CHAPTER

3

STORMWATER HYDROLOGY

3.1 Introduction to Hydrologic Methods

Hydrology is the science dealing with the characteristics, distribution, and movement of water on and below the earth's surface and in the atmosphere. Hydrology in this manual shall be limited to estimating flow peaks, volumes, and time distributions of stormwater runoff. The analysis of these parameters is fundamental to the design of stormwater management facilities, such as storm drainage systems and structural stormwater BMPs. In the hydrologic analysis of a development site, there are a number of variable factors that affect the nature of stormwater runoff from the site. Some of the factors that need to be considered include:

- rainfall amount and storm distribution;
- drainage area size, shape and orientation;
- ground cover and soil type;
- slopes of terrain and stream channel(s);
- antecedent moisture condition;
- storage potential (floodplains, ponds, wetlands, reservoirs, channels, etc.);
- watershed development potential; and
- characteristics of the local drainage system.

There are a number of empirical hydrologic methods that can be used to estimate runoff characteristics for a site or drainage subbasin; however, the following methods presented in this chapter have been selected to support hydrologic site analysis for the design methods and procedures included in the Manual:

- Rational Method;
- United States Geological Survey (USGS) and Tennessee Valley Authority (TVA) Regression Equations;
- Soil Conservation Service (SCS) Unit Hydrograph Method;
- Clark Unit Hydrograph;
- Water Quality Volume (WQv) Calculation; and
- Water Balance Calculations.

These methods were selected based upon their accuracy in duplicating local hydrologic estimates for a range of design storms and the availability of equations, nomographs, and computer programs to support them.

Table 3-1 lists the hydrologic methods and the circumstances for their use in various analysis and design applications, relevant to regulations and policies in Knox County. Table 3-2 provides some limitations on the use of several methods.

Table 3-1. Design Applications for Recommended Hydrologic Methods

Analysis or Design Application	Manual Section	Rational Method	USGS Equations	SCS Method	Clark Unit Hydrograph	Water Quality Volume	TVA Equations
Water Quality Volume (WQv)	2.2.3					✓	
Channel Protection Volume(CPv)	2.3			✓			
Overbank Flood Protection(Qp_2, Qp_{10},Qp_{25})	2.4.1			✓			
Extreme Flood Protection (Qp_{100})	2.4.2			✓			
Storage Facilities	3.2			✓			
Outlet Structures	3.3			✓			
Gutter Flow and Inlets	7.6	✓					
Storm Drain Pipes	7.2	✓	✓	✓			✓
Culverts	7.3	✓	✓	✓			✓
Small Ditches	7.4	✓	✓	✓			✓
Open Channels	7.4		✓	✓			✓
Energy Dissipation	7.5		✓	✓			✓
Flood Studies	8.4.3		✓	✓	✓		✓

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Table 3-2. Constraints on Using Recommended Hydrologic Methods

Method	Size Limitations[1]	Comments
Rational	0 – 5 acres	Method can be used for estimating peak flows and the design of small site or subdivision storm sewer systems. Not to be used for storage design.
USGS Rural	0.36 mi^2 to 21,400 mi^2	Method can be used for estimating peak flows for all design applications in rural areas.
USGS Urban	0.21 mi^2 to 24.3 mi^2	Method can be used for estimating hydrographs for all design applications in urban areas.
TVA	> 0.36 mi^2	Method can be used for estimating peak flows for storm system design applications such as culverts, channels, etc.
SCS[2,3]	0 – 2000 acres	Method can be used for estimating peak flows and hydrographs for all design applications.
Clark[2]	See Comments	Method may not be applicable to very large drainage basins. Large drainage basins may need to be subdivided to overcome limitations of this method.
Water Quality	Limits set for each Structural Control	Method used for calculating the WQv

1 - Size limitation refers to the drainage basin for the stormwater management facility (e.g., culvert, inlet).
2 - There are many readily available programs (such as HEC-1) that utilize this methodology.
3 - 2,000-acre upper size limit applies to single basin simplified peak flow only.

In general:

- the Rational Method is recommended for small, highly impervious drainage areas such as parking lots and roadways draining into inlets and gutters; and
- the USGS regression equations are recommended for drainage areas with characteristics within the ranges given for the equations. The USGS equations should be used with caution when there are significant storage areas within the drainage basin or where other drainage characteristics indicate that general regression equations might not be appropriate; and
- the TVA regression equations are used for stormwater system design (discussed in Chapter 7), choosing the more conservative solution from between the results of the applicable USGS regression equation and the TVA regression equation.

Note: Users must realize that any hydrologic analysis is only an approximation. The relationship between the amount of precipitation on a drainage basin and the amount of runoff from the basin is complex and too little data are available on the factors influencing the rainfall-runoff relationship to expect exact solutions.

3.1.1 Symbols and Definitions

To provide consistency within this section, the symbols listed in Table 3-3 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this manual, the symbol will be defined where it occurs in the text or equations.

Table 3-3. Symbols and Definitions for Stormwater Runoff

Symbol	Definition	Units
A	Drainage area	acres (or mi^2)
B_f	Baseflow	cfs
C	Runoff coefficient	-
C_f	Frequency factor	-
CN	SCS-runoff curve number	-
CPv	Channel protection volume	acre-feet
d	Time interval	hours
D_{wq}	Water quality runoff depth	in
E	Evaporation	ft
E_t	Evapotranspiration	ft
F_p	Pond and swamp adjustment factor	-
G_h	Hydraulic gradient	ft/ft
I or i	Runoff intensity	in/hr
IA	Percent of impervious cover	%
I	Infiltration	ft
I	Inflows	cfs
I_a	Initial abstraction from total rainfall	in
k_h	Infiltration rate	ft/day
L	Flow length	ft
n	Manning roughness coefficient (Manning's "n")	-
O_f	Overflow	acre-feet
O	Outflows	cfs
P	Accumulated rainfall	in
P_2	2-year, 24-hour rainfall	in
P_w	Wetted perimeter	ft

Symbol	Definition	Units
PF	Peaking factor	-
Q	Rate of runoff or depth of runoff	cfs or inches
Qp	Peak rate of discharge	cfs
Qp_2	2-year event peak discharge	cfs
Qp_{10}	10-year event peak discharge	cfs
Qp_{25}	25-year event peak discharge	cfs
Qp_{100}	100-year event peak discharge	cfs
Q_{wq}	Water quality peak discharge	cfs
Q_{wv}	Water quality runoff peak volume	in
q	Storm runoff during a time interval	in
q_u	Unit peak discharge	cfs (or cfs/mi^2/inch)
R	Clark watershed storage constant	hours
R	Hydraulic radius	ft
R_o	Runoff	acre-feet
Rv	Runoff coefficient	-
S	Ground slope	ft/ft or %
S	Potential maximum retention	in
S	Slope of hydraulic grade line	ft/ft
T_L	Lag time	hours
T_p	Time to peak	hours
T_t or t_t	Travel time	min or hours
t	Time	min
Tc	Time of concentration	min
TIA	Total impervious area	%
V	Velocity	ft/s
V	Pond volume	acre-feet
WQv	Water quality volume	acre-feet
W_s	Average ground surface slope as a percentage	%

3.1.2 Rainfall Estimation

The first step in any hydrologic analysis is an estimation of the rainfall that will fall on the site for a given time period. The amount of rainfall can be quantified with the following characteristics:

Duration (hours) – Length of time over which rainfall (storm event) occurs;

Depth (inches) – Total amount of rainfall occurring during the storm duration; and

Intensity (inches per hour) – Rate of rainfall or depth divided by the duration

The frequency of a rainfall event is the recurrence interval of storms having the same duration and volume (depth). This can be expressed either in terms of *exceedance probability* or *return period*.

Exceedance Probability – Probability that a storm event having the specified duration and volume will be exceeded in one given time period, typically 1-year.

Return Period – Average length of time between events that have the same duration and volume.

Thus, if a storm event with a specified duration and volume has a 1% chance of occurring in any given year, then it has an exceedance probability of 0.01 and a return period of 100-years. A design storm event over 24-hours with a 1% chance of occurring in any given year is often referred to as the 100-year, 24-hour storm. This design storm would be developed based on assumptions regarding intensity and distribution of the storm over the specified timeframe (24-hours for this scenario). Therefore, a design storm event is used to estimate actual storm events even though it would be very unlikely that an actual storm event would match up with all of the design storm event assumptions.

Rainfall intensities for Knox County are provided in Table 3-4 and should be used for all hydrologic analysis. The sources of the values in this table are the Weather Bureau Technical Papers TP-25 and TP-40 (Hershfield, 1961) and National Weather Service publication Hydro-35 (NOAA, 1977). The intensity values have been adjusted to produce smooth intensity-duration-frequency (IDF) curves and cumulative rainfall distributions. Table 3-5 shows the rainfall depths for hypothetical storm events.

Figure 3-1 shows the IDF curves for Knox County for the 1, 2, 5, 10, 25, and 100-year, 24-hour storms. These curves are plots of the tabular values. No values are given for times less than 5 minutes.

Table 3-4. Intensity-Duration-Frequency Curve Data

(Sources: Hershfield, 1961; NOAA, 1977)

ARI[1] (years)		24-Hour Precipitation Frequency Estimates (inches/hour) by Return Periods					
Hours	**Minutes**	**2-year**	**5-year**	**10-year**	**25-year**	**50-year**	**100-year**
0.083	5	4.60	5.55	6.25	7.30	7.90	8.60
0.170	10	3.70	4.60	5.25	6.20	6.80	7.49
0.250	15	3.19	3.98	4.60	5.45	6.00	6.60
0.330	20	2.82	3.50	4.10	4.90	5.45	6.02
0.420	25	2.48	3.12	3.70	4.45	4.95	5.50
0.500	30	2.22	2.80	3.34	4.03	4.53	5.03
0.580	35	2.02	2.55	3.06	3.67	4.14	4.62
0.670	40	1.86	2.35	2.82	3.38	3.80	4.24
0.750	45	1.73	2.18	2.62	3.14	3.53	3.93
0.830	50	1.62	2.04	2.46	2.94	3.30	3.67
0.920	55	1.53	1.92	2.32	2.77	3.10	3.45
1.000	60	1.45	1.82	2.20	2.62	2.93	3.26
1.500	90	1.06	1.36	1.64	1.95	2.18	2.45
2.000	120	0.86	1.09	1.31	1.55	1.71	1.95
3.000	180	0.66	0.80	0.97	1.13	1.23	1.38
6.000	360	0.41	0.50	0.58	0.66	0.75	0.83
12.000	720	0.24	0.30	0.34	0.39	0.43	0.48
24.000	1440	0.14	0.17	0.20	0.23	0.25	0.27

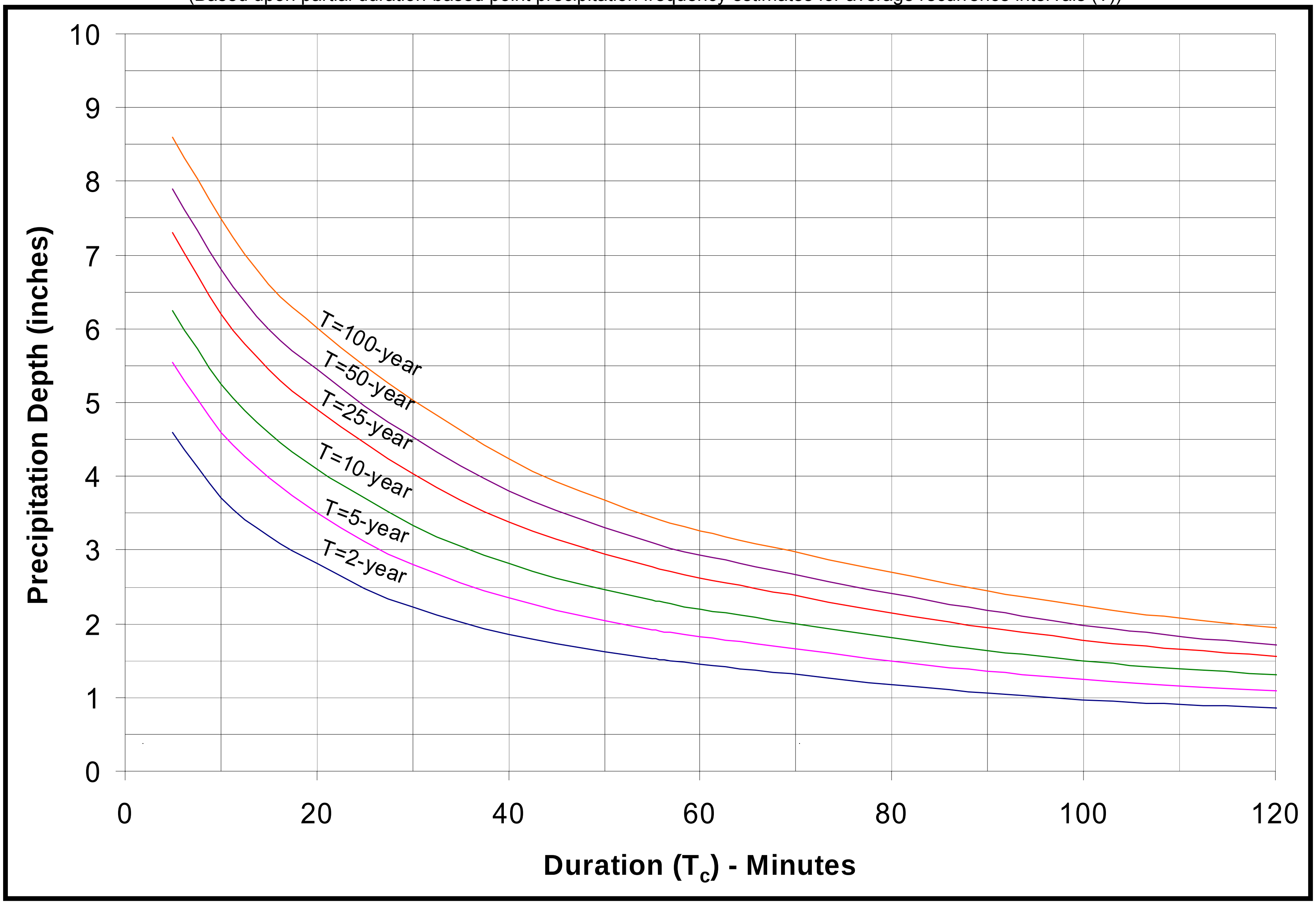
1 - ARI= Average Recurrence Interval

Table 3-5. Rainfall Depths for Hypothetical Storm Events

Rainfall Depths for Hypothetical Storm Events	
Storm Event	**24-Hr Depth (in)**
1-year	2.5
2-year	3.3
5-year	4.1
10-year	4.8
25-year	5.5
100-year	6.5

Figure 3-1. Intensity-Duration-Frequency-(IDF) Curves for Knox County 24-hour Storms

(Based upon partial duration-based point precipitation frequency estimates for average recurrence intervals (T))



3.1.3 Rational Method

A popular approach for determining the peak runoff rate is the Rational Formula. The Rational Method considers the entire drainage area as a single unit and estimates the peak discharge at the most downstream point of that area.

The Rational Formula follows the assumptions that:

- the rainfall is uniformly distributed over the entire drainage area and is constant over time;
- the predicted peak discharge has the same probability of occurrence (return period) as the used rainfall intensity (I);
- peak runoff rate can be represented by the rainfall intensity averaged over the same time period as the drainage area's time of concentration (Tc); and
- the runoff coefficient (C) is constant during the storm event.

When using the Rational Method some precautions should be considered:

- in determining the C value (runoff coefficient based on land use) for the drainage area, hydrologic analysis should take into account any future changes in land use that might occur during the service life of the proposed facility;
- if the distribution of land uses within the drainage basin will affect the results of hydrologic analysis (e.g., if the impervious areas are segregated from the pervious areas), the basin should be divided into sub-drainage basins. The single equation used for the Rational Method uses one composite C and one Tc value for the entire drainage area; and,
- the charts, graphs, and tables included in this section are given to assist the engineer in applying the Rational Method. The engineer shall use sound engineering judgment in applying these design aids and shall make appropriate adjustments when specific site characteristics dictate that these adjustments are appropriate.

3.1.3.1 Application

The Rational Method can be used to estimate stormwater runoff peak flows for the design of gutter flows, drainage inlets, storm drain pipe, culverts and small ditches. It is most applicable to small, highly impervious areas. Knox County policies regarding the use of the Rational Method are as follows:

- In Knox County, the Rational Method shall not be utilized for drainage areas greater than five (5) acres.
- The Rational Method shall not be used for storage design or any other application where a more detailed routing procedure is required.
- The Rational Method shall not be used for calculating peak flows downstream of bridges, culverts or storm sewers that may act as restrictions and impact the peak rate of discharge.

3.1.3.2 Equations

The Rational Method estimates the peak rate of runoff at a specific watershed location as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration, Tc. The Tc is the time required for water to flow from the most remote point of the basin to the location being analyzed.

The Rational Method is expressed in Equation 3-1. Further explanation of each variable in the Rational Method equation is presented in Sections 3.1.3.3 and 3.1.3.4.

Equation 3-1 $$Q = CIA$$

where:

- Q = maximum rate of runoff (cfs)
- C = runoff coefficient representing a ratio of runoff to rainfall
- I = average rainfall intensity for a duration equal to the Tc (in/hr)
- A = drainage area contributing to the design location (acres)

3.1.3.3 Runoff Coefficient

The runoff coefficient (C) is the variable of the Rational Method least susceptible to precise determination and requires judgment and understanding on the part of the design engineer. While engineering judgment will always be required in the selection of runoff coefficients, typical coefficients represent the integrated effects of many drainage basin parameters. Table 3-6 gives the recommended runoff coefficients for the Rational Method.

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage areas. Composites can be made with the values from Table 3-6 by using percentages of different land uses. In addition, more detailed composites can be made with coefficients for different surface types such as rooftops, asphalt, and concrete. The composite procedure can be applied to an entire drainage area or to typical “sample" blocks as a guide to the selection of reasonable values of the coefficient for an entire area.

It should be remembered that the Rational Method assumes that all land uses within a drainage area are uniformly distributed throughout the area. If it is important to locate a specific land use within the drainage area, then another hydrologic method should be used where hydrographs can be generated and routed through the drainage system.

Using only the impervious area from a highly impervious site (and the corresponding high C factor and shorter time of concentration) can in some cases yield a higher peak runoff value than by using the whole site. Peak flow calculations can be underestimated due to areas where the overland portion of flow is grassy (yielding a longer Tc).

Note that the coefficients given in Table 3-6 are applicable for storms of 5 to 10-year frequencies. Less frequent, higher intensity storms may require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff (Wright - McLaughlin Engineers, 1969). The adjustment of the Rational Method for use with major storms can be made by multiplying the right side of the Rational Formula by a frequency factor Cf. The Rational Formula for major storm events now becomes:

Equation 3-2 $$Q = C_f CIA$$

C_f values are listed in Table 3-6. The product of C_f times C shall not exceed 1.0.

Table 3-6. Frequency Factors for Rational Formula

Recurrence Interval (years)	C_f
10 or less	1.0
25	1.1
50	1.2
100	1.25

Table 3-7. Recommended Runoff Coefficient Values for Rational Method

Land Use	Runoff Coefficient (C) by Hydrologic Soil Group and Ground Slope											
	A			B			C			D		
	<2%	2 - 6%	>6%	<2%	2 - 6%	>6%	<2%	2 - 6%	>6%	<2%	2 - 6%	>6%
Forest	0.08	0.11	0.14	0.10	0.14	0.18	0.12	0.16	0.20	0.15	0.20	0.25
Meadow	0.14	0.22	0.30	0.20	0.28	0.37	0.26	0.35	0.44	0.30	0.40	0.50
Pasture	0.15	0.25	0.37	0.23	0.34	0.45	0.30	0.42	0.52	0.37	0.50	0.62
Farmland	0.14	0.18	0.22	0.16	0.21	0.28	0.20	0.25	0.34	0.24	0.29	0.41
Res. 1 acre	0.22	0.26	0.29	0.24	0.28	0.34	0.28	0.32	0.40	0.31	0.35	0.46
Res. 1/2 acre	0.25	0.29	0.32	0.28	0.32	0.36	0.31	0.35	0.42	0.34	0.38	0.46
Res. 1/3 acre	0.28	0.32	0.35	0.30	0.35	0.39	0.33	0.38	0.45	0.36	0.40	0.50
Res. 1/4 acre	0.30	0.34	0.37	0.33	0.37	0.42	0.36	0.40	0.47	0.38	0.42	0.52
Res. 1/8 acre	0.33	0.37	0.40	0.35	0.39	0.44	0.38	0.42	0.49	0.41	0.45	0.54
Industrial	0.85	0.85	0.86	0.85	0.86	0.86	0.86	0.86	0.87	0.86	0.86	0.88
Commercial	0.88	0.88	0.89	0.89	0.89	0.89	0.89	0.89	0.90	0.89	0.89	0.90
Streets: ROW	0.76	0.77	0.79	0.80	0.82	0.84	0.84	0.85	0.89	0.89	0.91	0.95
Parking	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97
Disturbed Area	0.65	0.67	0.69	0.66	0.68	0.70	0.68	0.70	0.72	0.69	0.72	0.75

3.1.3.4 Rainfall Intensity (I)

The rainfall intensity (I) is the average rainfall rate in in/hr for a selected return period that is based on a duration equal to the time of concentration (Tc). Once a particular return period has been selected for design and a time of concentration has been calculated for the drainage area, the rainfall intensity can be determined from rainfall-intensity-duration data given in Table 3-4 or Figure 3-1. Calculation of Tc is discussed in detail in the next section.

3.1.3.5 Time of Concentration

Use of the Rational Method requires calculating the time of concentration (Tc) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I). The basin time of concentration is defined as the time required for water to flow from the most remote part of the drainage area to the point of interest for discharge calculations. The time of concentration is computed as a summation of travel times within each flow path as follows:

Equation 3-3

$$T_C = t_{t1} + t_{t2} + t_{tm}$$

where:

Tc = time of concentration (hours)
t_t = travel time of segment (hours)
m = number of flow segments

Knox County policies regarding the calculation of Tc are as follows:

- The Tc shall be the longest sub-basin travel time when all flow paths are considered.
- The minimum Tc for all computations shall be five (5) minutes.

Time of concentration calculations are subject to the following limitations:

1. the equations presented in this section should not be used for overland (i.e., sheet) flow on impervious land uses where the flow length is longer than 50 feet; and
2. in watersheds with storm sewers, use care to identify the appropriate hydraulic flow path to estimate Tc.

Two common errors should be avoided when calculating time of concentration. First, in some cases runoff from a highly impervious portion of a drainage area may result in a greater peak discharge than the calculated peak discharge for the entire area. Second, the designer should consider that the overland flow path does not necessarily remain the same when comparing pre-development and post-development areas. Grading operations and development can alter the overland flow path and length. Selecting overland flow paths for impervious areas that are greater than 50 feet should be done only after careful consideration. For typical urban areas, the time of concentration consists of multiple flow paths including overland flow, shallow concentrated flow and the travel time in the storm drain, paved gutter, roadside ditch, or drainage channel.

Overland Flow:

Overland flow in urbanized basins occurs from the backs of lots to the street, across and within parking lots and grass belts, and within park areas, and is characterized as shallow, steady and uniform flow with minor infiltration effects. The travel time (T_t) for sheet flow over plane surfaces for distances of less than 100 lineal feet for unpaved surfaces (50 feet for paved surfaces) can be calculated using Manning's kinematic solution (Overton and Meadows, 1976), shown in Equation 3-4. Following the equation, Table 3-8 presents Manning's “n” roughness coefficients for use in Equation 3-4.

