area time of concentration, $t_{c_{imp}}$ = 0.09 hours, or 5.4 minutes.

9. Using $t_{c_{imp}}$, the stage-discharge relationship, the stage-storage relationship, and the impervious acreage (*RCN = 98*), route the water quality storm through the basin. The water quality storm for this calculation is the SCS Type 2, 24-hour storm, rainfall depth = 0.7 inches. (Note that the rainfall depth is established as the amount of rainfall required to generate 0.5 inches of runoff from the impervious area.)

   The water quality volume may be routed using a variety of computer programs such as TR-20, HEC-1, or other storage indication routing programs. Alternatively, it can be routed by hand using the storage indication routing procedure outlined in **Section 5-9** of this chapter.

10. Evaluate the discharge hydrograph to verify that the drawdown time from maximum storage to zero discharge is at least 30 hours. (Note that the maximum storage corresponds to the maximum rate of discharge on the discharge hydrograph.)

    The routing of the water quality volume using TR-20 results in a maximum storage elevation is 85.69 ft. versus the approximated 88.0 ft. The brim drawdown time is 17.5 hours (peak discharge occurs at 12.5 hours and .01 discharge occurs at 30 hours). For this example, the orifice size may be reduced to provide a more reasonable drawdown time and another routing performed to find the new water quality volume elevation.

METHOD 2: Water quality orifice design using *average hydraulic head* and *average discharge*.

The procedure described in Method 2 is presented in the next section. For the previous example, Method 2 results in a 2.5 inch orifice (versus a 3.0 inch orifice), and the design extended detention water surface elevation is set at 88 ft.(versus 85.69ft.). (It should be noted that trial two of Method 1 as noted above may result in a design water surface elevation closer to 88 ft.) If the basin is to control additional storms, such as the 2-year and/or 10-year storms, the additional storage volume would be "stacked" just above the water quality volume. The invert for the 2-year control, for example, would be set at 88.1 *ft.*

5-6.3 Extended-Detention Basins - Channel Erosion Control Volume and Orifice Design

Extended detention of a specified volume of stormwater runoff can also be incorporated into a basin design to protect downstream channels from erosion. Virginia’s Stormwater Management Regulations recommend 24-hour extended detention of the runoff from the 1-year frequency storm as an alternative to the 2-year peak rate reduction required by MS-19 of the Virginia Erosion and Sediment Control Regulations. A full discussion of this channel erosion criteria will be presented in a future Technical Bulletin, along with practical guidance from DCR on the effective implementation of the criteria. The discussion presented here is for the design of a channel erosion control extended-detention orifice.

The design of a channel erosion control extended-detention orifice is similar to the design of the water quality orifice in that two methods can be employed:

1. Use the *maximum hydraulic head* associated with the specified channel erosion control (V_{ce}) storage volume and calculate the orifice size needed to achieve the required draw down time and route the 1-year storm through the basin to verify the storage volume and the draw down time, or

2. Approximate the orifice size using the *average hydraulic head* associated with the channel erosion control volume (V_{ce}) and draw down time.

The routing procedure takes into account the discharge that occurs before maximum or *brim* storage of the channel erosion control volume (V_{ce}). The routing procedure simply provides a more accurate accounting of the storage volume used while water is flowing into and out of the basin, and results in less storage volume being used than the calculated brim storage volume associated with the maximum hydraulic head. The actual storage volume needed for extended detention of the runoff generated by the 1-year frequency storm will be approximately 60 percent of the calculated volume (V_{ce}) of runoff for curve numbers between 75 and 95 and time of concentration between 0.1 and 1 hour.

The following procedure illustrates the design of the extended-detention orifice for channel erosion control. Refer to **Chapter 6** for **Example 6.2** which includes the design of an extended-detention orifice for channel erosion control, Method 1, within the design of a multi-stage riser.

Method 2:

Procedure

1. Calculate the channel erosion control volume, V_{ce}.

   Determine the rainfall amount (inches) of the 1-year frequency storm for the local area where the project is located (**Appendix 4B**). With the rainfall amount and the runoff curve number (RCN), determine the corresponding runoff depth using the runoff Equation (**Chapter 4: Hydrologic Methods - SCS** TR-55) or the Rainfall - Runoff Depth Charts (**Appendix 4C**).

   From **Example 2**:

   1-year rainfall = 2.7 inches, RCN = 75; using **Appendix 4C**, the 1-year frequency depth of runoff = 0.8 inches, therefore:

   $$V_{ce} = 25\ ac. \times 0.8\ in. \times 1'/12'' = 1.66\ ac.ft.$$

   To account for the routing effect, reduce the channel erosion control volume:

   $$V_{ce} = (0.6)(1.66\ ac.ft.) = 1.0\ ac.ft. = 43{,}560\ ft.^3$$

2. Determine the *average hydraulic head*, h_{avg}, corresponding to the required channel erosion control volume.

   From **Example 2** - Stage - Storage Curve: 1.0 *ac.ft.* occurs at elevation 89.0 *ft.* Therefore,

   $$h_{avg} = (89 - 81) / 2 = 4.0\ ft.$$

3. Determine the *average discharge*, Q_{avg}, resulting from the 24-hour draw down requirement.

   The average discharge is calculated by dividing the required volume, in ft^3, by the required time, in seconds, to find the average discharge.

   From **Example 2**:

$$Q_{avg} = \frac{43{,}560\ ft^3}{(24\,hr.)(3{,}600\,sec./hr.)} = 0.5\ cfs$$

4. Determine the required orifice diameter by rearranging the **Orifice Equation, Equation 5-6** to solve for the orifice area, in ft^2, and then diameter, in *ft.*

   Insert the values for Q_{avg} and h_{avg} into the **Rearranged Orifice Equation, Equation 5-7** to solve for the orifice area, and then solve for the orifice diameter.

$$Q = Ca\sqrt{2gh}$$

Equation 5-6
Orifice Equation

$$a = \frac{Q}{C\sqrt{2gh}}$$

Equation 5-7
Rearranged Orifice Equation

where: Q = *discharge, cfs*
C = *dimensionless coefficient = 0.6*
a = *area of the orifice,* ft^2
g = *gravitational acceleration, 32.2* ft/sec^2
h = *head, ft.*

From **Example 2**:

For orifice area:

$$a = \frac{0.5}{0.6\sqrt{(2)(32.2)(4.0)}}$$

$$a = 0.052\ ft^2 = \pi r^2 = \pi d^2/4$$

For orifice diameter:

$$d = \sqrt{\frac{4a}{\pi}} = \sqrt{\frac{4(0.052\ ft^2)}{\pi}}$$

d = orifice diameter = 0.257 ft = 3.09 inches

Use 3.0-inch diameter channel erosion extended detention orifice

The use of Method 1, utilizing the maximum hydraulic head and a routing of the 1-year storm is illustrated in **Chapter 6: Example 6.2**. Methd 1 results in a 3.7" diameter orifice and a routed water surface elevation of 88.69 ft. Additional storms to may be "stacked" just above this volume if additional controls are desired.

5-7 MULTI-STAGE RISER DESIGN

A principal spillway system that controls the rate of discharge from a stormwater facility will often use a multi-stage riser for the drop inlet structure.

A multi-stage riser is a structure that incorporates separate openings or devices at different elevations to control the rate of discharge from a stormwater basin during multiple design storms. Permanent multi-stage risers are typically constructed of concrete to help increase their life expectancy; they can be precast or cast-in-place. The geometry of risers will vary from basin to basin. The engineer can be creative to provide the most economical and hydraulically efficient riser design possible. **Figure 3-02.1** in **Chapter 3** provides some examples of multi-stage riser structures.

In a stormwater management basin design, the multi-stage riser is of utmost importance since it controls the design water surface elevations. In designing the multi-stage riser, many iterative routings are usually required to arrive at a minimum structure size and storage volume that provides proper control. Each iterative routing requires that the facility's size (*stage-storage* curve) and outlet shape (*stage-discharge table* or *rating curve*) be designed and tested for performance. Prior to final design, it is helpful to approximate the required storage volume and outlet shape using one of the "shortcut" methods, as described in **Section 5-4**. In doing this, the number of iterations may be reduced. The following procedures outline methods for approximating and then completing the design of a riser structure. (These design procedures are illustrated in the examples found in **Chapter 6**.)

Information needed:

1. The hydrology for the watershed or drainage area to be controlled, calculated by using one of the methods outlined in **Chapter 4**, and

2. The allowable release rates for the facility, as established by ordinance or downstream conditions.

The design procedure provided here will incorporate the traditional 2-year and 10-year design storms and the pre-developed hydrology will establish the allowable discharge rates of the developed watershed. It should be noted that any design storm, 1-year, 5-year, etc., can be substituted into this design procedure, as required.

Procedure:

<u>STEP 1</u> Determine Water Quality or Extended Detention Requirements

Calculate the water quality volume and decide what method (extended-detention or retention) will be used to treat it, and/or calculate the channel erosion control volume for extended-detention, if required. (Virginia's Stormwater Management Regulations state that the **water quality volume is equal to the first 0.5 inch of runoff multiplied by the total impervious area of the land development project, and that the channel erosion control volume for extended detention is the runoff generated by the site during the 1-year frequency storm**.)

a. **Water Quality Extended-Detention Basin**: The water quality volume must be detained and released over 30-hours. The established pollutant removal efficiency is based on a 30-hour drawdown.

b. **Water Quality Retention Basin**: The volume of the permanent pool is established by the site impervious cover or the desired pollutant removal efficiency.

c. **Channel Erosion Control Extended-Detention Basin**: The channel erosion control volume must be detained and released over 24 hours.

Refer to **Chapter 3** for minimum BMP design standards and details.

<u>STEP 2</u> Compute Allowable Release Rates

Compute the pre- and post-developed hydrology for the watershed. Sometimes, the pre-developed hydrology will establish the allowable release rate from the basin. Other times, the release rate will be established by downstream conditions. In either case, the post-developed hydrology will provide the peak inflow into the basin, as a peak rate (*cfs*) or a runoff hydrograph. Refer to **Section 5-3, Allowable Release Rates**.

<u>STEP 3</u> Estimate the Required Storage Volume

Estimate the storage volume required using one of the "shortcut" volume estimate methods described in **Section 5-4**. The information required includes the developed condition peak rate of runoff, or runoff hydrograph, and the allowable release rates for each of the appropriate design storms.

STEP 4 **Grade the Basin; Create Stage-Storage Curve**

After considering the site geometry and topography, select a location for the proposed stormwater management basin. By trial and error, size the basin such that it will hold the approximate required storage volume. Ensure that the storage volume is measured from the lowest stage outlet. (Note: the storage volume can be computed by planimetering the contours and creating a stage-storage relationship as described in **Section 5-5**.) Remember that this is a preliminary sizing which needs to be fine-tuned during the final design.

STEP 5a **Design Water Quality Orifice (Extended-Detention)**

The procedure for sizing the water quality orifice for an extended-detention basin is covered in **Section 5-6.2** of this chapter. Using either Method 1 or Method 2, the designer establishes the size of the water quality or stream channel erosion control orifice and the design maximum water surface elevation.

The lowest stage outlet of an extended-detention basin is the invert of the extended-detention (or water quality) orifice, which corresponds to zero storage. **Section 5-6.2** provides a detailed discussion for sizing the water quality orifice and **Chapter 6** gives examples of the calculation procedure.

STEP 5b **Set Permanent Pool Volume (Retention)**

In a retention pond, the permanent pool volume, from **STEP 1**, establishes the lowest stage outlet for the riser structure (not including a pond drain, if provided). The permanent pool elevation, therefore, corresponds to "0" storage for the design of the "dry" storage volume stacked on top of the permanent pool.

STEP 6 **Size 2-Year Control Orifice**

(The 2-year storm is used here to show the design procedure. Other design storms or release requirements can be substituted into the procedure.)

Knowing the 2-year storm storage requirement, from design **STEP 3**, and the water quality volume, from design **STEP 1**, the engineer can do a preliminary design for the 2-year release opening in the multi-stage riser. To complete the design, some iterations may be required to meet the allowable release rate performance criteria. This procedure is very similar to the water quality orifice sizing calculations:

1. Approximate the 2-year maximum head, $h_{2_{max}}$.

Establish the approximate elevation of the 2-year maximum water surface elevation using the stage-storage curve and the preliminary sizing calculations. Subtract the water quality volume elevation from the approximate 2-year maximum water surface elevation to find the 2-year maximum head, $h_{2_{max}}$. If there are no water quality requirements, use the elevation of the basin bottom or invert.

2. Determine the maximum allowable 2-year discharge rate, $Q_{2_{allowable}}$, from **STEP 2**.

3. Calculate the size of the 2-year control release orifice using the **Rearranged Orifice Equation, Equation 5-7** and solve for the area, a, in ft^2.

The engineer may choose to use any one of a variety of orifice shapes or geometries. Regardless of the selection, the orifice will initially act as a weir until the top of the orifice is submerged. Therefore, the discharges for the first stages of flow are calculated using the weir equation:

$$Q_w = C_w L h^{1.5}$$

Equation 5-8
Weir Equation

where:
- Q_w = *weir flow discharge, cfs*
- C_w = *dimensionless weir flow coefficient, typically equal to 3.1 for sharp crested weirs. Refer to **Table 5-8**.*
- L = *length of weir crest, ft.*
- h = *head, ft., measured from the water surface elevation to the crest of the weir*

Flow through the rectangular opening will transition from *weir flow* to *orifice flow* once the water surface has risen above the top of the opening. This orifice flow is expressed by the orifice equation. The area, a, of a rectangular orifice is written as $a = L \times H$,

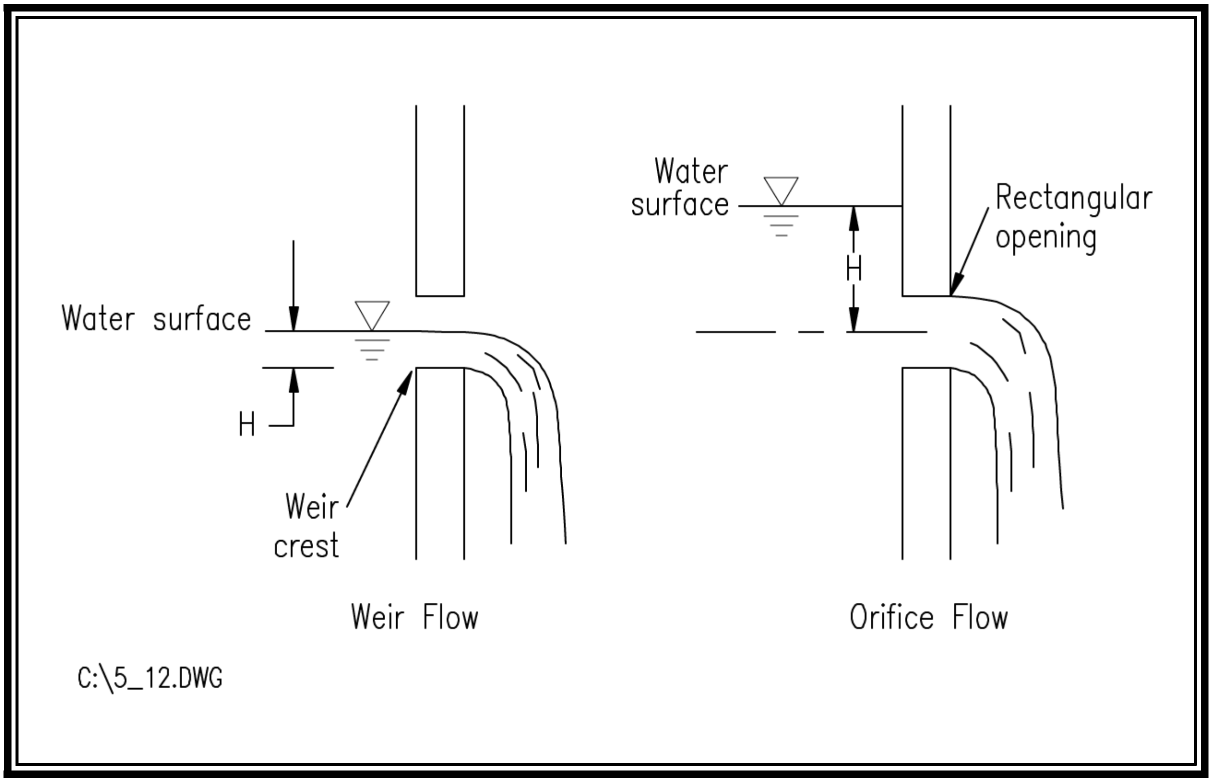
where:
- L = *length of opening, ft.*
- H = *height of opening, ft.*

Figure 5-12 shows a rectangular orifice acting as a weir at the lower stages and as an orifice after the water surface rises to height H, the height of the opening.

4. Develop the stage-storage-discharge relationship for the 2-year storm.

Calculate the discharge using the orifice equation and, if a rectangular opening is used, the weir equation as needed for each elevation specified on the stage-storage curve. Record the discharge on a Stage-Storage-Discharge Worksheet. **Figure 5-13** shows a completed Stage-Storage-Discharge Worksheet for **Example 2**. A blank worksheet is provided in **Appendix 5D**.

FIGURE 5 - 12
Weir and Orifice Flow



STEP 7 **Check Performance of 2-Year Opening**

(Note: This step may not be necessary if the design is to be completed using one of the shortcut routing procedures where the water surface elevations are established by the required storage volume and not by an actual routing.)

1. Check the performance of the 2-year control opening by a) *reservoir routing the 2-year storm through the basin using an acceptable reservoir routing computer program* or by b) *doing the long hand calculations outlined in **Section 5-9** of this chapter*. **Verify that the 2-year release rate is less than or equal to the allowable release rate**. If not, reduce the size of the opening or provide additional storage and repeat **STEP 6**.

This procedure presents just one of many riser configurations. The engineer may choose to use any type of opening geometry for controlling the design storms and, with experience, may come to recognize the most efficient way to configure the riser. Note that if a weir is chosen for the 2-year storm control, the procedures outlined here for the 10-year storm may be used by substituting with the appropriate values for the 2-year storm. Refer to **Figure 3-02.1** for several different riser shapes.

STEP 8 Size 10-Year Control Opening

The design of the 10-year storm control opening is similar to the procedure used in sizing the 2-year control opening:

1. From the routing results, identify the exact 2-year water surface elevation.

2. Set the invert of the 10-year control just above the 2-year design water surface elevation and determine the corresponding storage volume from the stage-storage curve. Add this elevation, storage, and 2-year discharge to the stage-storage-discharge worksheet, **Figure 5-13**.

> ***The 10-year control invert may be set at a small distance, such as 0.1 feet minimum, above the 2-year maximum water surface elevation. If the 2-year orifice is also to be used for the 10-year control, the head is measured from the maximum water surface elevation to the centerline of the 2-year orifice. See Figure 5-14.***

3. Establish the approximate 10-year maximum water surface elevation using the stage-storage curve and the preliminary sizing calculations. Subtract the invert elevation of the 10-year control (from Step 2 above) from the approximate 10-year maximum water surface elevation to find the 10-year maximum head, $h_{10_{max}}$.

4. Determine the maximum allowable 10-year discharge rate, $Q_{10_{allowable}}$, from **STEP 2**.

5. Calculate the required size of the 10-year release opening. The engineer may choose between a circular and rectangular orifice, or a weir. If a weir is chosen, the weir flow equation can be rearranged to solve for L as follows.

$$Q_W = C_W L\, h^{1.5}$$

Equation 5-8
Weir Equation

$$L = Q_{10_{allowable}} / C_W\, h^{1.5}$$

Equation 5-9
Rearranged Weir Equation

Where:
L = *length of weir required, ft.*
C_W = *dimensionless weir flow coefficient, see **Table 5-8***
$Q_{10_{allowable}}$ = *10-year allowable riser weir discharge, cfs*
h = *hydraulic head; water surface elevation minus the weir crest elevation*

6. Develop the stage-storage-discharge relationship for the 10-year storm. Calculate the discharge for each elevation specified on the stage-storage curve, and record the discharge on a Stage-Storage-Discharge Worksheet, as shown in **Figure 5-13**.

Any weir length lost to the trash rack or debris catcher must be accounted for. See Chapter 3 for Trash Rack Specifications and example riser configurations.

TABLE 5 - 8
Weir Flow Coefficients

WEIR FLOW COEFFICIENTS, C			
Measured head, *h*, (*ft.*)	**Breadth of weir crest (*ft.*)**		
	0.50	**0.75**	**1.00**
0.2	2.80	2.75	2.69
0.4	2.92	2.80	2.72
0.6	3.08	2.89	2.75
0.8	3.30	3.04	2.85
1.0	3.32	3.14	2.98
1.2	3.32	3.20	3.08
1.4	3.32	3.26	3.20
1.6	3.32	3.29	3.28
1.8	3.32	3.32	3.31
2.0	3.32	3.32	3.30
3.0	3.32	3.32	3.32
4.0	3.32	3.32	3.32
5.0	3.32	3.32	3.32

Source: Kings Handbook of Hydraulics

<u>STEP 9</u> Check Performance of 10-Year Opening

(Note: This step may not be necessary if the design is to be completed using one of the short-cut routing procedures where the water surface elevations are established by the required storage volume and not by an actual routing.)

Check the performance of the 10-year control opening by a) *reservoir routing the 2-year and 10-year storms through the basin using an acceptable reservoir routing computer program* (see **Appendix**) or by b) *doing the long hand calculations outlined in* ***Section 5-9***. **Verify that the 10-year release rate is less than or equal to the allowable release rate**. If not, reduce the size of the opening and/or provide additional storage and repeat **STEP 8**.

<u>STEP 10</u> Perform Hydraulic Analysis

At this point, several iterations may be required to calibrate and optimize the hydraulics of the riser and the riser and barrel system. Drop inlet spillways should be designed so that full flow is established in the outlet conduit and riser at the lowest head over the riser crest as is practical. Also, the structure should operate without excessive surging, noise, vibration, or vortex action at any stage. This requires the riser to have a larger cross-sectional area than the outlet conduit.

As the water passes over the rim of the riser, the riser acts as a weir (**Figure 5-15a**); this discharge is described as ***riser weir flow control***. However, when the water surface reaches a certain height over the rim of the riser, the riser will begin to act as a submerged orifice (**Figure 5-15b**); such discharge is called ***riser orifice flow control***. The engineer must compute the elevation at which this transition from riser weir flow control to riser orifice flow control takes place. (This transition usually occurs during high hydraulic head conditions, such as between the 10-*yr.* and 100-*yr.* design high water elevations.)

Note in **Figure 5-15a & b** that the riser crest controls the flow, not the barrel. Thus, either condition can be described as ***riser flow control***. **Figure 5-15c & d** illustrates ***barrel flow control***. Barrel flow control occurs when the barrel controls the flow at the upstream entrance to the barrel (*barrel inlet flow control*, **Figure 5-15c**), or along the barrel length (*barrel pipe flow control*, **Figure 5-15d**).

Barrel flow control conditions illustrated in **Figure 5-15c & d** are desirable because they reduce or even eliminate cavitation forces, or surging and vibration (as described above), in the riser and barrel system. Cavitation forces in the riser and barrel system can greatly reduce the design flow capacity of the system. Cavitation forces may also cause vibrations that can damage the riser (especially corrugated metal risers) and the connection between the riser and barrel. This connection may crack and lose its watertight seal. Additionally, if a concrete riser is excessively tall with a minimum amount of the riser secured in the embankment, the cavitation forces may cause the riser to rock on its foundation, risking possible structural failure.

The surging, vibrations, and other cavitation forces result when the riser is restricting flow to the barrel such that the riser is flowing full and the barrel is <u>not</u> flowing full. This condition occurs when the flow through the riser structure transitions from *riser weir flow control* to *riser orifice flow control* <u>before</u> the barrel controls. Therefore, the barrel and riser system should be designed so that as the storm continues and the hydraulic head on the riser increases, **the barrel controls the flow <u>before</u> the riser transitions from riser weir flow control to riser orifice flow control.** This can be accomplished by checking the flow rates for the riser weir, riser orifice, and barrel inlet and outlet flow control at each stage of discharge. **The lowest discharge for any given stage will be the controlling flow**.

The following procedures are for designing and checking riser and barrel system hydraulics.

a. Riser Flow Control

During the design of the control orifices and riser weir, the geometry of the riser is established. Subsequently, the riser must be checked to determine at what stage it transitions from *riser weir* to *riser orifice* flow control. The riser weir controls the flow initially, and then as the water rises, the top of the riser acts as a submerged horizontal orifice. Thus, the flow transitions from riser weir flow control to riser orifice flow control as the water in the basin rises. The flow capacity of the riser weir is determined using the **Weir Equation**, **Equation 5-8**, and the flow capacity of the riser orifice is determined using the **Orifice Equation**, **Equation 5-6**, for each elevation. **The smaller of the two flows for any given elevation is the controlling flow**.

1. Calculate the flow, in *cfs,* over the riser weir using the standard **Weir Equation**, **Equation 5-8**, for each elevation specified on the Stage-Storage-Discharge Worksheet, **Figure 5-13**. Record the flows on the worksheet.

The ***weir length***, *L*, is the circumference or length of the riser structure, measured at the crest, less any support posts or trash rack. The ***head*** is measured from the water surface elevation to the crest of the riser structure (refer to **Figure 5-14**).

2. Calculate the flow, in *cfs,* through the riser structure using the standard **Orifice Equation**, **Equation 5-6,** for each elevation specified on the Stage-Storage-Discharge Worksheet, **Figure 5-13**. Record the flows on the worksheet.

The ***Orifice flow area****, a,* is measured from the inside dimensions of the riser structure. The ***head*** is measured from the water surface elevation to the elevation of the orifice centerline, or, since the orifice is horizontal, to the elevation of the riser crest.