Equation 3-4

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} S^{0.4}}$$

where:

T_t = travel time (hours)
n = Manning's roughness coefficient (see Table 3-8)
L = flow length (ft)
P_2 = 2-year 24-hour rainfall (inches)
S = ground slope, (ft/ft)

Table 3-8. Roughness coefficients for Overland (Sheet) Flow (Manning's "n")[1]
(Source: Soil Conservation Service, 1986)

Surface Description	n
Smooth surfaces (concrete, asphalt, gravel or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses[2]	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods[3]:	
Light underbrush	0.40
Dense underbrush	0.80

[1] The n values are a composite of information by Engman (1986).
[2] Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue gamagrass, and native grass mixtures.
[3] When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

Shallow Concentrated Flow:
After a maximum of 100 feet (50 feet for paved areas), overland flow will normally become shallow concentrated flow. The average velocity of this flow can be determined from Figure 3-2, in which average velocity is a function of watercourse slope and type of channel. Equations 3-5 and 3-6 can be used to determine the average flow velocity on paved and unpaved surfaces for slopes less than the minimum slope in Figure 3-2 (0.005 ft/ft):

Equation 3-5 Unpaved $V = 16.13(S)^{0.5}$

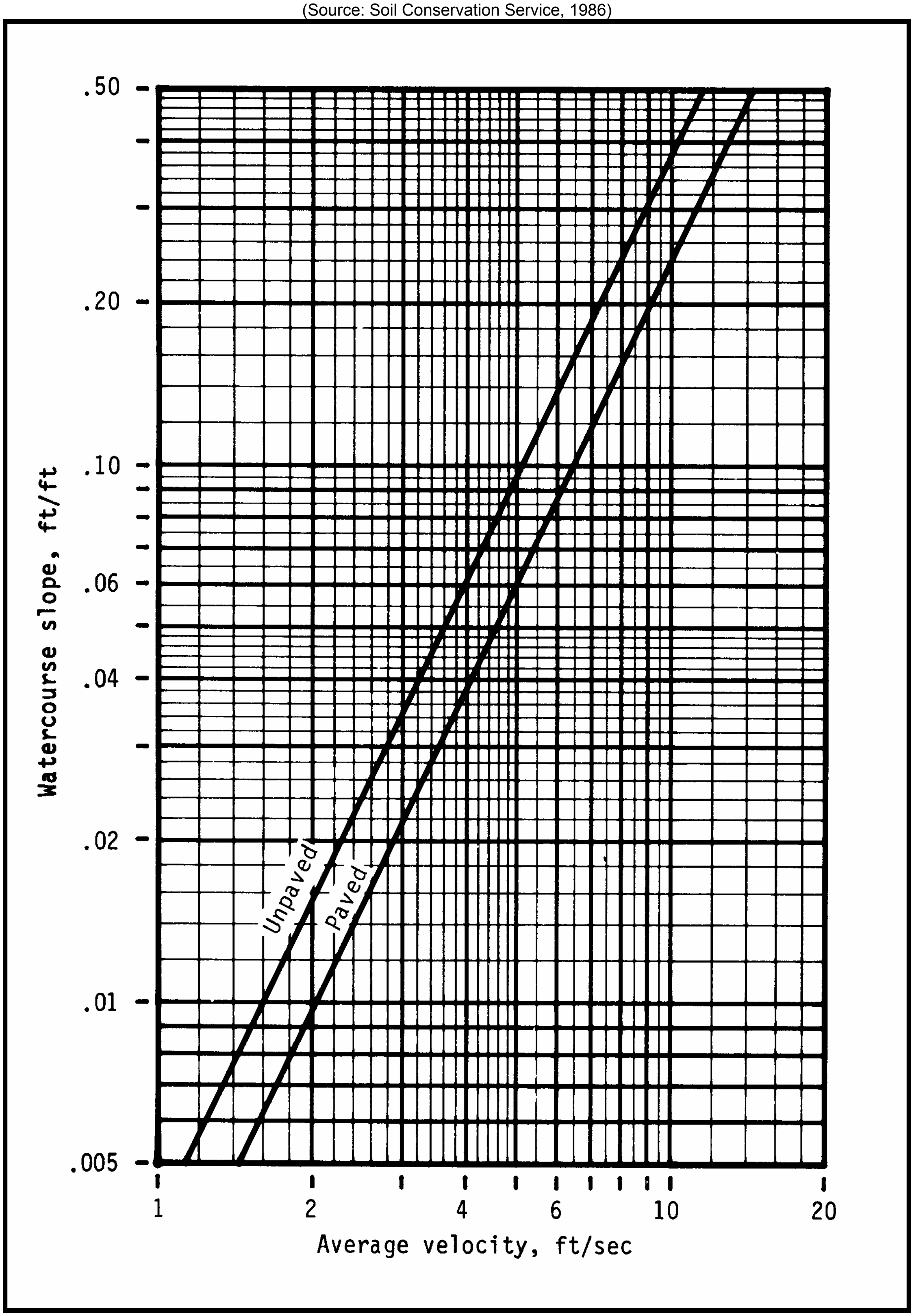
Equation 3-6 Paved $V = 20.33(S)^{0.5}$

where:

V = average velocity (ft/s), and
S = slope of hydraulic grade line (watercourse slope, ft/ft)

After determining average velocity, use Equation 3-7 to estimate travel time for the shallow concentrated flow segment.

Figure 3-2. Average Velocities - Shallow Concentrated Flow
(Source: Soil Conservation Service, 1986)



Equation 3-7

$$T_t = \frac{L}{60V}$$

where:

- T_t = travel time (min)
- L = reach length (ft)
- V = velocity in reach (ft/sec)

Paved Gutter and Open Channel Flow:
The travel time within the storm drain, gutter, swale, ditch, or other drainage way can be determined through an analysis of the hydraulic properties of these conveyance systems using Manning's equation (Equation 3-8).

Equation 3-8

$$V = \frac{1.49(R)^{2/3}(S)^{1/2}}{n}$$

where:

- V = average velocity (ft/s)
- R = hydraulic radius (feet) and equals A/P_w
- A = cross sectional flow area (sq.ft.)
- P_w = wetted perimeter (feet)
- S = slope of energy grade line (may be estimated as channel slope, ft/ft), and
- n = Manning's roughness coefficient for open channel flow

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, where channels have been identified by TDEC or Knox County, where blue lines (indicating streams) appear on USGS quadrangle sheets, or where defined channels are identified by field inspection or topographic map indications. Equation 3-8 or water surface profile information can be used to estimate average flow velocity. Average flow velocity for travel time calculations is usually determined for bankfull elevation assuming low vegetation winter conditions.

Values of Manning's "n" for use in Equation 3-8 may be obtained from standard design textbooks such as Chow (1959) and Linsley et al. (1949). These values are also included as a part of the discussion of Manning's equation within Chapter 7 of this Manual, *Stormwater Drainage System Design*.

SCS Lag equation:
Additionally, the SCS lag equation is an acceptable method for calculating the time of concentration (Tc) based on watershed lag time (T_L). The SCS lag equation shall be used only for predeveloped rural areas with less than 10% imperviousness. It shall never be used in postdeveloped time of concentration calculations. T_L is defined as the time between the center of mass of excess rainfall to the time of peak runoff (similar to an average flow time for a small homogeneous area). The following equations can be used to determine Tc:

Equation 3-9

$$T_c = 1.67 T_L$$

where:

- T_C = time of concentration of overland flow portion of flow path (hours)
- T_L = NRCS lag time (hours)

Equation 3-10

$$T_L = \frac{L^{0.8}(S+1)^{0.7}}{1900 W_s^{0.5}}$$

where:

T_L = SCS lag time (hours)
L = flow length for overland flow over the surface (feet)
S = potential maximum soil retention (inches) = 1000/CN-10
W_s = average ground surface slope as a percentage (%)

Example 3-1. Calculation of Peak Discharge Using Rational Method

Estimates of the maximum rate of runoff are needed at the inlet to a proposed culvert for a 25-year return period.

Site Data

From an example topographic map and a field survey, the area of the drainage basin upstream from the point in question is found to be 23 acres. In addition the following data were measured:

- Average overland slope = 2.0% = 0.02 ft/ft
- Length of overland flow = 50 ft
- Length of main basin channel = 2,250 ft
- Hydraulic Radius R taken from channel dimensions = 1.62
- Slope of channel = 0.018 ft/ft = 1.8%
- Roughness coefficient (n) of channel was estimated to be 0.040
- Roughness coefficient (n) of overland flow area was estimated to be 0.090
- From existing land use maps, land use for the drainage basin was estimated to be:
  - Residential ($^1/_2$ acre) - 80%
  - Pasture - sandy soil, 3% slope - 20%
- From existing land use maps, the land use for the overland flow area at the head of the basin was estimated to be: lawn – silty clay soil, 2% slope
- 2-year, 24-hour rainfall = 3.3 inches

Step 1: The overland flow time can be calculated using Equation 3-4:

$$T_t = 0.007[(0.090)(50)]^{0.8}/(3.30)^{0.5}(0.02)^{0.4}$$
$$= 0.061 \text{ hrs} = 3.7 \text{ minutes}$$

Step 2: Calculate the channel flow time by first calculating the main channel velocity using Equation 3-10:

$$V = 1.49(1.62)^{2/3}(0.018)^{1/2}/(0.040)$$
$$= 6.9 \text{ ft/s}$$

The flow time is calculated using Equation 3-9:

$$T_t = 2250/[(6.9)(60)]$$
$$= 5.4 \text{ minutes}$$

Step 3: Calculate Tc.

$$Tc = 3.7 + 5.4 = 9.1 \text{ min (use 9 min)}$$

Step 4: From Table 3-4, use interpolation to calculate the intensity for a duration equal to 9 minutes,

$$I_{25} = 6.42 \text{ in/hr}$$

Step 5: A weighted runoff coefficient (C) for the total drainage area is determined below by utilizing the C values from Table 3-6. Assume the silty clay soil specified is classified in hydrologic soil group C. See table on next page.

Step 6: The Rational Method estimate of peak runoff for a 25-yr design storm for the given basin is:

$Q_{25} = C_f CIA = (1.10)(0.364)(6.42)(23)$
$= 59.1$ cfs

1	2	3	4
Land Use	**Percent of Total Land Area**	**Runoff Coefficient**	**Weighted Runoff Coefficient[1]**
Residential (1/2 acre)	80	0.35	0.280
Pasture	20	0.42	0.084
Total Weighted Runoff Coefficient = 0.364			

1 - Column 4 equals Column 2 multiplied by Column 3.

3.1.4 Regression Methods

3.1.4.1 USGS Regressions Equations

Two sets of USGS Regression Equations are presented in this section. Table 3-9 presents urban equations intended for use in the preliminary design of culverts across streams that are depicted as blue lines (i.e., waters of the state) on USGS quadrangle maps.

Table 3-9. USGS Urban Peak Flow Regression Equations
(Source: United States Geological Survey, 1984)

Frequency[1]	Equations[2, 3]
2-year	$Q_2 = 1.76A^{0.74} IA^{0.48} P^{3.01}$
5-year	$Q_5 = 5.55A^{0.75} IA^{0.44} P^{2.53}$
10-year	$Q_{10} = 11.8A^{0.75} IA^{0.43} P^{2.12}$
25-year	$Q_{25} = 21.9A^{0.75} IA^{0.39} P^{1.89}$
50-year	$Q_{50} = 44.9A^{0.75} IA^{0.40} P^{1.42}$
100-year	$Q_{100} = 77.0A^{0.75} IA^{0.40} P^{1.10}$

A = drainage area, mi^2
IA = total impervious area, % (e.g., 30% would be input as 30 not 0.30)
P = 2-year, 24-hour rainfall (inches) = 3.30 inches for Knox County
1 - Extrapolation is required to determine the 500-year peak flow.
2 - These equations are applicable for drainage areas between 0.21 mi^2 and 24.3 mi^2.
3 - These equations are applicable for impervious areas between 4.7% and 74%.

Table 3-10 presents USGS rural equations and USGS urban “three parameter” estimating equations (USGS, 1983). These equations were utilized by TVA to calculate peak discharges for the 2006 Flood Insurance Study of Knox County, Tennessee (FEMA, not yet dated). The equations presented in Table 3-10 must be used for preparation of new, and/or updating of existing, flood elevation studies in Knox County. *Note: the designer may be required to utilize an existing HEC-1 model as opposed to using the equations presented in Table 3-10 to prepare or modify a flood elevation study in Knox County. Consult Knox County Engineering prior to beginning a flood elevation study to determine the appropriate peak discharge calculation method.*

The three parameter equations require the determination of the basin development factor (BDF) and equivalent rural discharge (RQ_x) prior to use of the equations. These parameters are discussed in the following paragraphs.

Basin Development Factor (BDF): The BDF is a somewhat subjective parameter that is intended to account for the effects of urbanization in a watershed (USGS, 1984). The BDF index range from a minimum value of zero for a drainage area with very little development, to a maximum value of 12 for a drainage area with a high level of development. Four urbanization factors that are considered in the development of a BDF are channel improvements, channel linings, storm drains and curbed streets. For drainage areas that have BDF values of zero, the rural regression equations should be used to determine peak discharges for flood elevation studies. The urban three parameter equations should be used for drainage areas that have a BDF that is greater than zero.

Table 3-10. USGS Rural and Urban Three Parameter Equations
(Source: United States Geological Survey, 1983)

Frequency	Rural Equations[1]	Three Parameter Equations[1, 2, 3]
2-year	$RQ_2 = 118A^{0.753}$	$Q_2 = 13.2A^{.21}(13\text{-}BDF)^{-.43}RQ_2^{.73}$
5-year	$RQ_5 = 198A^{0.736}$	$Q_5 = 10.6A^{.17}(13\text{-}BDF)^{-.39}RQ_5^{.78}$
10-year	$RQ_{10} = 259A^{0.727}$	$Q_{10} = 9.51A^{.16}(13\text{-}BDF)^{-.36}RQ_{10}^{.79}$
25-year	$RQ_{25} = 344A^{0.717}$	$Q_{25} = 8.68A^{.15}(13\text{-}BDF)^{-.34}RQ_{25}^{.80}$
50-year	$RQ_{50} = 413A^{0.711}$	$Q_{50} = 8.04A^{.15}(13\text{-}BDF)^{-.32}RQ_{50}^{.81}$
100-year	$RQ_{100} = 493A^{0.703}$	$Q_{100} = 7.70A^{.15}(13\text{-}BDF)^{-.32}RQ_{100}^{.82}$
500-year	$RQ_{500} = 670A^{0.694}$	extrapolation required

1 - A = drainage area, mi^2
2 - BDF = basin development factor (see discussion below)
3 - RQ_x = equivalent rural discharge for an X-year event (cfs)

When using the USGS three parameter estimating equations to update an existing flood elevation study, the nature and size of the development will determine if the BDF that was determined for the existing flood elevation study should be increased to reflect the increased urbanization of the drainage areas to the stream. Knox County Engineering should be consulted prior to peak discharge calculation to determine if existing BDF's should be increased. Consult the USGS reference document (USGS, 1984) for more information on the determination of the BDF for any one basin.

Equivalent Rural Discharge (RQ_x): The RQ_x parameter is determined using the USGS rural regression equations presented in Table 3-10.

3.1.4.2 TVA Regression Equations

TVA developed a set of regression equations in the 1970s that can be used to calculate peak discharges in Knox County. These equations, shown in Table 3-11, can be used for the preliminary design of culverts across streams that are depicted as blue lines (waters of the state) on USGS quadrangle maps.

Table 3-11. TVA Regional Regressions Relationships for Natural Streams
(Source: City of Knoxville, 2003)

Frequency[1]	Equations[2, 3]
2-year	$Q_2 = 107\ A^{.804}\ I^{0.30}$
10-year	$Q_{10} = 217\ A^{.802}\ I^{0.26}$
50-year	$Q_{50} = 344\ A^{.796}\ I^{0.22}$
100-year	$Q_{100} = 402\ A^{.796}\ I^{0.20}$
500-year	$Q_{500} = 556\ A^{.795}\ I^{0.16}$

A = drainage area, mi^2
I = percent of contributing drainage area that is impervious, %

3.1.5 SCS Hydrologic Method

The SCS* hydrologic method requires basic data similar to the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. However, the SCS approach is more sophisticated in that it also considers the time distribution of the rainfall and the initial rainfall losses due to interception and depression storage. A typical application of the SCS method includes the following basic steps:

1. determination of curve numbers that represent different land uses within the drainage area;
2. calculation of time of concentration to the study point;
3. use of the SCS Type II rainfall distribution in this area; and
4. use of the unit hydrograph approach to develop the hydrograph of direct runoff from the drainage basin.

The SCS method can be used for both the estimation of stormwater runoff peak rates and the generation of hydrographs for the routing of stormwater flows. The SCS method can be used for most design applications, including storage facilities and outlet structures, storm drain systems, culverts, small drainage ditches and open channels, and energy dissipators.

3.1.5.1 Equations and Concepts

The hydrograph of outflow from a drainage basin is the sum of the hydrographs from all the sub-areas of the basin, modified by the effects of transit time through the basin and storage in the stream channels. Since the physical basin characteristics including shape, size and slope are constant, the unit hydrograph approach assumes that there is considerable similarity in the shape of hydrographs from storms of similar rainfall characteristics. Thus, the unit hydrograph is a typical hydrograph for the basin with a runoff volume under the hydrograph equal to one (1.0) inch from a rainfall excess of specified duration. For a rainfall excess of the same duration but with a different amount of runoff, the hydrograph of direct runoff can be expected to have the same time base as the unit hydrograph and ordinates of flow proportional to the unit hydrograph's runoff volume. Therefore, a storm that produces two inches of runoff would have a hydrograph with a flow equal to twice the flow of the unit hydrograph. With 0.5 inches of runoff, the total flow of the hydrograph would be one-half of the flow of the unit hydrograph.

The following discussion outlines the equations and basin concepts used in the SCS method.

<u>Drainage Area</u> - The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas it might be necessary to divide the area into sub-drainage areas to account for major land use changes, obtain analysis results at different points within the drainage area, combine hydrographs from different sub-basins as applicable, and/or route flows to points of interest.

<u>Rainfall</u> - The SCS method applicable to Knox County is based on a storm event that has a Type II time distribution. This distribution is used to distribute the 24-hour volume of rainfall for the different storm frequencies.

<u>Rainfall-Runoff Equation</u> - A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous soils and vegetative cover conditions. The SCS runoff equation (Equation 3-12) is used to estimate direct runoff from 24-hour or 1-day storm rainfall.

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* The Soil Conservation Service is now known as the Natural Resources Conversation Service (NRCS)

Equation 3-12

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S}$$

where:

Q = accumulated direct runoff (in)
P = accumulated rainfall or potential maximum runoff (in)
I_a = initial abstraction including surface storage, interception, evaporation, and infiltration prior to runoff (in)
S = potential maximum soil retention (in) = (1000/CN)-10

An empirical relationship used in the SCS method for estimating I_a is presented in Equation 3-13. This is an average value that could be adjusted for flatter areas with more depressions if there are calibration data to substantiate the adjustment.

Equation 3-13

$$I_a = 0.2S$$

Substituting 0.2S for I_a in Equation 3-12, the SCS rainfall-runoff equation becomes Equation 3-14.

Equation 3-14

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$

where:

S = (1000/CN) - 10
CN = SCS curve number

Figure 3-3 presents a graphical solution of this equation. For example, 4.1 inches of direct runoff would result if 5.8 inches of rainfall occurs on a watershed with a curve number of 85.

Equation 3-14 can be rearranged so that the curve number can be estimated if the rainfall and runoff volume are known, as shown in Equation 3-15 (Pitt, 1994).

Equation 3-15

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

where:

CN = SCS curve number
P = accumulated rainfall or potential maximum runoff (in)
Q = accumulated direct runoff (in). Can be Q_{wv}, Q_2, Q_{10}, etc…

3.1.5.2 Runoff Factor/Curve Numbers

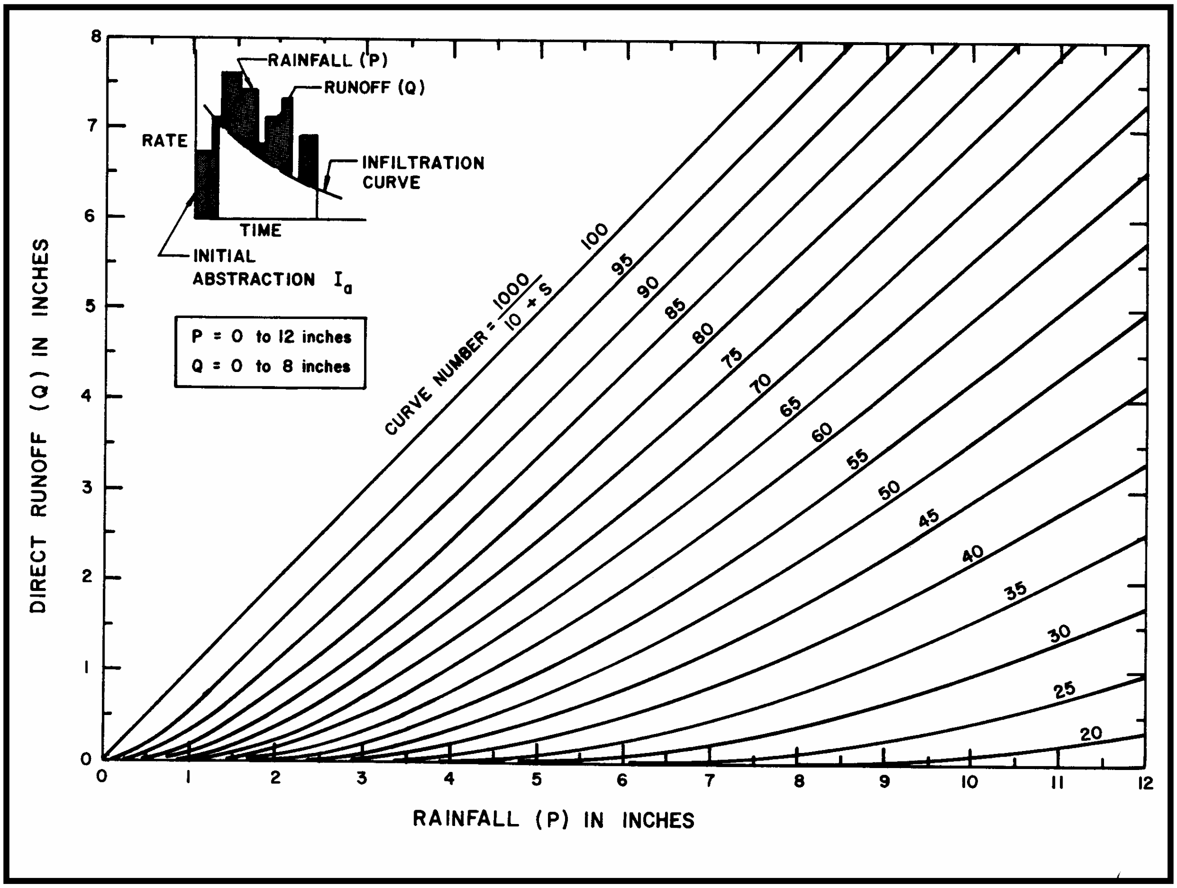
The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope. The SCS method uses a combination of soil conditions and land uses (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. The higher the CN, the higher the runoff potential. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Based on infiltration rates, the SCS has divided soils into four hydrologic soil groups (HSG).

Group A Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well-drained sands and gravels.

Group B Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Figure 3-3. SCS Solution of the Runoff Equation

(Source: Soil Conservation Service, 1986)



<u>Group C</u> Soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.

<u>Group D</u> Soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high water tables, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

A list of soils throughout Knox County and their hydrologic classification can be found in the reference SCS, 1986. Soil survey maps can be obtained from the local Natural Resources Conservation Service or the Knox County Soil Conservation office for use in estimating soil type.

Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. Also, runoff curve numbers vary with the antecedent soil moisture conditions. Antecedent runoff conditions (ARC II) are recommended for most hydrologic analyses, except in the design of developments in sinkhole drainage areas where ARC III may be allowed. Areas with high water table conditions may want to consider using ARC III antecedent soil moisture conditions. This should be considered a calibration parameter for modeling against real calibration data. Table 3-12 gives recommended curve number values for a range of different land uses assuming ARC II.

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Table 3-12. SCS Method Runoff Curve Numbers[1]

Cover Description	Cover Type and Hydrologic Condition	Average Percent Impervious Area[2]	Curve numbers for Hydrologic Soil Groups			
			A	B	C	D
Cultivated land:	without conservation treatment		72	81	88	91
	with conservation treatment		62	71	78	81
Pasture or range land:	poor condition		68	79	86	89
	good condition		39	61	74	80
Meadow	Generally mowed for hay		30	58	71	78
Wood or forest land:	thin stand, poor cover		45	66	77	83
	good cover		25	55	70	77
Open space (lawns, parks, golf course, cemeteries, etc.)[3]	poor condition (grass cover <50%)		68	79	86	89
	fair condition (grass cover 50% to 75%)		49	69	79	84
	good condition (grass cover > 75%)		39	61	74	80
Impervious areas:	paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:	paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
	paved; open ditches (including right-of-way)		83	89	92	93
	gravel (including right-of-way)		76	85	89	91
	dirt (including right-of-way)		72	82	87	89
Urban districts:	commercial and business	85%	89	92	94	95
	industrial	72%	81	88	91	93
Residential districts:	1/8 acre or less (town houses)	65%	77	85	90	92
	1/4 acre	38%	61	75	83	87
	1/3 acre	30%	57	72	81	86
	1/2 acre	25%	54	70	80	85
	1 acre	20%	51	68	79	84
	2 acres	12%	46	65	77	82
Developing urban areas and newly graded areas (pervious areas only, no vegetation)			77	86	91	94

1- Average runoff condition, and Ia = 0.2S

2- The average % impervious area shown was used to develop the composite CNs. Other assumptions are: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the SCS method has an adjustment to reduce the effect.

3- CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

Curve numbers are related to initial abstraction through equations 3-13, 3-14 and 3-15. This relationship is shown in Table 3-13.

Table 3-13. Initial Abstraction (I_a) for Runoff Curve Numbers

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

When a drainage area has more than one land use, a composite curve number can be calculated and used in the analysis. It should be noted that when composite curve numbers are used, the analysis does not take into account the location of the specific land uses, but sees the drainage area as a uniform land use represented by the composite curve number. Composite curve numbers for a drainage area can be calculated by using the weighted method as presented in Example 3-2.

Example 3-2. Composite Curve Number Calculation

Calculate the composite SCS curve number for a variable watershed. A watershed contains two primary land uses: 80% high density residential with HSG B soils and 20% meadow with HSG C soils. The watershed can be assumed to be divided into two sub-areas as shown in the table:

Step 1. Determine the curve number values for the given land uses and HSGs using Table 3-12.

Step 2. Calculate the weighted curve number for each sub-area of the watershed, and combine to obtain the composite curve number.

Land Use	% of Total Land Area	CN	Weighted CN (% area x CN)
Residential 1/8 acre Soil group B	0.8	85	68
Meadow Good condition Soil group C	0.2	71	14

The composite curve number =68 + 14 = 82.

Any number of land uses can be included. However, if the land use spatial distribution is important to the hydrologic analysis, then sub-basins should be developed and separate hydrographs developed and routed to the study point.

3.1.5.3 Urban Modifications of the SCS Method

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for developed areas. For example, consider whether the impervious areas connect directly to the drainage system or outlet onto lawns or other pervious areas where infiltration can occur. The curve number values given in Table 3-12 are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over pervious areas and then into a drainage system. It is possible to reduce curve number values from urban areas by not directly connecting impervious surfaces to the drainage system, but instead allowing runoff to flow as sheet flow over significant pervious areas. Chapter 5 (in Volume 2 of this manual) explains the benefits of using better site design techniques such as disconnected areas impervious area.

The following discussion will give some guidance for adjusting curve numbers for different percentages and types of impervious areas. The curve numbers provided in Table 3-12 for various land cover types were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that:

1. pervious urban areas are equivalent to pasture in good hydrologic condition, and
2. impervious areas have a CN of 98 and are directly connected to the drainage system.

If the typical values presented in Table 3-12 for impervious area percentages or the pervious land use assumptions are not applicable (e.g. the impervious areas are not directly connected), the following figures should be used to compute the composite CN value.

<u>For Site Impervious Area > 30%:</u>

- Figure 3-4 should be used to compute the composite CN value.

<u>For Site Impervious Area < 30%:</u>

- If all of the impervious area is directly connected to the drainage system: use Figure 3-4 to compute the composite CN value.
- If any of the impervious area is NOT directly connected (i.e., “disconnected”) to the drainage system: use Figure 3-5 to compute the composite CN value. (Enter the right half of Figure 3-5 with the percentage of total impervious area and the ratio of total disconnected impervious area to total impervious area.)

Figure 3-4. Composite CN for Total Impervious Area > 30% And Direct Connected Impervious Areas < 30%
(Source: Soil Conservation Service, 1986)

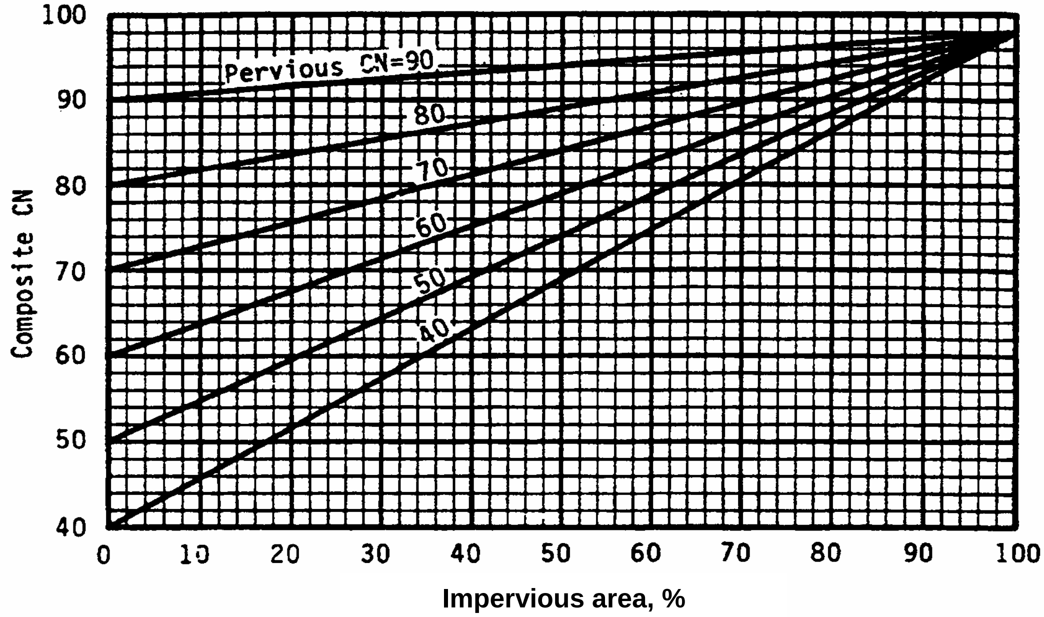
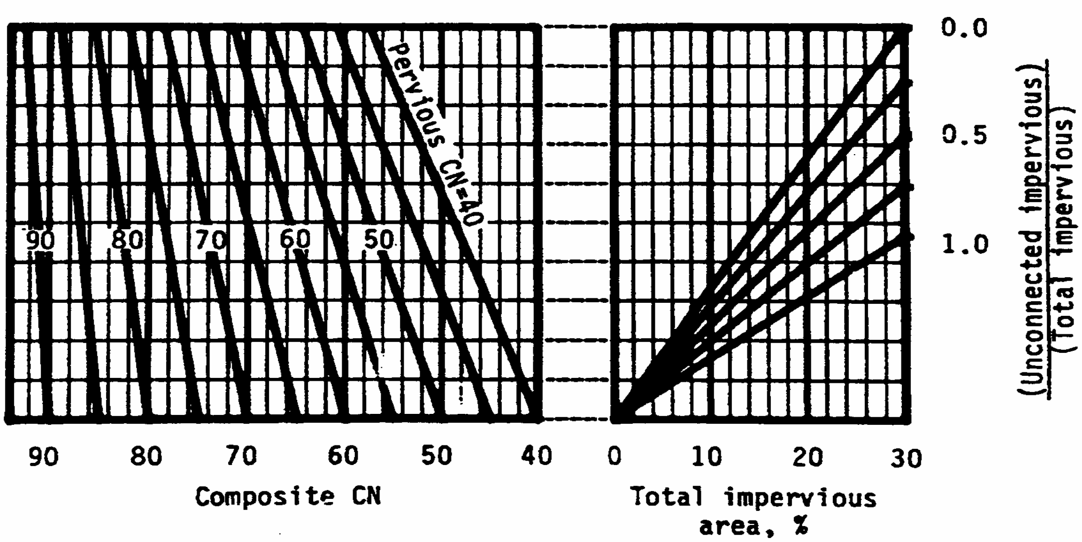


Figure 3-5. Composite CN with Disconnected Impervious Areas < 30%
(Source: Soil Conservation Service, 1986)



Examples 3-3 and 3-4 present the calculation of composite curve numbers for directly connected and disconnected impervious areas, respectively. (For additional information see TR-55, Chapter 2.)

Example 3-3. Curve Number Calculation for a Directly Connected Impervious Area Example

Assume a residential ½ acre lot with HSG B soils and an actual impervious area of 20%. Calculate the curve number for the directly connected area.

Step 1. Read the curve number of 70 for the given land use and HSG from Table 3-12. Note that this curve number is based on assumed impervious area of 25%.

Step 2. Adjust the curve number from the table to reflect less impervious area by using the connected impervious area of 20% and the pervious CN of 61 by using Figure 3-5. Enter Figure 3-5 along the bottom @ 20%, go vertically until the 61 CN line is met, then go to the left vertical axis to read the composite CN. The composite CN obtained from Figure 3-5 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

Example 3-4. Curve Number Calculation for an Unconnected Impervious Area Example

Assume a residential ½ acre lot with a pervious CN of 61 and 20% total impervious area (75% of which is unconnected). Calculate the composite curve number for the lot.

Step 1. Enter the right half of Figure 3-5 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. The ratio of unconnected impervious area to total impervious area is 0.75.

Step 2. Then, move left to the appropriate pervious CN and read down to find the composite CN. The composite CN is 66.

Step 3. If all of the impervious area is connected, the resulting CN (from Figure 3-4) would be 68.

3.1.5.4 Simplified SCS Peak Runoff Rate Calculation

The calculation presented in this section is applicable to drainage areas less than 2,000 acres that have homogeneous land uses that can be described by a single CN value (SCS, 1986). The SCS peak discharge equation is presented as Equation 3-16.

Equation 3-16

$$Q_p = q_u A Q F_p$$

where:

Q_p = peak discharge (cfs)
q_u = unit peak discharge (cfs/mi^2/in)
A = drainage area (mi^2)
Q = runoff (in)
F_p = pond and swamp adjustment factor

The computation sequence for the peak discharge method is presented in steps 1 through 6 below.

1. The 24-hour rainfall depth is determined from rainfall Table 3-5 for the selected location and return frequency.
2. The runoff curve number, CN, is estimated from Table 3-12 and direct runoff, Q, is calculated using Equation 3-14.

3. The CN value is used to determine the initial abstraction, I_a, from Table 3-13, and the ratio I_a/P is then computed (P = accumulated 24-hour rainfall).
4. The watershed time of concentration is computed using the procedures in Section 3.1.3.5 and is used with the ratio I_a/P to obtain the unit peak discharge, q_u, from Figure 3-6 for the Type II rainfall distribution. If the ratio I_a/P lies outside the range shown in the figure, either use the limiting values or use another peak discharge method. Note: Figure 3-6 is based on the Knox County standard peaking factor of 484. See Section 3.1.5.5 for additional information about peaking factor.
5. If pond and swamp areas are spread throughout the watershed and are not considered in the Tc computation, an adjustment is needed. The pond and swamp adjustment factor, F_p, is estimated from Table 3-14.

Table 3-14. Adjustment Factors for Ponds and Swamps

Pond and Swamp Areas (%[1])	F_p
0	1.00
0.2	0.97
1	0.87
3	0.75
5 or greater	0.72

[1] Percent of entire drainage basin

6. The peak runoff rate is computed using Equation 3-16.

Example 3-5. Calculate the 100-year peak discharge using the SCS Peak Discharge Equation.

Compute the 100-year peak discharge for a 50-acre wooded watershed located in Knox County, which will be developed as follows:

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

Other data include the following: Total impervious area = 13.2 acres, % of pond/swamp area = 0

Step 1. Calculate the rainfall excess:

The 100-year, 24-hour rainfall is 6.5 inches – From Table 3-5.
The calculation of the composite runoff coefficient for the watershed is shown in the following table.

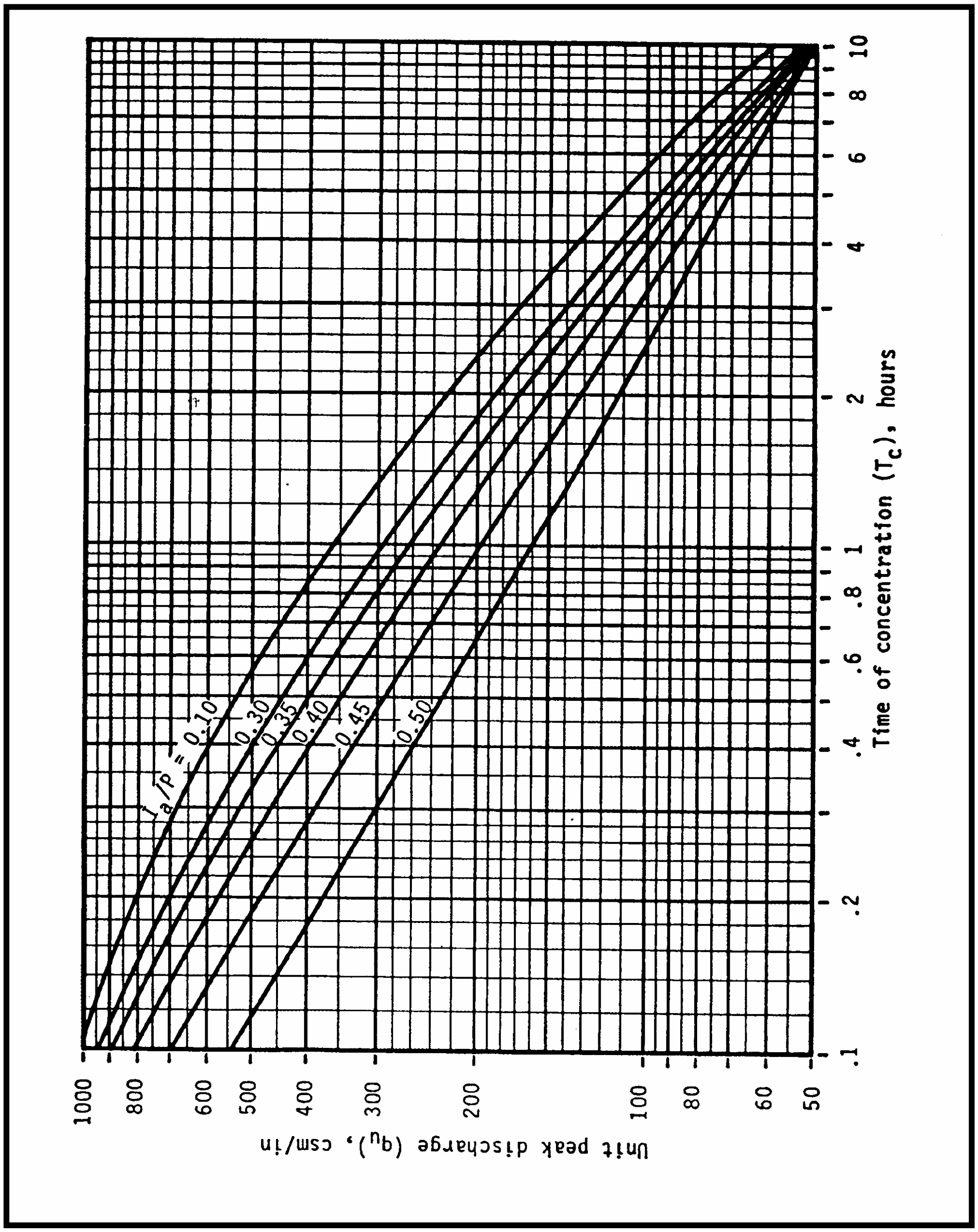
Dev. #	Area	% Total	CN[1]	Composite CN[2]
1	10 ac.	20	55	11.0
2	10 ac.	20	70	14.0
3	20 ac.	40	72	28.8
4	10 ac.	20	91	18.2
Total	50 ac.	100	-	72

[1] CN from Table 3-12
[2] Composite CN = % Total * CN

From Equation 3-14, Q (100-year) = 3.39 inches

Figure 3-6. SCS Type II Unit Peak Discharge Graph

(Source: Soil Conservation Service, 1986)



Step 2. Calculate time of concentration. The hydrologic flow path for this watershed = 1,890 ft

Segment	Type of Flow	Length (ft)	Slope (%)
1	Overland n = 0.24	40	2.0
2	Shallow channel	750	1.7
3	Main channel[1]	1100	0.5

[1] For the main channel, n = 0.06 (estimated), width = 10 feet, depth = 2 feet, rectangular channel

Segment 1 - Travel time from Equation 3-4 with P_2 = 3.36 inches (0.14 x 24 – Table 3-4)

$T_t = 0.007[(0.24)(40)]^{0.8}/(3.36)^{0.5}(0.02)^{0.4}$
$= 0.112$ hrs = 6.68 minutes

Segment 2 - Travel time from Figure 3-2 or Equation 3-7

V = 2.1 ft/s (from Equation 3-7)

$T_t = 750/[(60)(2.1)] = 5.95$ min

Segment 3 - Using Equation 3-10

$V = [(1.49)(1.43)^{0.67}(0.005)^{0.5}]/0.06$
$= 2.23$ ft/s

$T_t = 1100/60(2.23) = 8.22$ min

Therefore, using Equation 3-3

Tc = 6.68 + 5.95 + 8.22 = 20.85 min = 0.35 hrs

Step 3. Calculate I_a/P (for CN = 72), $I_a = 0.778$ (Table 3-13)

$I_a/P = (0.778/6.48) = 0.12$ (Note: Use $I_a/P = 0.10$ to facilitate use of Figure 3-6. Straight line interpolation could also be used.)

Step 4. Unit discharge q_u (100-year) from Figure 3-6= 650 csm/in,

Step 5. Calculate peak discharge with $F_p = 1$ using Equation 3-16

$Q_{100} = 650(50/640)(3.39)(1) = 172$ cfs

3.1.5.5 Hydrograph Generation

In addition to estimating the peak discharge, the SCS method can be used to estimate the entire hydrograph from a drainage area. The SCS has developed a Tabular Hydrograph procedure that can be used to generate the hydrograph for drainage areas less than 2,000 acres. The Tabular Hydrograph procedure uses unit discharge hydrographs that have been generated for a series of time of concentrations. In addition, SCS has developed hydrograph procedures to be used to generate composite flood hydrographs (SCS, 1986).

The unit hydrograph equations used in the SCS method for generating hydrographs include a constant to account for the general land slope in the drainage area. This constant, called a peaking factor, can be adjusted when using the method. A default value of 484 for the peaking factor represents rolling hills – a medium level of relief. SCS indicates that for mountainous terrain the peaking factor can go as high as 600, and as low as 300 for flat (coastal) areas. In Knox County, the default value of 484 must be used for the peaking factor.

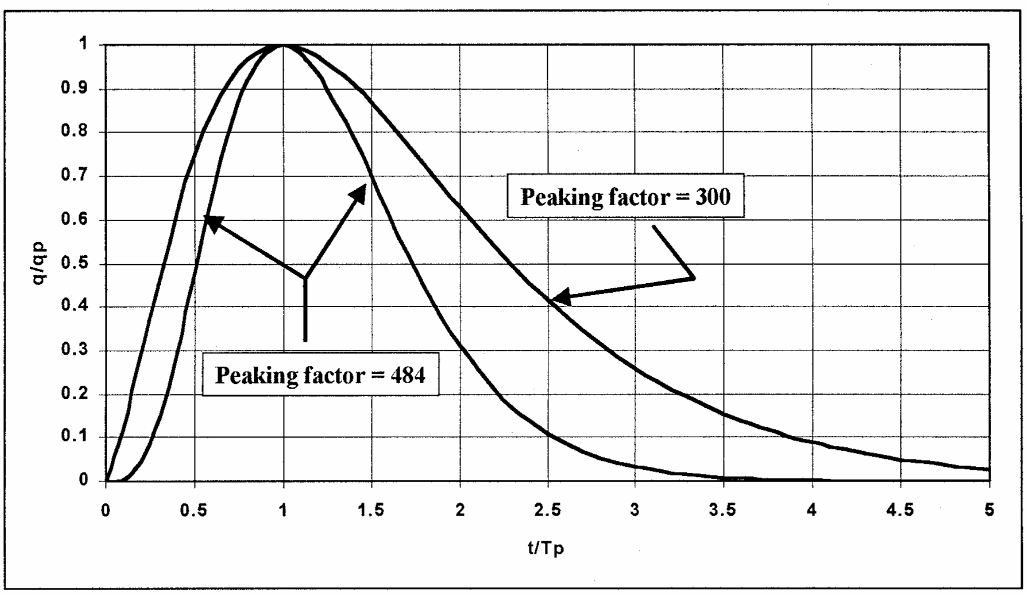
The development of a runoff hydrograph from a watershed is a laborious process not normally done by hand. For that reason this discussion is limited to an overview of the process and is given here to assist the designer in reviewing and understanding the input and output from a typical computer program. There are choices of computational interval, storm length (if the 24-hour storm is not going to be used) and other “administrative” parameters that are specific to each computer program.

The development of a runoff hydrograph for a watershed or one of many sub-basins within a more complex model involves the following steps:

1. Development or selection of a design storm hyetograph (a graph of the time distribution of rainfall over a watershed). Often, the SCS 24-hour storm described in Section 3.1.5.3 is used.
2. Development of curve numbers and lag times for the watershed using the methods described in Sections 3.1.5.4, 3.1.5.5, and 3.1.5.6.
3. Development of a unit hydrograph from the standard (peaking factor of 484) dimensionless unit hydrographs. See discussion below.
4. Step-wise computation of the initial and infiltration rainfall losses and, thus, the excess rainfall hyetograph using a derivative form of the SCS rainfall-runoff equation (Equation 3-12).
5. Application of each increment of excess rainfall to the unit hydrograph to develop a series of runoff hydrographs, one for each increment of excess rainfall (this is called "convolution").
6. Summation of the flows from each of the small incremental hydrographs (keeping proper track of time steps) to form a runoff hydrograph for that watershed or sub-basin.

Figure 3-7 and Table 3-16 can be used along with Equations 3-17 and 3-18 to assist the designer in using the SCS unit hydrograph in Knox County. The unit hydrograph with a peaking factor of 300 is shown in the figure for comparison purposes, but should not be used for areas in Knox County.

Figure 3-7. Dimensionless Unit Hydrographs for Peaking Factors of 484 and 300



Equation 3-17 is used to find the peak discharge, given the peaking factor, area, and time to peak. The peak discharge is then multiplied by the q/q_u values in Table 3-15 for each time step to give the flow at that time.

Equation 3-17

$$q_u = \frac{(PF)A}{T_p}$$

where:

q_u = unit hydrograph peak rate of discharge (cfs)
PF = peaking factor
A = area (mi^2)
T_p = time to peak = d/2 + 0.6 Tc (hours)
d = rainfall time increment (hours)

Table 3-15. Dimensionless Unit Hydrograph 484

t/T_p	484	
	q/q_u	Q/Qp
0.0	0.000	0.000
0.1	0.005	0.000
0.2	0.046	0.004
0.3	0.148	0.015
0.4	0.301	0.038
0.5	0.481	0.075
0.6	0.657	0.125
0.7	0.807	0.186
0.8	0.916	0.255
0.9	0.980	0.330
1.0	1.000	0.406
1.1	0.982	0.481
1.2	0.935	0.552
1.3	0.867	0.618
1.4	0.786	0.677
1.5	0.699	0.730
1.6	0.611	0.777
1.7	0.526	0.817
1.8	0.447	0.851
1.9	0.376	0.879
2.0	0.312	0.903
2.1	0.257	0.923
2.2	0.210	0.939
2.3	0.170	0.951
2.4	0.137	0.962
2.5	0.109	0.970
2.6	0.087	0.977
2.7	0.069	0.982
2.8	0.054	0.986
2.9	0.042	0.989
3.0	0.033	0.992
3.1	0.025	0.994
3.2	0.020	0.995
3.3	0.015	0.996
3.4	0.012	0.997
3.5	0.009	0.998
3.6	0.007	0.998
3.7	0.005	0.999
3.8	0.004	0.999
3.9	0.003	0.999
4.0	0.002	1.000

For ease of spreadsheet calculations, the dimensionless unit hydrograph using a peaking factor of 484 can be approximated using Equation 3-18.

Equation 3-18

$$\frac{q}{q_u} = \left[\frac{t}{T_p} e^{\left(1 - \frac{t}{T_p}\right)} \right]^X$$

where:

X = 3.79 for the PF = 484 unit hydrograph.

Example 3-6. Calculation of Unit Hydrograph

Compute the unit hydrograph for the 50-acre wooded watershed in Example 3-5.

Computations

Step 1. Calculate T_p (time to peak) and time increment

The time of concentration (Tc) is calculated to be 21 minutes for this watershed. If we assume a computer calculation time increment (d), duration of excess rainfall, of 3 minutes then:

T_p = d/2 + 0.6 Tc = 3/2 + 0.6(21) = 14.1 min = 0.235 hours

Step 2. Calculate q_u

q_u = (484)(50/640)/(0.235) = 161 cfs

Step 3. Calculate the hydrograph using unit hydrograph 484. The table below was derived based on spreadsheet calculations using Equations 3-17 and 3-18.

Time		484	
t/Tp	time (min)	q/q_u	Q
0.00	0	0.00	0.00
0.21	3	0.06	9.15
0.43	6	0.35	56.32
0.64	9	0.72	116.58
0.85	12	0.96	154.41
1.00	14	1.00	160.90
1.07	15	0.99	160.14
1.28	18	0.88	142.28
1.49	21	0.70	113.61
1.71	24	0.52	83.90
1.92	27	0.36	58.37
2.14	30	0.24	38.74
2.35	33	0.15	24.75
2.56	36	0.09	15.32
2.78	39	0.06	9.24
2.99	42	0.03	5.45
3.20	45	0.02	3.15
3.42	48	0.01	1.79
3.63	51	0.01	1.00
3.84	54	0.00	0.55

Time		484	
t/Tp	time (min)	q/q_u	Q
4.06	57	0.00	0.30
4.27	60	0.00	0.16
4.48	63	0.00	0.09
4.70	66	0.00	0.05
4.91	69	0.00	0.02

3.1.6 Clark Unit Hydrograph

In Knox County, use of the Clark Unit Hydrograph method is acceptable only for hydrologic calculations that are prepared for flood studies and flood elevation calculations. See Volume 2, Chapter 8 for more information on flood study preparation.

The Clark method defines a unit hydrograph for a given basin using the concept of the instantaneous unit hydrograph (IUH). An IUH is a theoretical hydrograph that would result when a single unit of rainfall excess was spread out evenly over an entire basin and allowed to run off. The IUH can be converted to a unit hydrograph of a desired duration by conventional techniques for developing unit hydrographs (Hoggan, 1997).

The Clark method is based on the effects of translation and attenuation as the primary forces involved in the flow of water through a watershed. Translation is defined as the 'downhill' flow of water as a result of the force of gravity. Attenuation is defined as the resistance of flow that is caused by either friction in the channel or water storage. According to Clark, translation in a watershed can be described with a time-area curve. This curve displays the portion of watershed area that is contributing runoff as a function of time. The curve should start at the point in which effective precipitation begins. Effective precipitation is any precipitation that does not infiltrate into the soil or is retained in a ponding area. Equation 3-19 presents these concepts.