3. Compare the riser weir flow discharges to the riser orifice flow discharges. **The smaller of the two discharges is the controlling flow for any given stage**.

b. Barrel Flow Control

Two types of barrel flow exist: 1) ***barrel flow with inlet control***, as shown in **Figure 5-15c**, and 2) ***barrel flow with outlet, or pipe flow control***, as shown in **Figure 5-15d**. For both types, different factors and formulas are used to compute the hydraulic capacity of the barrel. During barrel inlet flow control, the diameter of the barrel, amount of head acting on the barrel, and the barrel entrance shape play a part in controlling the flow. For barrel outlet, or pipe flow, control, consideration is given to the length, slope, and roughness of the barrel, and the elevation of the tailwater, if any, in the outlet channel.

1. Barrel Inlet Flow Control

Barrel inlet flow control means that the capacity of the barrel is controlled at the barrel entrance by the depth of headwater and the barrel entrance, which is acting as a submerged orifice. The flow through the barrel entrance can be calculated using the **Orifice Equation, Equation 5-6,** or by simply using the Pipe Flow Nomograph shown in **Figure 5-16.** This nomograph provides stage-discharge relationships for concrete culverts of various sizes. [Additional nomographs for other pipe materials and geometrics are available; refer to the U.S. Bureau of Public Roads (BPR) Hydraulic Engineering Circular (H.E.C.) 5.] The ***headwater***, or depth of ponding, is the vertical distance measured from the water surface elevation to the invert at the entrance to the barrel. Refer to **Figure 5-16** for ratios of *headwater* to *pipe diameter*, or *HW/D*. This nomograph, based on the orifice equation, provides flow rates for three possible hydraulic entrance shapes, as shown in **Figure 5-17**. During barrel inlet flow control, neither the barrel's length nor its outlet conditions are factors in determining the barrel's capacity. Note that when the *HW/D* design values exceed the chart values, the designer may use the orifice equation (**Equation 5-6**) to solve for the flow rate.

The inlet control nomographs are not truly representative of barrel inlet flow. These nomographs should be used carefully and with the understanding that they were developed to predict flow through highway culverts operating under inlet control. However, depending on the size relationship between the riser and outlet conduit, the inlet control nomograph may provide a

The following procedure outlines the steps to calculate the discharge during ***barrel inlet flow*** control conditions:

1. Determine the *entrance condition* of the barrel (see **Figure 5-17**).

2. Determine the *headwater to pipe diameter ratio* (*HW/D*) for each elevation specified on the stage-storage-discharge worksheet. *Headwater* is measured from the water surface elevation to the upstream invert of the barrel (see **Figures 5-14 and 5-18**).

3. Determine the *discharge*, *Q,* in *cfs,* using the inlet control nomograph for circular concrete pipe presented in **Figure 5-16** (or the BPR H.E.C. 5 pipe flow nomographs for other pipe materials), or the **Orifice Equation**, **Equation 5-6** (for *HW/D* values which exceed the range of the nomographs) for each elevation specified on the Stage-Storage-Discharge Worksheet. Enter the values on the worksheet.

2. Barrel Outlet Flow Control

Barrels flowing under outlet or pipe flow control experience full flow for all or part of the barrel length, as shown in **Figure 5-15d**.

The general pipe flow equation is derived by using the Bernouli and Continuity Principles and is simplified to:

$$Q = a\sqrt{\frac{2gh}{1 + K_m + K_p L}}$$

Equation 5 - 10
Pipe Flow Control Equation

Where:

Q = *discharge, cfs*
a = *flow area of the barrel,* ft^2
g = *acceleration due to gravity,* $ft./sec^2$
h = *elevation head differential, ft., see* ***Figure 5-18***
K_m = *coefficient of minor losses:* $K_e + K_b$
K_e = *entrance loss coefficient, see* ***Table 5-9***
K_b = *bend loss coefficient, typically = 0.5 for riser and barrel system*
K_p = *coefficient of pipe friction, see* ***Table 5-10***
l = *length of the barrel, ft.*

This equation is derived and further explained in the SCS's Engineering Field Manual, Chapter 3.

The following procedure outlines the steps to check for ***barrel outlet control***:

1. Determine the discharge for each elevation specified in the stage-storage-discharge table using the general **Pipe Flow Equation**, **Equation 5-10**.

2. Record the discharge on the stage-storage-discharge worksheet, **Figure 5-13**.

3. Compare the barrel inlet flow control discharges with the barrel outlet flow control discharges. **The smaller of the two discharges is the controlling flow for any given stage**.

STEP 11 Size 100-Year Release Opening or Emergency Spillway

It is recommended that all stormwater impoundment structures have a vegetated emergency spillway, if possible. This provides a degree of safety to prevent overtopping of the embankment if the principal spillway should become clogged, or otherwise inoperative. **If an emergency spillway is not practical due to site constraints, the 100-year storm must be routed through the riser and barrel system**.

100-Year Release Opening

The design procedure for sizing the 100-year release opening is the same as that of the 10-year design, except that the 100-year storm values are used instead of the 10-year values.

Emergency Spillway

Refer to **Minimum Standard 3.03, Vegetated Emergency Spillway** in **Chapter 3** for location and design requirements of an emergency spillway and to **Section 5-8** in this chapter for the design procedure. An emergency spillway is a broad crested weir. It can act as a control structure by restricting the release of flow, or it can be used to safely pass the 100-year storm flow with a minimum of storage. The impact of the 100-year storm on the required storage is lessened by using an emergency spillway due to the spillway's ability to pass significant volumes of flow with little head. If an emergency spillway is not used, additional storage may be needed since the riser and barrel will usually pass only a small portion of the 100-year inflow. This remains true unless the riser and barrel are sized for the 100-year storm, in which case they will be oversized for the 2- and 10-year storms.

The following procedure can be used to design an emergency spillway that will safely pass, or control, the rate of discharge from the 100-year storm.

1. Identify the 10-year maximum water surface elevation based on the routing from **STEP 9**. This elevation will be used to establish the elevation of the 100-year release structure.

2. Determine the storage volume that corresponds to the 100-year control elevation from the stage-storage curve. Add this elevation, storage, and appropriate storm discharges to the Stage-Storage-Discharge Worksheet.

3. Set the invert of the emergency spillway at the 10-year high water elevation.

4. Determine the 100-year developed inflow from the hydrology.

> ***A distance of 0.1 feet, minimum, is recommended between the 10-year high water mark and the invert of the emergency spillway.***

5. Using the design procedure provided in **Chapter 5-8**, determine the required bottom width of the spillway, the length of the spillway level section, and the depth of flow through the spillway that adequately passes the 100-year storm within the available free board. The minimum free board required is 1 foot from the 100-year water surface elevation to the settled top of embankment.

6. Develop the stage-storage-discharge relationship for the 100-year storm. Calculate the

discharge for each elevation specified on the stage-storage curve and record the discharge on the Stage-Storage-Discharge Worksheet, **Figure 5-13**. If a release rate is specified, then the TR-55 shortcut method can be used to calculate the approximate storage volume requirement. If a fixed storage volume is available, the same shortcut method can be used to decide what the discharge must be to ensure that the available storage is not exceeded. Refer to TR-55.

STEP 12 Calculate Total Discharge and Check Performance of 100-Year Control Opening

1. Calculate total discharge.

   The stage-storage-discharge table is now complete and the total discharge from the riser and barrel system and emergency spillway can be determined. The designer should verify that the barrel flow controls before the riser transitions from riser weir flow control to riser orifice flow control.

   The combined flows from the water quality orifice, the 2-year opening, the 10-year opening, and the riser will, at some point, exceed the capacity of the barrel. At this water surface elevation and discharge, the system transitions from riser flow control to barrel flow control. The total discharge for each elevation is simply the sum of the flows through the control orifices of the riser, or the controlling flow through the barrel and riser, whichever is **less**.

   In **Chapter 6,** the examples contain completed Stage-Storage-Discharge Worksheets. Notice that the flows that do not control are crossed out. The controlling flows are then summed in the total flow column to provide the total stage-storage-discharge relationship of the basin.

2. Check the performance of the 100-year control by a) *reservoir routing the 2-year, 10-year, and 100-year storms through the basin using an acceptable reservoir routing computer program* or by b) *doing the long hand calculations outlined in* ***Section 5-9***. **Verify that the design storm release rates are less than or equal to the allowable release rates, and that the 100-year design high water is:**

   a. at least 2 *ft.* lower than the settled top of embankment elevation if an emergency spillway is NOT used, or

   b. at least 1 *ft.* lower than the settled top of embankment if an emergency spillway is used.

Also, the designer should verify that the release rates for each design storm are not too low, which would result in more storage being provided than is required.

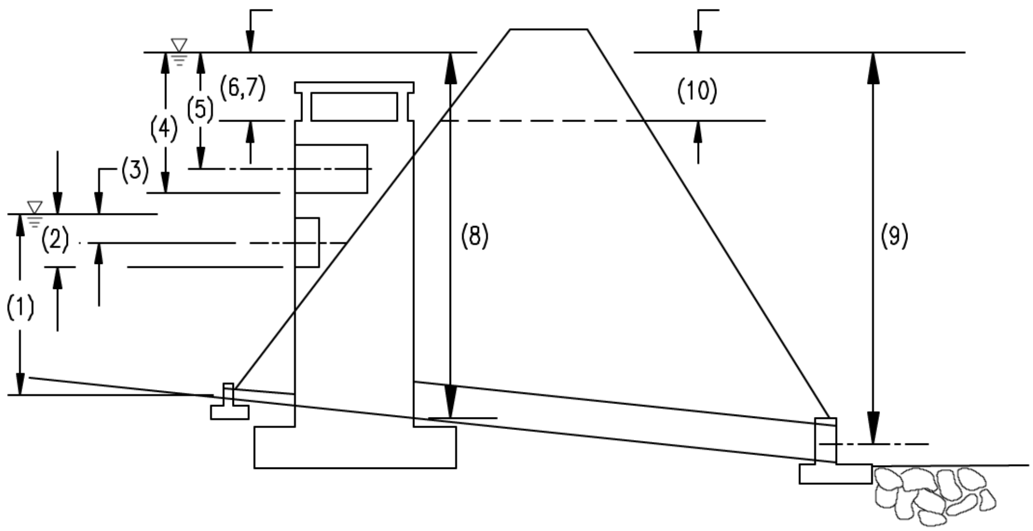
FIGURE 5 - 13
Stage - Storage - Discharge Worksheet, Example 1

ELEV (MSL)	STORAGE (ac.ft.)	1-yr. ext. det. ~~WATER QUALITY~~ ORIFICE□ (1)		TRIAL 2 10-YEAR CONTROL WEIR (4)		ORIFICE (5)		RISER STRUCTURE WEIR (6)		ORIFICE (7)		BARREL INLET (8)		OUTLET (9)		EMERGENCY SPILLWAY (10)		TOTAL Q (cfs)
		h	Q	h	Q	h	Q	h	Q	h	Q	HW/D	Q	h	Q	h	Q	
81	0	0	0															0
82	.02	1	0.3															0.3
84	.14	3	0.6															0.6
86	.33	5	0.7															0.7
88	.68	7	0.9															0.9
88.8	.90	7.8	0.98	0	0				0/0	0	0							1.0
89	.95	8	1.0	0.2	0.4				0/0.4	0.2	~~34.4~~	5.5	~~24~~	8.3	~~22.0~~			1.4
90	1.28	9	1.1	1.2	5.8				0/5.8	1.2	~~84.4~~	6.2	~~26~~	9.3	~~23.3~~			6.9
91	1.75	10	1.1	2.2	14.3				0/14.3	2.2	~~114.3~~	6.8	~~27~~	10.3	~~24.5~~			15.4
92	2.23	11	~~1.1~~	3.2	~~25.1~~				0/25.1	3.2	~~139.8~~	7.5	~~28~~	11.3	25.7			25.7
92.2	2.40	11.2	~~1.1~~	3.4	~~27.5~~			0	0/27.5	3.4	~~142.0~~	7.6	~~29~~	11.5	25.9	0	0	25.9
93	2.94	12.0	~~1.1~~	4.2	~~37.8~~			0.8	17.0/54.8	4.2	~~157.9~~	8.1	~~30~~	12.3	26.8	0.8	54	80.8
94	3.81	13.0	~~1.2~~	5.2	~~52.0~~			1.8	57.5/109.5	5.2	~~175.7~~	8.8	~~31~~	13.3	27.8	1.8	214	241.8

(6) Q for elevations 88.8 to 92.2 ft. represents 10-yr. weir flow (4)
Q for elevations 92.2 to 94 ft represents 10-yr weir flow plus the top of riser weir flow: $Q = 23.8\,(h)^{1.5}$ where h = wse - 92.2 ft.

(7) $Q = 77.03\,(h)^{1/2}$ where h = wse - 88.8 ft
(8) $Q = 8.5\,(h)^{1/2}$ where h = wse - 80.75 ft
(9) $Q = 7.64\,(h)^{1/2}$ where h = wse - 80.75 ft
(10) Figure 5-23: Design Data for Earth Spillways.

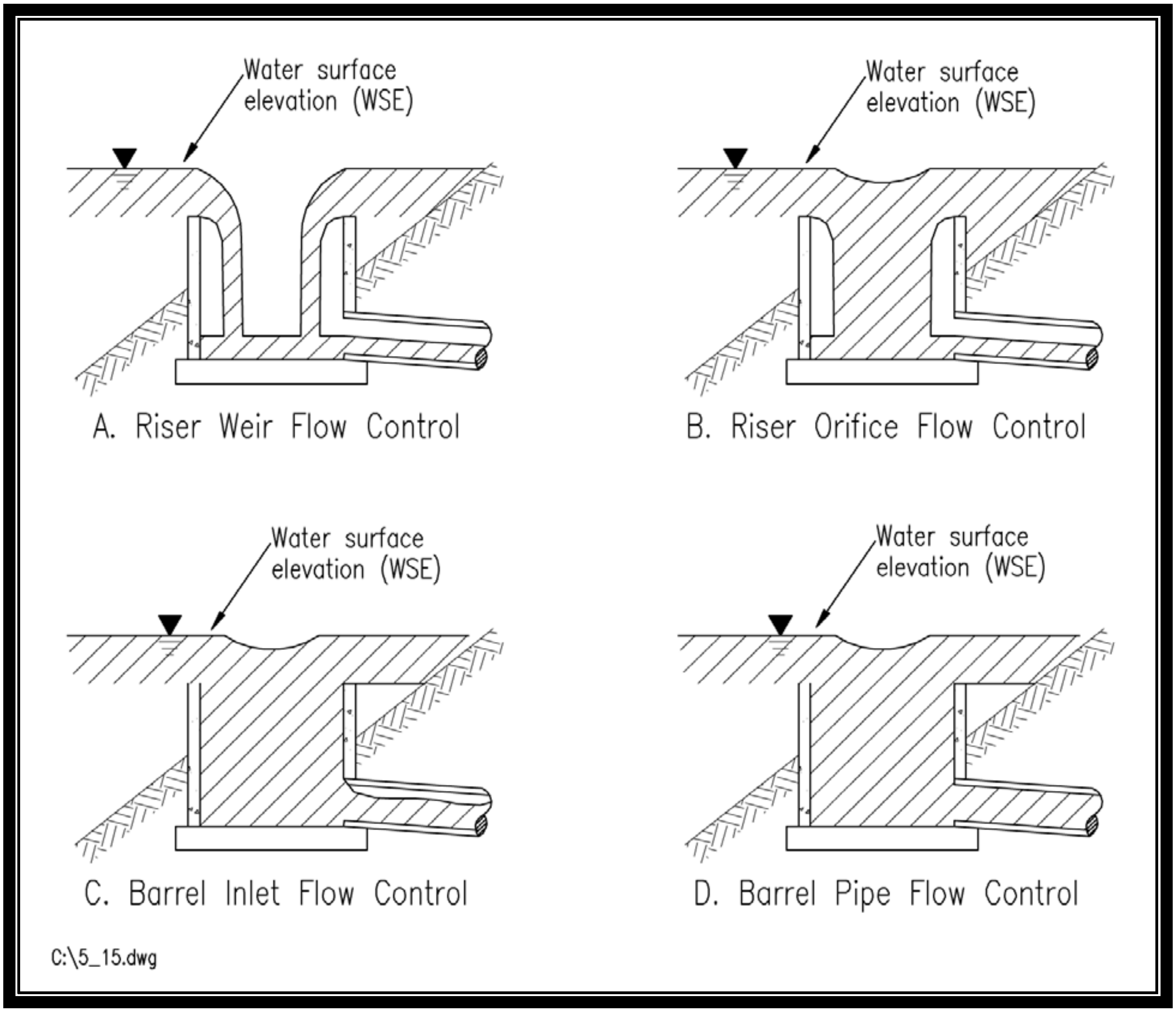
FIGURE 5 - 14
Typical Hydraulic Head Values - Multi-Stage Riser



1. WQ Orifice: H measured from water surface elevation (WSE) to centerline of pipe, or orifice. (orifice flow)
2. 2–year Control: Weir flow – H measured from WSE to invert of 2–year control weir.
3. 2–year Control: Orifice flow – H measured from WSE to centerline of opening (submerged).
4. 10–year Control: Weir flow – H measured from WSE to crest of opening.
5. 10–year Control: Orifice flow – H measured from WSE to centerline of opening.
6. Riser Structure: Weir flow – H measured from WSE to crest of riser top (if open).
7. Riser Structure: Orifice flow – H measured from WSE to crest of riser top, acting as horizontal orifice.
8. Barrel flow: Inlet control – H measured from WSE to upstream invert of outlet barrel.
9. Barrel flow: Outlet control – H measured from WSE to centerline of outlet of barrel or tailwater – whichever is higher.
10. Emergency Spillway: H measured from WSE to crest of Emergency Spillway.

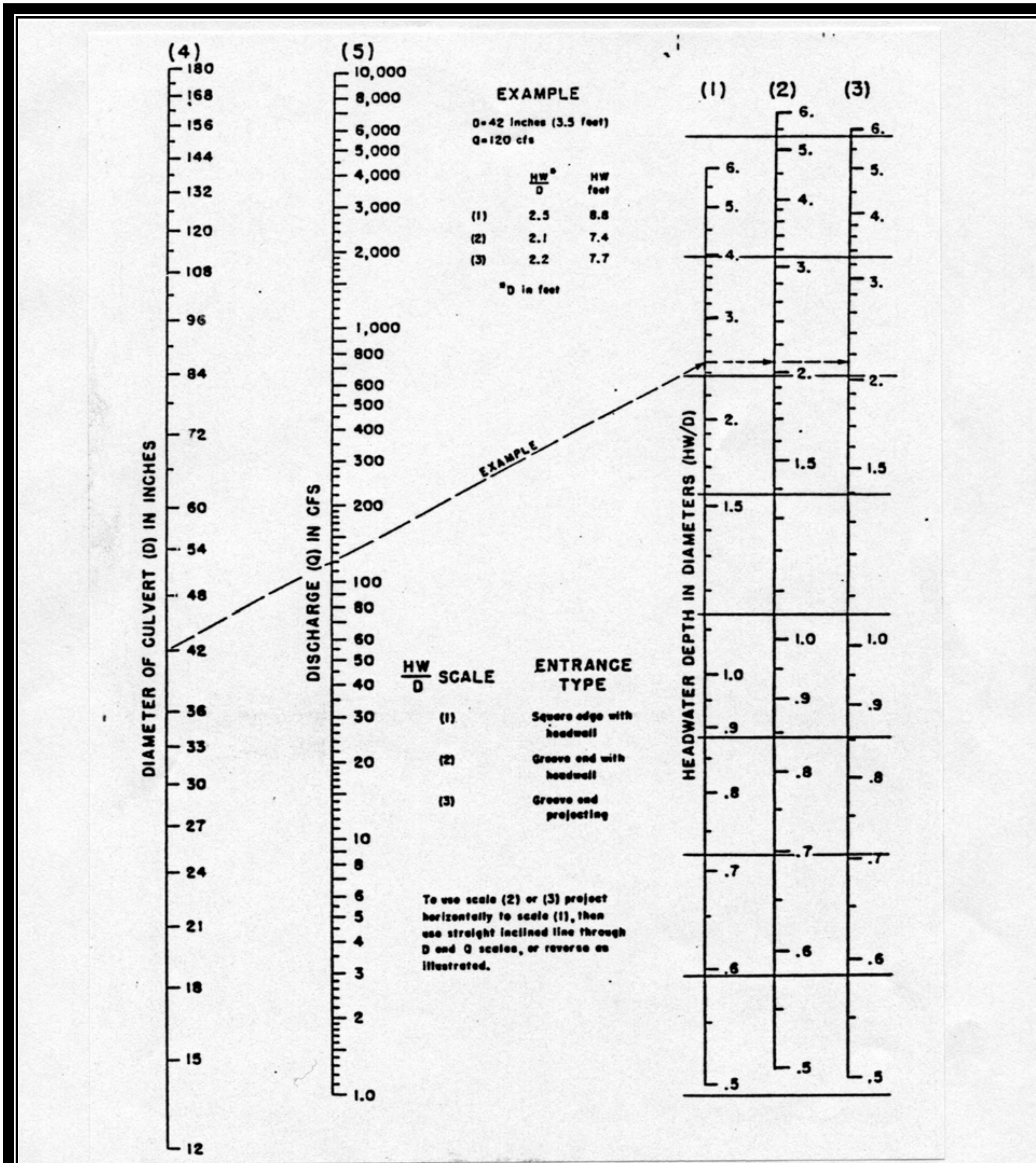
c:\5_14.dwg

FIGURE 5 - 15 a, b, c, & d
Riser Flow Diagrams



Source: SCS Engineering Field Manual - Chapter 6

FIGURE 5 - 16
Headwater Depth for Concrete Pipe Culverts With Inlet Control



Source: Bureau of Public Roads

FIGURE 5 - 17
Headwater Depth Entrance Conditions

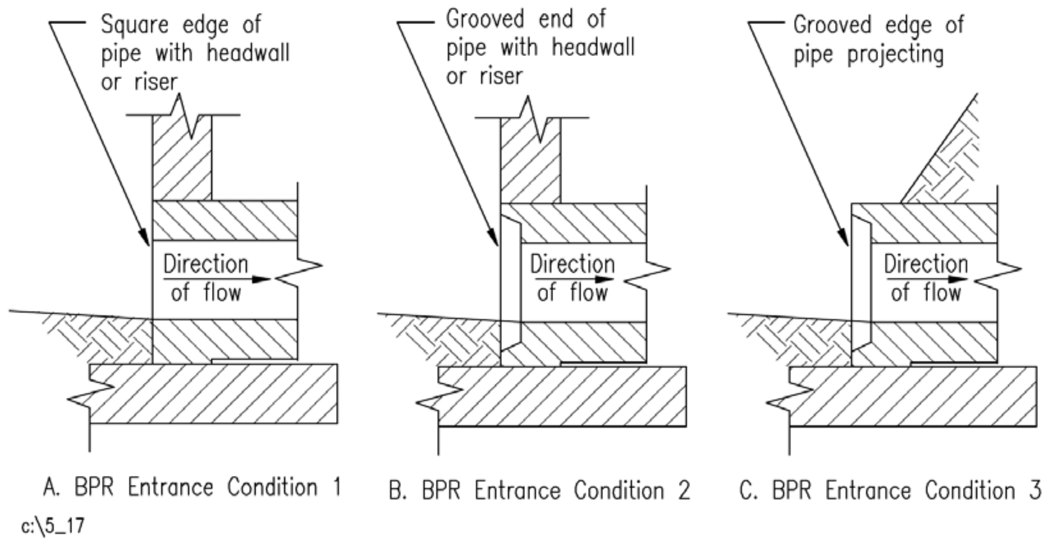


FIGURE 5 - 18
Hydraulic Head Values - Barrel Flow

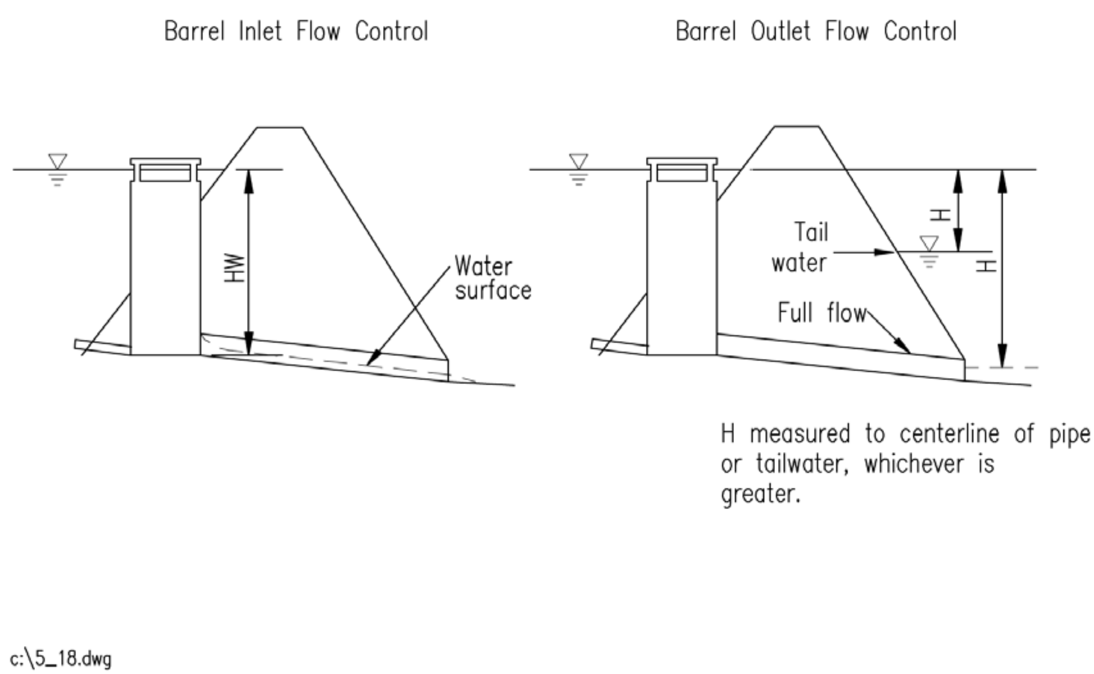


TABLE 5 - 9
Pipe Entrance Loss Coefficients - K_e

Type of Structure and Design of Entrance	Coefficient K_e
Pipe, Concrete	
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove end)	0.2
Square end	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End-section conforming to fill slope	0.5
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls	
Square end	0.5
Mitered to conform to fill slope	0.7
*End-section conforming to fill slope	0.5