Equation 3-19

$$S = RO$$

where:

S = Storage
R = Attenuation (Watershed Storage) Constant
O = Outflow

A synthetic hydrograph could be produced by proportionally routing an inch of direct runoff to the channel in accordance with the time-area curve. The runoff entering the channel would then be routed through a linear reservoir. More recent studies have indicated that it is not necessary to produce detailed time-area curves in order to produce accurate synthetic hydrographs. The dimensionless time-area curve included in HEC-1 and HEC-HMS hydrologic models (developed by the United States Army Corps of Engineers) have produced accurate synthetic hydrographs. In order to apply the Clark method in a HEC-1 or HEC-HMS model, the time of concentration (Tc) and a watershed storage constant (R) are required as inputs. In stormwater master plans prepared for Knox County in the late 1990's and early 2000's, research indicated that Equation 3-20, which equates to setting R = Tc, produced accurate estimates of peak discharges for small drainage areas. However, the engineer performing the flood study should determine the most appropriate equation to determine the value of R.

Equation 3-20

$$\frac{R}{T_c + R} = 0.5$$

where:

R = Attenuation (Watershed Storage) Constant
Tc = Time of concentration

3.1.7 Water Quality Calculations

3.1.7.1 Water Quality Volume Calculation

In Knox County, the Water Quality Volume (WQv) is the treatment volume required to remove 80% of the average annual, post-development total suspended solids (TSS) load. This is achieved by intercepting and treating a portion of the runoff from all storms and all the runoff from 85% of the storms that occur on average during the course of a year. The water quality treatment volume is calculated using Equation 3-21.

Equation 3-21

$$WQv = \frac{1.1 R_v A}{12}$$

where:

WQv = water quality volume (acre-feet)
1.1 = the 85th percentile rainfall depth in Knox County (inches)
Rv = volumetric runoff coefficient (see Equation 3-22)
A = total drainage area (acres)

The volumetric runoff coefficient (Rv) is directly proportional to the percent impervious cover of the development or drainage area. Rv is calculated using Equation 3-22.

Equation 3-22

$$Rv = 0.015 + 0.0092(I)$$

where:

I = percent of impervious cover (%)

3.1.7.2 Water Quality Peak Discharge Calculation

The peak rate of discharge for the water quality design storm (Q_{wq}, also called the water quality peak discharge) is needed to size off-line diversion structures, such as for sand filters and infiltration trenches. However, traditional peak discharge calculation methods are not appropriate for this application. For example, the use of the Rational Method for sizing water quality controls would require the choosing of an arbitrary storm event. Further, conventional SCS methods have been found to underestimate the volume and rate of runoff for rainfall events of less than two inches. This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff bypasses the structural control due to an inadequately sized diversion structure and leads to the design of undersized bypass channels.

The method employed to size water quality diversions uses the runoff coefficient to find the depth of runoff for the water quality storm of 1.1". The SCS method is then used to find a unit peak discharge that is combined with the runoff depth to find a peak runoff rate.

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

1. The runoff coefficient (Rv) is determined using Equation 3-22.
2. The depth of runoff that must be treated for the water quality storm is determined.

$$D_{wq} = 1.1Rv$$

where:

D_{wq} = water quality runoff depth (inches)
Rv = runoff coefficient

3. A CN reflecting the runoff characteristics during the water quality design storm is calculated utilizing a form of Equation 3-15:

$$CN = 1000/[10 + 5P + 10D_{wq} - 10(D_{wq}^{2} + 1.25\ D_{wq}P)^{0.5]}$$

where:

P = rainfall depth for water quality design storm = (1.1 inches)

4. Initial abstraction (Ia) is found from CN using Table 3-13. Ia/P is then calculated.
5. Concentration time (Tc) is computed per section 3.1.3.5.
6. Unit peak discharge (q_u) is determined from Tc and Ia using Figure 3-6.
7. The water quality peak discharge (Q_{wq}) is computed using drainage area (A), unit peak discharge (q_u) and water quality runoff depth (D_{wq}) using Equation 3-16.

Example 3-7. Calculation of Water Quality Peak Flow

Using the data and information from Example 3-5, calculate the WQv and the Q_{wv}.

Step 1: Compute volumetric runoff coefficient, Rv using Equation 3-22:

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

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

D_{wq} = 1.1Rv = 1.1(0.35) = 0.39 inches

Step 3: Compute the synthetic curve number (CN) using Equation 3-15:

CN = $1000/[10 + 5(1.1) + 10(0.39) – 10[(0.39)^2 +1.25(0.39)(1.1)]^{0.5}= 90$

Step 4: Find Ia from CN with Table 3-13:

Ia = 0.22 inches
Ia/P = 0.22/1.1 = 0.20

Step 5: Compute time of concentration, Tc:

Tc computed as 0.35 hours in Example 3-5.

Step 6: Find q_u, using Tc = 0.35 and I_a/P = 0.20 using Figure 3-6:

q_u = 580 cfs/mi²/in

Step 7: Compute water quality peak flow rate using Equation 3-16.

Q_{wq} = 580(50/640)(0.39)(1) = 17.67 cfs

3.1.8 Water Balance Calculations

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

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

3.1.8.1 Basic Equations

Water balance is defined as the change in volume of the permanent pool resulting from the total inflow minus the total outflow (actual or potential). Equation 3-23 presents this calculation.

Equation 3-23

$$\Delta V = \sum I - \sum O$$

where:

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

The inflows consist of rainfall, runoff and baseflow into the pond. The outflows consist of infiltration, evaporation, evapotranspiration, and surface overflow out of the pond or wetland. Equation 3-23 can be expanded to reflect these factors, as shown in Equation 3-24. Key variables in Equation 3-24 are discussed in detail below the equation.

Equation 3-24

$$\Delta V = PA + R_o + Bf - IA - EA - EtA - Of$$

where:

- P = precipitation (ft)
- A = area of pond (ac)
- R_o = runoff (ac-ft)
- B_f = baseflow (ac-ft)
- I = infiltration (ft)
- E = evaporation (ft)
- Et = evapotranspiration (ft)
- Of = overflow (ac-ft)

Rainfall (P) – Monthly rainfall values can be obtained from the National Weather Service climatology at *http://www.srh.noaa.gov/mrx/climat.htm*. Monthly values are commonly used for calculations of values over a season. Rainfall is the direct amount that falls on the pond surface for the period in question. When multiplied by the pond surface area (in acres) it becomes acre-feet of volume. Table 3-16 presents average monthly rainfall values for Knoxville based on a 30-year period of record.

Table 3-16. Average Rainfall Values in Inches for Knoxville, Tennessee

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
P (inches)	4.57	4.01	5.17	3.99	4.68	4.04	4.71	2.89	3.04	2.65	3.98	4.49
Annual Precipitation 48.2 inches												

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

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

Equation 3-21 gives a ratio of runoff to rainfall volume for a particular storm. If it can be assumed that the average storm that produces runoff has a similar ratio, then the Rv value can serve as the ratio of rainfall to runoff. Not all storms produce runoff in an urban setting. Typical initial losses (often called "initial abstractions") are normally taken between 0.1 and 0.2 inches. When compared to the rainfall records in Knox County, this is equivalent to about a 10% runoff volume loss. Thus, in a water balance calculation, a factor of 0.9 should be applied to the calculated Rv value to account for storms that produce no runoff. Equation 3-25 reflects this approach. Total runoff volume is then simply the product of runoff depth (Q) times the drainage area to the pond.

Equation 3-25 $$Q = 0.9PRv$$

where:

Q = runoff volume (in)
P = precipitation (in)
Rv = volumetric runoff coefficient [Equation 3-22]

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

Infiltration (I) – Infiltration is a very complex subject and cannot be covered in detail here. The amount of infiltration depends on soils, water table depth, rock layers, surface disturbance, the presence or absence of a liner in the pond, and other factors. The infiltration rate is governed by the Darcy equation, shown in Equation 3-26.

Equation 3-26 $$I = Ak_hG_h$$

where:

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

G_h can be set equal to 1.0 for pond bottoms and 0.5 for pond sides steeper than about 4:1. Infiltration rate can be established through testing, though not always accurately. Table 3-17 can be used for initial estimation of the saturated hydraulic conductivity.

Table 3-17. Saturated Hydraulic Conductivity
(Source: Ferguson and Debo, 1990)

Material	Hydraulic Conductivity	
	in/hr	ft/day
ASTM Crushed Stone No. 3	50,000	100,000
ASTM Crushed Stone No. 4	40,000	80,000
ASTM Crushed Stone No. 5	25,000	50,000
ASTM Crushed Stone No. 6	15,000	30,000
Sand	8.27	16.54
Loamy sand	2.41	4.82
Sandy loam	1.02	2.04
Loam	0.52	1.04

Material	Hydraulic Conductivity	
	in/hr	ft/day
Silt loam	0.27	0.54
Sandy clay loam	0.17	0.34
Clay loam	0.09	0.18
Silty clay loam	0.06	0.12
Sandy clay	0.05	0.10
Silty clay	0.04	0.08
Clay	0.02	0.04

Evaporation (E) – Evaporation is from an open lake water surface. Evaporation rates are dependent on differences in vapor pressure, which, in turn, depend on temperature, wind, atmospheric pressure, water purity, and shape and depth of the pond. It is estimated or measured in a number of ways, which can be found in most hydrology textbooks. Pan evaporation methods are also used, though there are no longer pan evaporation sites active in Knox County. Formerly pan evaporation methods were utilized at the Knoxville Experiment Station.

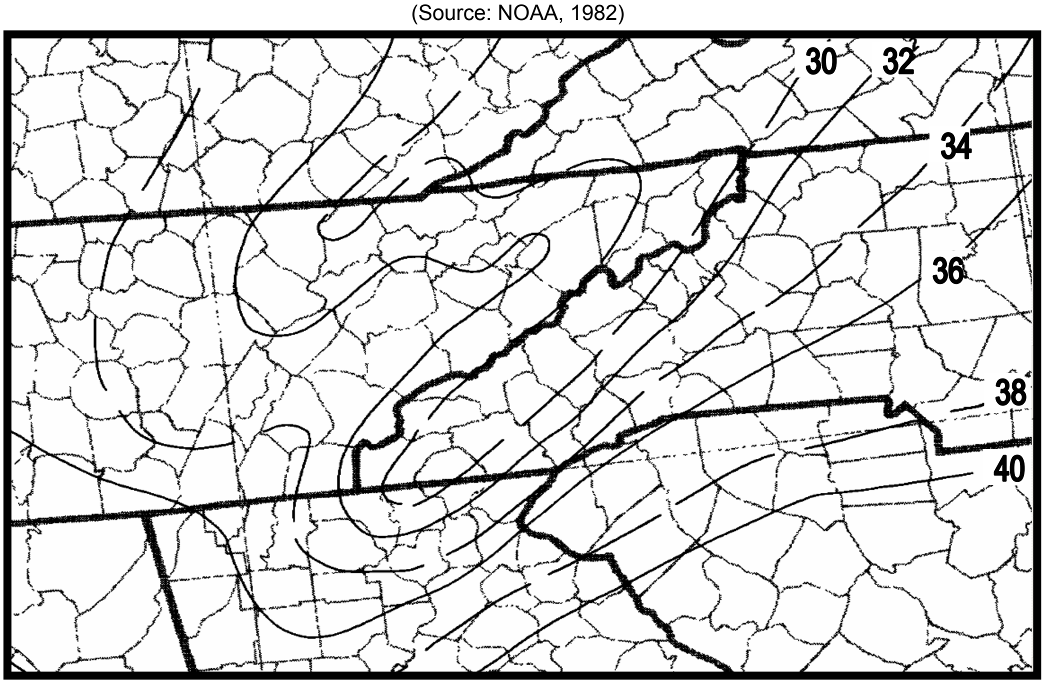
Table 3-18 presents pan evaporation rate distributions for a typical 12-month period based on pan evaporation information from one station in Knox County. Figure 3-8 depicts a map of annual free water surface (FWS) evaporation averages for Tennessee based on a National Oceanic and Atmospheric Administration (NOAA) assessment done in 1982. FWS evaporation differs from lake evaporation for larger and deeper lakes, but can be used as an estimate of it for the type of structural stormwater ponds and wetlands being designed in Knox County. Total annual values can be estimated from this map and distributed in accordance with the percentages presented in Table 3-18.

Table 3-18. Pan Evaporation Rates - Monthly Distribution

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2.9%	3.8%	7.2%	10.6%	13.1%	13.1%	13.2%	12.4%	9.8%	6.7%	4.1%	3.1%

Figure 3-8. Average Annual Free Water Surface Evaporation (in inches)

(Source: NOAA, 1982)



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

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

Example 3-8. Water Balance Calculation for Pond

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

Step 1: From Equation 3-22, Rv = 0.015 + 0.0092(75) = 0.71. With the correction factor of 0.9 the watershed efficiency is 0.64.
The annual lake evaporation from Figure 3-8 is about 32.5 inches.
For a sandy clay loam the infiltration rate is I = 0.34 ft/day (Table 3-17).
From a grading plan, it is known that 10% of the total pond area is sloped greater than 4:1.
Monthly rainfall for Knox County was found from the Web site provided above.

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

	Value	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	Days per Month	31	28	31	30	31	30	31	31	30	31	30	31
2	Precip. (in)	4.57	4.01	5.17	3.99	4.68	4.04	4.71	2.89	3.04	2.65	3.98	4.49
3	Evap. Dist. (%)	2.9	3.8	7.2	10.6	13.1	13.1	13.2	12.4	9.8	6.7	4.1	3.1
4	R_0 (ac-ft)	6.22	5.45	7.03	5.43	6.36	5.49	6.41	3.93	4.13	3.60	5.41	6.11
5	P (ac-ft)	0.19	0.17	0.22	0.17	0.20	0.17	0.20	0.12	0.13	0.11	0.17	0.19
6	E (ac-ft)	0.04	0.05	0.10	0.14	0.18	0.18	0.18	0.17	0.13	0.09	0.06	0.04
7	I (ac-ft)	5.01	4.52	5.01	4.85	5.01	4.85	5.01	5.01	4.85	5.01	4.85	5.01
8	Bal. (ac-ft)	1.36	1.05	2.14	0.60	1.38	0.64	1.42	-1.12	-0.72	-1.38	0.68	1.25
9	Run. Bal. (ac-ft)	1.36	2.00	2.00	2.00	2.00	2.00	2.00	0.88	0.16	0.00	0.68	1.92

Explanation of Table:

1. Days per month
2. Monthly precipitation from web site is shown in Table 3-16.
3. Distribution of evaporation by month from Table 3-18.
4. Ro = Watershed efficiency (0.64) * Rainfall (line 2) * Site Area w/o Pond (26-0.5) / 12 (conversion to acre-feet). From Equation 3-25.
5. Precipitation volume directly into pond = Rainfall (line 2) * Pond Area (0.5) / 12 (conversion to acre-feet).
6. Evaporation = Monthly % (line 3) / 100 (convert to decimal) * Annual Evaporation (32.5) * Pond Area (0.5) / 12 (conversion to acre-feet).
7. Infiltration = [infiltration rate (0.34) * 90% of Pond Area (0.90 * 0.5) + infiltration rate (0.34) * 10% of Pond Area (0.1 * 0.5) * 0.5 (Reduction factor for banks greater than 4:1)] * # of Days in month (line 1) (conversion from acre-feet/day to acre-feet).
8. Balance is lines (4 + 5) minus lines (6 + 7).
9. Running Balance is accumulated total from line 9 keeping in mind that all volume above 2 acre-feet overflows and is lost in the trial design. (Any value > 2, set = 2. Any value < 0, set = 0)

It can be seen that for this example the pond has potential to go dry in the fall. This can be remedied in a number of ways including compacting the pond bottom, placing a liner of clay of geosynthetics, and changing the pond geometry to decrease the surface area.

3.1.9 Calculating Downstream Impacts (the Ten Percent Rule)

In the Knox County Stormwater Management Manual, the “ten-percent” rule has been adopted as the approach for ensuring that stormwater quantity detention ponds maintain pre-development peak flows through the downstream conveyance system.

The ten-percent rule recognizes the fact that a structural control providing detention has a “zone of influence” downstream where its effectiveness can be observed. Beyond this zone of influence the structural control becomes relatively small and insignificant compared to the runoff from the total drainage area at that point. Based on studies and master planning results for a large number of sites, that zone of influence is considered to be the point where the drainage area controlled by the detention or storage facility comprises 10% of the total drainage area. For example, if the structural control drains 10 acres, the zone of influence ends at the point where the total drainage area is 100 acres or greater.

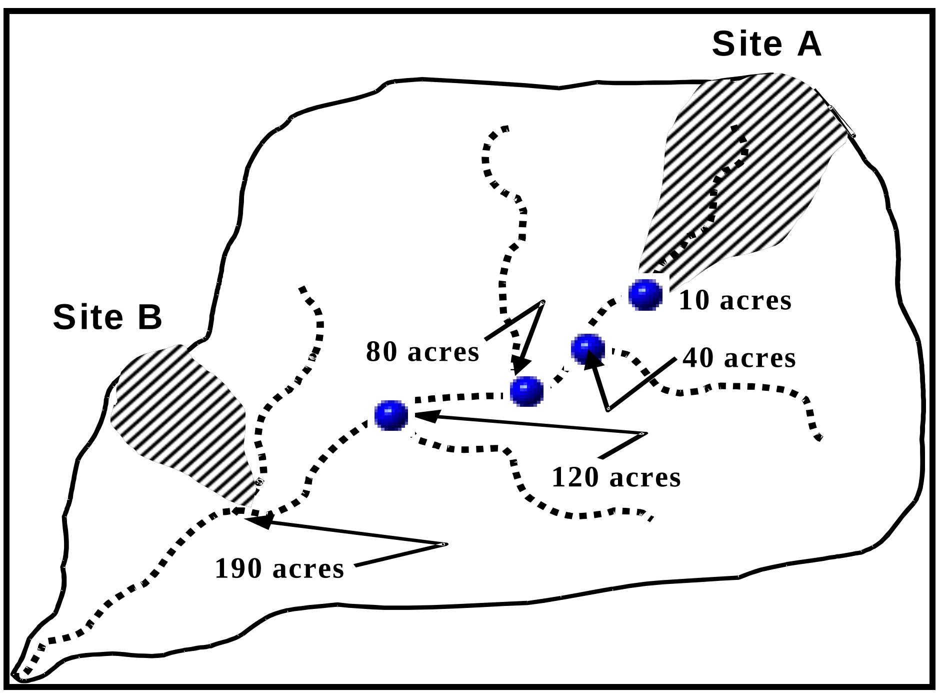
Typical steps in the application of the ten-percent rule are:

1. Using a topographic map determine the lower limit of the “zone of influence” (i.e., the 10% point), and determine all 10% rule comparison points (at the outlet of the site and at all downstream tributary junctions or other points of interest).
2. Using a hydrologic model determine the pre-development peak discharges (pre-Qp_2, pre-Qp_{10}, pre-Qp_{25}, and pre-Qp_{100}) and timing of those peaks at each tributary junction beginning at the pond outlet and ending at the next tributary junction beyond the 10% point.
3. Change the site land use to post-development conditions and determine the post-development peak discharges (post-Qp_2, post-Qp_{10}, post-Qp_{25}, and post-Qp_{100}). Design the structural control facility such that the post-development peak discharges from the site <u>for all storm events</u> do not increase the pre-development peak discharges at the outlet of the site and at each downstream tributary junction and each public or major private downstream stormwater conveyance structure located within the zone of influence.
4. If post-development conditions do increase the peak flow within the zone of influence, the structural control facility must be redesigned or one of the following options must be chosen:

- Control of the Qp_2, Qp_{10}, Qp_{25}, and/or Qp_{100} may be waived by the Director of Engineering and Public Works (the Director) if adequate overbank flood protection and/or extreme flood protection is suitably provided by a downstream or shared off-site stormwater facility, or if engineering studies determine that installing the required stormwater facilities would not be in the best interest of Knox County. However, a waiver of such controls does not eliminate the requirement to comply with the water quality and channel protection standards defined in the Ordinance and in this Stormwater Management Manual.
- The developer can coordinate with Knox County Engineering (and other state/federal agencies as appropriate) to determine other acceptable approaches to reduce the peak discharges (and, therefore the flow elevation) through the channel (e.g., conveyance improvements) for all design storm events.
- The developer can obtain a flow easement from downstream property owners through the zone of influence where the post-development peak discharges are higher than pre-development peak discharges.

Example 3-9. Ten Percent Rule Example

The figure below illustrates the concept of the ten-percent rule for two sites in a watershed.



Site A is a development of 10 acres, all draining to a wet ED stormwater pond. The overbank flooding and extreme flood portions of the design are going to incorporate the ten-percent rule. Looking downstream at each tributary in turn, it is determined that the analysis should end at the tributary marked "120 acres." The 100-acre (10%) point is in between the 80-acre and 120-acre tributary junction points.

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

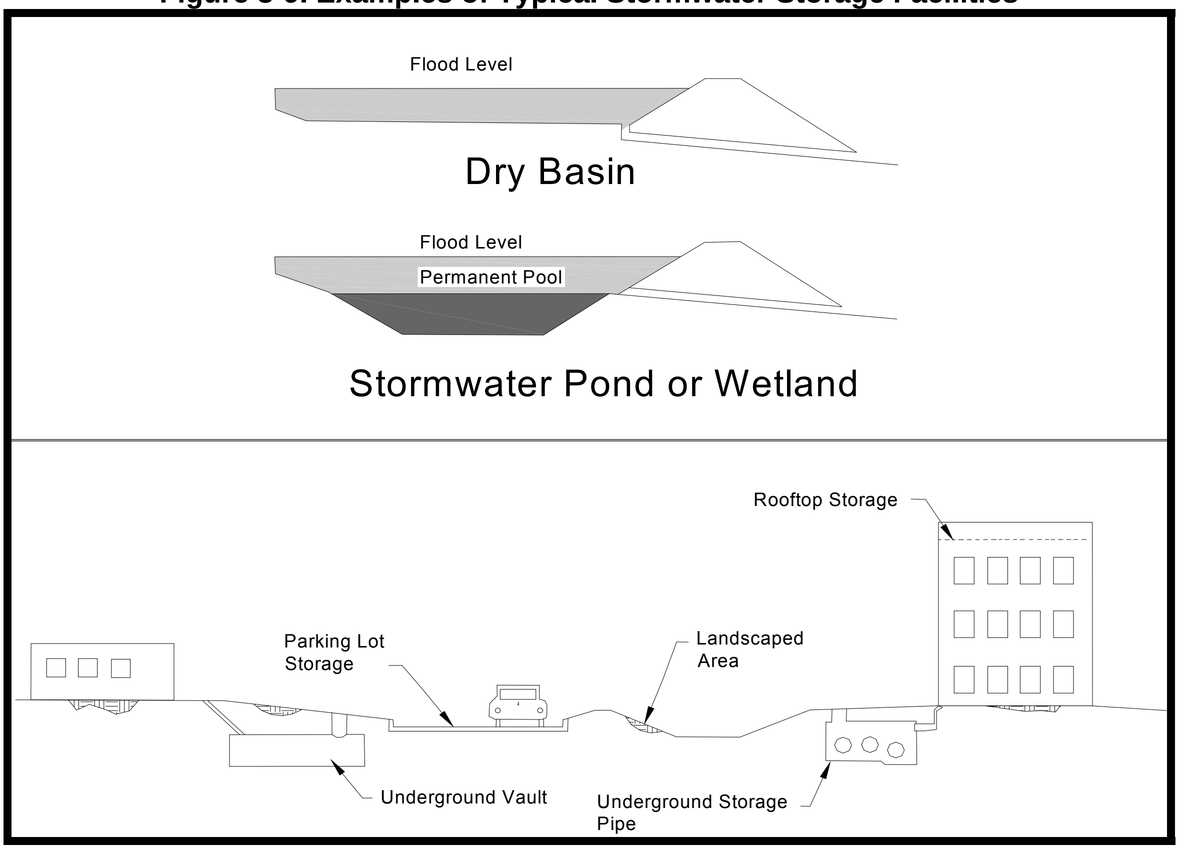
Site B is located downstream at the point where the total drainage area is 190 acres. The site itself is only 6 acres. The first tributary junction downstream from the 10% point is the junction of the site outlet with the stream. The total 190 acres is modeled as one basin with care taken to estimate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

3.2 Storage Design

3.2.1 General Storage Concepts

This section provides general guidance on stormwater runoff storage for meeting control of the WQv, CPv, Qp_2, Qp_{10}, Qp_{25} and the Qp_{100}. Storage of stormwater runoff within a stormwater management system is critical to providing the extended detention of flows for water quality treatment and downstream channel protection, as well as for peak flow attenuation of the larger overbank and extreme flood protection flows. Runoff storage can be provided within an on-site system through the use of structural stormwater BMPs and/or non-structural features and landscaped areas. Figure 3-9 illustrates various storage facilities that can be considered for a development site.

Figure 3-9. Examples of Typical Stormwater Storage Facilities

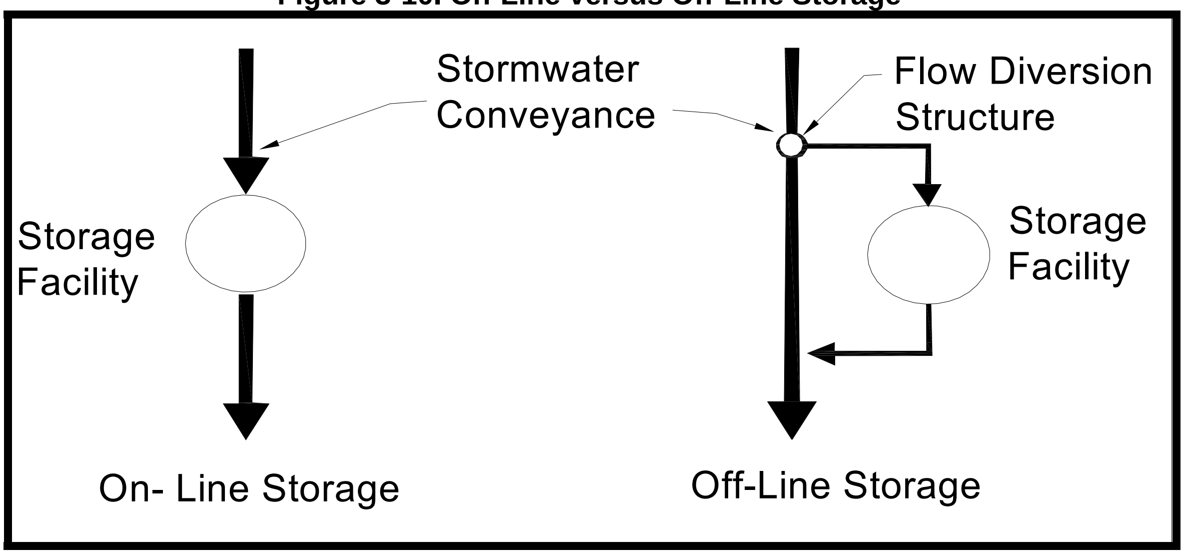


There are three main types of stormwater runoff storage: *detention, extended detention, and retention*. Stormwater *detention* is used to reduce the peak discharge and detain runoff for a specified short period of time. Detention basins are designed to completely drain after the design storm has passed. Detention is used to meet overbank flood protection criteria, and extreme flood criteria where required. *Extended detention* (ED) is used to drain a runoff volume over a specified period of time, typically 24 hours, and is used to meet channel protection criteria. Some structural BMP designs (wet ED pond, micropool ED pond, dry extended pond and shallow ED marsh) also include extended detention storage of a portion of the water quality volume. *Retention* facilities, such as stormwater ponds and wetlands, are designed to contain a permanent pool of water that is used for water quality treatment. Some facilities include one or more types of storage. An example of a combined storage facility is one that is sized to provide extended detention of the WQv as well as detention of the Qp_{100}.

Storage facilities are often classified on the basis of their location and size. *On-site* storage is constructed on individual development sites and most often only provides control of the runoff that discharges that individual site. *Regional* storage facilities are designed to manage stormwater runoff from multiple projects and/or properties, or are constructed at the lower end of a sub-basin within which multiple properties are located. Knox County Engineering will determine if the use of a regional storage facility is applicable on a case-by-case basis.

Storage can also be categorized as *on-line* or *off-line*. On-line storage uses a structural BMP facility that intercepts flows directly within a conveyance system or stream. Off-line storage is a separate storage facility to which flow is diverted from the conveyance system. Figure 3-10 illustrates on-line versus off-line storage.

Figure 3-10. On-Line versus Off-Line Storage



3.2.1.1 Stage-Storage Relationship

A stage-storage curve defines the relationship between the depth of water (stage) and storage volume in a storage facility. An example of a stage-storage curve is presented in Figure 3-11. This curve relationship allows the volume of storage to be calculated by using simple geometric formulas expressed as a function of depth. The storage volume for natural basins may be developed using a topographic map and the double-end area, frustum of a pyramid, prismoidal or circular conic section formulas.

<u>Double-end area method:</u> The double-end area method uses the areas of the planes at two given elevations to calculate the volume between the two area planes. This concept is presented in Figure 3-12. The double-end area equation is presented in Equation 3-27.

Equation 3-27

$$V_{1-2} = \left[\frac{(A_1 + A_2)}{2}\right] d$$

where:

V_{1-2} = storage volume (ft^3) between elevations 1 and 2
A_1, A_2 = surface area at elevation 1 and 2, respectively (ft^2)
d = change in elevation between points 1 and 2 (ft)

Figure 3-11. Stage-Storage Curve

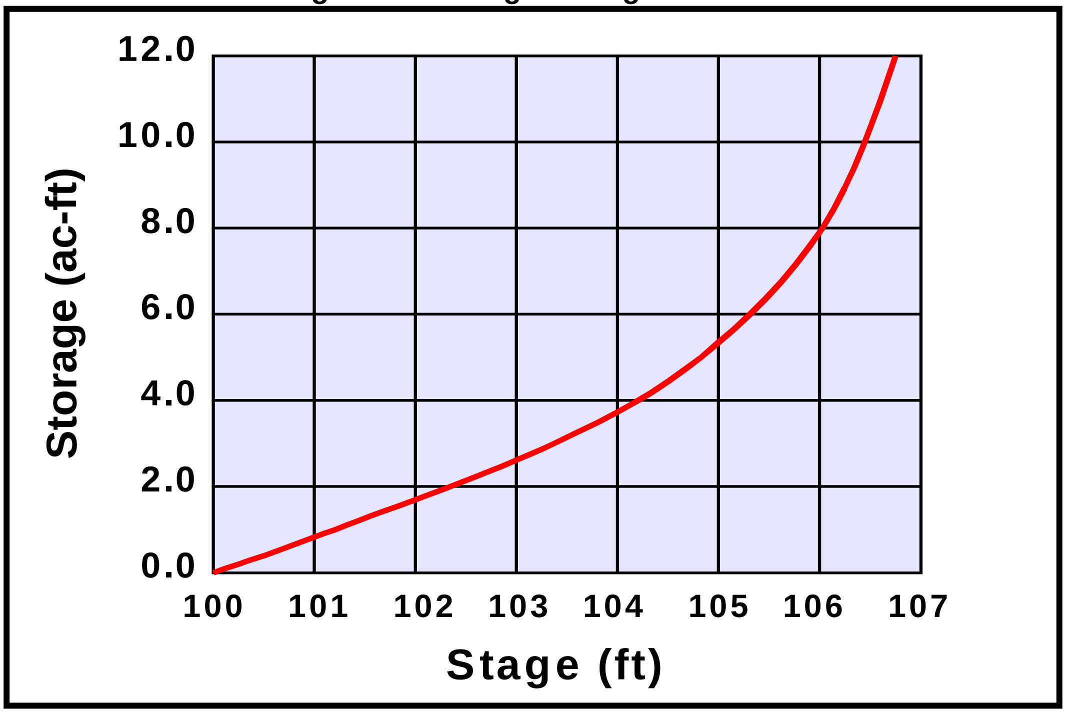
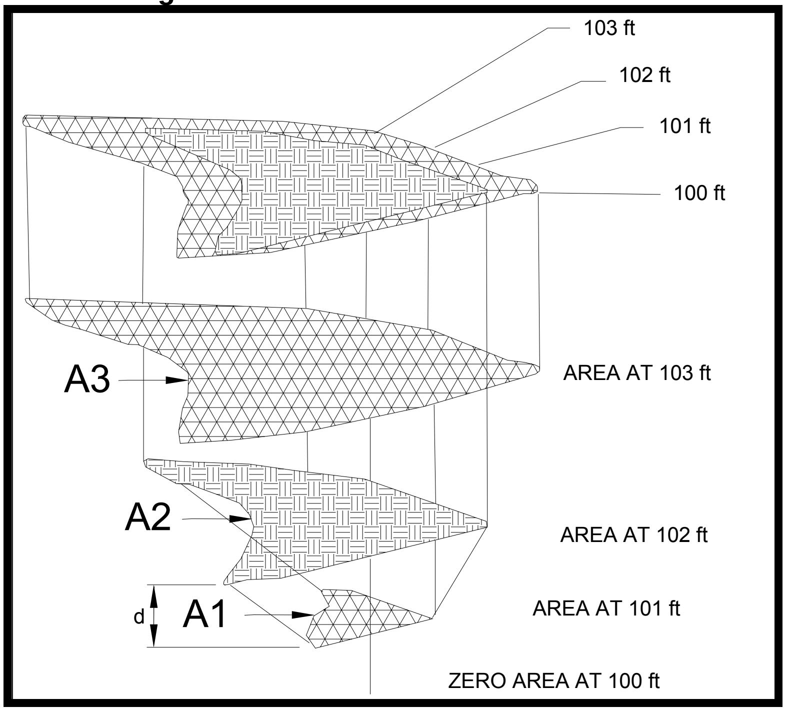


Figure 3-12. Double-End Area Method



Frustrum of a pyramid method: Another calculation involves treating the storage as a pyramid frustum, or a part of a pyramid. The frustum is formed by truncating the pyramid using two planes parallel to the pyramid base. The frustum of a pyramid can be calculated using Equation 3-28.

Equation 3-28

$$V = \frac{d}{3}\left[A_1 + \left(A_1 \times A_2\right)^{0.5} + A_2\right]$$

where:

V = volume of frustum of a pyramid (ft^3)
d = change in elevation between points 1 and 2 (ft)
A_1 = surface area at elevation 1 (ft^2)
A_2 = surface area at elevation 2 (ft^2)

Frustrum of a prismoid method: A trapezoidal basin can be represented as a prismoid since the volume is formed by the trapezoidal faces. Equation 3-29 presents the prismoidal equation for trapezoidal basins.

Equation 3-29

$$V = LWD + \left(L + W\right)ZD^2 + \frac{4}{3}Z^2D^3$$

where:

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

Frustrum of a cone or conic section method: Equations 3-30 and 3-31 present the calculation approach for the basin storage volume represented is as a circular cone.

Equation 3-30
$$V = 1.047D\left(R_1^{\ 2} + R_2^{\ 2} + R_1R_2\right)$$

Equation 3-31
$$V = 1.047D\left(3R_1^{\ 2} + 3ZDR_1 + Z_2D^2\right)$$

where:

V = volume of circular cone basin (ft^3)
R_1, R_2 = bottom and surface radii of the conic section (ft)
D = depth of basin (ft)
Z = side slope factor, ratio of horizontal to vertical

3.2.1.2 Stage-Discharge Relationship

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. Figure 3-13 presents an example stage-discharge curve. A typical storage facility has multiple outlets or spillways: a principal outlet that handles the range of design storms and design criteria and a secondary (or emergency) outlet. The principal outlet is usually designed with a capacity sufficient to convey the design discharges and volumes without allowing flow to enter the emergency spillway. Pipes, culverts, weirs, perforated risers and other appropriate outlets can be used in the principal spillway or outlet.

The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal outlet. This spillway should be designed taking into account the potential threat to downstream areas if the storage facility were to fail. The stage-discharge curve should take into account the discharge characteristics of both the principal spillway and the emergency spillway. For more details, see Section 3.3 of this chapter.

3.2.2 Symbols and Definitions

To provide consistency within this section, the symbols listed in Table 3-19 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Figure 3-13. Example Stage-Discharge Curve

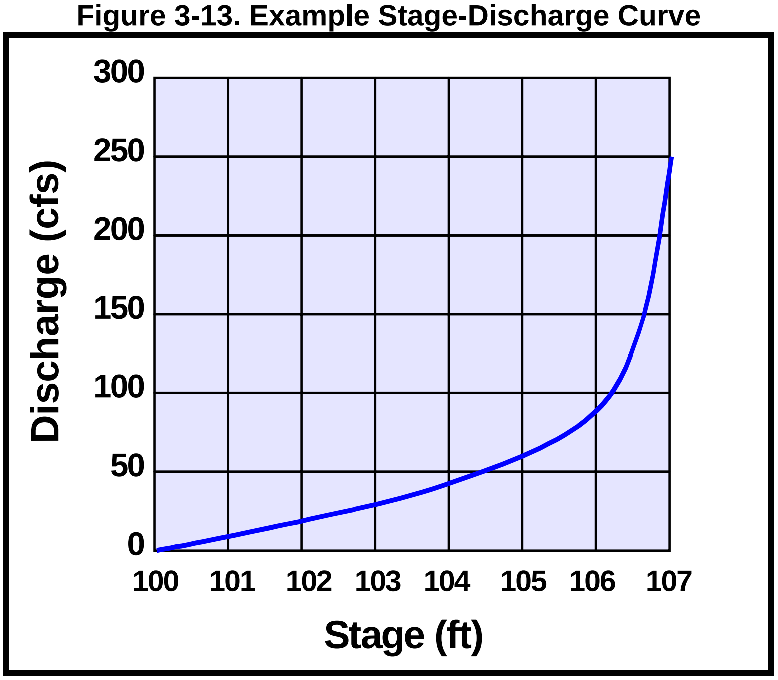


Table 3-19. Symbols and Definitions for Storage Design

Symbol	Definition	Units
a,b	Rainfall factors for Modified Rational Method	-
A	Cross sectional or surface area	ft^2
A	Drainage area	acres (or mi^2)
C_w	Weir coefficient	-
C	Rational Method Runoff Coefficient	-
CN	Curve number	-
CPv	Channel protection volume	acre-ft
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
g	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_C	Height of weir crest above channel bottom	ft
i	Rainfall intensity	in/hr
L	Length	ft
P_x	Storm depth for x duration storm	in
q_i,q_o	Peak inflow or outflow rate	cfs, in
R	Surface Radii	ft
Rv	Runoff coefficient	-
t	Routing time period	sec
t_b	Time base of hydrograph	sec, hr
Tc	Time of concentration	min
T_d	Critical storm duration	min
T_i	Duration of basin inflow	hr, min, sec
t_P	Time to peak of hydrograph	hr
T_t	Travel time	min
V, V_s	Storage volume	ft^3, acre-ft
V_r	Runoff volume	ft^3, acre-ft
W	Width of basin	ft
WQv	Water quality volume	acre-ft
Z	Side slope factor	-

3.2.3 Flood Protection Storage Design Procedures

This section discusses the general design procedures for designing storage to provide flood protection detention of stormwater runoff for the Qp_2, Qp_{10}, Qp_{25} and Qp_{100}. The design procedures for all storage facilities are the same whether or not they include a permanent pool of water. In the latter case, the permanent pool elevation is taken as the “bottom” of storage and is treated as if it were a solid basin bottom for routing purposes.

The location of a storage facility can have a sizeable impact on the effectiveness of such facilities to control downstream impacts. In addition, multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the downstream conveyance system, which could decrease or increase flood peaks in different downstream locations. Therefore, a downstream peak flow analysis (i.e., the 10% rule) should be performed as part of the storage facility design process. In multi-purpose multi-stage facilities such as stormwater ponds, the storage design must be integrated with the overall design for water quality treatment objectives. See Volume 2, Chapter 4 for further guidance and criteria for the design of structural best management practices (BMPs) for water quality control.

3.2.3.1 Design Procedure

The following data are needed for storage design and routing calculations:

- inflow hydrograph for all selected design storms;
- stage-storage curve for proposed storage facility; and
- stage-discharge curve(s) for all outlet control structures.

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

1. Compute inflow hydrographs for the 2, 10, 25 and 100-year, 24-hour design storms using the hydrologic methods outlined in Section 3.1. Both existing and post-development conditions hydrographs are required.
2. Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1.
3. Determine the physical basin dimensions necessary to hold the volumes determined in Step 2, including freeboard, which is defined as 1.0 foot above the Qp_{100} water surface elevation to the lowest point in the detention embankment, excluding the emergency spillway. The maximum storage requirement calculated from Step 2 should be used. From the selected basin shape, determine the maximum depth in the pond.
4. Select the type of outlet(s) and size each outlet structure. The outlet type and size will depend on the type of basin (detention, extended detention or retention) as well as the allowable discharge. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure(s) should be sized to convey the allowable discharge at this stage.
5. Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using a storage routing computer model. If the routed post-development peak discharges (Qp_2, Qp_{10}, Qp_{25} and the Qp_{100}) exceed the existing conditions peak discharges, then revise the available storage volume, outlet device(s), etc., and return to Step 3 until the basin size, basin depth, outlet type and outlet size meet the allowable discharge requirements.
6. Apply the 10% rule (i.e., downstream effects of detention outflows) for the 2-year, 10-year, 25-year and 100-year storms to ensure that the routed hydrograph does not cause peak flow increases, water level increases or downstream flooding problems.
7. Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

Routing hydrographs through storage facilities is critical to the proper facility design and is required in Knox County. Although storage design procedures have been developed that use inflow/outflow analysis without routing, these design procedures have not produced acceptable results in designing detention facilities for many areas of the country, including Knox County.

Although hand calculation procedures are available for routing hydrographs through storage facilities, these procedures are very time consuming, especially when several different designs are evaluated. Many standard hydrology and hydraulics textbooks give examples of hand-routing techniques. For this manual, it is assumed that designers will be using one of the many computer programs available for storage routing and thus other procedures and example applications will not be given here.

3.2.4 Preliminary Detention Calculations

Procedures for preliminary detention calculations are included here to provide a simple method that can be used to estimate storage needs and also provide a quick check on the results of using different computer programs. Standard routing should be used for actual (final) storage facility calculations and design.

3.2.4.1 Storage Volume Estimation

For small drainage areas, a preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular hydrograph shapes shown in Figure 3-14. The required storage volume may be estimated from the hatched area between the inflow and outflow hydrographs. This preliminary storage volume estimate can be calculated using Equation 3-32.

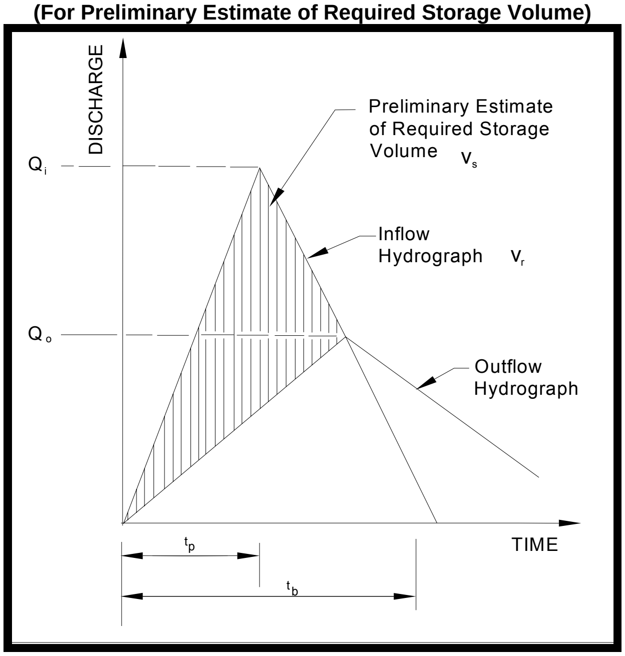
Equation 3-32

$$V_s = 0.5 t_b \left(Q_i - Q_o\right)$$

where:

V_s = storage volume estimate (ft^3)
t_b = time base of hydrograph (s)
Q_i = peak inflow rate (cfs)
Q_o = peak outflow rate (cfs)

Figure 3-14. Triangular-Shaped Hydrographs
(For Preliminary Estimate of Required Storage Volume)



3.2.4.2 Alternative Storage Volume Estimation Method

An alternative preliminary estimate of the storage volume required for a specified peak flow reduction can be obtained using the following regression equation procedure (Wycoff and Singh, 1976).

1. Determine input data, including the allowable peak outflow rate, Q_O; the peak flow rate of the inflow hydrograph, Q_i; the time base of the inflow hydrograph, t_b; and the time to peak of the inflow hydrograph, t_p.
2. Calculate a preliminary estimate of the ratio V_S/V_r using the input data from Step 1 and Equation 3-33.

Equation 3-33

$$\frac{V_s}{V_r} = \frac{1.291\left(1-\frac{Q_o}{Q_i}\right)^{0.753}}{\left(\frac{t_p}{t_b}\right)^{0.411}}$$

where:

V_s = volume of storage (in)
V_r = volume of runoff (in)
Q_o = outflow peak flow (cfs)
Q_i = inflow peak flow (cfs)
t_b = time base of the inflow hydrograph (hr) [Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak]
t_p = time to peak of the inflow hydrograph (hr)

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

3.2.4.3 Peak Flow Reduction Estimate

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

1. Determine volume of runoff, V_r; peak flow rate of the inflow hydrograph, Q_i; time base of the inflow hydrograph, t_b; time to peak of the inflow hydrograph, t_p; and storage volume V_s.
2. Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using Equation 3-34 (Wycoff and Singh, 1976):

Equation 3-34

$$\frac{Q_o}{Q_i} = 1 - 0.712\left(\frac{V_s}{V_r}\right)^{1.328}\left(\frac{t_b}{t_p}\right)^{0.546}$$

where:

Q_o = outflow peak flow (cfs)
Q_i = inflow peak flow (cfs)
V_s = volume of storage (in)
V_r = volume of runoff (in)
t_b = time base of the inflow hydrograph (hr) [Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak]
t_p = time to peak of the inflow hydrograph (hr)

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

3.2.5 Estimation of the Channel Protection Volume

The Simplified SCS Peak Runoff Rate Calculation approach (see Section 3.1.5.4) can be used for estimation of the channel protection volume (CPv) for storage facility design. The calculation procedure is as follows.

Step 1. 2.5 inches is used for the 1-year, 24-hour rainfall depth (P, in inches).

Step 2. A runoff curve number (CN) is then estimated according to the procedure in section 3.1.5.2.

Step 3. The CN value is used to determine the initial abstraction (I_a) from Table 3-13, and the ratio I_a/P is computed.

Step 4. The accumulated runoff (Q_d, inches) can then be calculated using Equation 3-12.

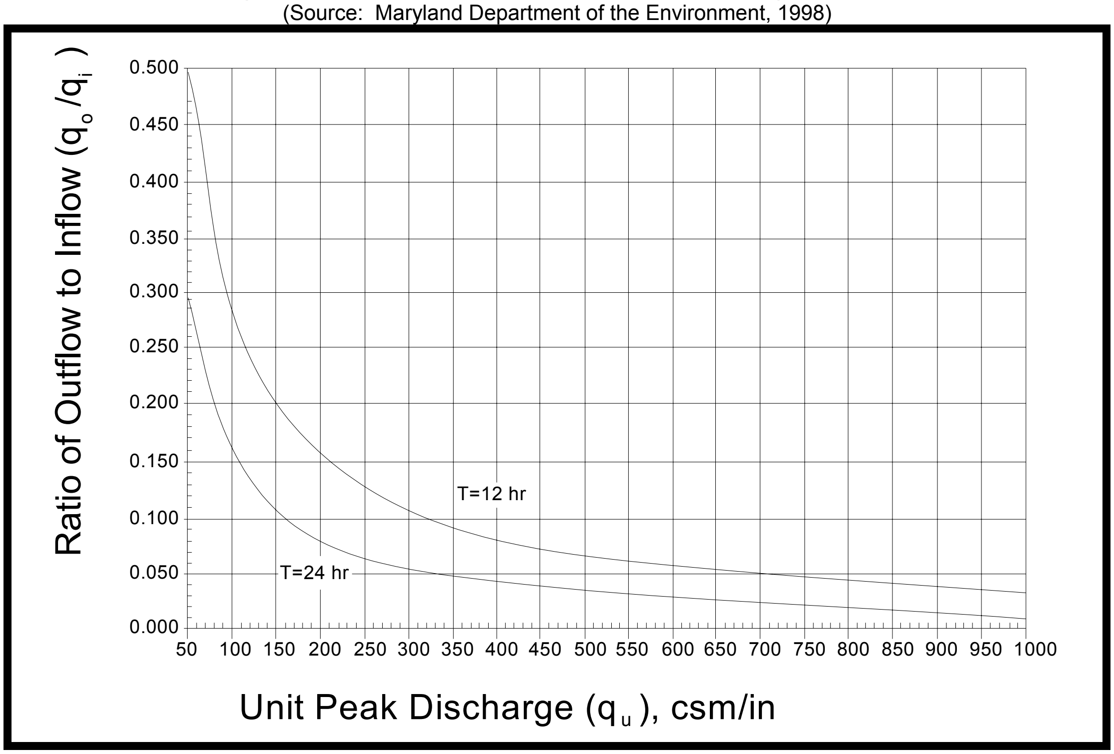
Step 5. Compute the drainage area time of concentration (t_c) for the post-development land use using the method outlined in section 3.1.3.5.

Step 6. Use t_c with the ratio I_a/P to obtain the unit peak discharge, q_u, from Figure 3-6 for the Type

II rainfall distribution. If the ratio I_a/P lies outside the range shown in the figure, either use the limiting values or use another peak discharge method.

Step 7. Knowing q_u and T (extended detention time, minimum of 24 hours and maximum of 72 hours); the q_o/q_i ratio (peak outflow discharge/peak inflow discharge) can be estimated from Figure 3-15.

Figure 3-15. Detention Time vs. Discharge Ratios
(Source: Maryland Department of the Environment, 1998)



Step 8. V_s/V_r is then determined from Figure 3-16 or Equation 3-35, which were developed from the SCS TR-55 hydrologic model using a Type II rainfall distribution.