**Note: "End-section conforming to fill slope" made of either metal or concrete, is the section commonly available from manufacturers. Based on limited hydraulic tests, it appears to be equivalent in operation to a headwall in either inlet or outlet control.*

Source - Federal Highway Administration, Bureau of Public Roads

TABLE 5 - 10

Head Loss Coefficients, K_p, for Circular and Square Conduits Flowing Full

HEAD LOSS COEFFICIENT, K_p, FOR CIRCULAR PIPE FLOWING FULL $K_p = \frac{5087 n^2}{d_i^{4/3}}$

Pipe diam. inches	Flow area sq. ft.	MANNING'S COEFFICIENT OF ROUGHNESS "n"															
		0.010	0.011	0.012	0.013	0.014	0.015	0.016	0.017	0.018	0.019	0.020	0.021	0.022	0.023	0.024	0.025
6	0.196	0.0467	0.0565	0.0672	0.0789	0.0914	0.1050	0.1194	0.1348	0.151	0.168	0.187	0.206	0.226	0.247	0.269	0.292
8	0.349	.0318	.0385	.0458	.0537	.0623	.0715	.0814	.0919	.1030	.1148	.1272	.140	.154	.168	.183	.199
10	0.545	.0236	.0286	.0340	.0399	.0463	.0531	.0604	.0682	.0765	.0852	.0944	.1041	.1143	.1249	.136	.148
12	0.785	.0185	.0224	.0267	.0313	.0363	.0417	.0474	.0535	.0600	.0668	.0741	.0817	.0896	.0980	.1067	.1157
14	1.069	.0151	.0182	.0217	.0255	.0295	.0339	.0386	.0436	.0488	.0544	.0603	.0665	.0730	.0798	.0868	.0942
15	1.23	.0138	.0166	.0198	.0232	.0270	.0309	.0352	.0397	.0446	.0496	.0550	.0606	.0666	.0727	.0792	.0859
16	1.40	.0126	.0153	.0182	.0213	.0247	.0284	.0323	.0365	.0409	.0455	.0505	.0556	.0611	.0667	.0727	.0789
18	1.77	.01078	.0130	.0155	.0182	.0211	.0243	.0276	.0312	.0349	.0389	.0431	.0476	.0522	.0570	.0621	.0674
21	2.41	.00878	.01062	.0126	.0148	.0172	.0198	.0225	.0254	.0284	.0317	.0351	.0387	.0425	.0464	.0506	.0549
24	3.14	.00735	.00889	.01058	.0124	.0144	.0165	.0188	.0212	.0238	.0265	.0294	.0324	.0356	.0389	.0423	.0459
27	3.98	.00628	.00760	.00904	.01061	.0123	.0141	.0161	.0181	.0203	.0227	.0251	.0277	.0304	.0332	.0362	.0393
30	4.91	.00546	.00660	.00786	.00922	.01070	.01228	.0140	.0158	.0177	.0197	.0218	.0241	.0264	.0289	.0314	.0341
36	7.07	.00428	.00518	.00616	.00723	.00839	.00963	.01096	.0124	.0139	.0154	.0171	.0189	.0207	.0226	.0246	.0267
42	9.62	.00348	.00422	.00502	.00589	.00683	.00784	.00892	.01007	.01129	.0126	.0139	.0154	.0169	.0184	.0201	.0218
48	12.57	.00292	.00353	.00420	.00493	.00572	.00656	.00747	.00843	.00945	.01053	.01166	.0129	.0141	.0154	.0168	.0182
54	15.90	.00249	.00302	.00359	.00421	.00488	.00561	.00638	.00720	.00808	.00900	.00997	.01099	.0121	.0132	.0144	.0156
60	19.63	.00217	.00262	.00312	.00366	.00426	.00487	.00554	.00626	.00702	.00782	.00866	.00955	.01048	.0115	.0125	.0135

HEAD LOSS COEFFICIENT, K_c, FOR SQUARE CONDUIT FLOWING FULL $K_c = \frac{29.16 n^2}{r^{4/3}}$

Conduit Size feet	Flow area sq. ft.	MANNING'S COEFFICIENT OF ROUGHNESS "n"				
		0.012	0.013	0.014	0.015	0.016
2×2	4.00	0.01058	0.01242	0.01440	0.01653	0.01880
2½×2½	6.25	0.00786	0.00922	0.01070	0.01228	0.01397
3×3	9.00	.00616	.00723	.00839	.00963	.01096
3½×3½	12.25	.00502	.00589	.00683	.00784	.00892
4×4	16.00	.00420	.00493	.00572	.00656	.00746
4½×4½	20.25	.00359	.00421	.00488	.00561	.00638
5×5	25.00	.00312	.00366	.00425	.00487	.00554
5½×5½	30.25	.00275	.00322	.00374	.00429	.00488
6×6	36.00	.00245	.00287	.00333	.00382	.00435
6½×6½	42.25	.00220	.00258	.00299	.00343	.00391
7×7	49.00	.00199	.00234	.00271	.00311	.00354
7½×7½	56.25	.00182	.00213	.00247	.00284	.00323
8×8	64.00	.00167	.00196	.00227	.00260	.00296
8½×8½	72.25	.00154	.00180	.00209	.00240	.00273
9×9	81.00	.00142	.00167	.00194	.00223	.00253
9½×9½	90.25	.00133	.00156	.00180	.00207	.00236
10×10	100.00	.00124	.00145	.00168	.00193	.00220

$$H_1 = (K_p \text{ or } K_c)\, L\, \frac{v^2}{2g}$$

Nomenclature:

- a = Cross-sectional area of flow in sq. ft.
- d_i = Inside diameter of pipe in inches.
- g = Acceleration of gravity = 32.2 ft. per sec.
- H_1 = Loss of head in feet due to friction in length L.
- K_c = Head loss coefficient for square conduit flowing full.
- K_p = Head loss coefficient for circular pipe flowing full.
- L = Length of conduit in feet.
- n = Manning's coefficient of roughness.
- Q = Discharge or capacity in cu. ft. per sec.
- r = Hydraulic radius in feet.
- v = Mean velocity in ft. per sec.

Example 1: Compute the head loss in 300 ft. of 24 in. diam. concrete pipe flowing full and discharging 30 c.f.s. Assume $n = 0.015$

$$v = \frac{Q}{a} = \frac{30}{3.14} = 9.55 \text{ f.p.s.}; \quad \frac{v^2}{2g} = \frac{(9.55)^2}{64.4} = 1.42 \text{ ft.}$$

$$H_1 = K_p L \frac{v^2}{2g} = 0.0165 \times 300 \times 1.42 = 7.03 \text{ ft.}$$

Example 2: Compute the discharge of a 250 ft., 3×3 square conduit flowing full if the loss of head is determined to be 2.25 ft. Assume $n = 0.014$.

$$H_1 = K_c L \frac{v^2}{2g}; \quad \frac{v^2}{2g} = \frac{H_1}{K_c L} = \frac{2.25}{0.00839 \times 250} = 1.073 \text{ ft.}$$

$$v = \sqrt{64.4 \times 1.073} = 8.31; \quad Q = 9 \times 8.31 = 74.8 \text{ c f s.}$$

Head loss coefficients for circular and square conduits flowing full (Ref. NEH Section 5, ES-42)

STEP 13 **Design Outlet Protection**

With the total discharge known for the full range of design storms, adequate outlet protection can now be designed. Protection is necessary to prevent scouring at the outlet and to help reduce the potential for downstream erosion by reducing the velocity and energy of the concentrated discharge. The most common form of outlet protection is a riprap-lined apron, constructed at zero grade for a specified distance, as determined by the outlet flow rate and tailwater elevation. The design procedure follows:

> ***Note that this procedure is for riprap outlet protection at the downstream end of an embankment conduit. It DOES NOT apply to continuous rock linings of channels or streams. Refer to Figure 5 -19.***

1. Determine the tailwater depth, for the appropriate design storm, immediately below the discharge pipe.

Typically, the discharge pipe from a stormwater management facility is sized to carry the allowable discharge from the 10-year frequency design storm. Manning's equation can be used to find the water surface elevation in the receiving channel for the 10-year storm, which represents the ***tailwater elevation***. If the tailwater depth is less than half the outlet pipe diameter, it is called a ***minimum tailwater condition***. If the tailwater depth is greater than half the outlet pipe diameter, it is called a ***maximum tailwater condition***. Stormwater basins that discharge onto flat areas with no defined channel may be assumed to have a *minimum tailwater condition.*

> ***Outflows from stormwater management facilities must be discharged to an adequate channel. Basins discharging onto a flat area with no defined channel will usually require a channel to be provided which can convey the design flows.***

2. Determine the required riprap size, D_{50}, and apron length, L_a.

Enter the appropriate figure, either **Figure 5-20: Minimum Tailwater Condition**, or **Figure 5-21: Maximum Tailwater Condition**, with the design discharge of the pipe spillway to read the required apron length, L_s. (The apron length should not be less than 10 feet.)

3. Determine the required riprap apron width, W.

When the pipe discharges directly into a well-defined channel, the apron shall extend across the channel bottom and up the channel banks to an elevation 1 foot above the maximum tailwater depth or the top of bank, whichever is less.

If the pipe discharges onto a flat area with no defined channel, the width of the apron shall be determined as follows:

a. The upstream end of the apron, next to the pipe, shall be 3 times wider than the diameter of the outlet pipe.

b. For a ***minimum tailwater condition***, the width of the apron's downstream end shall equal the pipe diameter plus the length of the apron.

c. For a ***maximum tailwater condition***, the width of the apron's downstream end shall equal the pipe diameter plus 0.4 times the length of the apron.

Using the same figure as in Step 2, above, determine the D_{50} riprap size and select the appropriate class of riprap, as shown in **Table 5-11**. Values falling between the table values should be rounded up to the next class size.

4. Determine the required depth of the rip rap blanket.

   The depth of the rip rap blanket is approximated as: 2.25 x D_{50}

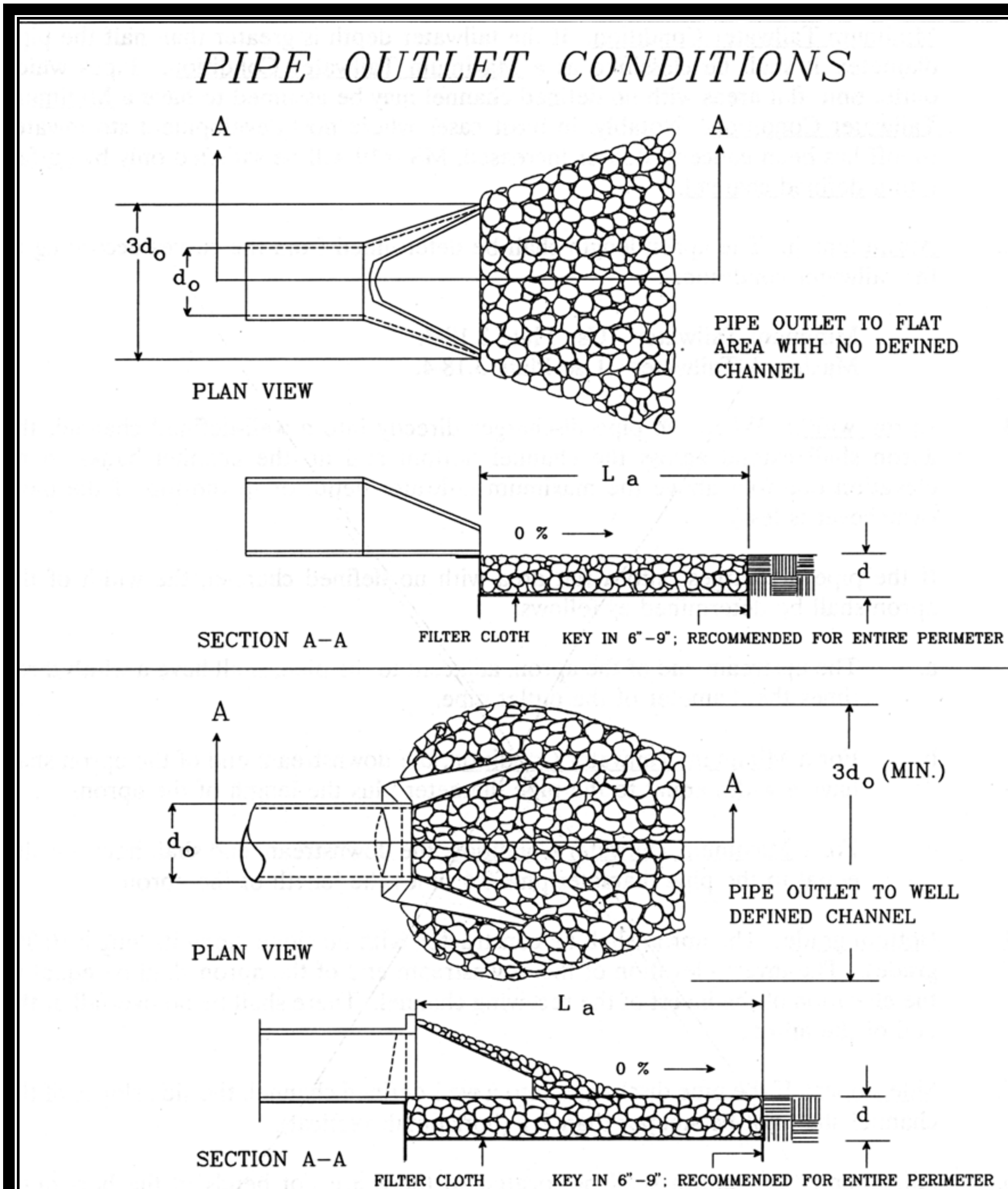
Additional design considerations and specifications can be found in **Minimum Standard 3.02, Principal Spillway** and Std. and Spec. 3.18 and 3.19 of the Virginia Erosion and Sediment Control Handbook, 1992 edition.

TABLE 5 - 11
Graded Riprap Design Values

Riprap Class	D_{15} Weight *(lbs.)*	Mean D_{15} Spherical Diameter *(ft.)*	Mean D_{50} Spherical Diameter *(ft.)*
Class AI	25	0.7	0.9
Class I	50	0.8	1.1
Class II	150	1.3	1.6
Class III	500	1.9	2.2
Type I	1,500	2.6	2.8
Type I	6,000	4.0	4.5

Source: VDOT Drainage Manual

FIGURE 5 - 19
Outlet Protection Detail



NOTES:
1. APRON LINING MAY BE RIPRAP, GROUTED RIPRAP, GABION BASKET, OR CONCRETE.
2. La IS THE LENGTH OF THE RIPRAP APRON AS CALCULATED USING FIG 5-20 AND FIG 5-21.
3. d = 1.5 TIMES THE MAXIMUM STONE DIAMETER, BUT NOT LESS THAN 6 INCHES.

FIGURE 5 - 20
Minimum Tailwater Condition

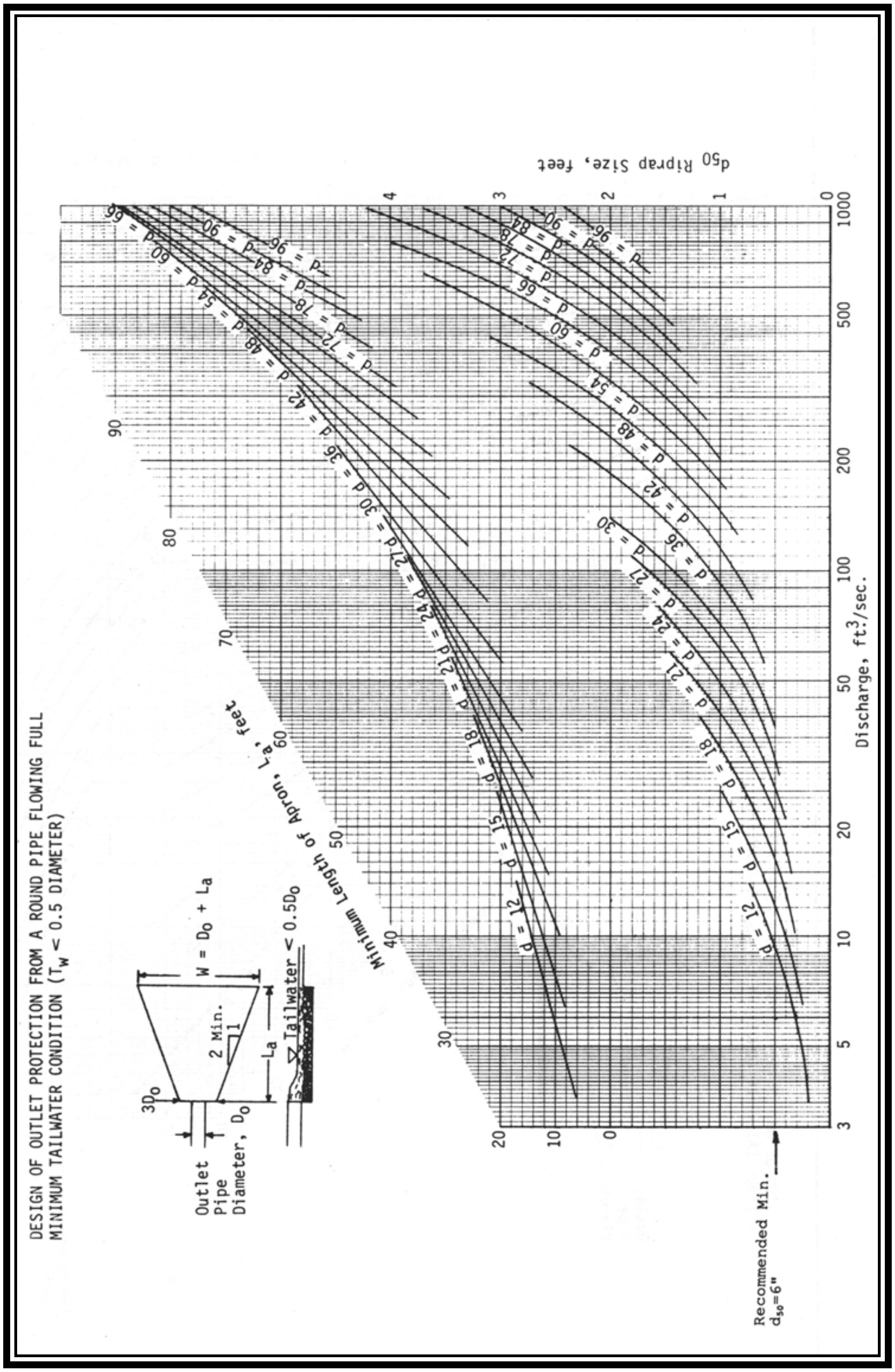
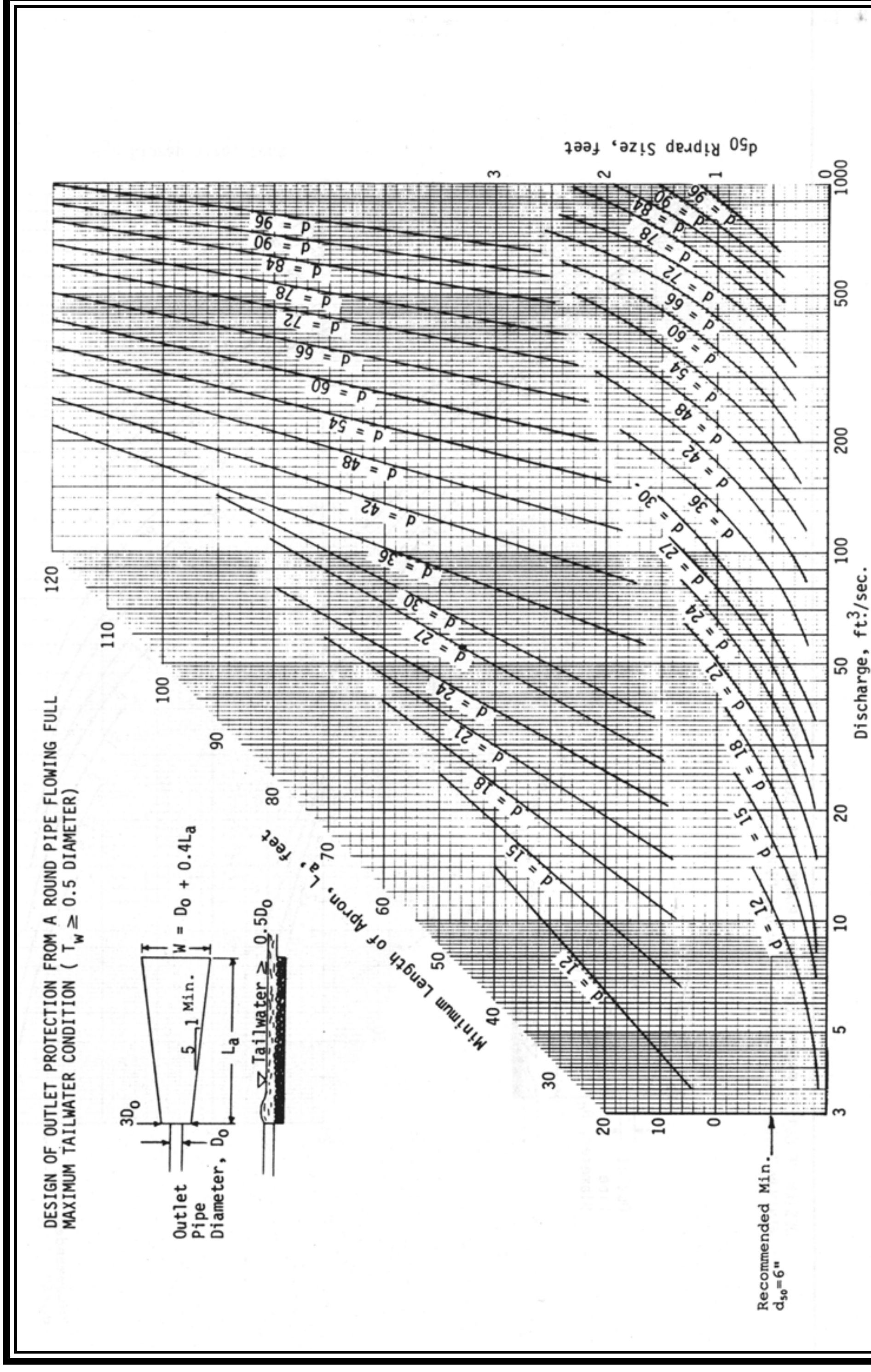


FIGURE 5 - 21
Maximum Tailwater Condition



STEP 14 Perform Buoyancy Calculation

The design of a multi-stage riser structure must include a buoyancy analysis for the riser and footing. When the ground is saturated and ponded runoff is at an elevation higher than the footing of the riser structure, the riser structure acts like a vessel. During this time, the riser is subject to uplifting, buoyant forces that are relative in strength to the volume of water displaced. **Flotation will occur when the weight of the structure is less than or equal to the buoyant force exerted by the water**. Flotation forces on the riser can lead to failure of the connection between the riser structure and barrel, and any other rigid connections. Eventually, this can also lead to the failure of the embankment.

A buoyancy calculation is the summation of all forces acting on the riser. The upward force is the weight of the water, or 62.4 lb/ft^3. The downward force includes the weight of the riser structure, any components, such as trash racks, and the weight of the soil above the footing. Note that conventional reinforced concrete weighs about 150 lb/ft^3 and the unit weight of soil is approximately 120 lb/ft^3 . The weight of components such as trash racks, anti-vortex devices, hoods, etc. is very specific to each structure and, depending upon the design, may or may not be significant in comparison to the other forces. If an extended base footing is used below the ground surface to support the control structure, then the weight of the soil above the footing may also be a significant force.

The outlet pipe is excluded from the buoyancy analysis for the control structure. However, the barrel should be analyzed separately to insure that it is not subject to flotation. The method used to attach the control structure to the outlet pipe is considered to have no bearing on the potential for these components to float.

The following procedure compares the upward force (buoyant force) to the downward force (structure weight). To maintain adequate stability, **the downward force should be a minimum of 1.25 times the upward force**.

1. Determine the buoyant force.

The buoyant force is the total volume of the riser structure and base, using outside dimensions (i.e., *the total volume displacement of the riser structure*) multiplied by the unit weight of water (62.4 lb/ft^3).

2. Determine the downward or resisting force.

The downward force is the total volume of the riser walls below the crest, including any top slab, footing, etc., less the openings for any pipe connections, multiplied by the unit weight of reinforced concrete (150 lb/ft^3). Additional downward forces from any components may also be added, including the weight of the soil above the extended footing.

3. Decide if the downward force is greater than the buoyant force by a factor of 1.25 or more.

If the downward force is not greater than the buoyant force by a factor of 1.25 or more, then additional weight must be added to the structure. This can be done by sinking the riser footing deeper into the ground and adding concrete to the base. Note that this will also increase the buoyant force, but since the unit weight of concrete is more than twice that of water, the net result will be an increase in the downward force. The downward and buoyant forces should be adjusted accordingly, and step 3 repeated.