Equation 3-35

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

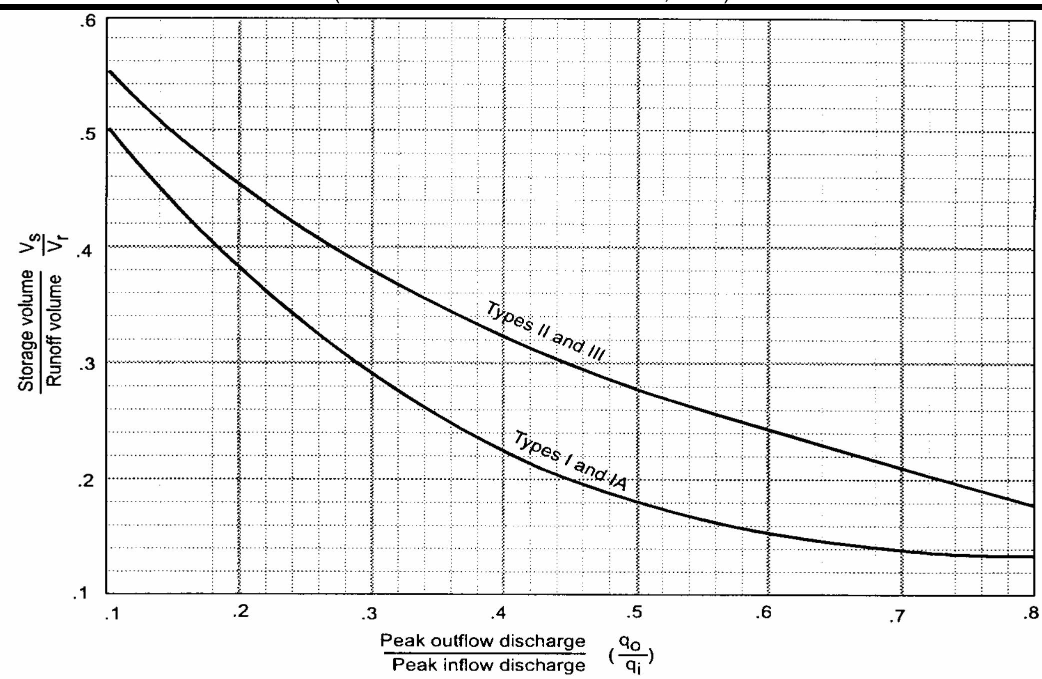
where:

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

This equation is only reliable when $0.1 < q_o/q_i < 0.8$.

Figure 3-16. Approximate Detention Basin Routing for Rainfall Types I, IA, II, and III

(Source: Soil Conservation Service, 1986)



Step 9. The required storage volume (CPv in this case) can then be calculated using Equation 3-36. To check the CPv estimate, the volume must be incorporated into a BMP design and the 1-year 24-hour storm routed through the BMP. The CPv is adequate when the 1-year 24-hour design storm is detained for 24 hours, measured from the centroid of the inflow hydrograph to the centroid of the outflow hydrograph.

Equation 3-36

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

where:

V_s and V_r are defined above
Q_d = the developed runoff for the design storm (inches) (see Figure 3-3)
A = total drainage area (acres)

<u>Example 3-10. Estimation of CPv</u>

Estimate the CPv necessary for a 50-acre wooded watershed located in Knox County, which will be developed as follows:
Forest land - good cover (hydrologic soil group B) = 10 ac
Forest land - good cover (hydrologic soil group C) = 10 ac
Residential with 1/3 acre lots (hydrologic soil group B) = 20 ac
Industrial development (hydrological soil group C) = 10 ac
Total impervious area = 18 acres
% of pond and swamp area = 0

<u>Step 1</u> Calculate the rainfall excess.
The 1-year, 24 hour rainfall is 2.5 inches (Table 3-5).

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

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

1 – Composite CN = $\frac{\text{\% Total}}{100}$ * CN.

From Equation 3-12 or Figure 3-3, Q_2= 0.53

Step 3 Calculate Ia/P for CN = 72
Ia = 0.778 (Table 3-13)
Ia/P = 0.778/2.5 = 0.31

Step 4 Calculate Qd for 1-year 24-hour storm using Equation 3-12
Qd = (2.5-0.778)2/(2.5-0.778+5*0.778) = 0.53 inches

Step 5 Calculate Tc.
The hydrologic flow path for this watershed = 1,890 ft. It is divided into segments as shown in the table below.

Segment	Type of Flow	Length (ft)	Slope (%)
1	Overland n = 0.24	40	2.00
2	Shallow channel	750	1.70
3	Main channel*	1100	0.50

* For the main channel, n = 0.06 (estimated), width = 10 feet, depth = 2 feet, rectangular channel

Segment 1 – Travel time from Equation 3-4 with P_2 = 3.30 in (0.138 x 24 from Table 3-4)
T_t = $0.007[(0.24)(40)]^{0.8}/(3.30)^{0.5}(0.02)^{0.4}$
= 0.115 hrs = 6.75 minutes

Segment 2 – Travel time from Figure 3-2 or Equation 3-7
V = 2.1 ft/s
T_t = 750/(60)(2.1) = 5.95 min

Segment 3 - Using Equations 3-10 and 3-9
V = $[(1.49)(0.06)(1.43)]^{0.67}/(0.005)^{0.5}$ = 2.23 ft/s
T_t = 1100/60(2.23) = 8.22 min

Therefore, adding the three segments using Equation 3-3:
Tc = 6.75 + 5.95 + 8.22 = 20.92 min = 0.35 hours

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

Step 7 Estimate channel protection volume (CPv = V_S)
Knowing q_u (1-year) = 540 csm/in from Step 6 and T (extended detention time of 24 hours), find q_o/q_i from Figure 3-15.
q_o/q_i = 0.035

Step 8 Estimate storage/runoff using Equation 3-35,

$V_s/V_r = 0.682 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$

$V_s/V_r = 0.682 - 1.43(0.035) + 1.64(0.035)^2 - 0.804(0.035)^3 = 0.63$

Step 9 The necessary channel protection volume is then calculated using Equation 3-36

$CPv = Vx \approx (0.64)(0.53)(50)/12 \approx 1.39$ ac-ft

3.3 Outlet Structures

3.3.1 Symbols and Definitions

To provide consistency within this section as well as throughout this manual, the symbols listed in Table 3-20 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 3-20. Symbols and Definitions for Outlet Structures

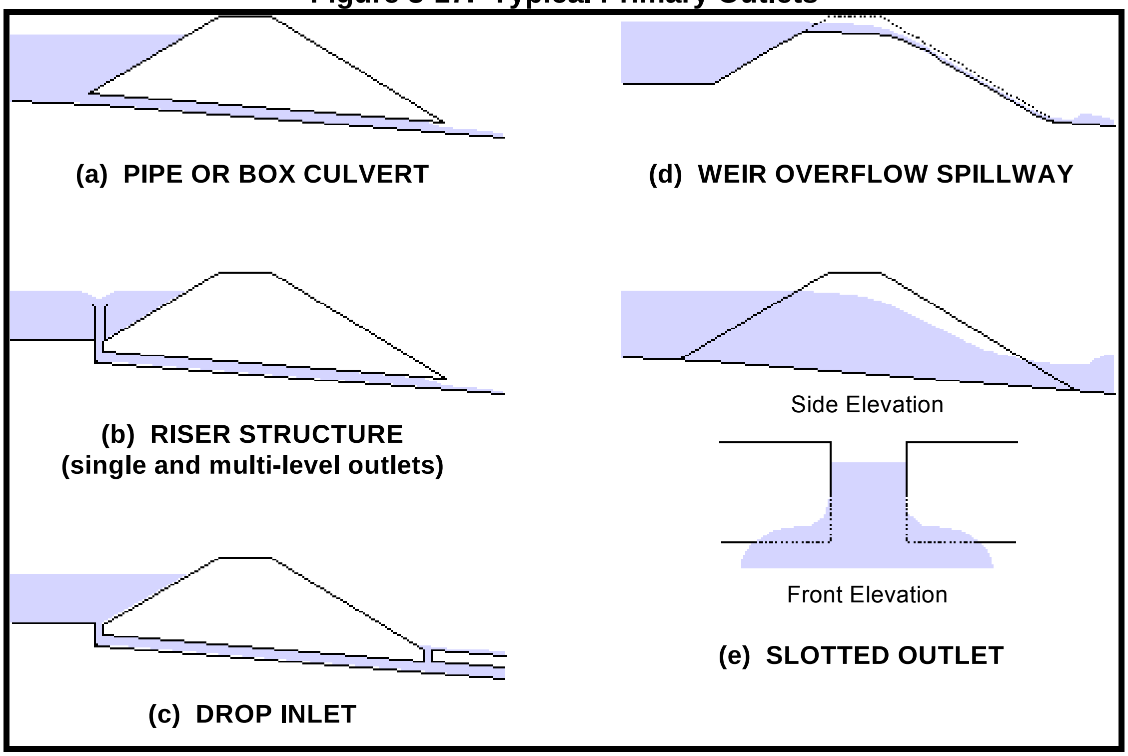
Symbol	Definition	Units
A,a	Cross sectional or surface area	ft^2
A_m	Drainage area	acres (or mi^2)
A_p	Cross sectional area of all holes (perforated riser)	ft^2
B	Breadth of weir	ft
C_w	Weir coefficient or Discharge coefficient	-
C_p	Discharge coefficient for perforations	-
CPv	Channel protection volume	ac-ft
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
g	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_C	Height of weir crest above channel bottom	ft
k_m	Coefficient of minor losses (1.0)	-
k_p	Pipe friction coefficient	-
K_g	Bar shape factor	-
L	Length	ft
n	Manning's "n"	-
Q,q	Peak inflow or outflow rate	cfs, in
Q_f	Free flow	cfs
Q_S	Submergence flow	cfs
V_u	Approach velocity	ft/s
WQv	Water quality volume	ac-ft
w	Maximum cross sectional bar width facing the flow	in
x	Minimum clear spacing between bars	in
θ	Angle of v-notch	degrees
θ_γ	Angle of the grate with respect to the horizontal	degrees

3.3.2 Primary Outlets

Primary outlets provide the critical function of the regulation of flow for structural stormwater BMPs. The different types of outlets consist of a single stage outlet structure, or several outlet structures combined to provide multi-stage outlet control. Figure 3-17 shows several typical primary outlets. For a single stage system, the stormwater facility can be designed as a simple pipe or culvert. For multi-stage control structures such as an extended detention pond used for control of WQv, CPv, Qp_2, Qp_{10}, Qp_{25}, and Qp_{100}, the inlet is designed considering a range of design flows. A stage-discharge curve is developed for the full range of flows that the structure would experience. The outlets may be housed in a riser or several pipes or culverts in a single location.

This section provides an overview of outlet structure hydraulics and design for stormwater storage facilities. The design engineer is referred to an appropriate hydraulics text for additional information on outlet structures not contained in this section.

Figure 3-17. Typical Primary Outlets



3.3.2.1 Outlet Structure Types

There are a wide variety of outlet structure types, the most common of which are covered in this section. Descriptions and equations are provided for the following outlet types for use in stormwater facility design.

- orifice
- perforated riser
- pipe/culvert
- sharp-crested weir
- broad-crested weir
- V-notch weir
- proportional weir
- combination outlet

Each of these outlet types has a different design purpose and application. The control of WQv and CPv flows are normally handled with smaller, more protected outlet structures such as reverse slope pipes, hooded orifices, orifices located within screened pipes or risers, perforated plates or risers, and V-notch weirs. Larger discharges, such as Qp_2, Qp_{10}, Qp_{25} and Qp_{100} flows, are typically handled through a riser with different sized openings, through an overflow at the top of a riser (drop inlet structure), or a flow over a broad crested weir or spillway. Overflow weirs can also be of different heights and configurations to handle control of multiple design flows.

3.3.2.2 Orifices

An orifice is a circular or rectangular opening of a prescribed shape and size. The flow rate is dependant on the height of the water above the opening and the size and edge treatment of the orifice. For a single orifice, as illustrated in Figure 3-18(a), the orifice discharge can be determined using the standard orifice equation shown in Equation 3-37 below. Figure 3-18(c) shows a perforated riser that has multiple openings.

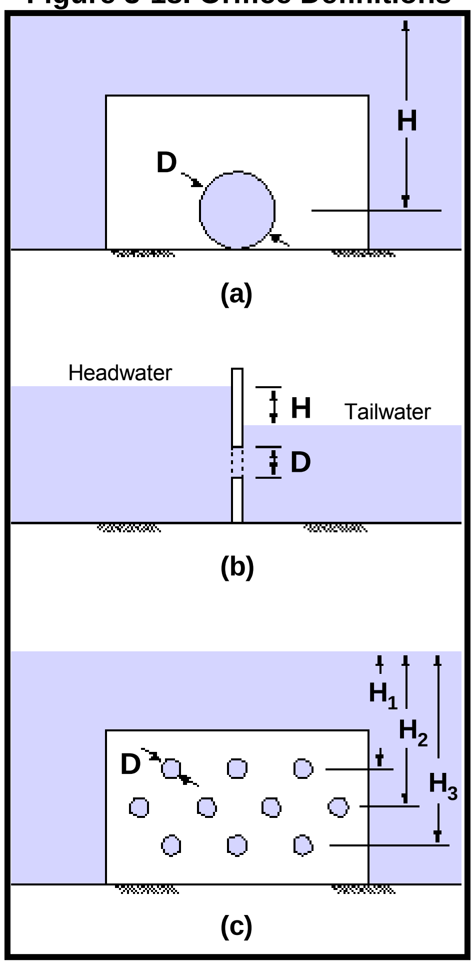
Equation 3-37

$$Q = CA(2gH)^{0.5}$$

where:

- Q = the orifice flow discharge (cfs)
- C = discharge coefficient
- A = cross-sectional area of orifice or pipe (ft^2)
- g = acceleration due to gravity (32.2 ft/s^2)
- H = effective head on the orifice, from the center of orifice to the water surface

Figure 3-18. Orifice Definitions



When the material used for the orifice is thinner than the orifice diameter (i.e., it has sharp edges), a discharge coefficient of 0.6 should be used. When the material is thicker than the orifice diameter a coefficient of 0.80 should be used. If the edges are rounded, a coefficient of 0.92 can be used. Equation 3-38 presents a simplification of the orifice equation that can be used for a round orifice with square-edged entrance conditions:

Equation 3-38 $$Q = 0.6A(2gH)^{0.5} = 3.78D^2H^{0.5}$$

where:

- Q = the orifice flow discharge (cfs)
- C = discharge coefficient
- A = cross-sectional area of orifice or pipe (ft^2)
- g = acceleration due to gravity (32.2 ft/s^2)
- H = effective head on the orifice, from the center of orifice to the water surface
- D = diameter of orifice or pipe (ft)

If the orifice discharges as a free outfall, then the effective head is measured from the center of the orifice to the upstream (headwater) surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the headwater and tailwater surfaces as shown in Figure 3-18(b).

Flow through multiple orifices, such as the perforated plate shown in Figure 3-18(c), can be computed by summing the flow through individual orifices. For multiple orifices of the same size and under the influence of the same effective head, the total flow can be determined by multiplying the discharge for a single orifice by the number of openings.

Perforated orifice plates for the control of discharge can be of any size and configuration. However, the Denver Urban Drainage and Flood Control District has developed standardized dimensions that have worked well. These are presented in Table 3-21.

Table 3-21. Circular Perforation Sizing

(Source: Urban Drainage and Flood Control District, 1999)

Hole Diameter (in)	Minimum Column Hole Centerline Spacing (in)	Flow Area per Row (in^2)		
		N=1	N=2	N=3
0.25	1	0.05	0.1	0.15
0.3125	2	0.08	0.15	0.23
0.375	2	0.11	0.22	0.33
0.4375	2	0.15	0.3	0.45
0.50	2	0.2	0.4	0.6
0.5625	3	0.25	0.5	0.75
0.625	3	0.31	0.62	0.93
0.6875	3	0.37	0.74	1.11
0.75	3	0.44	0.88	1.32
0.8125	3	0.52	1.04	1.56
0.875	3	0.6	1.2	1.8
0.9375	3	0.69	1.38	2.07
1.0	4	0.79	1.58	2.37
1.0625	4	0.89	1.78	2.67
1.125	4	0.99	1.98	2.97
1.1875	4	1.11	2.22	3.33
1.25	4	1.23	2.46	3.69
1.3125	4	1.35	2.7	4.05
1.375	4	1.48	2.96	4.44
1.4375	4	1.62	3.24	4.86
1.50	4	1.77	3.54	5.31
1.5625	4	1.92	3.84	5.76
1.625	4	2.07	4.14	6.21
1.6875	4	2.24	4.48	6.72
1.75	4	2.41	4.82	7.23
1.8125	4	2.58	5.16	7.74

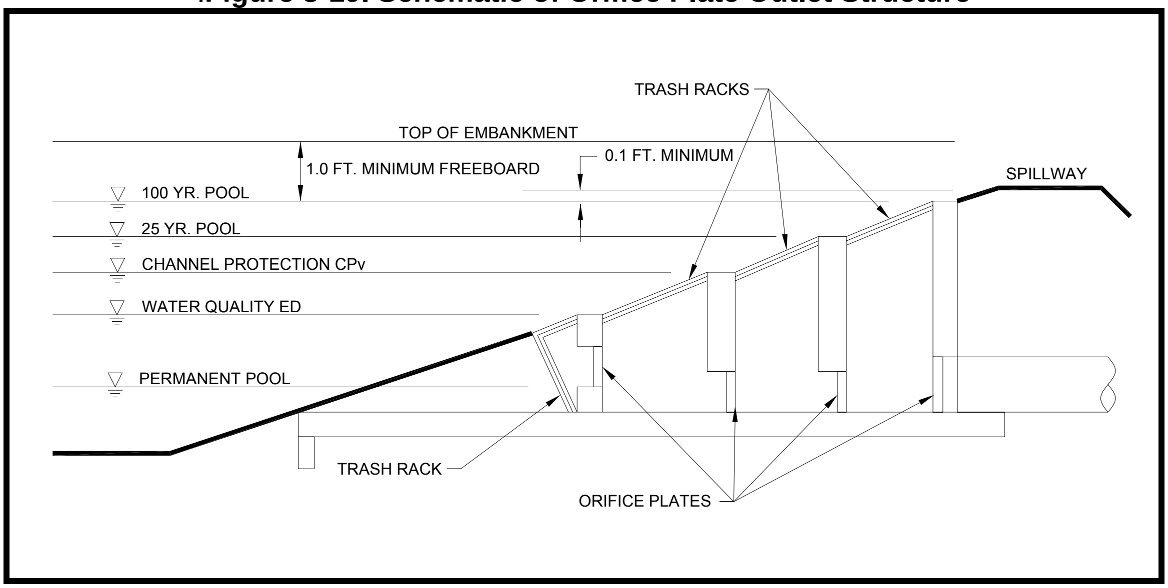
Hole Diameter (in)	Minimum Column Hole Centerline Spacing (in)	Flow Area per Row (in^2)		
		N=1	N=2	N=3
1.875	4	2.76	5.52	8.28
1.9375	4	2.95	5.9	8.85
2.0	4	3.14	6.28	9.42
N = Number of columns of perforations				
Minimum steel plate thickness		0.25”	0.3125”	0.375”

The vertical spacing between orifice row hole centerlines is always 4-inches.

Only one column of rectangular slots is allowed unless additional columns are hydraulically independent.

Figure 3-19 provides a schematic of an orifice plate outlet structure for a wet ED pond showing the design pool elevations and the flow control mechanisms. For simplicity, the outlets for the 2-year and 10-year pools are not shown.

.**Figure 3-19. Schematic of Orifice Plate Outlet Structure**



3.3.2.3 Perforated Risers

A special kind of orifice is a perforated riser as illustrated in Figure 3-20. In the perforated riser, an orifice plate at the bottom of the riser, or in the outlet pipe just downstream from the elbow at the bottom of the riser, controls the flow. It is important that the perforations in the riser convey more flow than the orifice plate so that the perforations do not become the control. Referring to Figure 3-20, Equation 3-39 presents a formula that has been developed to estimate the total flow capacity of the perforated section (McEnroe, 1988).

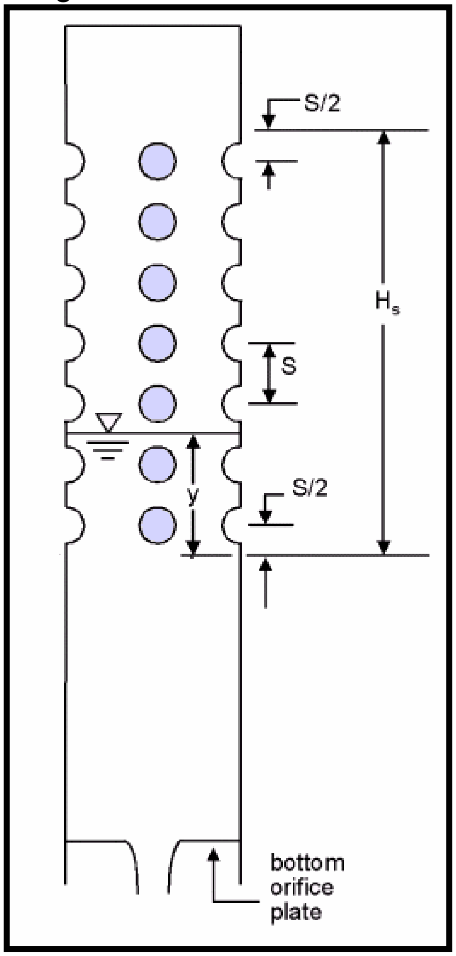
Equation 3-39

$$Q = C_p \frac{2A_p}{3H_s}\sqrt{2g}H^{3/2}$$

where:

- Q = discharge (cfs)
- C_p = discharge coefficient for perforations (normally 0.61)
- A_p = cross-sectional area of all the holes (ft^2)
- g = acceleration due to gravity (32.2 ft/s^2)
- H_s = distance from S/2 below the lowest row of holes to S/2 above the top row (ft)

Figure 3-20. Perforated Riser



3.3.2.4 Pipes and Culverts

Discharge pipes are often used as outlet structures for stormwater control facilities. The design of these pipes can be for either single or multi-stage discharges. A reverse-slope, underwater pipe is often used for water quality or channel protection outlets.

Pipes smaller than 12 inches in diameter may be analyzed as a submerged orifice as long as *H/D* is greater than 1.5. *Note: For low flow conditions, when the flow reaches and begins to overflow the pipe, discharge over the weir is the controlling flow. As the stage increases the controlling flow will transition to the flow through the orifice.* Pipes greater than 12 inches in diameter should be analyzed as a discharge pipe with headwater and tailwater effects taken into account. The outlet hydraulics for pipe flow can be determined from the outlet control culvert nomographs and procedures given in Chapter 7 or by using Equation 3-39 (NRCS, 1984). Equation 3-40 is a general pipe flow equation that is derived through the use of the Bernoulli and continuity principles, and should only be used for pipes flowing full. The use of FHWA HDS-5 methodology, and HY-8 or similar software is highly recommended for analysis of culverts. Concrete pipe is required for outflow pipes of all flood protection BMPs.

Equation 3-40

$$Q = a\left[\frac{(2gH)}{(1 + k_m + k_p L)}\right]^{0.5}$$

where:

Q	= discharge (cfs)
a	= pipe cross sectional area (ft^2)
g	= acceleration of gravity (ft/s^2)
H	= elevation head differential (ft)
k_m	= coefficient of minor losses (use 1.0)
k_p	= Manning's "n" friction head loss / unit length = $5087 * n^2/\text{dia(in)}^{4/3}$
L	= pipe length (ft)

3.3.2.5 Sharp-Crested Weirs

If the overflow portion of a weir has a sharp, thin leading edge such that the water springs clear of the weir plate as it overflows, the overflow is termed a *sharp-crested* weir. The weir's crest is the edge where the water flows over the weir. If the sides of the weir also cause the flow through the weir to contract, it is termed an *end-contracted* sharp-crested weir. Sharp-crested weirs have stable stage-discharge relations and are often used as a flow measurement device. A sharp-crested weir with no end contractions is illustrated in Figure 3-21(a). The discharge equation for this configuration is presented in Equation 3-41 (Chow, 1959).

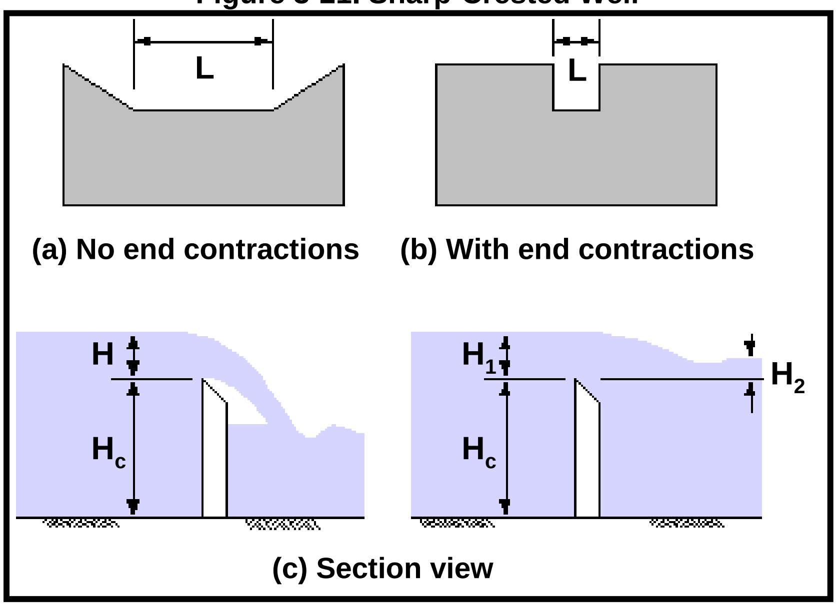
Equation 3-41

$$Q = \left[3.27 + 0.4\left(\frac{H}{H_c}\right)\right] L H^{1.5}$$

where:

Q	= discharge (cfs)
H	= head above weir crest excluding velocity head (ft)
H_c	= height of weir crest above channel bottom (ft)
L	= horizontal weir length (ft)

Figure 3-21. Sharp-Crested Weir



A sharp-crested weir with two end contractions is illustrated in Figure 3-21(b). Equation 3-42 presents the discharge equation for this configuration (Chow, 1959).

Equation 3-42

$$Q = \left[3.27 + 0.4\left(\frac{H}{H_c}\right)\right] (L - 0.2H) H^{1.5}$$

where:

- Q = discharge (cfs)
- H = head above weir crest excluding velocity head (ft)
- H_c = height of weir crest above channel bottom (ft)
- L = horizontal weir length (ft)

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. Equation 3-43 presents the discharge equation for a submerged sharp-crested weir (Brater and King, 1976).

Equation 3-43

$$Q_s = Q_f \left[1 - \left(\frac{H_2}{H_1}\right)^{1.5}\right]^{0.385}$$

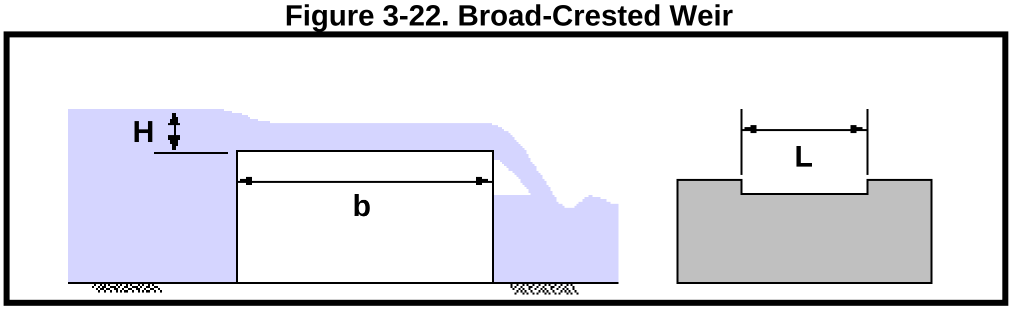
where:

- Q_s = submergence flow (cfs)
- Q_f = free flow (cfs)
- H_1 = upstream head above crest (ft)
- H_2 = downstream head above crest (ft)

3.3.2.6 Broad-Crested Weirs

A weir in the form of a relatively long raised channel control crest section is a *broad-crested* weir as shown in Figure 3-22. The flow control section can have different shapes, such as triangular or circular. True broad-crested weir flow occurs when upstream head above the crest is between the limits of about 1/20 and 1/2 the crest length in the direction of flow. For example, a thick wall or a flat stop log can act like a sharp-crested weir when the approach head is large enough that the flow springs from the upstream corner. If upstream head is small enough relative to the top profile length, the stop log can act like a broad-crested weir (USBR, 1997).

Figure 3-22. Broad-Crested Weir



Equation 3-44 presents the discharge equation for a broad-crested weir (Brater and King, 1976).

Equation 3-44

$$Q = CLH^{1.5}$$

where:

- Q = discharge (cfs)
- C = broad-crested weir coefficient
- L = broad-crested weir length perpendicular to flow (ft)
- H = head above weir crest (ft)

If the upstream edge of a broad-crested weir is so rounded as to prevent flow contraction and if the slope of the crest is as great as the friction head loss, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.32. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Information on C values as a function of weir crest breadth and head is given in Table 3-22.

Table 3-22. Broad-Crested Weir Coefficient (C) Values

(Source: Brater and King, 1976)

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

1 - Measured at least 2.5H upstream of the weir.

3.3.2.7 V-Notch Weirs

Equation 3-45 presents the discharge equation for a V-notch weir (Brater and King, 1976). Figure 3-23 presents an example V-notch weir.

Equation 3-45

$$Q = 2.5 \tan\left(\frac{\theta}{2}\right) H^{2.5}$$

where:

Q = discharge (cfs)
θ = angle of V-notch (degrees)
H = head on apex of notch (ft)

3.3.2.8 Proportional Weirs

Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary non-linearly with head. A typical proportional weir is shown in Figure 3-24.

Figure 3-23. V-Notch Weir

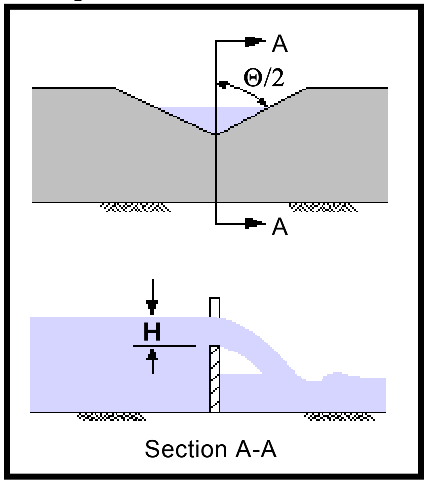
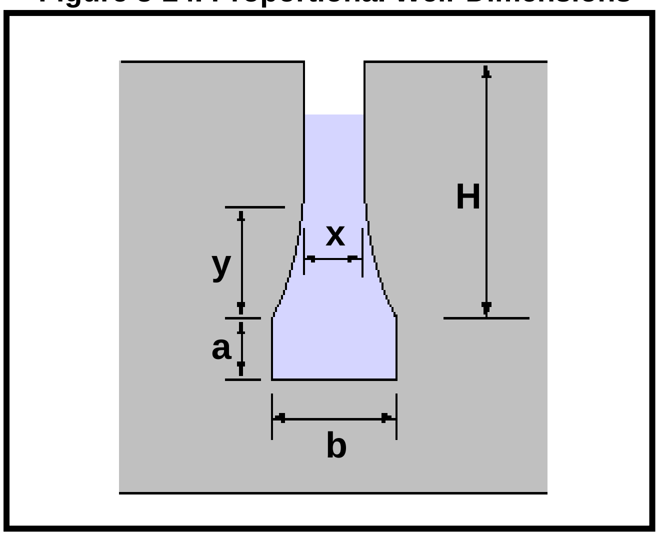


Figure 3-24. Proportional Weir Dimensions



Equations 3-46 and 3-47 present the design equations for proportional weirs (Sandvik, 1985).

Equation 3-46
$$Q = 4.97a^{0.5}b\left(H - \frac{a}{3}\right)$$

Equation 3-47
$$\frac{x}{b} = 1 - \left(\frac{1}{3.17}\right)\left[\arctan\left(\frac{y}{a}\right)^{0.5}\right]$$

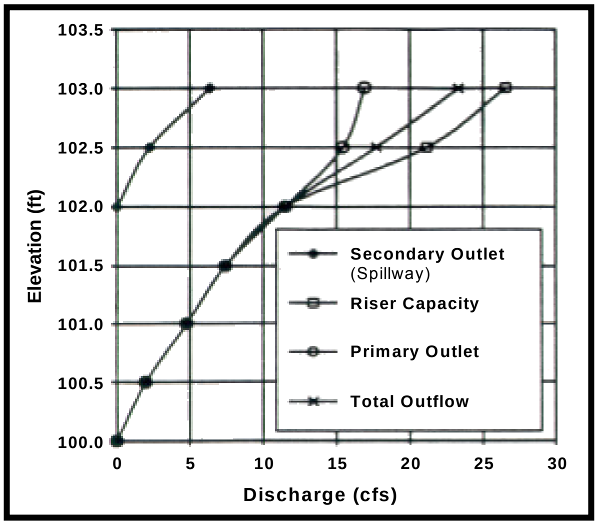
where:

Q = discharge (cfs)
Dimensions a, b, H, x, and y are shown in Figure 3-24

3.3.2.9 Combination Outlets

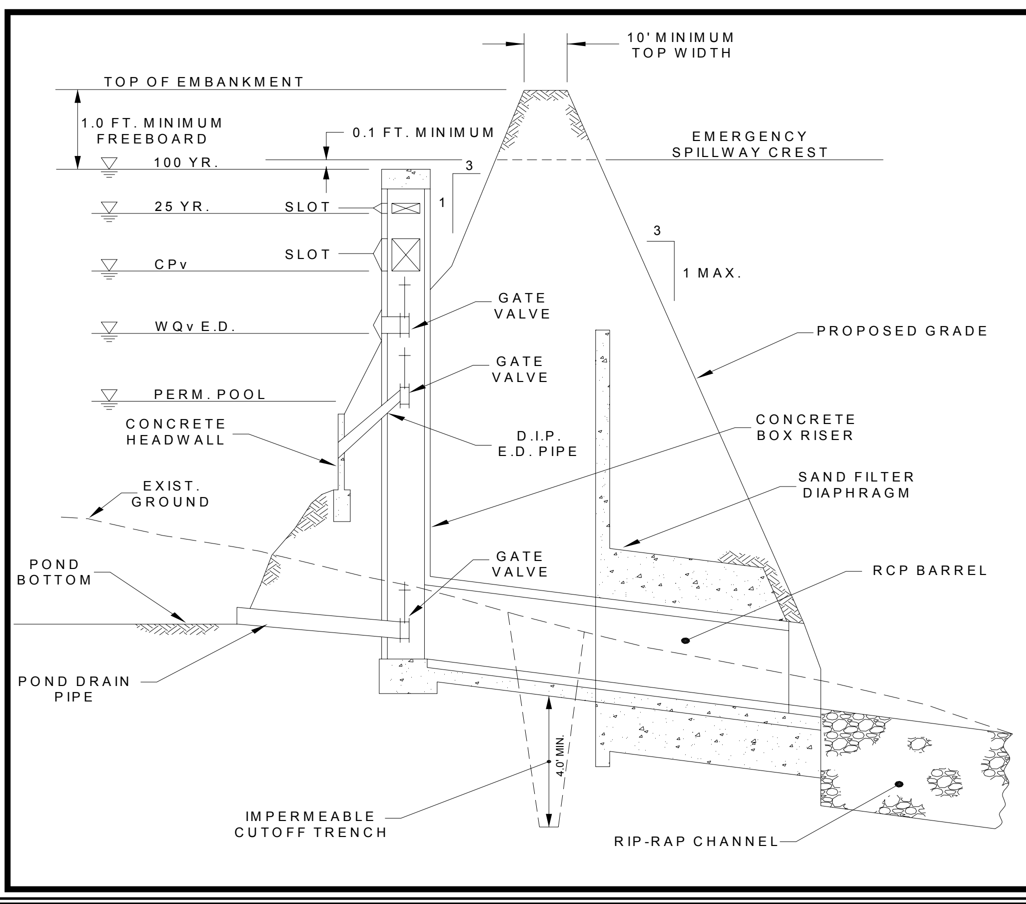
Combinations of orifices, weirs and pipes are typically used to provide multi-stage outlet control for different control volumes/discharges within a storage facility (i.e., WQv, CPv, Qp_2, Qp_{10}, Qp_{25} and Qp_{100}). The use of a combination outlet requires the construction of a composite stage-discharge curve suitable for control of multiple storm flows. An example composite curve is presented in Figure 3-25. The design of multi-stage combination outlets is discussed in detail in Section 3.3.4.

Figure 3-25. Composite Stage-Discharge Curve



There are generally of two types of combination outlets: *shared outlet control structures and separate outlet controls*. *Shared outlet control* is typically a number of individual outlet openings (orifices), weirs or drops at different elevations on a riser pipe or box which all flow to a common larger conduit or pipe. Figure 3-26 shows an example of a riser designed for a wet pond. The orifice plate outlet structure in Figure 3-20 is another example of a combination outlet.

Figure 3-26. Schematic of Shared Outlet Control Structure



Separate outlet controls are less common and may consist of several pipe or culvert outlets at different levels in the storage facility that are either discharged separately or are combined to discharge at a single location.

3.3.3 The Design of Extended Detention Outlets

3.3.3.1 Outlet Sizing

Extended detention (ED) orifice sizing is required in design applications that provide extended detention for the ED portion of the water quality volume (WQv). In both cases an extended detention orifice or reverse slope pipe can be used for the outlet. For a structural control facility providing both WQv extended detention and CPv control (wet ED pond, micropool ED pond, and shallow ED wetland), there will be a need to design two outlet orifices. The water quality control outlet will be sized using drawdown time principles and the channel protection outlet will be sized through routing to achieve a minimum of 24 hours separation between the centroid of the inflow hydrograph and the outflow hydrograph for the 1-year 24-hour design storm.

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

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

1. Use the maximum hydraulic head associated with the storage volume and maximum flow, and approximate the orifice size needed to achieve the required drawdown time.
2. Use a drawdown analysis to determine the drawdown time.

These two procedures are outlined in the examples below.

Example 3-11. ED Outlet Design Method 1: Maximum Hydraulic Head

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

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

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

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

Step 2. Determine the required orifice diameter by using the orifice equation (Equation 3-37) and Q_{max} and H_{max}:

$Q = CA(2gH)^{0.5}$, or $A = Q/C(2gh)^{0.5}$
$A = 0.76/0.6[(2)(32.2)(5.0)]^{0.5} = 0.071\ ft^2$

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

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

Example 3-12. ED Outlet Design Method 2: Drawdown Analysis

Using the data from the previous example (Example 3-11) use Method 2 to calculate the size of the outlet orifice. Use of a spreadsheet is highly recommended.

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

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

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

Step 3. Assume an orifice size:
Orifice diameter = 1"
Orifice area = $(\pi/4)*(\text{Diam}/12)^2$
Orifice area = $(3.14/4)*(1/12)^2 = 0.00545$ ft^2

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

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

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

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

3.3.3.2 Outlet Protection

Small low flow orifices such as those used for extended detention applications can easily clog, preventing the structural control from meeting its design purpose(s) and potentially causing adverse impacts. Therefore, extended detention orifices need to be adequately protected from clogging. There are a number of different anti-clogging designs, including:

- The use of a *reverse slope pipe* attached to a riser for a stormwater pond or wetland with a permanent pool. This configuration is presented in Figure 3-27. The inlet is submerged 1 foot below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond.
- The use of a *hooded outlet* for a stormwater pond or wetland with a permanent pool. This configuration is shown in Figures 3-28 and 3-29.

- Internal orifice protection through the use of an *over-perforated vertical stand pipe with ½-inch orifices or slots* that are protected by wirecloth and a stone filtering jacket. This configuration is shown in Figure 3-30.

- Internal orifice protection through the use of adjustable gate valves can achieve an equivalent orifice diameter. This configuration is not shown in a figure.

Figure 3-27. Reverse Slope Pipe Outlet

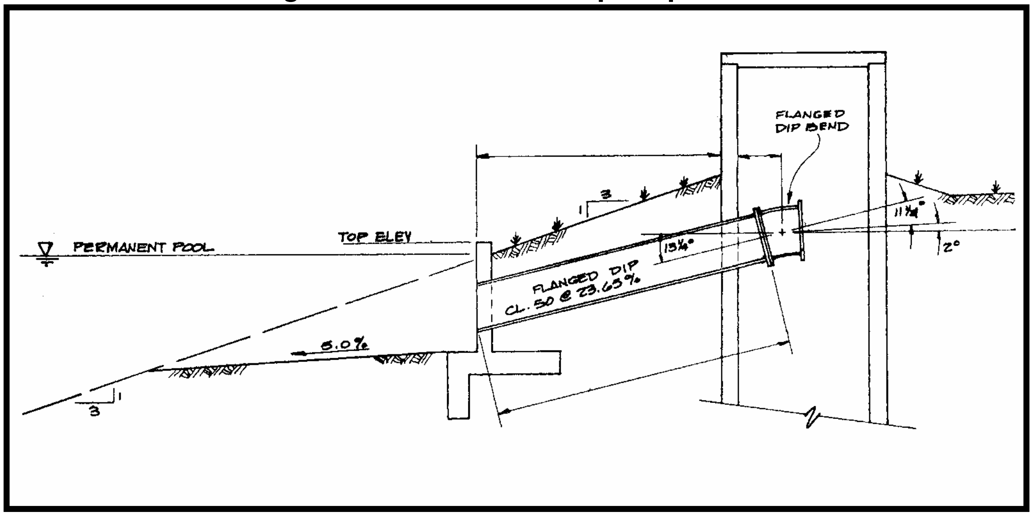


Figure 3-28. Hooded Outlet

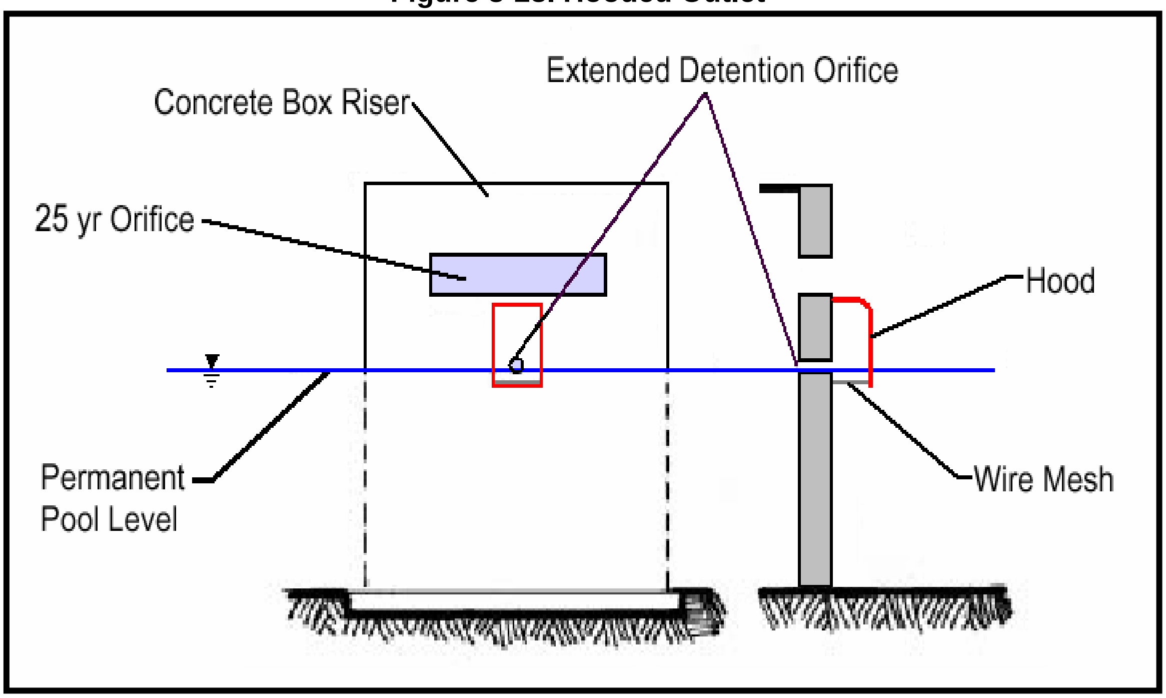


Figure 3-29. Half-Round CMP Orifice Hood

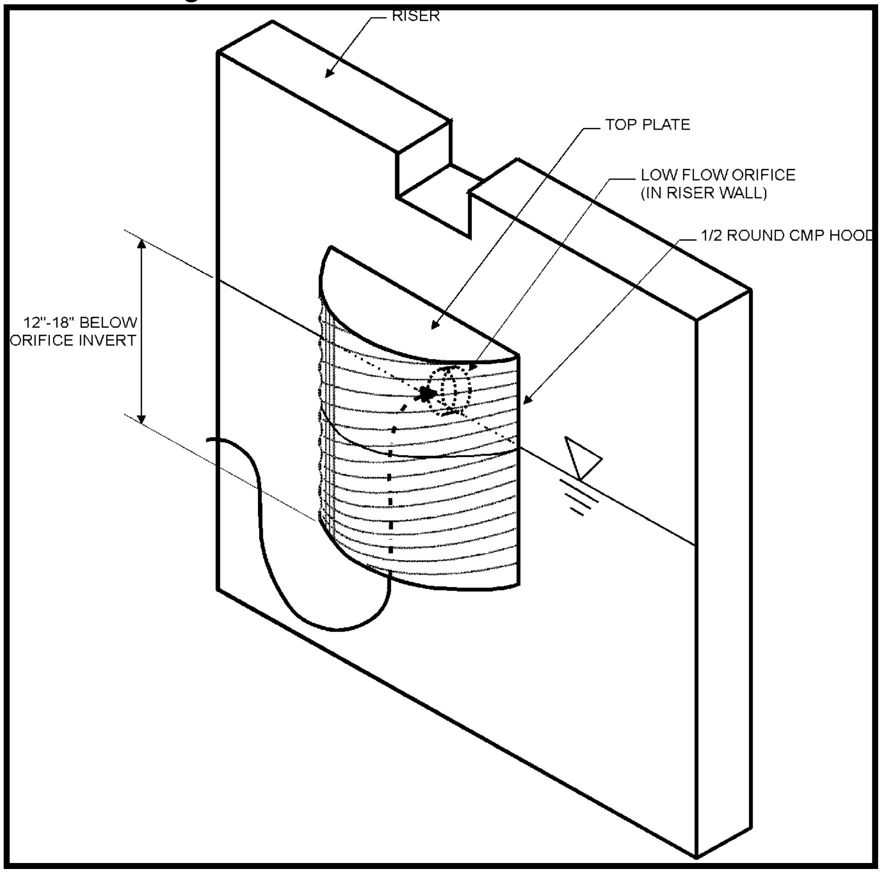
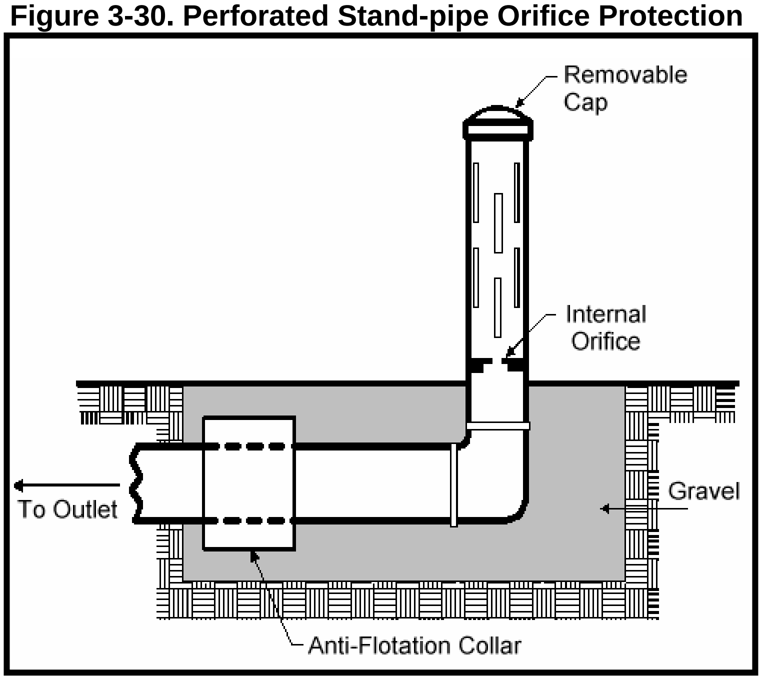


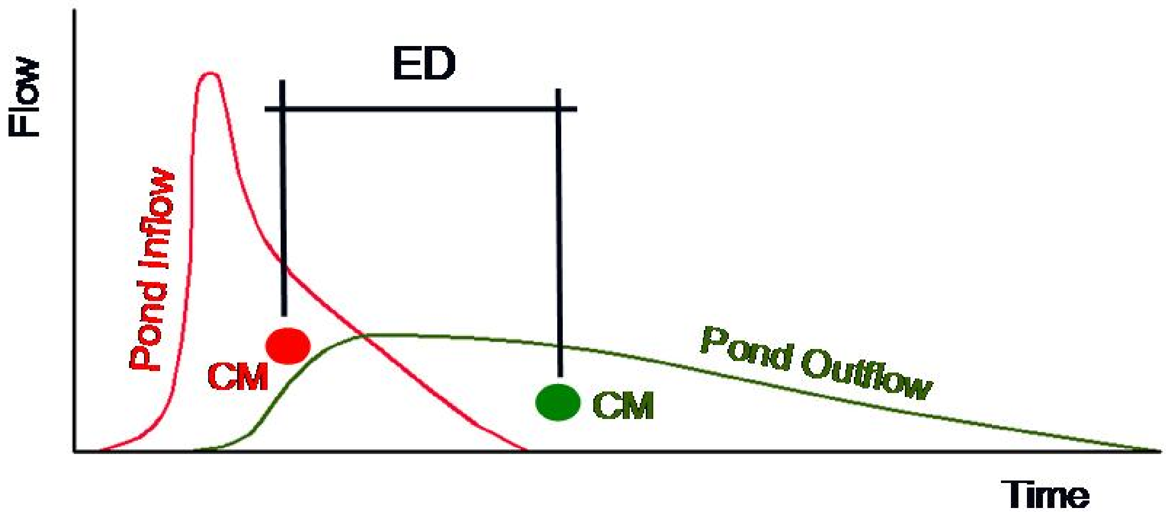
Figure 3-30. Perforated Stand-pipe Orifice Protection



3.3.4 The Design of Channel Protection Outlets

The purpose of channel protection outlets is to prevent the erosive channel-forming flows that occur during the 1 to 2 year storm. This purpose is accomplished by extending the detention of the 1-year 24-hour design storm to 24 hours. The detention time is measured from the centroid of the inflow hydrograph to the centroid of the outflow hydrograph as shown below.

Figure 3-31. Channel Protection Hydrograph



3.3.4.1 Outlet Sizing

Channel protection outlets, then, must be sized using hydrograph routing techniques. The channel protection volume estimated in section 3.2.5 will have a channel protection outlet placed at the bottom of it. The size of the outlet can only be estimated initially. Routing the 1-year 24-hour inflow hydrograph through the pond will provide an outflow hydrograph. If the detention time is less than 24 hours, the channel protection orifice must be made smaller. The water quality orifice may preclude reaching the 24 hour detention time, in which case, the water quality orifice must be made smaller. The water quality and channel protection orifices can be combined so long as both water quality and channel protection criteria are met.