STEP 15 Provide Seepage Control

Seepage control should be provided for the pipe through the embankment. The two most common devices for controlling seepage are 1) *filter and drainage diaphragms* and 2) *anti-seep collars*. The use of these devices is discussed in detail in **Minimum Standard 3.02, Principal Spillway**. Note that filter and drainage diaphragms are preferred over anti-seep collars for controlling seepage along pipe conduits.

a. Filter & Drainage Diaphragms

The design of filter and drainage diaphragms depends on the foundation and embankment soils and is outside the scope of this manual. When filter and drainage diaphragms are warranted, their design and construction should be supervised by a registered professional engineer. Design criteria and construction procedures for filter and drainage diaphragms can be found in the following references:

- USDA SCS TR-60
- USDA SCS Soil Mechanics Note No. 1: Guide for Determining the Gradation of Sand and Gravel Filters*
- USDA SCS Soil Mechanics Note No. 3: Soil Mechanics Consideration for Embankment Drains*
- U.S. Department of the Interior ACER Technical Memorandum No. 9: Guidelines for Controlling Seepage Along Conduits Through Embankments

* *These publications include design procedures and examples and are provided in **Appendix 5B**.*

b. Anti-Seep Collars

The Bureau of Reclamation, the U.S. Army Corps of Engineers and the Soil Conservation Service no longer recommend the use of anti-seep collars. In 1987, the Bureau of Reclamation issued Technical Memorandum No. 9 that states:

> *"When a conduit is selected for a waterway through an earth or rockfill embankment, cutoff [anti-seep] collars will not be selected as the seepage control measure."*

Alternative measures to anti-seep collars include *graded filters* (or *filter diaphragms*) and *drainage blankets*. These devices are not only less complicated and more cost-effective to construct than cutoff collars, but also allow for easier placement of the embankment fill. Despite this information, anti-seep collars may be appropriate for certain situations. A design procedure is provided below. Criteria for the use and placement of anti-seep collars are presented in **Minimum Standard 3.02, Principal Spillway**.

1. Determine the length of the barrel within the saturated zone using the following equation:

$$L_s = Y\,(Z + 4)\left[1 + \frac{S}{0.25 - S}\right]$$

Equation 5 - 11
Barrel Length in Saturated Zone

Where:
- L_s = *length of the barrel in the saturated zone, ft.*
- Y = *the depth of water at the principal spillway crest (10-year frequency storm water surface elevation), ft.*
- Z = *slope of the upstream face of the embankment, in Z ft. horizontal to 1 ft. vertical (Z ft. H:1V).*
- S = *slope of the barrel, in feet per foot.*

The length of pipe within the saturated zone can also be determined graphically on a *scale profile* of the embankment and barrel. The saturated zone of the embankment can be approximated as follows: starting at a point where the 10-year storm water surface elevation intersects the embankment slope, extend a line at a *4H:1V* slope downward until it intersects the barrel. The area under this line represents the ***theoretical zone of saturation*** (refer to **Figure 5-22**).

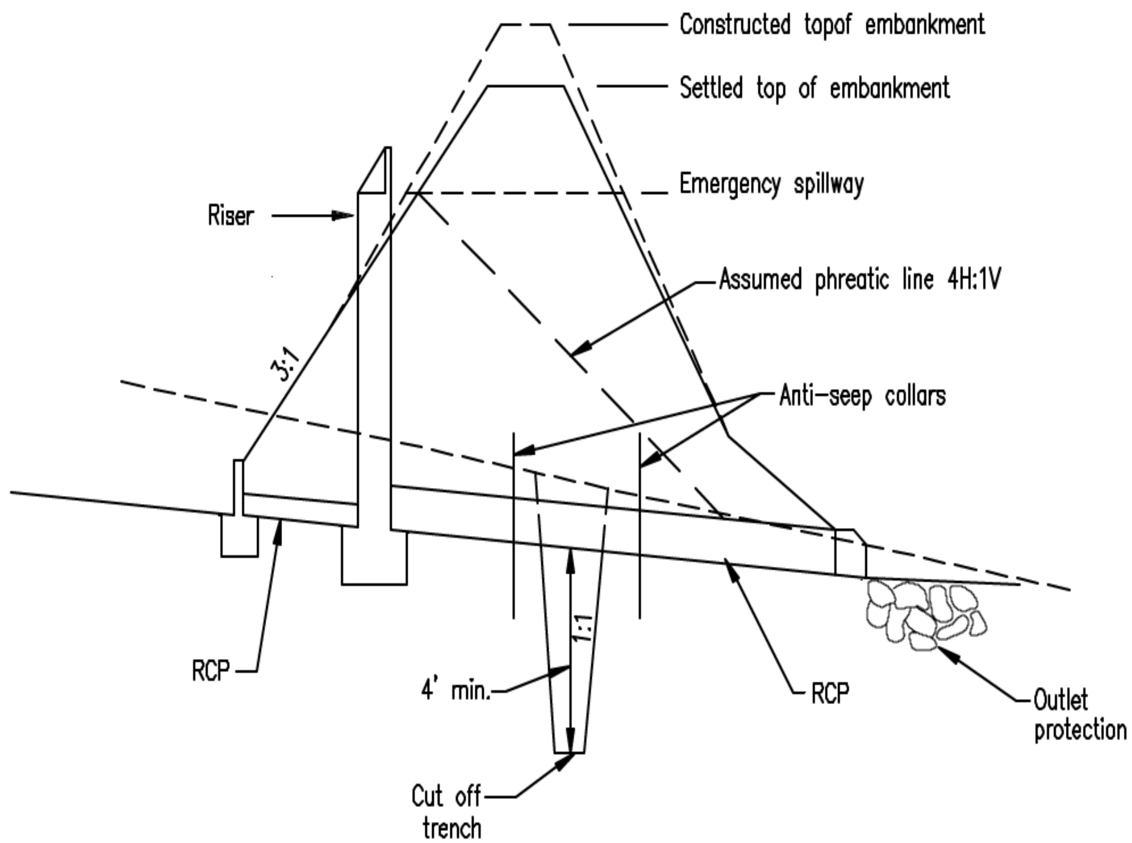
2. Determine the length required by multiplying 15% times the seepage length: $0.15\ L_s$. The increase in seepage length represents the total collar projection. This can be provided for by one or multiple collars.

3. Choose a collar size that is at least 4 feet larger than the barrel diameter (2 feet above and 2 feet below the barrel). For example, a 7-feet square collar would be selected for a 36-inch diameter barrel.

4. Determine the collar projection by subtracting the pipe diameter from the collar size.

5. Determine the number of collars required. The number of collars is found by dividing the seepage length increase, found in Step 2, by the collar projection from Step 4. To reduce the number of collars required, the collar size can be increased. Alternatively, the collar size can be decreased by providing more collars.

FIGURE 5 - 22
Phreatic Line Graphical Determination



SUMMARY
MULTI-STAGE RISER DESIGN PROCEDURE

STEP 1:	Determine Water Quality Volume Requirements a. Extended-Detention b. Retention
STEP 2:	Compute Allowable Release Rates
STEP 3:	Estimate the Required Storage Volume
STEP 4:	Grade the Basin; Create Stage-Storage Curve
STEP 5a:	Design Water Quality Orifice (Extended-Detention)
STEP 5b:	Set Permanent Pool Volume (Retention)
STEP 6:	Size 2-Year Control Orifice
STEP 7:	Check Performance of 2-Year Opening
STEP 8:	Size 10-Year Control Opening
STEP 9:	Check Performance of 10-Year Opening
STEP 10:	Perform Hydraulic Analysis a. Riser Flow Control b. Barrel Flow Control 1. Barrel Inlet Flow Control 2. Barrel Outlet Flow Control
STEP 11:	Size 100-Year Release Opening or Emergency Spillway
STEP 12:	Calculate Total Discharge and Check Performance of 100-Year Control Opening
STEP 13:	Design Outlet Protection
STEP 14:	Perform Buoyancy Calculation
STEP 15:	Provide Seepage Control

5-8 EMERGENCY SPILLWAY DESIGN

A vegetated emergency spillway is designed to convey a predetermined design flood volume without excessive velocities and without overtopping the embankment.

Two design methods are presented here. The first (Procedure 1) is a conservative design procedure which is also found in The Virginia Erosion & Sediment Control Handbook, 1992 edition, Std. & Spec. 3.14. This procedure is typically acceptable for stormwater management basins. The second method (Procedure 2) utilizes the roughness, or retardance, and durability of the vegetation and soils within the vegetated spillway. This second design is appropriate for larger or regional stormwater facilities where construction inspection and permanent maintenance are more readily enforced. These larger facilities typically control relatively large watersheds and are located such that the stability of the emergency spillway is essential to safeguard downstream features.

The following design procedures establish a stage-discharge relationship (H_p versus Q) for a vegetated emergency spillway serving a stormwater management basin (refer to **Figure 5-23**).

The information required for these designs includes the determination of the hydrology for the watershed draining to the basin. Any of the methods, as outlined in **Chapter 4**, may be used. The design should include calculations for the ***allowable release rate*** from the basin if the spillway is to be used to control a design frequency storm. Otherwise, the ***design peak flow rate*** should be calculated based on the spillway design flood, or downstream conditions.

(In general, a vegetated emergency spillway should not be used as an outlet for any storm less than the 100-year frequency storm, unless it is armored with a non-erodible material. The designer must consider the depth of the riprap blanket when riprap is used to armor the spillway. As noted previously, Class I riprap would require a blanket thickness or stone depth of 30" which may add considerable height to the embankment.)

The design maximum water surface elevations for the emergency spillway should be at least 1 foot lower than the settled top of the embankment. Refer to Minimum Standard 3.03, Vegetated Emergency Spillways.

Procedure 1:

1. Determine the ***design peak rate of inflow*** from the spillway design flood into the basin using the developed condition hydrology **or** determine the ***allowable design peak release rate, Q,*** from the basin based on downstream conditions or watershed requirements.

2. Estimate the maximum water surface elevation and calculate the maximum flow through the

riser and barrel system at this elevation (refer to the stage-storage-discharge table). Subtract this flow volume from the ***design peak rate of inflow*** to determine the desired maximum spillway design discharge.

3. Determine the crest elevation of the emergency spillway. This is usually a small increment (0.1 feet) above the design high water elevation of the next smaller storm, typically the 10-year frequency storm.

4. Enter **Table 5-12** with the maximum *Hp* value (maximum design water surface elevation from Step 2, less the crest elevation of the emergency spillway), and read across for the desired maximum spillway design discharge (from Step 2 above). Read the design bottom width of the emergency spillway (in feet) at the top of the table, and verify the minimum exit slope (s) and length (x), **or**;

   If a maximum bottom width (b) is known due to grading or topographic constraints, enter **Table 5-12** at the top with the desired bottom width and read down to find the desired discharge, Q, and then read across to the left to determine the required flow depth, *Hp*.

5. Add the appropriate *Hp* and discharge Q values to the stage-storage-discharge table.

Example Procedure 1:

Given: Q = *250 cfs* (determined from post-developed condition hydrology)
s_o = *4%* (slope of exit channel)
L = *50 ft.*(length of level section)

Find: Width of spillway, b, velocity, v, and depth of water above the spillway crest, H_p.

Solution: Complete Steps 1 through 5 of design **Procedure 1** for vegetated emergency spillways by using the given information as follows:

1. Peak rate of inflow: <u>given</u> Q = *250 cfs.*

2. The flow through the riser and barrel at the estimated maximum water surface elevation is calculated to be *163 cfs*. The desired maximum spillway design discharge is *250 cfs - 163 cfs = 87 cfs*, at a H_p value of *1.3 ft*.

3. Emergency spillway excavated into undisturbed material. The slope of the exit channel and length and elevation of level section: <u>given</u>, s_o = *4%, L = 50 ft.*, elevation = 100.0' (given).

4. Enter **Table 5-12** with the desired H_p value of *1.3 ft.* And read across to *86 cfs*, and read up to a bottom width of *24 ft*. at the top of the table. The minimum exit channel slope is *2.7%* which

is less than the *4%* provided, and the length of exit channel is required to be *63 ft*. The velocity within the exit channel is *4.7 ft/s* at an exit channel slope of *2.7%*. Since the provided exit channel slope is *4.0%*, erosive velocities may warrant special treatment of the exit channel.

5. Add the elevation corresponding to *1.3 ft*. above the crest of the emergency spillway to the Stage-Storage-Discharge Worksheet.

Procedure 2:

1. Determine the ***design peak rate of inflow*** from the spillway design flood into the basin, using the developed condition hydrology, **<u>or</u>** determine the ***allowable design peak release rate***, Q, from the basin based on downstream conditions or watershed requirements.

2. Estimate the maximum water surface elevation and calculate the associated flow through the riser and barrel system for this elevation. Subtract this flow value from the ***design peak rate of inflow*** to determine the desired maximum spillway design discharge.

3. Position the emergency spillway on the basin grading plan at an embankment abutment.

4. Determine the slope, s_o, of the proposed exit channel, and the length, L, and elevation of the proposed level section from the basin grading plan.

5. Classify the natural soils around the spillway as ***erosion resistant*** or ***easily erodible*** soils.

6. Determine the type and height of vegetative cover to be used to stabilize the spillway.

7. Determine the permissible velocity, v, from **Table 3-03.1**, based on the vegetative cover, soil classification, and the slope of the exit channel, s_o.

8. Determine the retardance classification of the spillway based on the type and height of vegetative cover from **Table 3-03.2**.

9. Determine the unit discharge of the spillway, q, in *cfs/ft*, from **Table 5-13(a-d)** for the selected retardance, the maximum permissible velocity, v, and the slope of the exit channel, s_o.

10. Determine the required bottom width of the spillway, in *ft.*, by dividing the allowable or design discharge, Q, by the spillway unit discharge, q:

$$\frac{Q(cfs)}{q(cfs/ft)} \; ' \; ft.$$

11. Determine the depth of flow, H_p, upstream of the control section based on the length of the

level section, L, from **Table 5-13(a-d)**.

12. Enter the stage-discharge information into the stage-storage-discharge table.

The following examples use **Tables 3-03.1, 3-03.2** and **5-13** to find the capacity of a vegetated emergency spillway.

Example Procedure 2:

Given: $Q = 250\ cfs$ (determined from post-developed condition hydrology)
$s_o = 4\%$ (slope of exit channel)
$L = 50\ ft.$(length of level section)
Erosion resistant soils
Sod forming grass-legume mixture cover, 6 to 10-inch height

Find: Permissible velocity, v, width of spillway, b, depth of water above the spillway crest, H_p.

Solution: Complete Steps 1 through 12 of design **Procedure 2** for vegetated emergency spillways by using the given information as follows:

1. Peak rate of inflow: given $Q = 250\ cfs$.

2. The flow through the riser and barrel at the estimated maximum water surface elevation is calculated to be *163 cfs*. The desired maximum spillway design discharge is $250\ cfs - 163\ cfs = 87\ cfs$.

3. Emergency spillway excavated into undisturbed material.

4. Slope of exit channel, and length and elevation of level section: given, $s_o = 4\%$, $L = 50\ ft.$, elevation = 100.0 feet (given).

5. Soil classification: given, erosion resistant soils.

6. Vegetative cover: given, sod-forming grass-legume mixture.

7. Permissible velocity $v = 5\ ft/s$ from **Table 3-03.1** for sod-forming grass-legume mixtures, erosion resistant soils, and exit channel slope $s_o = 4\%$.

8. Retardance classification, C, from **Table 3.03.2** for sod-forming grass-legume mixtures, *expected height = 6 to 10 inches*.

9. The unit discharge of the spillway $q = 3\ cfs/ft$ from **Table 5-13c** for Retardance C, maximum permissible velocity $v = 5\ ft/s$, and exit channel slope $s_o = 4\%$.

10. The required bottom width $b = Q \,' q = 87\ cfs/\ 3\ \ cfs/ft = 29\ ft$.

11. The depth of flow, $H_{p,}$ from **Table 5-13c** for Retardance *C* ; enter at $q = 3\ cfs/ft$, find $H_p = 1.4\ ft.$ for level section $L = 50\ ft.$

12. The stage-discharge relationship: at stage elevation 1.4 feet above the spillway crest (*101.4'*), the discharge is 87 *cfs*.

Example Procedure 2:

Given: $Q\ = 175\ cfs$ (determined from post-developed hydrology)
$s_o\ =\ 8\ \%$ (slope of exit channel)
$L\ =\ 25\ ft$. (length of level section)
Easily erodible soil
Bahiagrass, good stand, 11 to 24 inches expected

Find: Permissible velocity, *v*, width of spillway, *b*, depth of water above the spillway crest, H_p.
Analyze the spillway for stability during the vegetation establishment period, and capacity once adequate vegetation is achieved.

Solution: Complete Steps 1 through 12 of the design **Procedure 2** for vegetated emergency spillways by using the given information as follows:

1. $Q = 175\ cfs.$

2. The flow through the riser and barrel at the estimated maximum water surface elevation is calculated to be *75 cfs*. The desired maximum spillway design discharge is $175\ cfs - 75\ cfs = 100\ cfs$.

3. Emergency spillway in undisturbed ground.

4. $s_o = 8\ \%;\quad L = 25\ ft.$, elevation = *418.0 feet* (given)

5. Easily erodible soils.

6. Bahiagrass, good stand, 11 to 24 inches expected.

7. Permissable velocity, $v = 5\ ft/s,$ from **Table 3-03.1**.

8. a) Retardance used for **stability** during the establishment period - good stand of vegetation 2 to 6 inches; Retardance D**.**

   b) Retardance used for **capacity** - good stand of vegetation 11 to 24 inches; Retardance B**.**

9. Unit discharge $q = 2\ cfs/ft$ for stability. From **Table 5-13d** for Retardance D, permissable velocity, $v = 5\ ft/s.$, and $s_o = 8\%$

10. Bottom width $b = Q/\ q = 100\ cfs/\ 2\ cfs/\ ft = 50\ ft.$ (stability)

11. The depth of flow, H_p, for capacity. From **Table 5-13b** for Retardance B, enter at $q = 2\ cfs/ft$, find $H_p = 1.4\ ft.$ for $L = 25\ ft$.

12. The stage-discharge relationship: at stage (elevation) 1.4 *ft.* above the spillway crest (*419.4'*), the discharge, *Q*, is 100 *cfs*.

TABLE 5-12
Design Data for Earth Spillways

STAGE (Hp) IN FEET	SPILLWAY VARIABLES	BOTTOM WIDTH (b) IN FEET																
		8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
0.5	Q	6	7	8	10	11	13	14	15	17	18	20	21	22	24	25	27	28
	V	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	S	3.9	3.9	3.9	3.9	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
	X	32	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33
0.6	Q	8	10	12	14	16	18	20	22	24	26	28	30	32	34	35	37	39
	V	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	S	3.7	3.7	3.7	3.7	3.6	3.7	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	X	36	36	36	36	36	36	37	37	37	37	37	37	37	37	37	37	37
0.7	Q	11	13	16	18	20	23	25	28	30	33	35	38	41	43	44	46	48
	V	3.2	3.2	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3
	S	3.5	3.5	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
	X	39	40	40	40	41	41	41	41	41	41	41	41	41	41	41	41	41
0.8	Q	13	16	19	22	26	29	32	35	38	42	45	46	48	51	54	57	60
	V	3.5	3.5	3.5	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	S	3.3	3.3	3.3	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2
	X	44	44	44	44	45	45	45	45	45	45	45	45	45	45	45	45	45
0.9	Q	17	20	24	28	32	35	39	43	47	51	53	57	60	64	68	71	75
	V	3.7	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
	S	3.2	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1
	X	47	47	48	48	48	48	48	48	48	48	49	49	49	49	49	49	49
1.0	Q	20	24	29	33	38	42	47	51	56	61	63	68	72	77	81	86	90
	V	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
	S	3.1	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	X	51	51	51	51	52	52	52	52	52	52	52	52	52	52	52	52	52
1.1	Q	23	28	34	39	44	49	54	60	65	70	74	79	84	89	95	100	105
	V	4.2	4.2	4.2	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3
	S	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
	X	55	55	55	55	55	55	55	56	56	56	56	56	56	56	56	56	56
1.2	Q	28	33	40	45	51	58	64	69	76	80	86	92	98	104	110	116	122
	V	4.4	4.4	4.4	4.4	4.4	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5
	S	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
	X	58	58	59	59	59	59	59	59	60	60	60	60	60	60	60	60	60
1.3	Q	32	38	46	53	58	65	73	80	86	91	99	106	112	119	125	133	140
	V	4.5	4.6	4.6	4.6	4.6	4.6	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7
	S	2.8	2.8	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	X	62	62	62	63	63	63	63	63	63	63	63	64	64	64	64	64	64
1.4	Q	37	44	51	59	66	74	82	90	96	103	111	119	127	134	142	150	158
	V	4.7	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.9	4.9	4.9	4.9	4.9	4.9	49	4.9
	S	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6
	X	65	66	66	66	66	67	67	67	67	67	67	68	68	68	68	68	69
1.5	Q	41	50	58	66	75	85	92	101	108	116	125	133	142	150	160	169	178
	V	4.8	4.9	4.9	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.1	5.1	5.1
	S	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.5	2.5	2.5
	X	69	69	70	70	71	71	71	71	71	71	71	72	72	72	72	72	72
1.6	Q	46	56	65	75	84	94	104	112	122	132	142	149	158	168	178	187	197
	V	5.0	5.1	5.1	5.1	5.1	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2
	S	2.6	2.6	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	X	72	74	74	75	75	76	76	76	76	76	76	76	76	76	76	76	76
1.7	Q	52	62	72	83	94	105	115	126	135	145	156	167	175	187	196	206	217
	V	5.2	5.2	5.2	5.3	5.3	53	5.3	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4
	S	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	X	76	78	79	80	80	80	80	80	80	80	80	80	80	80	80	80	80
1.8	Q	58	69	81	93	104	116	127	138	150	160	171	182	194	204	214	226	233
	V	5.3	5.4	5.4	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.6	5.6	5.6	5.6	5.6	5.6
	S	2.5	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	X	80	82	83	84	84	84	84	84	84	84	84	84	84	84	84	84	84
1.9	Q	64	76	88	102	114	127	140	152	164	175	188	201	213	225	235	248	260
	V	5.5	5.5	5.5	5.6	5.6	5.6	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7
	S	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	X	84	85	86	87	88	88	88	88	88	88	88	88	88	88	88	88	88
2.0	Q	71	83	97	111	125	138	153	164	178	193	204	218	232	245	256	269	283
	V	5.6	5.7	5.7	5.7	5.8	5.8	5.8	5.8	5.8	5.8	5.8	5.9	5.9	5.9	5.9	5.9	5.9
	S	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	88	90	91	91	91	91	92	92	92	92	92	92	92	92	92	92	92
2.1	Q	77	91	107	122	135	149	162	177	192	207	220	234	250	267	276	291	305
	V	5.7	5.8	5.9	5.9	5.9	5.9	5.9	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
	S	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	92	93	95	95	95	95	95	95	95	96	96	96	96	96	96	96	96
2.2	Q	84	100	116	131	146	163	177	194	210	224	2.38	253	269	288	301	314	330
	V	5.9	5.9	6.0	6.0	6.0	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.2	6.2	6.2	6.2
	S	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	96	98	99	99	99	99	99	100	100	100	100	100	100	100	100	100	100
2.3	Q	90	108	124	140	158	175	193	208	226	243	258	275	292	306	323	341	354
	V	6.0	6.1	6.1	6.1	6.2	6.2	6.2	6.2	6.3	6.3	6.3	6.3	6.3	6.3	6.3	6.3	6.3
	S	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2
	X	100	102	102	103	103	103	104	104	104	105	105	105	105	105	105	105	105
2.4	Q	99	116	136	152	170	189	206	224	241	260	275	294	312	327	346	364	378
	V	6.1	6.2	62	6.3	6.3	6.3	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4
	S	2.3	2.3	23	2.3	2.3	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2
	X	105	105	106	107	107	108	108	108	108	109	109	109	109	109	109	109	109

Source: USDA - SCS

TABLE 5 - 13a

H_p and Slope Range for Discharge, Velocity and Crest Length - Retardance A

Max. Velocity, *v* (*ft/s*)	Unit Discharge, *q* (*cfs/ft*)	Depth of Water Above Spillway Crest, H_p (*ft.*) / Length of Level Section, *L* (*ft.*)				Slope Range, s_o %	
		25	50	100	200	Min.	Max.
3	3	2.3	2.5	2.7	3.1	1	11
4	4	2.3	2.5	2.8	3.1	1	12
4	5	2.5	2.6	2.9	3.2	1	7
5	6	2.6	2.7	3.0	3.3	1	9
6	7	2.7	2.8	3.1	3.5	1	12
7	10	3.0	3.2	3.4	3.8	1	9
8	12.5	3.3	3.5	3.7	4.1	1	10

Source: SCS Engineering Field Manual

TABLE 5 - 13b
H_p and Slope Range for Discharge, Velocity, and Crest Length - Retardance B

Max. Velocity, v (ft/s)	Unit Discharge, q (cfs/ft)	Depth of Water Above Spillway Crest, H_p ($ft.$) — Length of Level Section, L ($ft.$)				Slope Range, s_o %	
		25	50	100	200	Min.	Max.
2	1	1.2	1.4	1.5	1.8	1	12
2	1.25	1.3	1.4	1.6	1.9	1	7
3	1.5	1.3	1.5	1.7	1.9	1	12
3	2	1.4	1.5	1.7	1.9	1	8
4	3	1.6	1.7	1.9	2.2	1	9
5	4	1.8	1.9	2.1	2.4	1	8
6	5	1.9	2.1	2.3	2.5	1	10
7	6	2.1	2.2	2.4	2.7	1	11
8	7	2.2	2.4	2.6	2.9	1	12

Source: SCS Engineering Field Manual

TABLE 5 - 13c
H_p and Slope Range for Discharge, Velocity, and Crest Length - Retardance C

Max. Velocity, v (ft/s)	Unit Discharge, q (cfs/ft)	Depth of WaterAbove Spillway Crest, H_p ($ft.$) — Length of Level Section, L ($ft.$)				Slope Range, s_o %	
		25	50	100	200	Min.	Max.
2	0.5	0.7	0.8	0.9	1.1	1	6
2	1	0.9	1.0	1.2	1.3	1	3
3	1.25	0.9	1.0	1.2	1.3	1	6
4	1.5	1.0	1.1	1.2	1.4	1	12
4	2	1.1	1.2	1.4	1.6	1	7
5	3	1.3	1.4	1.6	1.8	1	6
6	4	1.5	1.6	1.8	2.0	1	12
8	5	1.7	1.8	2.0	2.2	1	12
9	6	1.8	2.0	2.1	2.4	1	12
9	7	2.0	2.1	2.3	2.5	1	10
10	7.5	2.1	2.2	2.4	2.6	1	12

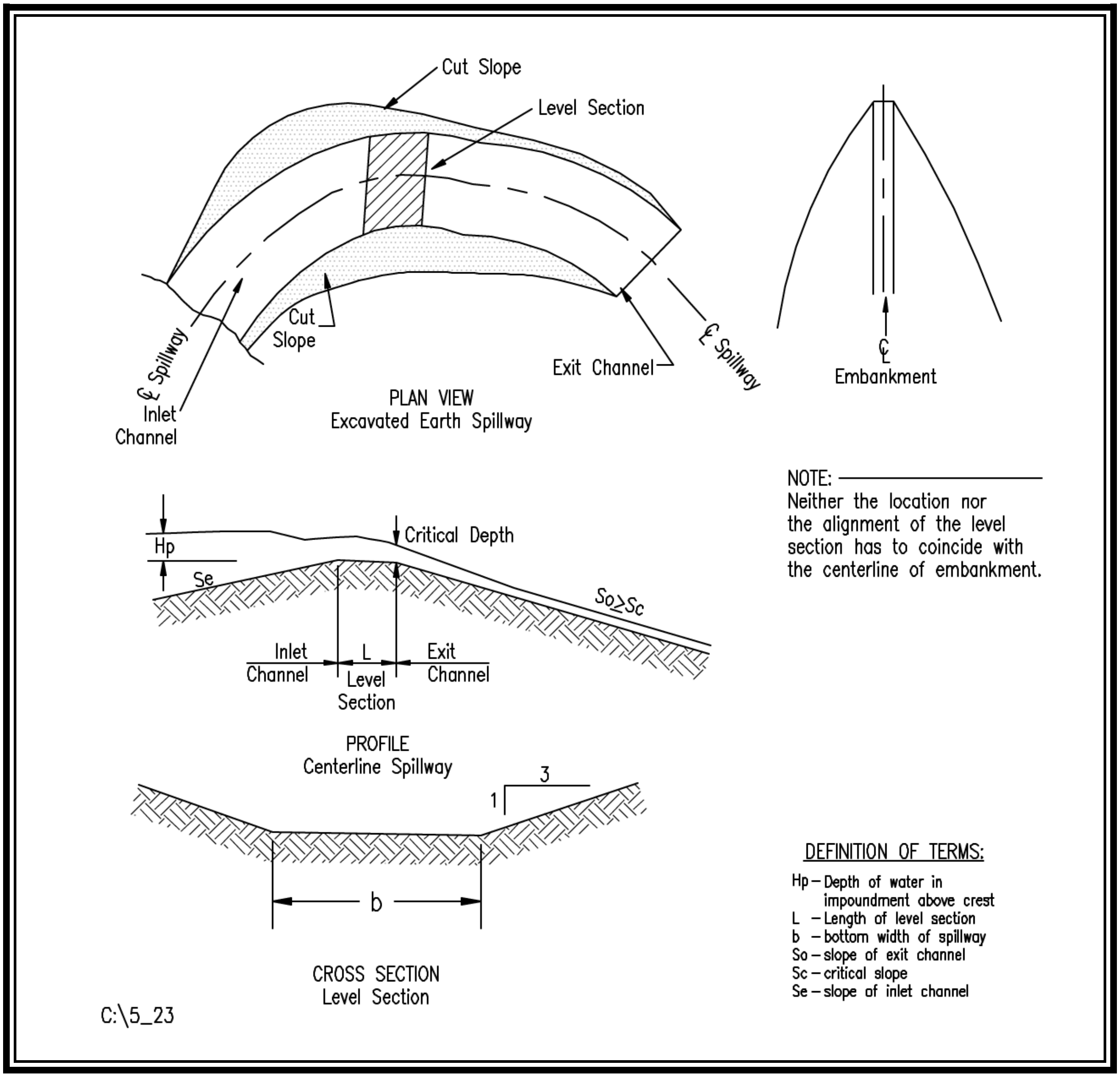
Source: SCS Engineering Field Manual

TABLE 5 - 13d
H_p and Slope Range for Discharge, Velocity, and Crest Length, Retardance D

Max. Velocity, *v* (*ft/s*)	**Unit Discharge, *q* (*cfs/ft*)**	**Depth of Water Above Spillway Crest, H_p (*ft.*) — Length of Level Section, *L (ft.)***				**Slope Range, s_o %**	
		25	**50**	**100**	**200**	**Min.**	**Max.**
2	0.5	0.6	0.7	0.8	0.9	1	6
3	1	.8	.9	1.0	1.1	1	6
3	1.25	.8	.9	1.0	1.2	1	4
4	1.5	.8	.9	1.0	1.2	1	10
4	2	1.0	1.1	1.3	1.4	1	4
5	1.5	.9	1.0	1.2	1.3	1	12
5	2	1.0	1.2	1.3	1.4	1	9
5	3	1.2	1.3	1.5	1.7	1	4
6	2.5	1.1	1.2	1.4	1.5	1	11
6	3	1.2	1.3	1.5	1.7	1	7
7	3	1.2	1.3	1.5	1.7	1	12
7	4	1.4	1.5	1.7	1.9	1	7
8	4	1.4	1.5	1.7	1.9	1	12
8	5	1.6	1.7	1.9	2.0	1	8
10	6	1.8	1.9	2.0	2.2	1	12

Source: SCS Engineering Field Manual

FIGURE 5 - 23
Vegetated Emergency Spillways: Typical Plan and Section



5-9 HYDROGRAPH ROUTING

This section presents the methodology for routing a runoff hydrograph through an existing or proposed stormwater basin. The "level pool" or storage indication routing technique is one of the simplest and most commonly used methods, and is based on the continuity equation:

$$I \quad - \quad O \quad = \quad ds / dt$$

Inflow - Outflow = Change in Storage over time

The goal of the routing process is to create an outflow hydrograph that is the result of the combined effects of the outlet device and the available storage. This will allow the designer to evaluate the performance of the outlet device or the basin storage volume, or both. When multiple iterations are required to create the most efficient basin shape, the routing procedure can be time consuming and cumbersome, especially when done by hand using the methods presented in this section. It should be noted that several computer programs are available to help complete the routing procedure.

A step-by-step procedure for routing a runoff hydrograph through a stormwater basin is given below. Note that the first four steps are part of the multi-stage riser design of the previous section. Due to the complexity of this procedure, **Example 1** from **Chapter 6** will be used. Note that the water quality volume is **not** considered and only one design storm will be routed, the 2-year storm. Other design frequency storms can be easily analyzed with the same procedure. Blank worksheets for this procedure are provided in **Appendix 5D**.

Procedure:

1. Generate a post-developed condition inflow hydrograph. The runoff hydrograph for the 2-year frequency storm, post-developed condition from **Example 1**, as calculated by the SCS TR-20 computer program and shown in **Figure 5-1**, will be used for the inflow hydrograph. (Refer to **Chapter 6** for details on the hydrology from **Example 1**. Refer to **Chapter 4** for information on the hydrologic methods used.)

2. Develop the stage-storage relationship for the proposed basin. The hydrologic calculations and the hydrograph analysis for **Example 1**, in **Section 5-3** and **Section 5-4.1**, revealed that the storage volume required to reduce the 2-year, post-developed peak discharge back to the pre-developed rate was 35,820 ft^3. Therefore, a preliminary grading plan should have a stormwater basin with this required storage volume, as a minimum, to control the 2-year frequency storm. The stage-storage relationship of the proposed stormwater facility can be generated by following the procedures outlined in **Section 5-5**. **Figure 5-10** shows the completed Storage Volume Calculations Worksheet, and **Figure 5-11** shows the stage vs. storage curve.

3. Size the outlet device for the design frequency storm and generate the stage-discharge relationship. An outlet device or structure must be selected to define the stage-discharge relationship. This procedure is covered in **Section 5-7, STEP 6** of the multi-stage riser design. Using the procedure within **STEP 6** from **Section 5-7** and **Example 1**, the procedure is as follows (from **STEP 6, Section 5-7**):

   1. Approximate the 2-year maximum head, $h_{2_{max}}$.

   Enter the stage-storage curve, **Figure 5-11**, with the 2-year required storage: 35,820 ft^3 and read the corresponding elevation: 88.5 *ft*. Then, $h_{2_{max}}$ *= 88.5 ft. - 81.0 ft. (bottom of basin) = 7.5 ft*. Note that this is an approximation because it ignores the centerline of the orifice as the point from which the head is measured. The head values can be adjusted when the orifice size is selected.

   2. Determine the maximum allowable 2-year discharge rate, $Q_{2_{allowable}}$.

   From the pre-developed hydrologic analysis, the 2-year allowable discharge from the basin was found to be 8.0 *cfs*. (This assumes that watershed conditions or local ordinance limit the developed rate of runoff to be # the pre-developed rate.)

   3. Calculate the size of the 2-year controlled release orifice.

   Solve for the area, *a*, in ft^2 by inserting the allowable discharge *Q = 8.0 cfs* and $h_{2_{max}}$ *= 7.5 ft.* into the **Rearranged Orifice Equation**, **Equation 5-7**. This results in an orifice diameter of 10 inches.

$$a = \frac{Q}{C\sqrt{2gh}}$$

Equation 5-7
Rearranged Orifice Equation

Where:
- a = *required orifice area,* ft^2
- Q = *maximum allowable discharge = 8.0 cfs*
- C = *orifice coefficient = 0.6*
- g = *gravitational acceleration = 32.2 ft/sec*
- h = *maximum 2-year hydraulic head,* $h_{2_{max}}$ *= 7.5 ft.*

$$a = \frac{8.0}{0.6\sqrt{(2)(32)(7.5)}}$$

$$a = 0.61\ ft^2$$

For orifice diameter:

$$a = 0.61\ ft^2 = \pi\left(\frac{d}{2}\right)^2$$

$$d = 0.88\ ft. = 10.6\ inches$$

***Use a 10-inch diameter orifice*.**

4. Develop the stage-storage-discharge relationship for the 2-year storm.

   Substituting the 10-inch orifice size into the **Orifice Equation**, **Equation 5-6**, and solving for the discharge, Q, at various stages provides the information needed to plot the stage vs. discharge curve and complete the Stage-Storage-Discharge Worksheet.

$$Q = C_o a\sqrt{2gh}$$

Equation 5-6
Orifice Equation

Where: $a = a_{10''} = 0.545\ ft^2$

$$Q = (0.6)(0.545)\sqrt{(2)(32.2)(h)}$$

$$Q_2 = 2.62\ (h)^{0.5}$$

Where: h = *water surface elev. - (81.0 + 0.83/2)*
h = *water surface elev. - 81.4*

Note that the *h* is measured to the centerline of the 10-inch orifice.

Figure 5-24 shows the result of the calculations: the stage vs. discharge curve and table.

Continuing with the Hydrograph Routing Procedure:

5. Develop the relationship *2S/Δt vs. O* and plot *2S/Δt vs. O.*

The plot of the curve *2S/Δt vs. 0* is derived from the continuity equation. The continuity equation is rewritten as:

$$\frac{I_n + I_{n+1}}{2} - \frac{O_n + O_{n+1}}{2} = \frac{S_{n+1} - S_n}{\Delta t}$$

Equation 5-12
Continuity Equation

where:
I_n & I_{n+1} = *inflow at time n=1 and time n=2*
O_n & O_{n+1} = *outflow at time n=1 and time n=2*
S_n & S_{n+1} = *storage at time n=1 and time n=2*
Δt = *time interval (n=2 - n=1)*

This equation describes the *change in storage over time* as the difference between the average inflow and outflow at that given time. Multiplying both sides of the equation by 2 and rearranging allows the equation to be re-written as:

$$I_n + I_{n+1} + \left(\frac{2S_n}{\Delta t} - O_n\right) = \frac{2S_{n+1}}{\Delta t} + O_{n+1}$$

Equation 5-13
Rearranged Continuity Equation

The terms on the left-hand side of the equation are known from the inflow hydrograph and from the storage and outflow values of the previous time interval. The unknowns on the right hand side, O_{n+1} and S_{n+1} , can be solved interactively from the previously determined stage vs. storage curve, **Figure 5-11**, and stage vs. discharge curve, **Figure 5-24**.

First, however, the relationship between *2S/Δt + O* and *O* must be developed. This relationship can best be developed by using the stage vs. storage and stage vs. discharge curves to fill out the worksheet shown in **Figure 5-25**, as follows:

a) Columns 1, 2, and 3 are completed using the stage vs. discharge curve.

b) Columns 4 and 5 are completed using the stage vs. storage curve.

c) Column 6 is completed by determining the time step increment used in the inflow hydrograph. (For Example 1, $\Delta t = 1\ hr. = 3{,}600\ sec.$) Δt is in seconds to create units of cubic feet per second (*cfs*) for the $2S/\Delta t$ calculation.

d) Column 7 is completed by adding Columns 3 and 6. The completed table is presented in **Figure 5-26**, and **Example 1** in **Chapter 6**, along with the plotted values from Column 3, O or outflow, and Column 7, $2S/\Delta t + O$.

6. Route the inflow hydrograph through the basin and 10-inch diameter orifice. The routing procedure is accomplished by use of another worksheet, **Figure 5-27**, Hydrograph Routing Worksheet. Note that as the work is completed for each value of *n,* it becomes necessary to jump to the next row for a value. The table is completed by the following steps:

   a. Complete Column 2 and Column 3 for each time *n*. These values are taken from the inflow hydrograph. The inflow hydrograph is provided in tabular form in **Figure 5-29**. This information is either taken from the plot of the inflow hydrograph or read directly from the tabular version of the inflow hydrograph (TR-20, TR-55, etc.).

   b. Complete Column 4 for each time *n* by adding two successive inflow values from Column 3. Therefore, $Column\ 4_n = Column\ 3_n + Column\ 3_{n+1}$.

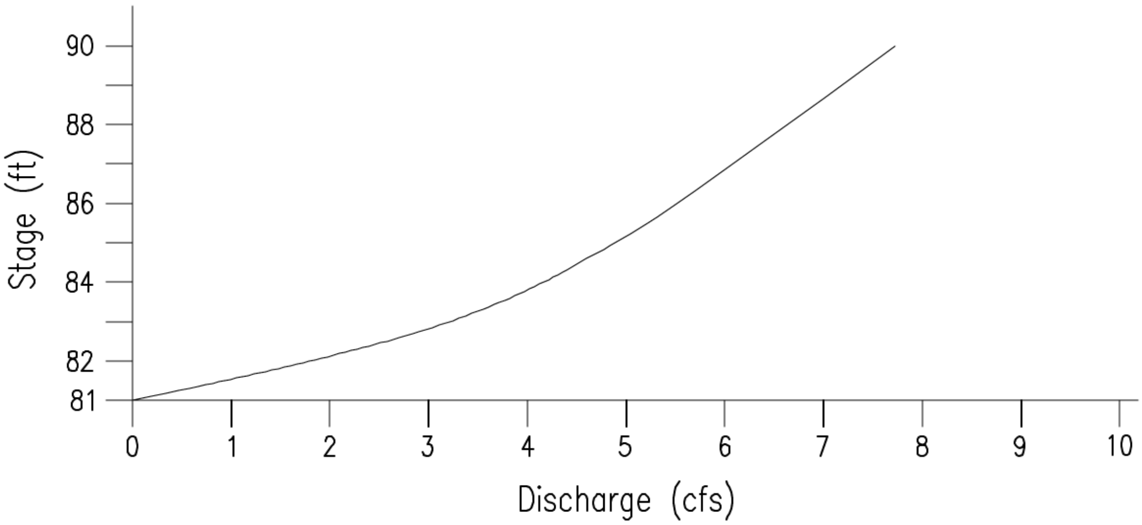
   c. Compute the values in Column 6 by adding Columns 4 and 5 from the previous time step. Note that for $n = 0$, Columns 5, 6, and 7 are given a value of zero before starting the table. Therefore, $Column\ 6_{n=2} = Column\ 4_{n=1} + Column\ 5_{n=1}$. (Note that this works down the table and not straight across.)

   d. Column 7 is read from the $2S/\Delta t + O$ *vs.* O curve by entering the curve with the value from Column 6 to obtain the outflow, O.

   e. Now backtrack to fill Column 5 by subtracting twice the value of Column 7 (from step d) from the value in Column 6. $Column\ 5_n = Column\ 6_n - 2(Column\ 7_n)$.

   f. Repeat steps c through e until the discharge (O, Column 7) reaches zero.

FIGURE 5 - 24
Stage vs. Discharge Curve, Example 1



Stage	(h)	(Q)
81.4	0	0
82	0.6	2.0
84	2.6	4.2
86	4.6	5.6
88	6.6	6.7
90	8.6	7.7

c:\g5_24.dwg

FIGURE 5 - 25

Storage Indication Hydrograph Routing (2S/Δt + O) vs. O Worksheet

1	2	3	4	5	6	7
elevation (ft)	***stage (ft)***	***outflow (cfs)***	***storage (cf)***	***2S (cf)***	***2S/Δt (cfs)***	***2S/Δt + O (cfs)***
from plan	$elev_n$ - $elev_o$	*based on outflow device & stage*	*based on stage*	*2 × Col 4*	*Col 5 /Δt of hydrograph*	*Col 3 + Col 6*

FIGURE 5 - 26
Storage Indication Hydrograph Routing (2S/Δt + O) vs. O Worksheet, Example 1, Curve & Table

1	2	3	4	5	6	7
Elev	Stage	Outflow (0 cfs)	Storage (S cf)	2S (cf)	2S/Δt (cfs)	2S/Δt + 0
81	0	0	0	0	0	0
8	0.6	2.0	900	1800	0.50	2.5
84	2.6	4.2	5940	11,880	3.3	7.5
86	4.6	5.6	14,354	28,708	7.97	13.6
88	6.6	6.7	29,582	59,164	16.4	23.1
90	8.6	7.7	55,564	111,128	30.9	38.6

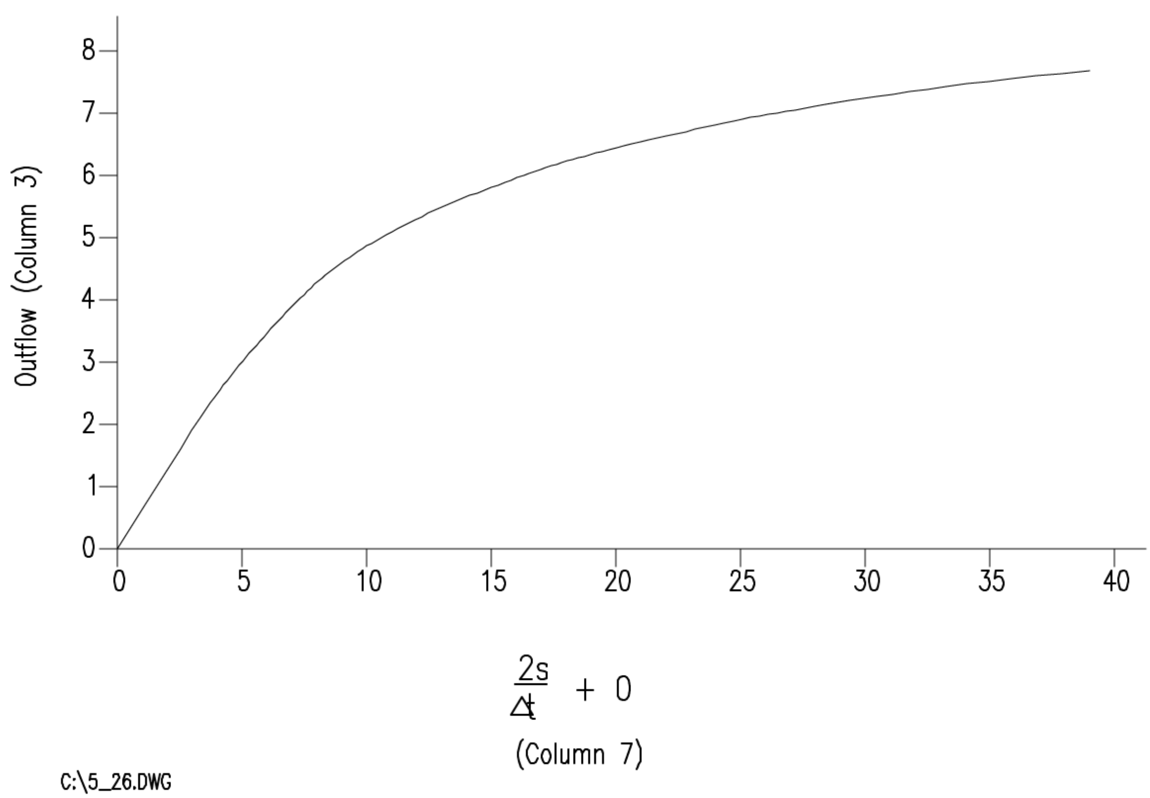


FIGURE 5 - 27
Storage Indication Hydrograph Routing Worksheet

1	2	3	4	5	6	7
n	*Time (min)*	I_n *(cfs)*	$I_n + I_{n+1}$ *(cfs)*	$2S_n/\Delta t - O_n$ *(cfs)*	$2S_{n+1}/\Delta t + O_{n+1}$ *(cfs)*	O_{n+1} *(cfs)*
	from hydrograph		*Col* 3_n *+Col* 3_{n+1}	*Col* 6_n *- 2(Col* 7_n*)*	*Col* 4_{n-1} *+ Col* 5_{n-1}	*from chart; use Col* 6_n
0				0	0	0

The above steps are repeated here for the first four time steps in **Example 1** and displayed in the completed Hydrograph Routing Worksheet, **Figure 5-28**.

1. Columns 2 and 3 are completed for each time step using the inflow hydrograph.

2. Column 4 is completed as follows:

   Column 4_n *= Column* 3_n *+ Column* 3_{n+1}
   for n = 1: Column $4_{n=1}$ *= Column* $3_{n=1}$ *+ Column* $3_{n=2}$
   Column $4_{n=1}$ *= 0 + 0.32 = 0.32*
   for n = 2: Column $4_{n=2}$ *= 0.3 + 23.9 = 24.2*
   for n = 3: Column $4_{n=3}$ *= 23.9 + 4.6 = 28.5*
   for n = 4: Column $4_{n=4}$ *= 4.6 + 2.4 = 7.0*
   etc.

n = 1

3. *Column* $6_{n=1}$ *= 0.* *n = 1* is at time 0. The first time step has a value of zero.

4. *Column* $7_{n=1}$ *= 0.* Entering the $2S/\Delta t$ *vs.* O curve with a value of zero gives $O = 0$ cfs. (The discharge is always zero at time $t=0$ unless a base flow exists.)

5. *Column* $5_{n=1}$ *= Column* $6_{n=1}$ *- 2 (Column* $7_{n=1}$*) Column* $5_{n=1}$ *= 0 - 0 = 0.*

n = 2

3. *Column* $6_{n=2}$ = *Column* $4_{n=1}$ + *Column* $5_{n=1}$.
   Column $6_{n=2}$ = *0.3 + 0 = 0.3.*

4. *Column* $7_{n=2}$ = *0.3.* Enter the $2S/\Delta t + O$ *vs.* O curve with $2S/\Delta t + O = 0.3$ (from Column 6) and read $O = 0.3$.

5. *Column* $5_{n=2}$ = *Column* $6_{n=2}$ - *2(Column* $7_{n=2}$*).*
   Column $5_{n=2}$ = *0.3 - 2(0.3) = -0.3 = 0.* (A negative outflow is unacceptable.)

n = 3

3. *Column* $6_{n=3}$ = *24.2 + 0 = 24.2.*

4. *Column* $7_{n=3}$ = *6.8.* Enter $2S/\Delta t + O$ *vs.* O curve with 24.2, read $O = 6.8$.

5. *Column* $5_{n=3}$ = *24.2 - 2(6.8) = 10.6.*

n = 4

3. *Column* $6_{n=4}$ = *28.5 + 10.6 = 39.1*

4. *Column* $7_{n=4}$ = *7.7.* Enter $2S/\Delta t = O$ *vs.* O curve with 39.1, read $O = 7.7$.

5. *Column* $5_{n=4}$ = *39.1 - 2(7.7) =23.7.*

n = 5, etc.

This process is continued until the discharge (*O*, Column 7) equals "0". The values in Column 7 can then be plotted to show the *outflow rating curve*, or *discharge hydrograph*, as shown in **Figure 5-29**. **The designer should verify that the maximum discharge from the basin is less than the allowable release**. If the maximum discharge is greater than or much less than the allowable discharge, the designer should try a different outlet size or basin shape.