3.3.5 Multi-Stage Outlet Design

A combination outlet such as a multiple orifice plate system or multi-stage riser is often used to provide adequate hydraulic outlet controls for the different design requirements (e.g., WQv, CPv, Qp_2, Qp_{10}, Qp_{25} and Qp_{100}) for stormwater ponds, stormwater wetlands and detention-only facilities. Separate openings or devices at different elevations are used to control the rate of discharge from a facility during multiple design storms. Figure 3-26 (shown previously) is an example of multi-stage combination outlet systems.

A design engineer must be creative to provide the most economical and hydraulically efficient outlet design possible in designing a multi-stage outlet. Many iterative routings are usually required to arrive at a minimum structure size and storage volume that provides proper control. The stage-discharge table or rating curve is a composite (previously shown in Figure 3-25) of the different outlets that are used for different elevations within the multi-stage riser.

3.3.5.1 Multi-Stage Outlet Design Procedure

Below are the steps for designing a multi-stage outlet. Note that if a structural control facility will not control one or more of the required storage volumes (WQv, CPv, Qp_2, Qp_{10}, Qp_{25}, and Qp_{100}), then that step in the procedure is skipped.

1. Determine Stormwater Control Volumes. Using the procedures from Sections 3.1 and 3.2, estimate the required storage volumes for water quality treatment (WQv), channel protection (CPv), and overbank flood control (Qp_2, Qp_{10}, Qp_{25}) and extreme flood control (Qp_{100}).

2. Develop Stage-Storage Curve. Using the site geometry and topography, develop the stage-storage curve for the facility in order to provide sufficient storage for the control volumes involved in the design.

3. Design the Water Quality Volume Outlet. Design the water quality orifice. If a permanent pool is incorporated into the design of the facility, a portion of the storage volume for water quality may be above the elevation of the permanent pool. The outlet can be protected using either a reverse slope pipe, a hooded protection device, or another acceptable method. Design the water quality volume outlet using either Method 1 or 2 from subsection 3.3.3.

4. Design the Channel Protection Volume Outlet. For this design, the storage needed for channel protection may be "stacked" on top of the water quality volume storage elevation determined in Step 3. The total stage-discharge rating curve at this point will include the water quality control orifice and the outlet used for stream channel protection. Use hydrograph routing software to ensure that the 1-year 24-hour storm has been detained for a minimum of 24 hours, centroid to centroid. The water quality outlet may be too large for the channel protection criteria to be met, in which case the water quality outlet must be made smaller and the routing re-run. The outlet should be protected in a manner similar to that for the water quality orifice.

5. Design Overbank Flood Protection Outlet. The overbank protection volume is added above the water quality and channel protection storage. Establish the Qp_2, Qp_{10}, Qp_{25} maximum water surface elevations using the stage-storage curve to find the 2-year, 10-year and 25-year maximum head. Select an outlet type and calculate the initial size and geometry based upon maintaining the pre-development 2-year, 10-year and 25-year peak discharge rates. Develop a stage-discharge curve for the combined set of outlets (WQv, CPv and Qp_2, Qp_{10}, Qp_{25}). This procedure is repeated for control (peak flow attenuation) of the 100-year storm (Qp_{100}).

6. Check Performance of the Outlet Structure. Perform a hydraulic analysis of the multi-stage outlet structure using reservoir routing to ensure that all outlets will function as designed. Ensure that post-developed peak flows are less than pre-developed peak flows for all design storms at the pond outlet, and downstream per the 10% rule. Also, check that the CPv criteria are still being met. Several iterations may be required to calibrate and optimize the pond shape and outlets that are used. Also, the structure should operate without excessive surging, noise, vibration, or vortex action at any stage. This usually requires that the structure have a larger cross-sectional area than the outlet conduit. The hydraulic analysis of the design must take into account the hydraulic changes that will occur as depth of storage changes for the different design storms. As shown in Figure 3-32, as the water passes over the rim of a riser, the riser acts as a weir. However, when the water surface reaches a certain height over the rim of a riser, the riser will begin to act as a submerged orifice. The designer must compute the elevation at which this transition from riser weir flow control to riser orifice flow control takes place for an outlet where this change in hydraulic conditions will occur. Also note in Figure 3-32 that as the elevation of the water increases further, the control can change from barrel inlet flow control to barrel pipe flow control. Figure 3-33 shows another condition where weir flow can change to orifice flow, which must be taken into account in the hydraulics of the rating curve as different design conditions result in changing water surface elevations.

Figure 3-32. Riser Flow Diagrams
(Source: Virginia Department of Conservation and Recreation, 1999)

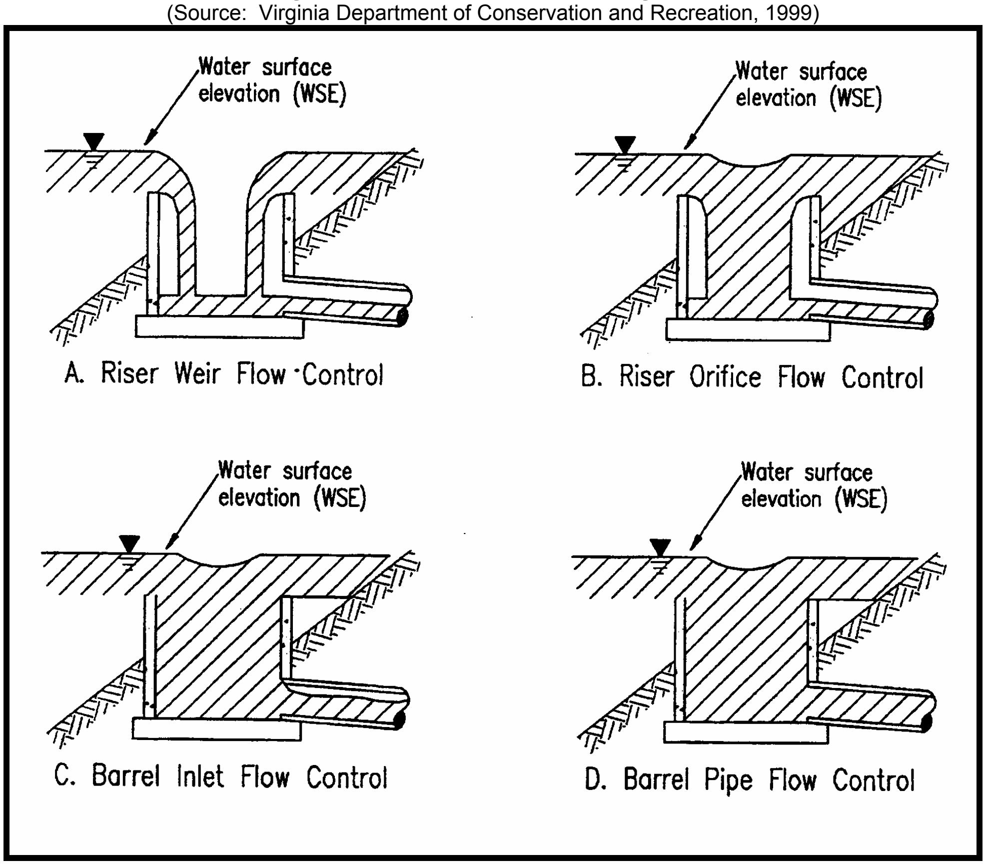
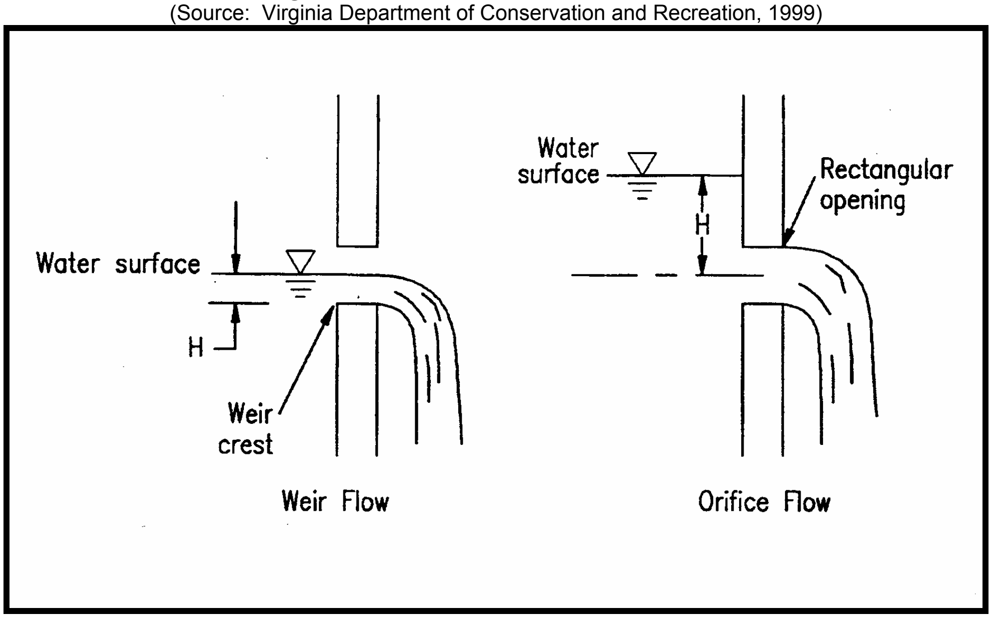


Figure 3-33. Weir and Orifice Flow
(Source: Virginia Department of Conservation and Recreation, 1999)



7. Size the Emergency Spillway. It is recommended that all stormwater impoundment structures have an emergency spillway. An emergency spillway provides a degree of safety to prevent overtopping of an embankment if the primary outlet or principal spillway should become clogged, or otherwise inoperative. The 100-year storm should be routed through the outlet devices and emergency spillway to ensure the hydraulics of the system will operate as designed.
8. Design Outlet Protection. Design necessary outlet protection and energy dissipation facilities to avoid erosion problems downstream from outlet devices and emergency spillway(s). Refer to Chapter 7 for more information.
9. Perform Buoyancy Calculations. Perform buoyancy calculations for the outlet structure and footing. Flotation will occur when the weight of the structure is less than or equal to the buoyant force exerted by the water.
10. Provide Seepage Control. Seepage control should be provided for the outflow pipe or culvert through an embankment. The two most common devices for controlling seepage are (1) filter and drainage diaphragms and (2) anti-seep collars.

3.3.6 Trash Racks and Safety Grates

The susceptibility of larger inlets to clogging by debris and trash needs to be considered when estimating their hydraulic capacities. In most instances, trash racks will be needed. Trash racks and safety grates are a critical element of outlet structure design and serve several important functions. When properly designed, installed, and maintained, trash racks:

- keep debris away from the entrance to the outlet works where the debris will not clog the critical portions of the structure;
- capture debris in such a way that relatively easy removal is possible;
- ensure that people and large animals are kept out of confined conveyance and outlet areas; and,
- provide a safety system that prevents humans and animals from being drawn into the outlet and allows them to climb to safety.

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

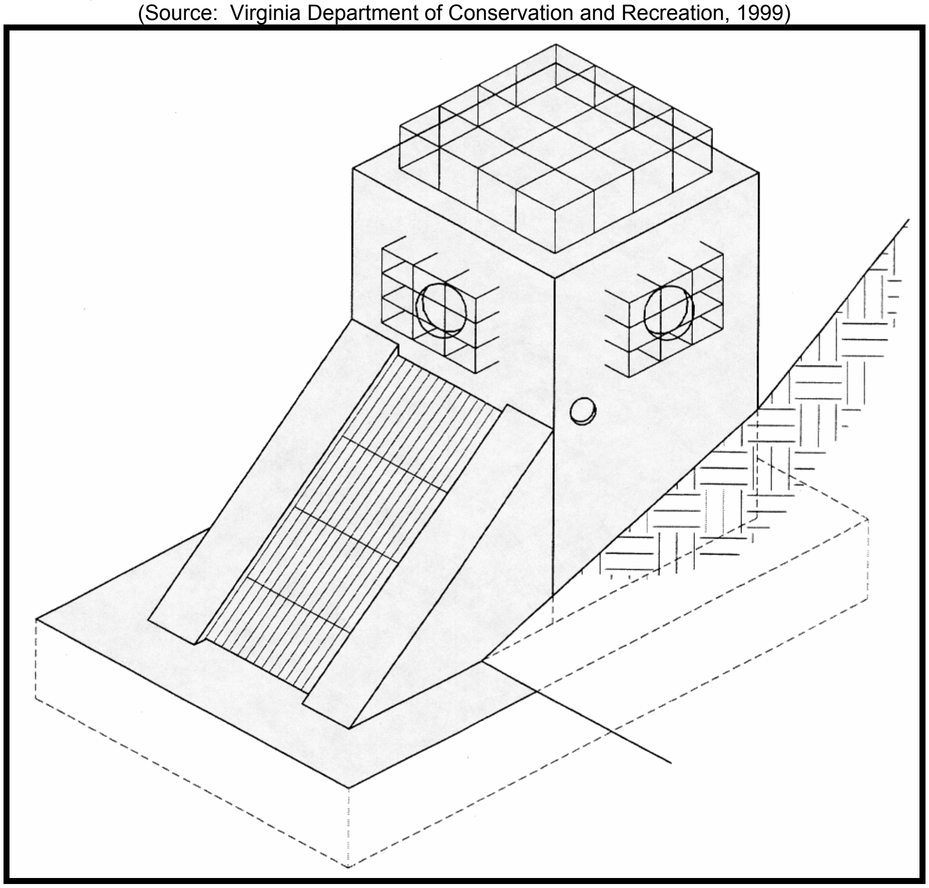
3.3.6.1 Trash Rack Design

An example of trash racks used on a riser outlet structure is shown in Figure 3-34. The inclined bar rack is most effective for lower stage outlets. Debris will ride up the trash rack as water levels rise. This design also allows for removal of accumulated debris with a rake while standing on top of the structure.

Trash racks must be large enough such that partial plugging will not adversely restrict flows reaching the control outlet. There are no universal guidelines for the design of trash racks to protect detention basin outlets, although a commonly used "rule-of-thumb" is to have the trash rack area at least ten times larger than the control outlet orifice.

The surface area of all trash racks should be maximized and the trash racks should be located a suitable distance from the protected outlet to avoid interference with the hydraulic capacity of the outlet. The spacing of trash rack bars must be proportioned to the size of the smallest outlet protected. However, where a small orifice is provided, a separate trash rack for that outlet should be used, so that a simpler, sturdier trash rack with more widely spaced members can be used for the other outlets. Spacing of the rack bars should be wide enough to avoid interference, but close enough to provide the level of clogging protection required.

Figure 3-34. Trash Racks Used on a Riser Outlet Structure
(Source: Virginia Department of Conservation and Recreation, 1999)



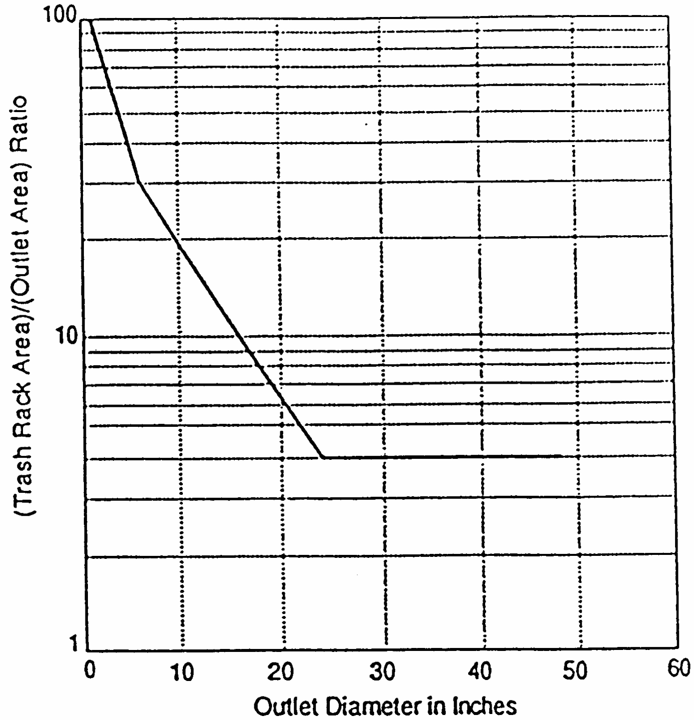
To facilitate removal of accumulated debris and sediment from around the outlet structure, the racks should have hinged connections. If the rack is bolted or set in concrete it will preclude removal of accumulated material and will eventually adversely affect the outlet hydraulics.

Since sediment will tend to accumulate around the lowest stage outlet, the inside of the outlet structure for a dry basin should be depressed below the ground level to minimize clogging due to sedimentation. Depressing the outlet bottom to a depth below the ground surface at least equal to the diameter of the outlet is recommended.

Trash racks at entrances to pipes and conduits should be sloped at about 3H:1V to 5H:1V to allow trash to slide up the rack with flow pressure and rising water level—the slower the approach of flow, the flatter the trash rack angle. Rack opening rules-of-thumb are found in literature. Figure 3-35 gives opening estimates based on outlet diameter (Urban Drainage and Flood Control District, 1999). Judgment should be used in that an area with higher debris (e.g., a wooded area) may require more opening space.

Figure 3-35. Minimum Rack Size vs. Outlet Diameter

(Source: Urban Drainage and Flood Control District, 1999)



The bar opening space for small pipes should be less than the pipe diameter. For larger diameter pipes, openings should be 6 inches or less. Collapsible racks have been used in some places if clogging becomes excessive or a person becomes pinned to the rack. Alternately, debris for culvert openings can be caught upstream from the opening by using pipes placed in the ground or a chain safety net (United States Bureau of Reclamation, 1978; Urban Drainage and Flood Control District, 1999). Racks can be hinged on top to allow for easy opening and cleaning.

The control for the outlet should not shift to the grate, nor should the grate cause the headwater to rise above planned levels. Therefore, either head losses through the grate should be calculated or Figure 3-35 used to ensure adequate hydraulic capacity through the trash rack. A number of empirical loss equations exist though many have difficult to estimate variables. Two will be given to allow for comparison.

Equation 3-48 can be used to determine the head loss through a trash rack (Metcalf and Eddy, 1972). In the use of Equation 3-48, grate openings should be calculated assuming a certain percentage blockage as a worst case to determine losses and upstream head. Often 40 to 50% blockage is chosen as a working assumption.

Equation 3-48

$$H_g = K_{g1}\left(\frac{w}{x}\right)^{4/3}\left(\frac{V_u^{\,2}}{2g}\right)\sin\theta_g$$

where:

H_g = head loss through grate (ft)

K_{g1} = bar shape factor:

2.42 – sharp-edged rectangular

1.83 - rectangular bars with semicircular upstream faces

1.79 - circular bars
1.67 - rectangular bars with semicircular up- and downstream faces
w = maximum cross-sectional bar width facing the flow (in)
x = minimum clear spacing between bars (in)
V_u = approach velocity (ft/s)
θ_g = angle of the grate with respect to the horizontal (degrees)
g = acceleration of gravity (32.2 ft/s^2)

The United States Army Corps of Engineers has developed a similar head loss equation for trash racks, shown in Equation 3-49 (USACE, 1988). This equation is for vertical racks, but presumably can be adjusted through multiplication by the sine of the angle of the grate with respect to the horizontal, in a manner similar to the previous equation.

Equation 3-49

$$H_g = \frac{K_{g2} V_u^{\,2}}{2g}$$

where:

H_g = head loss through grate (ft)
K_{g2} = defined as described below
V_u = approach velocity (ft/s)
g = acceleration of gravity (32.2 ft/s^2)

K_{g2} is defined from a series of fit curves, as shown in Table 3-23.

Table 3-23. Fit Curves to Determine K_{g2}

Grate Type	Length/thickness	Curve
Sharp-edged rectangular	10	K_{g2} = 0.00158 - 0.03217 A_r + 7.1786 A_r^2
Sharp-edged rectangular	5	K_{g2} = -0.00731 + 0.69453 A_r + 7.0856 A_r^2
Round edged rectangular	10.9	K_{g2} = -0.00101 + 0.02520 A_r + 6.0000 A_r^2
Circular cross-section	not applicable	K_{g2} = 0.00866 + 0.13589 A_r + 6.0357 A_r^2

3.3.7 Secondary Outlets

The purpose of a secondary outlet (emergency spillway) is to provide a controlled overflow for flows in excess of the maximum design storm for a storage facility. Figure 3-36 shows an example of an emergency spillway.

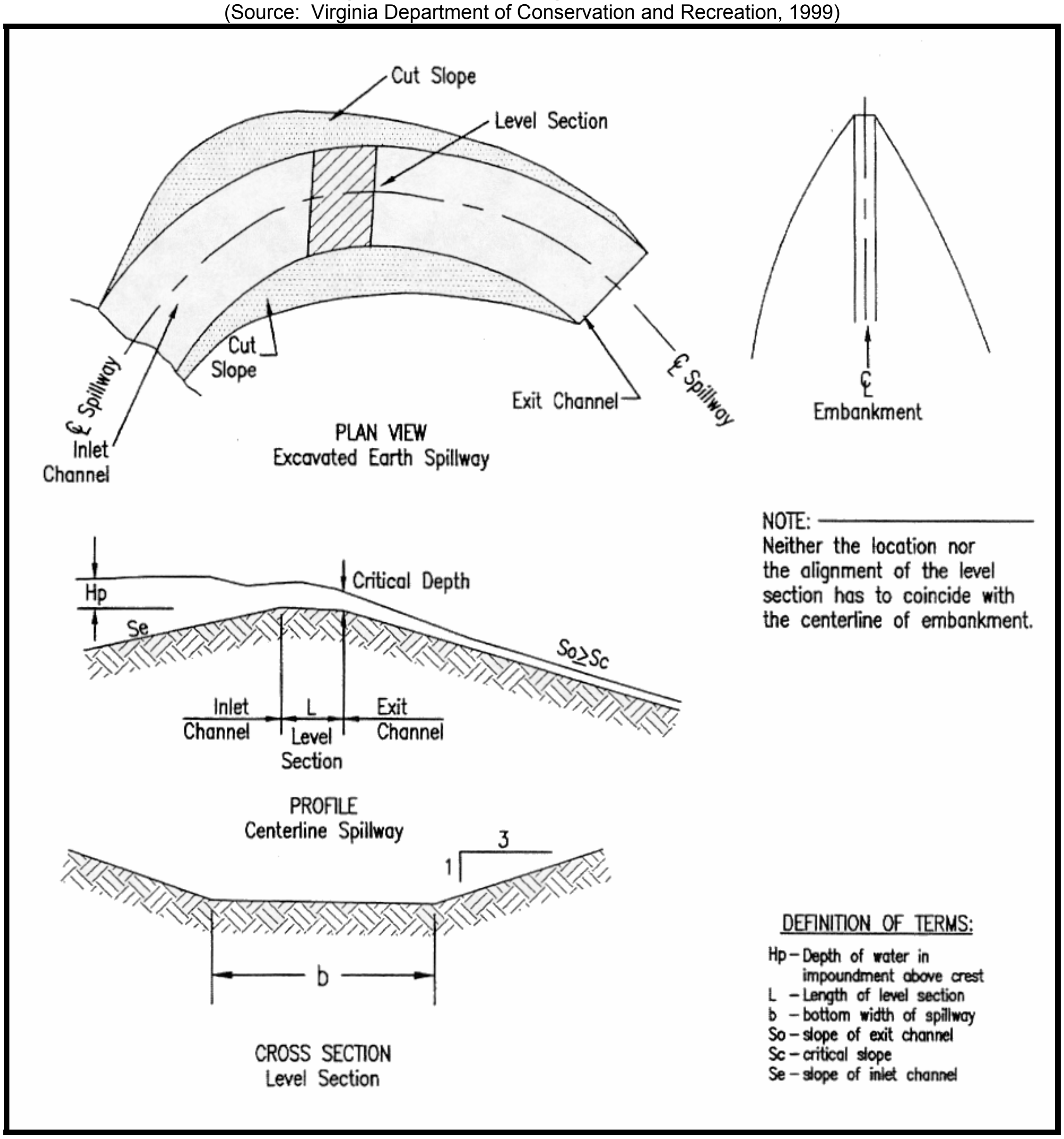
In many cases, on-site stormwater storage facilities do not warrant elaborate studies to determine spillway capacity. While the risk of damage due to failure is a real one, it normally does not approach the catastrophic risk involved in the overtopping or breaching of a major reservoir. By contrast, regional facilities with homes immediately downstream could pose a significant hazard if failure were to occur, in which case emergency spillway considerations are a major design factor.

3.3.7.1 Emergency Spillway Design

Emergency spillway designs are open channels, usually trapezoidal in cross section, and consist of an inlet channel, a control section, and an exit channel (see Figure 3-36). The emergency spillway is proportioned to pass flows in excess of the 100-year flood without allowing excessive velocities and without overtopping of the embankment. Flow in the emergency spillway is open channel flow. Normally, it is assumed that critical depth occurs at the control section. Volume 2, Chapter 7 provides more information on open channel hydraulics.

SCS manuals provide guidance for the selection of emergency spillway characteristics for different soil conditions and different types of vegetation. The selection of degree of retardance for a given spillway depends on the vegetation. Knowing the retardance factor and the estimated discharge rate, the emergency spillway bottom width can be determined. For erosion protection during the first year, assume minimum retardance. Both the inlet and exit channels should have a straight alignment and grade. Spillway side slopes should be no steeper the 3:1 (horizontal to vertical). The most common type of emergency spillway used is a broad-crested overflow weir cut through original ground next to the embankment. The transverse cross section of the weir cut is typically trapezoidal in shape for ease of construction. Such an excavated emergency spillway is illustrated below.

Figure 3-36. Emergency Spillway
(Source: Virginia Department of Conservation and Recreation, 1999)



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