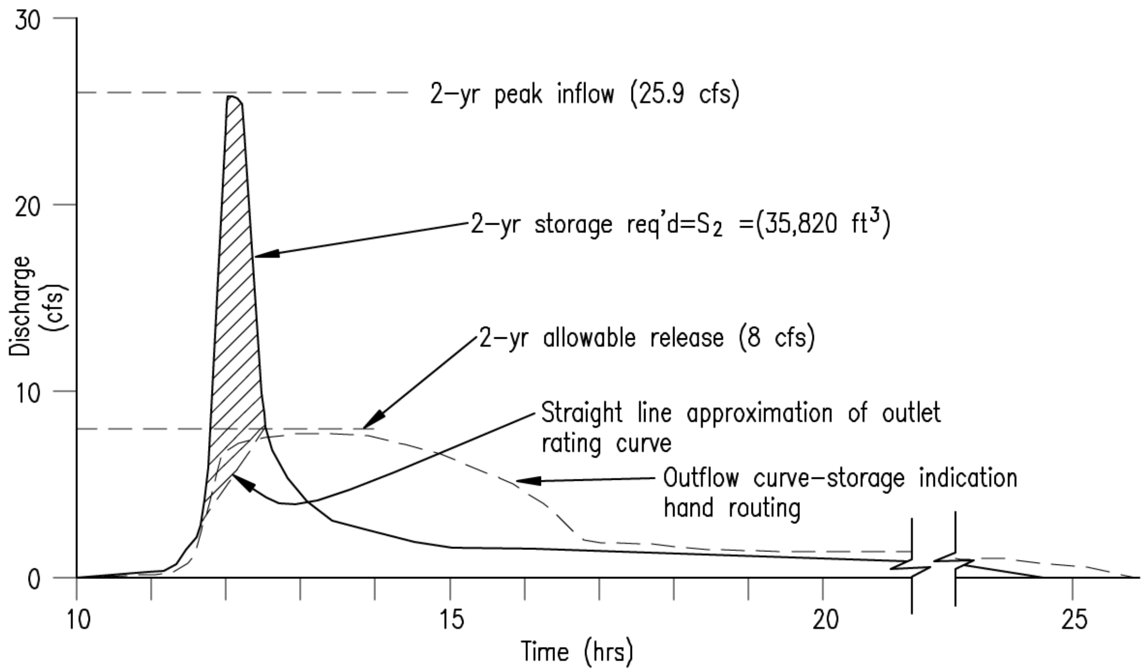
FIGURE 5 - 28
Storage Indication Hydrograph Routing Worksheet, Example 1

1	2	3	4	5	6	7
n	*Time (min)*	I_n *(cfs)*	$I_n + I_{n+1}$ *(cfs)*	$2S_n / \Delta t - O_n$ *(cfs)*	$2S_{n+1} / \Delta t + O_{n+1}$ *(cfs)*	O_{n+1} *(cfs)*
	from hydrograph		*Col* 3_n + *Col* 3_{n+1}	*Col* 6_n - 2(*Col* 7_n)	*Col* 4_{n-1} + *Col* 5_{n-1}	*from chart; use Col* 6_n
1	0	0	0.32	0	0	0
2	60	0.32	24.2	0 (-0.3)	0.3	0.3
3	120	23.9	28.5	10.6	24.2	6.8
4	180	4.6	7.0	23.7	39.1	7.7
5	240	2.4	4.0	15.7	30.7	7.5
6	300	1.6	3.0	6.9	19.7	6.4
7	360	1.4	2.6	0.3	9.9	4.8
8	420	1.2	2.3	0 (-0.7)	2.9	1.8
9	480	1.1	2.1	0 (-0.7)	2.3	1.5
10	540	1.0	1.9	0 (-0.7)	2.1	1.4
11	600	0.9	1.6	0 (-0.7)	1.9	1.3
12	660	0.7	1.4	0 (-0.6)	1.6	1.1
13	720	0.7	1.4	0 (-0.6)	1.4	1.0
14	780	0.7	1.3	0 (-0.6)	1.4	1.0
15	840	0.6	0.6	0 (-0.5)	1.3	0.9
16	900	0	0	0 (-0.4)	0.6	0.5
17	960	0	0		0	0

FIGURE 5 - 29
Inflow and Discharge Hydrographs, Example 1

hr.	Qi	Qo	hr.	Qi	Qo	hr.	Qi	Qo
10	0	0	16	1.4	4.8	22	0.7	1.0
11	0.3	0.3	17	1.2	1.8	23	0.7	1.0
12	23.9	6.8	18	1.1	1.5	24	0.6	0.9
13	4.6	7.7	19	1.0	1.4	25	0.0	0.5
14	2.4	7.5	20	0.9	1.3	26	0	0
15	1.6	6.4	21	0.7	1.1			

Inflow and Outflow Hydrograph Values
Qi taken from TR-20 Hydrology Computer Run
Qo taken from Routing Worksheet – Fig. 5-28



Peak 25.9 cfs, 12.13 hrs

Calculations=
$(0.40\ in^2)(10\ cfs/in)(2.5\ hrs/in)(3600\ sec/hr)=35{,}820\ ft^3$
$=0.82$ ac-ft

c:\g5_29.dwg

5-10 WATER QUALITY CALCULATION PROCEDURES

This section presents procedures for complying with the water quality criterion outlined in the stormwater management regulations. The water quality criterion represent a consolidation of the requirements of three state agencies charged with the responsibility of monitoring and improving the water resources of the Commonwealth: The Department of Conservation and Recreation (DCR), the Department of Environmental Quality (DEQ), and the Chesapeake Bay Local Assistance Department (CBLAD). The specific responsibilities of these agencies are presented in **Chapter 1**.

The stormwater management water quality regulations require compliance by either a **performance-based water quality criteria** or a **technology-based water quality criteria**. The performance-based water quality criteria requires the designer to implement a Best Management Practice (BMP) or combination of BMPs which effectively remove the anticipated increase in pollutant load from a development site. This approach requires the designer to calculate the pollutant load to be removed, implement a BMP strategy, and then calculate the performance of that strategy, based on the effectiveness or pollutant removal efficiency of the selected BMP(s).

The technology-based water quality criteria simply states that for land uses of given amounts of impervious cover, measured in percent, there are best available technologies with which to remove the anticipated pollutant load increase.

These two criterion are considered to be equivalent when implemented as described in this handbook. A more detailed discussion of these water quality criterion and the selection of water quality BMPs is presented in **Chapter 2**.

5-10.1 Performance-Based Water Quality Criteria

This procedure is for determining compliance with the performance-based water quality criteria of the Commonwealth's stormwater management regulations. The **Performance-based water quality criteria** is defined as follows:

For land development, the calculated post-development nonpoint source pollutant runoff load shall be compared to the calculated pre-development load based upon the average land cover condition <u>or</u> the existing site condition. A BMP(s) shall be located, designed, and maintained to achieve the target pollutant removal efficiencies specified in **Table 5-14** and to effectively reduce the pollutant load to the required level based upon the four applicable land development situations for which the performance criteria apply. (Refer to **STEP 3** for a discussion of the development situations.)

The "nonpoint source pollutant runoff load" or "pollutant discharge" is defined as the average amount of a particular pollutant(s) measured in pounds per year, delivered in a diffuse manner by stormwater runoff. The calculation procedure described herein uses the contaminant **phosphorous** for the purposes of calculating pollutant discharge in order to determine compliance with the performance-based water quality criteria. **However, other pollutants may be targeted if**

determined to be more appropriate for the intended land use. Refer to **Chapter 2** for a discussion of urban nonpoint source pollution.

The accepted calculation procedure for the determining the pre- and post-developed pollutant loads from development sites is referred to as the Simple Method. A more detailed discussion and derivation of the Simple Method can be found in *Appendix A* of Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs, published by the Metropolitan Washington Council of Governments. The simple method uses impervious cover as the key variable in calculating the levels of pollutant export. (It should be noted that other more data intensive methods for calculating pollutant loads are available. DCR will evaluate the option of utilizing these methods in the future.

Equation 5-14 presents the Simple Method General Pollutant Load Equation.

$$L = P \times P_j \times [0.05 + (0.009 \times I)] \times C \times A \times 2.72 \div 12$$

Equation 5-14
Simple Method Pollutant Load (L)

where:
L = *relative total phosphorous load (pounds per year)*
P = *average annual rainfall depth (inches), assumed to be 43 inches for Virginia**
P_j = *unitless correction factor for storm with no runoff = 0.9*
I = *percent impervious cover (percent expressed in whole numbers)*
C = *flow-weighted mean pollutant concentration = 0.26 milligrams per liter*
A = *applicable area (acres)*
Note: 12 and 2.72 are conversion factors
* - The annual rainfall depth may vary across the commonwealth based on locally collected rainfall data. The designer should verify actual rainfall values which may be required in the local jurisdiction. Also note that the use of the same value in the pre- and post-developed computations allows for the cancellation of this and other values as discussed below.

The purpose of this calculation is to provide a comparison between the pre- and post-development pollutant loads. Therefore, in an effort to simplify **Equation 5-14**, any value which will not change with the development of land, such as rainfall (P) and the flow weighted mean pollutant concentration (C), and any constants, such as the correction factor(P_j) and conversion factors, can be multiplied through. Thus **Equation 5-14** simplifies to:

$$L = [0.05 + (0.009 \times I)] \times A \times 2.28$$

Equation 5-15
Simple Method Pollutant Load (L), Simplified

where: L = *relative total phosphorous load (pounds per year)*
I = *percent impervious cover (percent expressed in whole numbers)*
A = *applicable area (acres)*

The Performance-based criteria requires that a pre- and post-developed condition pollutant load be calculated in order to determine the relative increase. A consistent, calculated pre-developed annual load (L_{pre}), or base annual load, with which to compare the calculated post-developed annual load (L_{post}) is therefore required. The Chesapeake Bay Local Assistance Department has determined a base line annual load of phosphorous for Tidewater Virginia and has established a corresponding baseline impervious value, or average land cover condition ($I_{watershed}$), of 16%. A locality may choose to adopt this value as the pre-developed default for the entire locality. **Or** the locality may choose to calculate a watershed or locality-wide pre-developed annual load and corresponding impervious value, and designate a watershed-specific or locality-specific average land cover condition.

Localities have the following options when determining average land cover conditions:

Option 1: A locality may designate specific watersheds within its jurisdiction and calculate the average land cover condition ($I_{watershed}$) and associated average total phosphorous loading for those watersheds (**Table 5-15** presents representative land uses and associated percent impervious cover and phosphorous export values); or

Option 2: A locality may assume the Chesapeake Bay default value for total phosphorous loading of 0.45 pounds/acre/year (F_{VA}) and an equivalent impervious cover ($I_{watershed}$) of 16 percent for its entire jurisdiction.

The calculation of watershed-specific average total phosphorous loadings must be based upon the following:

1. existing land use data at time of local program adoption,
2. watershed size, and
3. determination of equivalent values of impervious cover for non-urban land uses which contribute nonpoint source pollution, such as agriculture, silviculture, etc.

Some localities may begin with *Option 2* while they gather the necessary data for *Option 1*. The average land cover condition, once established for a locality (or watershed), **should not change**, and the designer simply uses that value as the existing condition baseline value for the specific watershed or locality in which the project is located.

5-10.2 Performance-Based Water Quality Calculation Procedure

The following steps represent the performance-based water quality calculation procedure:

<u>STEP 1</u> **Determine the applicable area (A) and the post-developed impervious cover (I_{post}).**

<u>STEP 2</u> **Determine the existing impervious cover ($I_{existing}$) <u>or</u> use the average land cover condition ($I_{watershed}$)**

<u>STEP 3</u> **Determine the appropriate development situation.**

<u>STEP 4</u> **Determine the relative pre-development pollutant load (L_{pre}).**

<u>STEP 5</u> **Determine the relative post-development pollutant load (L_{post}).**

<u>STEP 6</u> **Determine the relative pollutant removal requirement (RR).**

<u>STEP 7</u> **Identify best management practice (BMP) options for the site.**

The following discussion presents each step of the calculation procedure:

<u>STEP 1</u> **Determine the applicable area (A) and the post-developed impervious cover (I_{post}).**

<u>Applicable Area</u>

The applicable area (A) is the parcel of land being developed. For large developments such as subdivisions, shopping centers, or office / institutional campus style developments, use of the entire parcel or development areas can result in unreasonable water quality requirements. In these cases, the designation of a *planning area* may be more appropriate. A planning area is a designated portion of the parcel of land, measured in acres, on which the development project is located. The planning area may be established by drainage areas or development areas. A designated planning area can be helpful when analyzing developments where the density of impervious cover, construction phasing, or other factors vary across the total site and create distinctly separate areas of analysis. (The concept and advantages of planning areas are discussed further in **Chapter 2**.)

The use of planning areas must be preceeded by the development of a master plan to ensure that the entire development is accounted for, as well as document the consistent application of the designated planning areas (land can not be included in more than one planning area).

Post-development Impervious Cover (I_{post})

The designer must determine the amount of post-development impervious cover (I_{post}), in percent, within the applicable area. The zoning classifications or proposed density of a site will allow the designer to estimate impervious cover. It is important that the roadways, sidewalks, and other public or common ground improvements are included in the overall total impervious cover calculations when calculating the average lot size and the associated impervious cover. Compliance and final engineering calculations, however, should be based on impervious cover shown on the final site or subdivision plan. A locality may set minimum acceptable impervious percentages for particular land uses, and may also require a determination of the actual proposed impervious cover and **use the higher value**. Representative land use categories and associated average impervious cover values are shown in **Table 5-15**.

STEP 2 Determine the existing impervious cover ($I_{existing}$) or use the average land cover condition ($I_{watershed}$) as determined by the locality.

Existing Impervious Cover ($I_{existing}$):

The existing impervious cover ($I_{existing}$) is the percentage of the site that is occupied by impervious cover prior to the development of the proposed project. For new construction there is typically no existing impervious cover and therefore the average land cover condition or the watershed-specific value is used. Two of the four development situations presented in this standard, however, are based on the presence of existing site features or previous development and use the existing impervious cover as the basis for determining the pre-development total phosphorous load (L_{pre}).

Average Land Cover Condition ($I_{watershed}$):

A locality must establish the base pollutant load for specific watersheds or for the locality as a whole based on all of the land uses within the established boundary and, in turn, must determine the corresponding average land cover condition ($I_{watershed}$) measured in percent impervious cover. The average land cover condition, therefore, will be a watershed- or locality-specific value, or the Chesapeake Bay default value of 16%. The average land cover condition, once established for a locality (or watershed), should not change, and the designer simply uses that value as the pre-developed or existing average land cover condition for the specific watershed or locality in which the project is located.

STEP 3 Determine the appropriate development situation.

The performance-based criteria is applied through the use of four development situations. The application of each of these situations uses the same development characteristic (impervious cover) to determine the post-development pollutant load (L_{post}). However, the pre-development pollutant load (L_{pre}) is determined using either the average land cover condition ($I_{watershed}$) or the existing impervious cover ($I_{existing}$), depending on the development situation. The situations are as follows:

Situation 1: Land development where the existing percent impervious cover ($I_{existing}$) is less than or equal to the average land cover condition ($I_{watershed}$) and the proposed improvements will create a total percent impervious cover (I_{post}) which is less than the average land cover condition ($I_{watershed}$).

Requirement: No reduction in the after development pollutant discharge (L_{post}) is required.

Situation 2: Land development where the existing percent impervious cover ($I_{existing}$) is less than or equal to the average land cover condition ($I_{watershed}$) and the proposed improvements will create a total percent impervious cover (I_{post}) which is greater than the average land cover condition ($I_{watershed}$).

Requirement: The pollutant discharge after development (L_{post}) shall not exceed the existing pollutant discharge based on the average land cover condition ($L_{pre(watershed)}$).

Situation 3: Land development where the existing percent impervious cover ($I_{existing}$) is greater than the average land cover condition ($I_{watershed}$).

Requirement: The pollutant discharge after development (L_{post}) shall not exceed 1) the pollutant discharge based on existing conditions ($L_{pre(existing)}$) less 10%; or 2) the pollutant discharge based on the average land cover condition ($L_{pre(watershed)}$), whichever is greater.

Situation 4: Land development where the existing percent impervious cover ($I_{existing}$) is served by an existing stormwater management BMP(s) that addresses water quality.

Requirement: The pollutant discharge after development (L_{post}) shall not exceed the existing pollutant discharge based on the existing percent impervious cover while served by the existing BMP ($L_{pre(existingBMP)}$). The existing BMP shall be shown to have been designed and constructed in accordance with proper design standards and specifications, and to be in proper functioning condition.

If the proposed development meets the criteria for development Situation 1, than the low density development is considered to be the BMP and no pollutant removal is required. The calculation procedure for Situation 1 stops here. Development Situations 2 through 4 proceed to STEP 4.

STEP 4 **Determine the relative pre-development pollutant load (L_{pre}).**

The pre-developed pollutant load is based on either the average land cover condition ($L_{pre(watershed)}$): Situation 2; or the existing site conditions ($L_{pre(existing)}$): Situation 3; **or** the existing site conditions while being served by a water quality BMP ($L_{pre(existingBMP)}$): Situation 4.

The simplified version of the Simple Method Pollutant Load Equation (**Equation 5-15**) is modified by inserting the specific values of I ($I_{watershed}$ or $I_{existing}$) to calculate the relative pre-development total phosphorous load for the different development situations (2 through 4). The Simple Method Pollutant Load Equation is applied to the development situations as follows:

Situation 2:

The treatment requirement for Situation 2 states that the pollutant discharge after development (L_{post}) shall not exceed the existing pollutant discharge based on the average land cover condition ($L_{pre(watershed)}$). Therefore, the Simple Method Pollutant Load Equation is slightly modified to calculate the relative pre-development pollutant load ($L_{pre(watershed)}$) as follows:

$$L_{pre(watershed)} = [0.05 + (0.009 \times I_{watershed})] \times A \times 2.28$$

Equation 5-16
Pollutant Load Based on Average Land Cover Conditions ($L_{pre(watershed)}$)

where:
- $L_{pre(watershed)}$ = *relative pre-development total phosphorous load (pounds per year)*
- $I_{watershed}$ = *average land cover condition for specific watershed or locality **or** the Chesapeake Bay default value of 16% (percent expressed in whole numbers)*
- A = *applicable area (acres)*

Situation 3:

The treatment requirement for Situation 3 states that the pollutant discharge after development (L_{post}) shall not exceed the greater of: 1) the pollutant discharge based on existing conditions ($L_{pre(existing)}$) less 10%; or 2) the pollutant discharge based on the average land cover condition ($L_{pre(watershed)}$).

The pre-development pollutant discharge must be calculated twice in order to determine compliance with this requirement: first based on the existing impervious cover ($I_{existing}$) to calculate the pre-development load ($L_{pre(existing)}$) (**Equation 5-17**); and again based on the average land cover condition ($I_{watershed}$) to calculate the pre-development load ($L_{pre(watershed)}$) (**Equation 5-16**). The Simple Method Pollutant Load Equation is used as follows:

$$L_{pre(existing)} = [0.05 + (0.009 \times I_{existing})] \times A \times 2.28$$

Equation 5-17
Pollutant Load Based on Existing Site Conditions ($L_{pre(existing)}$)

where: $L_{pre(existing)}$ = *relative pre-development total phosphorous load (pounds per year)*
$I_{existing}$ = *existing site impervious cover (percent expressed in whole numbers)*
A = *applicable area (acres)*

The existing pollutant discharge based on the average land cover condition ($L_{pre(watershed)}$) is calculated the same as was done in **STEP 2** using **Equation 5-16**. The comparison of $L_{pre(existing)}$ less 10% and $L_{pre(watershed)}$ is made in **STEP 5** of this procedure.

Situation 4:

The requirement for Situation 4 states that the pollutant discharge after development (L_{post}) shall not exceed the existing pollutant discharge based on the existing percent impervious cover while served by the existing BMP(s) ($L_{pre(existingBMP)}$). The existing BMP(s) shall be shown to have been designed and constructed in accordance with proper design standards and specifications, and to be in proper functioning condition.

This requirement assumes that either all or a portion of the pollutant load generated by the existing impervious cover on a development is being reduced by one or more BMPs designed and constructed for that purpose. It becomes the responsibility of the designer or applicant to demonstrate that the facility was designed and constructed in accordance with the proper design standards and specifications, and is in proper functioning condition in order to justify the pollutant removal efficiency attributed to that particular BMP. Acceptable pollutant removal efficiency values attributed to some of the more commonly used BMPs for which there is adequate performance data are presented in **Table 5-14**. **Chapter 3** provides the design and maintenance requirements for these BMPs.

It should be noted that there may be more than one existing BMP. The drainage area to each BMP must be evaluated independantly. All areas being evaluated should be clearly documented on an existing conditon drainage area map.

The pre-developed total phosphorous load based on existing site conditions ($L_{pre(existing)}$) is calculated using **Equation 5-17**. The designer must then determine how much of the existing impervious cover is captured by the existing BMP(s), and the relative pollutant load removed. The Simple Method Pollutant Load Equation is therefore applied independently to each BMP drainage area of the site to determine the relative pollutant load of the area draining to the existing BMP(s) (**Equation 5-18**) and then the efficiency of each BMP is applied to the respective load to determine the load removed (**Equation 5-19**) as follows:

$$L_{pre(BMP)} = [0.05 + (0.009 \times I_{pre(BMP)})] \times A_{existBMP} \times 2.28$$

Equation 5-18
Pollutant Load to Existing BMP ($L_{pre(BMP)}$)

where: $L_{pre(BMP)}$ = *relative pre-development total phosphorous load entering existing BMP (pounds per year)*
$I_{pre(BMP)}$ = *existing impervious cover to existing BMP (percent expressed in whole numbers)*
$A_{existBMP}$ = *drainage area to existing BMP (acres)*

The relative pollutant load removed by the existing BMP ($L_{removed(existingBMP)}$) is determined as follows:

$$L_{removed(existingBMP)} = L_{pre(BMP)} \times EFF_{existBMP}$$

Equation 5-19
Pollutant Load Removed by Existing BMP ($L_{removed(existingBMP)}$)

where: $L_{removed(existingBMP)}$ = *relative pre-development total phosphorous load removed by existing BMP (pounds per year)*
$L_{pre(BMP)}$ = *relative pre-development total phosphorous load entering existing BMP, **Equation 5-18** (pounds per year)*
$EFF_{existBMP}$ = *documented pollutant removal efficiency of existing BMP (expressed in decimal form)*

Equations 5-18 and **5-19** are thus applied independantly to each existing BMP on the site.

The relative pre-development pollutant load from the site can now be calculated using **Equation 5-20** as follows:

$$L_{pre(existingBMP)} = L_{pre(existing)} \ ! \ (L_{removed(existingBMP1)} + L_{removed(existingBMP2)} + L_{removed(existingBMP3)})$$

Equation 5-20
Pollutant Load Based on Existing BMP Removal Efficiency ($L_{pre(existingBMP)}$)

where: $L_{pre(existingBMP)}$ = *relative pre-development total phosphorous load while being served by an existing BMP (pounds per year)*
$L_{pre(existing)}$ = *relative pre-development total phosphorous load based on existing site conditions, **Equation 5-17** (pounds per year)*
$EFF_{existBMP}$ = *documented pollutant removal efficiency of existing BMP (expressed in decimal form)*
$L_{removed(existingBMP)}$ = *relative pre-development total phosphorous load removed by existing BMP, **Equation 5-19** (pounds per year)*

<u>STEP 5</u> Determine the relative post-development pollutant load (L_{post}).

The post-development pollutant load (L_{post}) is calculated based on the proposed impervious cover for each development situation. The Simple Method Pollutant Load Equation based on the proposed post-development impervious cover (I_{post}) is used as follows:

$$L_{post} = [0.05 + (0.009 \times I_{post})] \times A \times 2.28$$

Equation 5-21
Pollutant Load Based on Post-Development Site Conditions (L_{post})

where:
- L_{post} = *relative post-development total phosphorous load (pounds per year)*
- I_{post} = *post-development impervious cover (percent expressed in whole numbers)*
- A = *applicable area (acres)*

<u>STEP 6</u> Determine the relative pollutant removal requirement (RR).

The pollutant removal requirement (RR) is defined as the relative amount of the keystone pollutant (in pounds per year) which must be removed by a BMP. The development situations discussed in **<u>STEP 3</u>** present the different removal or treatment requirements for each situation. There is no treatment requirement for Situation 1 due to the low density of development (proposed impervious cover less than the average land cover condition). The requirements for Situations 2, 3, and 4 are as follows:

Situation 2: $RR = L_{post} \; ! \; L_{pre(watershed)}$

Situation 3: $RR = L_{post} \; ! \; (0.9 \times L_{pre(existing)})$; **<u>or</u>**
$RR = L_{post} \; ! \; L_{pre(watershed)}$, which ever value of RR is less.

Situation 4: $RR = L_{post} \; ! \; L_{pre(existingBMP)}$

If the calculated RR value is less than or equal to zero, no BMPs are required. If the RR value greater than zero, continue on with <u>STEP 7</u>.

<u>STEP 7</u> Identify best management practice (BMP) options for the site.

The selection criteria for choosing an appropriate BMP for any given development site is often dictated by the physical characteristics of the site, such as soil types, topography, and drainage area. In addition, the pollutant removal requirement (RR) for the site may dictate that a BMP with a high removal efficiency (EFF_{BMP}) be used, while the physical characteristics of the site may dictate that a combination of strategically located BMPs be used. Specific siting and design criterion, as well as the accepted pollutant removal efficiencies for generally acceptable BMPs, are discussed in **Chapter 3: BMP Minimum Standards**.

The first step in determining which BMP may satisfy the pollutant removal requirement is to determine the necessary BMP pollutant removal efficiency. When the entire development is to be served by one BMP, this can be calculated using the following equation:

$$EFF = (RR \div L_{post}) \times 100$$

Equation 5-22
Required Pollutant Removal Efficiency (EFF)

where:
- EFF = *required pollutant removal efficiency*
- RR = *pollutant removal requirement (pounds per year)*
- L_{post} = *relative post-development total phosphorous load, Equation 5-21 (pounds per year)*

If more than one BMP will be used on the site, the removal requirement (RR) and post-development total load (L_{post}) must be calculated for each area using **Equation 5-22**. The designer can then use the required pollutant removal efficiency (RR) value to make a preliminary BMP(s) selection from **Table 5-15**. This is a preliminary selection since the specific siting and design criteria for the selected BMP must now be satisfied. Refer to **Chapter 3** for more information.

Once the BMP is selected and sited the designer must verify that the BMP(s) satisfies the removal requirement (RR) for the development. This is done by applying the pollutant removal efficiency (EFF_{BMP}) of the selected BMP to the post-developed pollutant load **entering the BMP as sited** (L_{BMP}). If the entire site drains to the proposed BMP, then the post-development pollutant load entering the BMP (L_{BMP}) is that which was calculated in **STEP 5** ($L_{post} = L_{BMP}$). In many cases, however, the topographic constraints of the site, or siting constraints of the specific BMP chosen, may result in some impervious areas not draining to the proposed BMP. Therefore, the Simple Method General Pollutant Load Equation must be applied to the actual drainage area of the BMP(s) as follows:

$$L_{BMP} = [0.05 + (0.009 \times I_{BMP})] \times A_{propBMP} \times 2.28$$

Equation 5-23
Pollutant Load Entering Proposed BMP (L_{BMP})

where:
- L_{BMP} = *relative post-development total phosphorous load entering proposed BMP(pounds per year)*
- I_{BMP} = *post-development percent impervious cover to proposed BMP (percent expressed in whole numbers)*
- $A_{propBMP}$ = *drainage area to proposed BMP (acres)*

The load removed by the BMP is then calculated as follows:

$$L_{removed} = Eff_{BMP} \times L_{BMP}$$

Equation 5-24
Pollutant Load Removed by Proposed BMP ($L_{removed}$)

where:

$L_{removed}$ = *post-development total phosphorous load removed by proposed BMP (pounds per year)*
Eff_{BMP} = *pollutant removal efficiency of BMP (expressed in decimal form)*
L_{BMP} = *relative post-development total phosphorous load entering proposed BMP,* ***Equation 5-23*** *(pounds per year)*

The calculation in this step is performed for each BMP and the various $L_{removed}$ values for the existing and proposed BMPs are summed for the total pollutant load removal as follows:

$$L_{removed/total} = L_{removed/BMP1} + L_{removed/BMP2} + L_{removed/BMP3} + \ldots + L_{removed(existingBMP1)} + L_{removed(existingBMP2)} + L_{removed(existingBMP3)}$$

Equation 5-25
Total Pollutant Load Removed by Proposed BMPs ($L_{removed/total}$)

where:

$L_{removed/total}$ = ***total*** *pollutant load removed by proposed BMPs (pounds per year)*
$L_{removed/BMP1}$ = *pollutant load removed by proposed BMP No. 1,* ***Equation 5-24***
$L_{removed/BMP2}$ = *pollutant load removed by proposed BMP No. 2,* ***Equation 5-24***
$L_{removed/BMP3}$ = *pollutant load removed by proposed BMP No. 3,* ***Equation 5-24***
$L_{removed(existingBMP)}$ = *pollutant load removed by existing BMP No. 1,* ***Equation 5-19***
$L_{removed(existingBMP)}$ = *pollutant load removed by existing BMP No. 2,* ***Equation 5-19***
$L_{removed(existingBMP)}$ = *pollutant load removed by existing BMP No. 3,* ***Equation 5-19***

The BMP or combination of BMPs is determined to be adequate if the total pollutant load removed ($L_{removed/total}$) is greater than or equal to the removal requirement (RR) calculated in **STEP 6**: $L_{removed/total} \geq RR$

If the total load removed is less than the removal requirement (RR) than an alternate BMP or combination of BMPs must be selected. It may be possible to simply increase the drainage area to the BMP(s) (if the entire site does not already drain to the BMP) in order to increase the overall pollutant removal from the site. Another option may be to reduce the impervious cover of the development in order to lower the removal requirement. The designer may also investigate the opportunities to capture off-site impervious area drainage in the proposed BMP to compensate for on-site areas which cannot be captured. In all cases the designer should contact the local program authority to determine if options are available in the local program as a result of a watershed or regional BMP plan.

Table 5-14
Water Quality BMP Pollutant Removal Efficiencies

Water Quality BMP*	Target Pollutant Removal Efficiency	Percent Impervious Cover
Vegetated filter strip Grassed swale	10% 15%	16-21%
Constructed wetlands Extended detention (2 x WQ Vol) Retention basin I (3 x WQ Vol)	30% 35% 40%	22 -37%
Bioretention basin Bioretention filter Extended detention-enhanced Retention basin II (4 x WQ Vol) Infiltration (1 x WQ Vol)	50% 50% 50% 50% 50%	38 -66%
Sand filter Infiltration (2 x WQ Vol) Retention basin III (4 x WQ Vol with aquatic bench)	65% 65% 65%	67 -100%

* Innovative or alternative BMPs not included in this table may be allowed at the discretion of the local program administrator, the plan approving authority, or the Department

Table 5-15
Simple Method General Pollutant Load Equation Solved for Incremental Impervious Cover Values (Urban Land Uses)

Representative Land Uses	Average Impervious Cover	Annual Pollutant Load (lb/ac/yr)
	0	0.11
2-5 Acre Residential	5	0.22
	10	0.32
	15	0.42
1 Acre Residential	20	0.52
½ Acre Residential	25	0.63
1/3 Acre Residential	30	0.73
1/4 Acre Residential	35	0.83
	40	0.94
1/8 Acre Residential	45	1.04
	50	1.14
Townhouses/ Garden Apartments	55	1.24
	60	1.35
	65	1.45
Light Industrial	70	1.55
	75	1.65
Heavy Industrial/ Commercial	80	1.76
	85	1.86
	90	1.96
	95	2.06
Pavement	100	2.17

Note: The average impervious cover values may be used for estimating or planning purposes when considering the representative land use as shown. When possible, final design calculations should be based on actual percent impervious cover as measured from the site plan.

Table 5-15 (Cont.)
Simple Method General Pollutant Load Equation Solved for Incremental Impervious Cover Values
(Non-Urban Land Uses)
(in pounds/acre/year)

Land Use	Silt Loam Soils	Loam Soils	Sandy Loam Soils
Conventional Tillage Cropland	3.71	2.42	0.83
Conservation Tillage Cropland	2.32	1.52	0.52
Pasture Land	0.91	0.59	0.20
Forest Land	0.19	0.12	0.04

CHAPTER 5

APPENDIX

APPENDIX 5A

a b Constants for Virginia

a b Constants for Virginia

		2 YEAR		10 YEAR		100 YEAR	
COUNTY	#	a	b	a	b	a	b
ARLINGTON	00	119.34	17.86	178.78	20.66	267.54	22.32
ACCOMACK	01	107.75	14.69	175.90	20.64	277.44	24.82
ALBEMARLE	02	106.02	15.51	161.60	18.73	244.82	20.81
ALLEGHENY	03	95.47	13.98	145.89	17.27	220.94	19.29
AMELIA	04	112.68	15.11	173.16	18.81	266.77	22.13
AMHERST	05	106.72	15.39	162.75	18.83	245.52	21.02
APPOMATTOX	06	109.11	15.39	167.44	19.12	254.03	21.61
AUGUSTA	07	84.21	10.44	135.74	14.54	210.02	16.99
BEDFORD	09	114.59	17.21	171.51	20.47	258.17	22.80
BLAND	10	105.33	16.56	162.75	20.41	247.84	22.87
BOTETOURT	11	110.32	16.95	164.94	20.01	247.92	22.16
BRUNSWICK	12	126.74	17.27	190.73	21.52	287.02	24.46
BUCHANAN	13	87.14	13.22	128.51	15.15	189.98	16.22
BUCKINGHAM	14	109.95	15.41	168.28	19.11	254.59	21.47
CAMPBELL	15	110.26	15.76.	167.27	19.18	252.65	21.56
CAROLINE	16	121.21	17.33	182.56	20.88	275.65	23.30
CARROLL	17	119.79	18.65	188.13	23.81	288.94	27.06
CHARLES CITY	18	124.23	17.14	186.52	21.05	281.04	23.85
CHARLOTTE	19	109.87	14.71	171.75	19.25	265.18	22.56
CHESTERFIELD	20	124.66	17.55	186.15	21.03	277.94	23.26
CLARKE	21	94.13	12.88	141.03	15.39	210.66	16.85
CRAIG	22	106.67	16.54	166.19	20.94	251.27	22.95
CULPEPER	23	111.90	16.25	169.78	19.51	255.26	21.52
CUMBERLAND	24	111.34	15.29	172.73	19.29	271.55	24.02

COUNTY	#	a	b	a	b	a	b
DICKENSON	25	87.03	13.10	128.09	14.82	190.08	15.98
DINWIDDIE	26	125.08	17.29	189.77	21.51	284.68	24.02
ESSEX	28	119.70	16.76	180.50	20.18	271.79	22.58
FAIRFAX	29	117.06	17.34	178.32	20.49	269.23	22.40
FAUQUIER	30	116.55	17.52	172.47	20.02	255.06	21.38
FLOYD	31	121.22	19.16	185.59	23.38	281.91	26.26
FREDERICK	34	93.79	13.15	141.02	15.77	211.40	17.42
GILES	35	106.14	16.72	165.04	20.80	252.79	23.46
GLOUCESTER	36	119.62	16.09	182.54	20.40	276.43	23.35
GOOCHLAND	37	114.42	15.95	177.24	19.93	269.07	22.27
GRAYSON	38	119.29	18.94	176.02	22.06	262.24	24.25
GREEN	39	105.71	15.10	159.92	18.20	241.18	20.34
GREENSVILLE	40	129.97	17.80	194.08	22.01	291.37	24.83
HALIFAX	41	111.92	15.14	173.81	19.52	267.09	22.70
HANOVER	42	122.80	17.29	185.01	20.91	278.40	23.40
HENRICO	43	123.51	17.35	185.51	21.13	277.61	23.44
HENRY	44	116.19	17.33	177.84	21.34	270.32	24.01
HIGHLAND	45	90.13	12.61	134.38	15.02	199.74	16.50
ISLE OF WIGHT	46	125.69	17.02	190.34	21.71	287.14	24.73
JAMES CITY	47	121.86	16.58	185.06	20.81	279.14	23.67
KING GEORGE	48	120.31	17.28	181.05	20.50	273.29	22.83
KING & QUEEN	49	113.84	15.29	179.09	19.95	275.98	23.15
KING WILLIAM	50	114.92	15.58	180.36	20.13	277.03	23.26
LANCASTER	51	109.80	14.49	170.27	18.72	259.78	21.41
LEE	52	93.78	14.40	143.28	17.58	215.10	19.22
LOUDOUN	53	104.05	14.91	157.67	17.71	237.83	19.65

COUNTY	#	a	b	a	b	a	b
LOUISA	54	112.63	15.89	174.35	19.72	265.20	22.11
LUNENBERG	55	122.01	16.82	184.70	20.80	278.38	23.48
MADISON	56	106.87	15.33	161.43	18.49	242.78	20.62
MATHEWS	57	118.61	15.83	180.56	20.17	274.12	23.29
MECKLENBERG	58	121.77	16.55	184.54	20.74	278.33	23.48
MIDDLESEX	59	110.72	14.57	172.76	19.15	264.49	22.13
MONTGOMERY	60	118.78	19.21	176.95	22.39	262.93	24.17
NELSON	62	103.46	14.52	160.23	18.36	245.04	20.89
NEW KENT	63	121.03	16.58	183.93	20.72	277.89	23.51
NORFOLK	64	124.88	17.02	190.64	22.14	288.73	25.60
NORTHAMPTON	65	111.07	14.78	173.72	19.63	267.48	23.04
NORTHUMBERLAND	66	111.20	14.99	171.55	19.00	260.59	21.63
NOTTOWAY	67	122.38	17.06	183.97	20.87	275.78	23.19
ORANGE	68	116.77	16.63	178.14	20.19	270.55	22.72
PAGE	69	84.19	10.29	135.43	14.29	209.57	16.86
PATRICK	70	123.68	19.26	189.08	23.60	284.78	26.12
PITTSYLVANIA	71	112.30	16.02	173.58	20.27	263.51	22.98
POWHATAN	72	114.14	15.64	175.93	19.65	266.86	22.15
PRINCE EDWARD	73	111.01	15.06	172.73	19.29	264.28	22.20
PRINCE GEORGE	74	126.22	17.46	188.62	21.39	283.12	24.09
VIRGINIA BEACH	75	129.20	17.84	196.25	22.74	294.74	26.33
PRINCE WILLIAM	76	116.04	17.08	176.18	20.19	266.75	22.36
PULASKI	77	117.44	18.71	182.33	23.39	279.39	26.49
RAPPAHANNOCK	78	104.86	15.05	159.40	18.34	239.30	20.19
RICHMOND	79	117.41	16.23	177.35	19.85	267.20	22.24
ROANOKE	80	117.53	18.79	174.97	21.80	261.95	23.81

COUNTY	#	a	b	a	b	a	b
ROCKBRIDGE	81	84.23	10.46	143.41	15.89	229.43	19.56
ROCKINGHAM	82	83.83	10.55	128.80	13.37	195.24	15.29
RUSSELL	83	92.64	14.17	143.00	17.32	216.40	19.36
SCOTT	84	92.64	14.17	143.00	17.32	216.40	19.35
SMYTH	86	106.19	16.57	169.30	21.37	262.49	24.57
SOUTHAMPTON	87	129.91	17.77	195.84	22.34	294.40	25.43
SPOTSYLVANIA	88	117.31	16.86	179.21	20.48	269.84	22.55
STAFFORD	89	118.72	17.34	179.62	20.64	270.74	22.79
SURRY	90	124.79	16.97	188.62	21.39	283.36	24.16
SUSSEX	91	130.37	18.03	193.23	21.91	287.99	24.56
TAZEWELL	92	91.25	13.56	141.61	17.04	217.59	19.48
WARREN	93	89.03	11.53	137.69	14.73	210.46	16.87
WASHINGTON	95	106.65	16.86	162.19	20.02	244.60	21.98
WESTMORELAND	96	114.40	15.76	174.96	19.47	266.16	22.12
WISE	97	89.83	13.49	132.05	15.44	194.10	16.35
WYTHE	98	116.78	18.83	174.91	22.13	261.68	24.25
YORK	99	122.93	16.72	186.78	21.22	282.80	24.39

		2 YEAR		10 YEAR		100 YEAR	
CITIES	#'s	a	b	a	b	a	b
RICHMOND	127/43	122.47	17.10	185.51	21.13	278.85	23.60
HAMPTON	114/27	123.93	16.94	186.78	21.22	283.18	24.56
LYNCHBURG	118/15	107.39	15.15	166.87	19.37	255.02	22.08
SUFFOLK	133/61	129.97	17.80	196.63	22.61	298.69	26.35
NEWPORT NEWS	121/94	126.11	17.37	189.27	21.62	285.24	24.71

Source: Virginia Department of Transportation

APPENDIX 5B

Filter and Drainage Diaphragm Design

- **USDA-SCS Soil Mechanics Note No. 1: Guide for Determining the Gradation of Sand and Gravel Filters (AVAILABLE UPON REQUEST)**

- **USDA-SCS Soil Mechanics Note No. 3: Soil Mechanics Considerations for Embankment Drains (AVAILABLE UPON REQUEST)**

APPENDIX 5C

Water Balance Analysis

Water Balance Analysis

The water balance analysis helps determine if a drainage area is large enough to support a permanent pool during normal conditions. The maximum draw down due to evaporation and infiltration is checked against the anticipated inflows during that same period. The anticipated drawdown during an extended period of no appreciable rainfall is checked as well. This will also help establish a planting zone for vegetation which can tolerate the dry conditions of a periodic draw down of the permanent pool.

The water balance is defined as the change in volume of the permenant pool resulting from the potential total inflow less the potential total outflow.

change volume = inflows ! outflows

where: *inflows* = *runoff*, *baseflow*, and *rainfall*.
outflows = *infiltration*, *surface overflow*, *evaporation*, and *evapotranspiration*.

This procedure will assume no inflow from baseflow, and because only the permanent pool volume is being evaluated, no losses for surface overflows. In addition, infiltration should be addressed by a geotechnical report. A clay liner should be specified if the analysis of the existing soils indicates excessive infiltration. In many cases, the permeability of clayey soils will be reduced to minimal levels due to the clogging of the soil pores by the fines which eventually settle out of the water column. This may be considered in the water balance equation by assuming the permeability of a clay liner: 1×10^{-6} cm/s (3.94×10^{-7} in/sec.) per specifications. Therefore, the change in storage = runoff ! evaporation ! infiltration.

Example

Given:

Drainage Area:	*85 ac.* (Average 65% impervious cover)
SCS RCN:	*72*
Precipitation P (2-year storm):	*3.1* inches
Runoff, Q:	*1.1* inches
Permanent Pool Volume:	*0.65 x 85 ac. = 55 ac.* impervious cover
	WQ volume = *(0.5in.) (55 ac.) (12in./ft.) = 2.29 ac.ft.*
	Retention Basin II (*4* x WQ vol.) = *4 x 2.29 = 9.16 ac.ft.*
Permanent Pool Surface area:	*2.4 ac.*
Infiltration (clay liner per specs.):	1×10^{-6} *cm/s* (3.94×10^{-7} *in/sec.*)

Find:

a) Draw down during highest period of evaporation.
b) Draw down during extended period of no appreciable rainfall.

Solution:

a) Draw down during highest period of evaporation: July

$$Inflow = \text{Monthly Runoff} = P \times E$$

Where P = precipitation
E = efficiency of runoff (assumed to be ratio of SCS runoff depth to rainfall depth for 2 year storm)
$= 1.1\ in./\ 3.1'' = 0.35$

	April	May	June	July	Aug.	Sept.
Precip. (in)	*2.96*	*3.84*	*3.62*	***5.03***	*4.40*	*3.34*
Evap. (in)	*2.28*	*3.89*	*5.31*	***6.23***	*5.64*	*3.92*

(From **Table 5C-1** and **5C-2**)

Inflow: Runoff $= 5.03\ in. \times 0.35 = 1.76\ in. = 1.76\ in. \times 85\ ac. \div 12\ in./ft. = \underline{12.5\ ac.ft.}$

Outflow: Evaporation $= 2.4\ ac. \times 6.23\ in. \div 12\ in./ft. = \underline{1.24\ ac.ft.}$

Infiltration (w/ liner)= $2.4\ ac. \times (3.94 \times 10^{-7}\ in./sec.)\ (3600\ sec./hr.)\ (24\ hr./day)\ (31\ days) \div (12\ in./ft.) = \underline{0.21\ ac.\ ft.}$

Water balance (w/ liner) = $(inflow) - (outflow) = (12.5\ ac.ft.) - (1.24 + 0.21)\ ac.ft. = \underline{+\ 11.05\ ac.ft}.$

Infiltration (w/o liner); assume infiltration rate of .02 in./hr. (clay/silty clay) =
$2.4\ ac. \times .02\ in./hr. \times (24\ hr./day)\ (31\ days) \div 12 in./ft. = \underline{2.97\ ac.ft}.$

Water balance (w/o liner) = $(12.5\ ac.ft.) - (2.97 + 0.21)\ ac.ft. = \underline{+\ 9.32\ ac.ft}.$

b) Drawdown during period of no appreciable rainfall. Assume 45 day period during July and August with no rainfall.

Inflow: runoff *= 0"*

Outflow: Evaporation = Avg. evaporation (July-Aug.) = *6.23 in. + 5.64 in.* ' *2 = 5.93 in.*
Avg. daily evaporation = *5.93 in.* ' *31 days =* *0.191 in./day*
Evaporation for 45 days = *45 days × 0.191 in./day = 8.61 in.*
Total evaporation = *2.4 ac. × 8.61 in.* ' *12 in./ft. =* *1.7 ac.ft.*

Infiltration (w/ liner): *2.4 ac. × (3.94 x* 10^{-7} *in./sec.) (3600 sec./hr.) (24 hr./day) (45 days)* ' *12 in./ft. = 0.30 ac.ft.*

Water balance (w/ liner): *(0)* ! *(1.7 + 0.30) ac.ft. =* !*2.0 ac.ft.*

Specify drawdown tolerant plants in areas corresponding to a depth of 2.0 ac.ft. (use stage storage curve).

Infiltration (w/o liner): *2.4 ac.× (.02 in./hr.) (24 hr./day) (45 day)* ' *12 in./ft.*
= 4.32 ac.ft.

Water balance (w/o liner): *(0)* ! *(1.7 + 4.32) ac.ft. =* ! *6.02 ac.ft.*

This basin (with out a liner) will experience a significant draw down during drought conditions. Over time, the rate of infiltration may decrease due to the clogging of the soil pores. However, the aquatic and wetland plants may not survive the potential drought conditions and subsequent draw down during the first few years, and eventually give way to invasive species.

Note: A permanent pool volume of *9.16 ac.ft. = 1.29* watershed inches. A rainfall event yielding 1.29" or more of runoff will fill the pool volume.

Table 5C-1
Monthly Precipitation Normals (Inches)

Station	April	May	June	July	August	Sept.
Charlottesville	3.34	4.88	3.74	4.75	4.71	4.10
Danville	3.24	3.85	3.65	4.42	3.80	3.39
Farmville	3.03	4.05	3.41	4.34	3.99	3.18
Fredericksburg	3.05	3.85	3.35	3.65	3.61	3.49
Hot Springs	3.43	4.15	3.36	4.49	3.70	3.39
Lynchburg	3.09	3.91	3.45	4.16	3.59	3.24
Norfolk	3.06	3.81	3.82	5.06	4.81	3.90
Page County	3.84	4.77	4.41	4.50	4.34	4.81
Pennington Gap	4.25	4.83	4.09	4.77	3.76	3.67
Richmond	2.98	3.84	3.62	5.03	4.40	3.34
Roanoke	3.25	3.98	3.19	3.91	4.15	3.50
Staunton	2.82	3.60	2.95	3.49	3.67	3.46
Wash. National Airport	2.31	3.66	3.38	3.80	3.91	3.31
Williamsburg	3.01	4.52	4.03	4.96	4.72	4.25
Winchester	3.08	3.74	3.87	3.89	3.46	3.11
Wytheville	3.09	3.95	3.03	4.20	3.44	3.09

Source: Department of Environmental Services, Virginia State Climatology Office, Charlottesville, Virginia

Table 5C-2
Potential Evapotranspiration (Inches) *

Station	April	May	June	July	August	Sept.
Charlottesville	2.24	3.84	5.16	6.04	5.45	3.87
Danville	2.35	3.96	5.31	6.23	5.69	3.91
Farmville	2.34	3.81	5.13	6.00	5.41	3.71
Fredericksburg	2.11	3.80	5.23	6.11	5.46	3.83
Hot Springs	1.94	3.41	4.50	5.14	4.69	3.33
Lynchburg	2.21	3.72	4.99	5.85	5.31	3.70
Norfolk	2.20	3.80	5.37	6.34	5.79	4.14
Page County	1.68	3.06	4.09	4.71	4.26	3.05
Pennington Gap	2.14	3.59	4.72	5.45	4.97	3.60
Richmond	2.28	3.89	5.31	6.23	5.64	3.92
Roanoke	2.20	3.75	4.99	5.85	5.30	3.67
Staunton	2.00	3.52	4.77	5.52	4.95	3.47
Wash. National Airport	2.13	3.87	5.50	6.51	5.84	4.06
Williamsburg	2.27	3.86	5.23	6.14	5.61	3.97
Winchester	2.07	3.68	4.99	5.82	5.26	3.67
Wytheville	2.01	3.43	4.46	5.17	4.71	3.39

Source:Department of Environmental Services, Virginia State Climatology Office, Charlottesville, Virginia

***** Calculated using the Thornthwaite method

APPENDIX 5D

Worksheets

C **Stage Storage Worksheet**

C **Stage-Storage-Discharge Worksheet**

C **Storage Indication Hydrograph Routing Worksheet;** ***$2S/\Delta t + O$ Vs. O***

C **Storage Indication Hydrograph Routing Worksheet**

C **Performance-Based Water Quality Calculations Worksheet 1**

C **Performance-Based Water Quality Calculations Worksheet 2: Situation 2**

C **Performance-Based Water Quality Calculations Worksheet 3: Situation 3**

C **Performance-Based Water Quality Calculations Worksheet 4: Situation 4**

Stage-Storage Worksheet

PROJECT: ____ SHEET _____ OF _____

COUNTY: ___________________ COMPUTED BY:______________ DATE:________

DESCRIPTION: ____

ATTACH COPY OF TOPO: SCALE - 1" = ______ *ft.*

1	2	3	4	5	6	7	8
ELEV.	AREA *(in²)*	AREA *(ft²)*	AVG. AREA *(ft²)*	INTERVAL	VOL. *(ft³)*	TOTAL VOLUME *(ft³)*	*(ac.ft.)*

Stage - Storage - Discharge Worksheet

ELEV *(MSL)*	STORAGE *(ac.ft..)*	WATER QUALITY ORIFICE		2-YEAR CONTROL				10-YEAR CONTROL				RISER STRUCTURE				BARREL				EMERGENCY SPILLWAY		TOTAL *Q* *(cfs)*
		(1)		WEIR (2)		ORIFICE (3)		WEIR (4)		ORIFICE (5)		WEIR (6)		ORIFICE (7)		INLET (8)		OUTLET (9)		(10)		
		h	*Q*	*h*	*Q*	*h*	*Q*	*h*	*Q*	*h*	*Q*	*h*	*Q*	*h*	*Q*	*HW/D*	*Q*	*h*	*Q*	*h*	*Q*	

Storage Indication Hydrograph Routing Worksheet
2S/Δt + O Vs. O

1	**2**	**3**	**4**	**5**	**6**	**7**
elevation (ft)	*stage (ft)*	*outflow (cfs)*	*storage (cf)*	*2S (cf)*	*2S/Δt (cfs)*	*2S/Δt + O (cfs)*
from plan	*$elev_n$ - $elev_o$*	*based on outflow device & stage*	*based on stage*	*2 × Col 4*	*Col 5 /Δt of hydrograph*	*Col 3 + Col 6*

Storage Indication Hydrograph Routing Worksheet

1	**2**	**3**	**4**	**5**	**6**	**7**
n	*Time (min)*	I_n *(cfs)*	$I_n + I_{n+1}$ *(cfs)*	$2S_n /\Delta t - O_n$ *(cfs)*	$2S_{n+1} /\Delta t + O_{n+1}$ *(cfs)*	O_{n+1} *(cfs)*
	from hydrograph		*Col* 3_n + *Col* 3_{n+1}	*Col* 6_n - 2(*Col* 7_n)	*Col* 4_{n-1} + *Col* 5_{n-1}	*from chart; use Col* 6_n

Worksheet 1
Page 1 of 3

STEP 1 **Determine the applicable area (A) and the post-developed impervious cover (I_{post}).**

Applicable area (A)* = ________ acres

Post-development impervious cover:

structures =__________acres

parking lot =__________acres

roadway =__________acres

other:

______________=__________acres

______________=__________acres

Total = __________acres

I_{post} = (total post-development impervious cover ÷ A) × 100 = ________%

* The area subject to the criteria may vary from locality to locality. Therefore, consult the locality for proper determination of this value.

STEP 2 **Determine the average land cover condition ($I_{watershed}$) or the existing impervious cover ($I_{existing}$).**

Average land cover condition ($I_{watershed}$):
If the locality has determined land cover conditions for individual watersheds within its jurisdiction, use the watershed specific value determined by the locality as $I_{watershed}$.

$I_{watershed}$ = ________%

Otherwise, use the Chesapeake Bay default value:

$I_{watershed}$ = 16%

Worksheet 1
Page 2 of 3

Existing impervious cover ($I_{existing}$):

Determine the existing impervious cover of the development site if present.

Existing impervious cover:

structures = __________ acres

parking lot = __________ acres

roadway = __________ acres

other:

______________ = __________ acres

______________ = __________ acres

Total = __________ acres

$I_{existing}$ = (total existing impervious cover ÷ A*) × 100 = _______ %

* The area should be the same as used in STEP 1.

STEP 3 **Determine the appropriate development situation.**

The site information determined in STEP 1 and STEP 2 provide enough information to determine the appropriate development situation under which the performance criteria will apply. Check (U) the appropriate development situation as follows:

_____ **Situation 1**: This consists of land development where the existing percent impervious cover ($I_{existing}$) is less than or equal to the average land cover condition ($I_{watershed}$) and the proposed improvements will create a total percent impervious cover (I_{post}) which is less than or equal to the average land cover condition ($I_{watershed}$).

$\mathbf{I_{post}}$ ________ % # $\mathbf{I_{watershed}}$ ________ %

Worksheet 1
Page 3 of 3

_____ **Situation 2**: This consists of land development where the existing percent impervious cover ($I_{existing}$) is less than or equal to the average land cover condition ($I_{watershed}$) and the proposed improvements will create a total percent impervious cover (I_{post}) which is greater than the average land cover condition ($I_{watershed}$).

$\mathbf{I_{existing}}$ ________% # $\mathbf{I_{watershed}}$ ________%; **and**

$\mathbf{I_{post}}$ ________% > $\mathbf{I_{watershed}}$ ________%

_____ **Situation 3**: This consists of land development where the existing percent impervious cover ($I_{existing}$) is greater than the average land cover condition ($I_{watershed}$).

$\mathbf{I_{existing}}$ ________% > $\mathbf{I_{watershed}}$ ________%

_____ **Situation 4**: This consists of land development where the existing percent impervious cover ($I_{existing}$) is served by an existing stormwater management BMP(s) that addresses water quality.

If the proposed development meets the criteria for development Situation 1, than the low density development is considered to be the BMP and no pollutant removal is required. The calculation procedure for Situation 1 stops here. If the proposed development meets the criteria for development Situations 2, 3, or 4, then proceed to STEP 4 on the appropriate worksheet.

Worksheet 2 : Situation 2

Page 1 of 4

Summary of Situation 2 criteria: from calculation procedure **STEP 1** thru **STEP 3**, Worksheet 1:

Applicable area (A)* = ________ acres

$\mathbf{I_{post}}$ = (total post-development impervious cover ÷ A) × 100 = ________%

$\mathbf{I_{watershed}}$ = ________% **or** $I_{watershed}$ = 16%

$\mathbf{I_{existing}}$ = (total existing impervious cover ÷ A*) × 100 = ________%

$\mathbf{I_{existing}}$ ________% # $\mathbf{I_{watershed}}$ ________%; **and**

$\mathbf{I_{post}}$ ________% > $\mathbf{I_{watershed}}$ ________%

STEP 4 **Determine the relative pre-development pollutant load (L_{pre}).**

$\mathbf{L_{pre(watershed)}} = [0.05 + (0.009 \times I_{watershed})] \times A \times 2.28$ (**Equation 5-16**)

where: $L_{pre(watershed)}$ = relative pre-development total phosphorous load (pounds per year)

$I_{watershed}$ = average land cover condition for specific watershed or locality **or** the Chesapeake Bay default value of 16% (percent expressed in whole numbers)

A = applicable area (acres)

$\mathbf{L_{pre(watershed)}}$ = [0.05 + (0.009 × ________)] × ________ × 2.28

= __________ pounds per year

Worksheet 2 : Situation 2

Page 2 of 4

STEP 5 Determine the relative post-development pollutant load (L_{post}).

$$\mathbf{L_{post}} = [0.05 + (0.009 \times I_{post})] \times A \times 2.28 \quad \textbf{(Equation 5-21)}$$

where:
L_{post} = relative post-development total phosphorous load (pounds per year)
I_{post} = post-development percent impervious cover (percent expressed in whole numbers)
A = applicable area (acres)

$\mathbf{L_{post}}$ = [0.05 + (0.009 × ________)] × ________ × 2.28

= __________ pounds per year

STEP 6 Determine the relative pollutant removal requirement (RR).

RR = L_{post} ! $L_{pre(watershed)}$

RR = ________ ! ________

= __________ pounds per year

STEP 7 Identify best management practice (BMP) for the site.

1. Determine the required pollutant removal efficiency for the site:

$$\mathbf{EFF} = (RR \div L_{post}) \times 100 \qquad \textbf{(Equation 5-22)}$$

where:
EFF = required pollutant removal efficiency (percent expressed in whole numbers)
RR = pollutant removal requirement (pounds per year)
L_{post} = relative post-development total phosphorous load (pounds per year)

EFF = (________ ÷ ________) × 100

= __________%

Worksheet 2 : Situation 2

Page 3 of 4

2. Select BMP(s) from **Table 5-15** and locate on the site:

BMP 1:____

BMP 2:____

BMP 3:____

3. Determine the pollutant load entering the proposed BMP(s):

$\mathbf{L_{BMP}} = [0.05 + (0.009 \times I_{BMP})] \times A \times 2.28$ **(Equation 5-23)**

where: L_{BMP} = relative post-development total phosphorous load entering proposed BMP (pounds per year)

I_{BMP} = post-development percent impervious cover of BMP drainage area (percent expressed in whole numbers)

A = drainage area of proposed BMP (acres)

$\mathbf{L_{BMP1}} = [0.05 + (0.009 \times$ ________$)] \times$ _______ $\times 2.28$

= __________ pounds per year

$\mathbf{L_{BMP2}} = [0.05 + (0.009 \times$ ________$)] \times$ ________ $\times 2.28$

= __________ pounds per year

$\mathbf{L_{BMP3}} = [0.05 + (0.009 \times$ ________$)] \times$ ________ $\times 2.28$

= __________ pounds per year

Worksheet 2 : Situation 2

Page 4 of 4

4. Calculate the pollutant load removed by the proposed BMP(s):

$\mathbf{L_{removed}} = Eff_{BMP} \times L_{BMP}$ **(Equation 5-24)**

where: $L_{removed}$ = Post-development pollutant load removed by proposed BMP (pounds per year)
Eff_{BMP} = pollutant removal efficiency of BMP (expressed in decimal form)
L_{BMP} = relative post-development total phosphorous load entering proposed BMP (pounds per year)

$\mathbf{L_{removed/BMP1}}$ = ________ × ________ = __________ pounds per year

$\mathbf{L_{removed/BMP2}}$ = ________ × ________ = __________ pounds per year

$\mathbf{L_{removed/BMP3}}$ = ________ × ________ = __________ pounds per year

5. Calculate the total pollutant load removed by the BMP(s):

$\mathbf{L_{removed/total}} = L_{removed/BMP1} + L_{removed/BMP2} + L_{removed/BMP3} + \ldots$ **(Equation 5-25)**

where: $L_{removed/total}$ = **total** pollutant load removed by proposed BMPs
$L_{removed/BMP1}$ = pollutant load removed by proposed BMP No. 1
$L_{removed/BMP2}$ = pollutant load removed by proposed BMP No. 2
$L_{removed/BMP3}$ = pollutant load removed by proposed BMP No. 3

$\mathbf{L_{removed/total}}$ = __________ + __________ + __________ + . . .

= __________ pounds per year

6. Verify compliance:

$\mathbf{L_{removed/total}}$ \$ **RR**

__________ \$ __________

Worksheet 3 : Situation 3

Page 1 of 5

Summary of Situation 3 criteria: from calculation procedure **STEP 1** thru **STEP 3**, Worksheet 1:

Applicable area (A)* = ________ acres

$\mathbf{I_{post}}$ = (total post-development impervious cover ÷ A) × 100 = ________%

$\mathbf{I_{watershed}}$ = ________% **or** $I_{watershed}$ = 16%

$\mathbf{I_{existing}}$ = (total existing impervious cover ÷ A*) × 100 = ________%

$\mathbf{I_{existing}}$ ________% > $\mathbf{I_{watershed}}$ ________%

STEP 4 **Determine the relative pre-development pollutant load (L_{pre}).**

1. Pre-development pollutant load based on the existing impervious cover:

$\mathbf{L_{pre(existing)}} = [0.05 + (0.009 \times I_{existing})] \times A \times 2.28$ **(Equation 5-17)**

where:

$L_{pre(existing)}$ = relative pre-development total phosphorous load (pounds per year)
$I_{existing}$ = existing site impervious cover (percent expressed in whole numbers)
A = applicable area (acres)

$\mathbf{L_{pre(existing)}}$ = [0.05 + (0.009 × ________)] × ________ × 2.28

= __________ pounds per year

Worksheet 3 : Situation 3

Page 2 of 5

2. Pre-development pollutant load based on the average land cover condition:

$$\mathbf{L_{pre(watershed)}} = [0.05 + (0.009 \times I_{watershed})] \times A \times 2.28 \quad (\textbf{Equation 5-16})$$

where:

$L_{pre(watershed)}$ = relative pre-development total phosphorous load (pounds per year)
$I_{watershed}$ = average land cover condition for specific watershed or locality **or** the Chesapeake Bay default value of 16% (percent expressed in whole numbers)
A = applicable area (acres)

$$\mathbf{L_{pre(watershed)}} = [0.05 + (0.009 \times \underline{\hspace{2cm}})] \times \underline{\hspace{2cm}} \times 2.28$$

$$= \underline{\hspace{2.5cm}} \text{ pounds per year}$$

STEP 5 **Determine the relative post-development pollutant load (L_{post}).**

$$\mathbf{L_{post}} = [0.05 + (0.009 \times I_{post})] \times A \times 2.28 \quad (\textbf{Equation 5-21})$$

where:

L_{post} = relative post-development total phosphorous load (pounds per year)
I_{post} = post-development percent impervious cover (percent expressed in whole numbers)
A = applicable area (acres)

$$\mathbf{L_{post}} = [0.05 + (0.009 \times \underline{\hspace{1.5cm}})] \times \underline{\hspace{1.5cm}} \times 2.28$$

$$= \underline{\hspace{2.5cm}} \text{ pounds per year}$$

STEP 6 **Determine the relative pollutant removal requirement (RR).**

$$\mathbf{RR} = L_{post} \; ! \; (0.9 \times L_{pre(existing)})$$

$$= \underline{\hspace{2cm}} \; ! \; (0.9 \times \underline{\hspace{2cm}}) = \underline{\hspace{2.5cm}} \text{ pounds per year}$$

or

$$\mathbf{RR} = L_{post} \; ! \; L_{pre(watershed)}$$

$$= \underline{\hspace{2cm}} \; ! \; \underline{\hspace{2cm}} = \underline{\hspace{2.5cm}} \text{ pounds per year}$$

Worksheet 3 : Situation 3
Page 3 of 5

The pollutant removal requirement (RR) for Situation 3 is the lesser of the two values calculated above:

RR = __________ pounds per year

STEP 7 **Identify best management practice (BMP) for the site.**

1. Determine the required pollutant removal efficiency for the site:

EFF = (RR ÷ L_{post}) × 100 **(Equation 5-22)**

where: EFF = required pollutant removal efficiency (percent expressed in whole numbers)
RR = pollutant removal requirement (pounds per year)
L_{post} = relative post-development total phosphorous load (pounds per year)

EFF = (________ ÷ ________) × 100

= __________%

2. Select BMP(s) from **Table 5-15** and locate on the site:

BMP 1:____

BMP 2:____

BMP 3:____

Worksheet 3 : Situation 3

3. Determine the pollutant load entering the proposed BMP(s):

$\mathbf{L_{BMP}} = [0.05 + (0.009 \times I_{BMP})] \times A \times 2.28$ **(Equation 5-23)**

where:
L_{BMP} = relative post-development total phosphorous load entering proposed BMP (pounds per year)
I_{BMP} = post-development percent impervious cover of BMP drainage area (percent expressed in whole numbers)
A = drainage area of proposed BMP (acres)

$\mathbf{L_{BMP1}} = [0.05 + (0.009 \times$ ________$)] \times$ _______ $\times 2.28$

= __________ pounds per year

$\mathbf{L_{BMP2}} = [0.05 + (0.009 \times$ ________$)] \times$ ________ $\times 2.28$

= __________ pounds per year

$\mathbf{L_{BMP3}} = [0.05 + (0.009 \times$ ________$)] \times$ ________ $\times 2.28$

= __________ pounds per year

4. Calculate the pollutant load removed by the proposed BMP(s):

$\mathbf{L_{removed}} = Eff_{BMP} \times L_{BMP}$ **(Equation 5-24)**

where:
$L_{removed}$ = Post-development pollutant load removed by proposed BMP (pounds per year)
Eff_{BMP} = pollutant removal efficiency of BMP (expressed in decimal form)
L_{BMP} = relative post-development total phosphorous load entering proposed BMP (pounds per year)

$\mathbf{L_{removed/BMP1}}$ = ________ × ________ = __________ pounds per year

$\mathbf{L_{removed/BMP2}}$ = ________ × ________ = __________ pounds per year

$\mathbf{L_{removed/BMP3}}$ = ________ × ________ = __________ pounds per year

Worksheet 3 : Situation 3

Page 5 of 5

5. Calculate the total pollutant load removed by the BMP(s):

$\mathbf{L_{removed/total}} = L_{removed/BMP1} + L_{removed/BMP2} + L_{removed/BMP3} + \ldots$ **(Equation 5-25)**

where: $L_{removed/total}$ = **total** pollutant load removed by proposed BMPs
$L_{removed/BMP1}$ = pollutant load removed by proposed BMP No. 1
$L_{removed/BMP2}$ = pollutant load removed by proposed BMP No. 2
$L_{removed/BMP3}$ = pollutant load removed by proposed BMP No. 3

$\mathbf{L_{removed/total}}$ = __________ + __________ + __________ + . . .

= __________ pounds per year

6. Verify compliance:

$\mathbf{L_{removed/total}} \ \$ \ \mathbf{RR}$

_________ \$ _________

<u>Worksheet 4 : Situation 4</u>

Page 1 of 6

Summary of Situation 3 criteria: from calculation procedure **<u>STEP 1</u>** thru **<u>STEP 3</u>**, Worksheet 1:

Applicable area (A) = ________ acres

$\mathbf{I_{post}}$ = (total post-development impervious cover ÷ A) × 100 = ________%

$\mathbf{I_{watershed}}$ = ________% **or** $I_{watershed}$ = 16%

$\mathbf{I_{existing}}$ = (total existing impervious cover ÷ A*) × 100 = ________%

$\mathbf{I_{existing}}$ ________% > $\mathbf{I_{watershed}}$ ________%

Summary of existing BMP:

Existing BMP drainage area ($\mathbf{A_{existBMP}}$) = ________ acres

$\mathbf{I_{pre(BMP)}}$ = (total pre-development impervious cover ÷ $A_{existBMP}$) × 100 = ________%

$\mathbf{EFF_{existBMP}}$ = documented pollutant removal efficiency of existing BMP (expressed in decimal form)

<u>STEP 4</u> Determine the relative pre-development pollutant load ($\mathbf{L_{pre}}$).

1. Calculate pre-development pollutant load based on the existing impervious cover:

$\mathbf{L_{pre(existing)}} = [0.05 + (0.009 \times I_{existing})] \times A \times 2.28$ **(Equation 5-17)**

where:

$L_{pre(existing)}$ = relative pre-development total phosphorous load (pounds per year)
$I_{existing}$ = existing site impervious cover (percent expressed in whole numbers)
A = applicable area (acres)

$\mathbf{L_{pre(existing)}} = [0.05 + (0.009 \times$ ________$)] \times$ ________ $\times 2.28$

= __________ pounds per year

Worksheet 4 : Situation 4

Page 2 of 6

2. Calculate pre-development pollutant load to existing BMP:

$\mathbf{L_{pre(BMP)}} = [0.05 + (0.009 \times I_{pre(BMP)})] \times A_{existBMP} \times 2.28$ **(Equation 5-18)**

where:

$L_{pre(BMP)}$ = relative pre-development total phosphorous load to existing BMP (pounds per year)

$I_{pre(BMP)}$ = existing impervious cover to existing BMP (percent expressed in whole numbers)

$A_{existBMP}$ = drainage area of existing BMP (acres)

$L_{pre(BMP)} = [0.05 + (0.009 \times$ ________$)] \times$ ________ $\times 2.28$

= __________ pounds per year

3. Calculate pre-development pollutant load removed by existing BMP:

$\mathbf{L_{removed(existingBMP)}} = L_{pre(BMP)} \times EFF_{existBMP}$ **(Equation 5-19)**

where:

$L_{removed(existingBMP)}$ = relative pre-development total phosphorous load removed by existing BMP (pounds per year)

$L_{pre(BMP)}$ = relative pre-development total phosphorous load entering existing BMP, **Equation 5-18** (pounds per year)

$EFF_{existBMP}$ = documented pollutant removal efficiency of existing BMP (expressed in decimal form)

$\mathbf{L_{removed(existingBMP)}}$ = ________ $\times$ ________

= __________ pounds per year

Steps 2 and 3 are repeated for each existing BMP on the site.

Worksheet 4: Situation 4
Page 3 of 6

4. Calculate the pre-development pollutant load while being served by existing BMP(S):

$$\mathbf{L_{pre(existingBMP)}} = L_{pre(existing)} \;!\; (L_{removed(existingBMP1)} + L_{removed(existingBMP2)} + L_{removed(existingBMP3)})$$

Equation 5-20

where:

$L_{pre(existingBMP)}$ = relative pre-development total phosphorous load while being served by an existing BMP (pounds per year)

$L_{pre(existing)}$ = relative pre-development total phosphorous load based on existing site conditions, **Equation 5-17** (pounds per year)

$EFF_{existBMP}$ = documented pollutant removal efficiency of existing BMP (expressed in decimal form)

$L_{removed(existingBMP)}$ = relative pre-development total phosphorous load removed by existing BMP, **Equation 5-19** (pounds per year)

$\mathbf{L_{pre(existingBMP)}}$ = _________ ! (_________ + _________ + _________)

= ____________ pounds per year

STEP 5 **Determine the relative post-development pollutant load (L_{post}).**

$\mathbf{L_{post}} = [0.05 + (0.009 \times I_{post})] \times A \times 2.28$ (**Equation 5-21**)

where:

L_{post} = relative post-development total phosphorous load (pounds per year)

I_{post} = post-development percent impervious cover (percent expressed in whole numbers)

A = applicable area (acres)

$\mathbf{L_{post}}$ = [0.05 + (0.009 × ______)] × ______ × 2.28

= __________ pounds per year

STEP 6 **Determine the relative pollutant removal requirement (RR).**

RR = L_{post} ! $L_{pre(existingBMP)}$

= ________ ! ________

RR = __________ pounds per year

Worksheet 4 : Situation 4
Page 4 of 6

2. Pre-development pollutant load based on the average land cover condition:

$\mathbf{L_{pre(watershed)}} = [0.05 + (0.009 \times I_{watershed})] \times A \times 2.28$ (**Equation 5-16**)

where: $L_{pre(watershed)}$ = relative pre-development total phosphorous load (pounds per year)
$I_{watershed}$ = average land cover condition for specific watershed or locality **or** the Chesapeake Bay default value of 16% (percent expressed in whole numbers)
A = applicable area (acres)

$\mathbf{L_{pre(watershed)}} = [0.05 + (0.009 \times$ ________$)] \times$ ________ $\times 2.28$

= __________ pounds per year

STEP 7 **Identify best management practice (BMP) for the site.**

1. Determine the required pollutant removal efficiency for the site:

EFF = $(RR \div L_{post}) \times 100$ **(Equation 5-22)**

where: EFF = required pollutant removal efficiency (percent expressed in whole numbers)
RR = pollutant removal requirement (pounds per year)
L_{post} = relative post-development total phosphorous load (pounds per year)

EFF = (________ ÷ ________) × 100

= __________%

2. Select BMP(s) from **Table 5-15** and locate on the site:

BMP 1:____

BMP 2:____

BMP 3: ____

Worksheet 4 : Situation 4
Page 5 of 6

3. Determine the pollutant load entering the proposed BMP(s):

$\mathbf{L_{BMP}} = [0.05 + (0.009 \times I_{BMP})] \times A \times 2.28$ **(Equation 5-23)**

where: L_{BMP} = relative post-development total phosphorous load entering proposed BMP (pounds per year)
I_{BMP} = post-development percent impervious cover of BMP drainage area (percent expressed in whole numbers)
A = drainage area of proposed BMP (acres)

$\mathbf{L_{BMP1}} = [0.05 + (0.009 \times$ ________$)] \times$ _______ $\times 2.28$

= __________ pounds per year

$\mathbf{L_{BMP2}} = [0.05 + (0.009 \times$ ________$)] \times$ ________ $\times 2.28$

= __________ pounds per year

$\mathbf{L_{BMP3}} = [0.05 + (0.009 \times$ ________$)] \times$ ________ $\times 2.28$

= __________ pounds per year

4. Calculate the pollutant load removed by the proposed BMP(s):

$\mathbf{L_{removed}} = Eff_{BMP} \times L_{BMP}$ **(Equation 5-24)**

where: $L_{removed}$ = Post-development pollutant load removed by proposed BMP (pounds per year)
Eff_{BMP} = pollutant removal efficiency of BMP (expressed in decimal form)
L_{BMP} = relative post-development total phosphorous load entering proposed BMP (pounds per year)

$\mathbf{L_{removed/BMP1}}$ = ________ × ________ = __________ pounds per year

$\mathbf{L_{removed/BMP2}}$ = ________ × ________ = __________ pounds per year

$\mathbf{L_{removed/BMP3}}$ = ________ × ________ = __________ pounds per year

Worksheet 4 : Situation 4

Page 6 of 6

5. Calculate the total pollutant load removed by the existing and proposed BMP(s):

$$\mathbf{L_{removed/total}} = L_{removed/BMP1} + L_{removed/BMP2} + L_{removed/BMP3} + $$

$$+ \; L_{removed(existingBMP1)} + L_{removed(existingBMP2)} + L_{removed(existingBMP3)} \qquad \textbf{(Equation 5-25)}$$

where: $L_{removed/total}$ = **total** pollutant load removed by proposed BMPs

$L_{removed/BMP1}$ = pollutant load removed by proposed BMP No. 1, **Equation 5-24**

$L_{removed/BMP2}$ = pollutant load removed by proposed BMP No. 2, **Equation 5-24**

$L_{removed/BMP3}$ = pollutant load removed by proposed BMP No. 3, **Equation 5-24**

$L_{removed(existingBMP)}$ = pollutant load removed by existing BMP No. 1, **Equation 5-19**

$L_{removed(existingBMP)}$ = pollutant load removed by existing BMP No. 2, **Equation 5-19**

$L_{removed(existingBMP)}$ = pollutant load removed by existing BMP No. 3, **Equation 5-19**

$\mathbf{L_{removed/total}}$ = __________ + __________ + __________ + . . .

= __________ pounds per year

6. Verify compliance:

$\mathbf{L_{removed/total}} \; \$ \; \mathbf{RR}$

__________ $\$$ __________