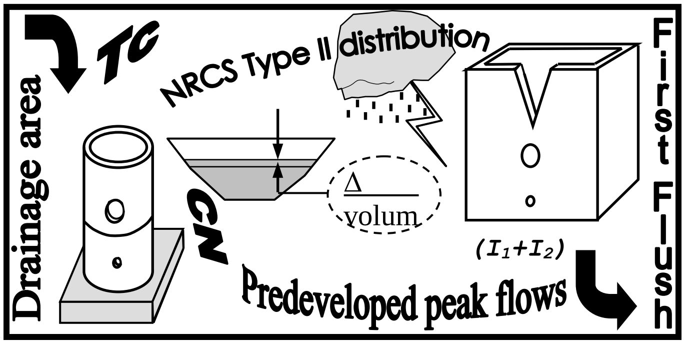
ACTIVITY: Detention Computations

ST – 10



CITY OF KNOXVILLE

Description

This stormwater treatment BMP will describe the purpose, basic formulas, minimum criteria, computational methods, and types of structures that are needed for stormwater detention calculations. Stormwater detention is a necessary component of most stormwater treatment BMPs, and is required for most types of site development and redevelopment by the Knoxville Stormwater and Street Ordinance.

Approach

The Stormwater and Street Ordinance of the City of Knoxville is posted at the City Engineering website and must be read carefully by anyone who attempts to perform stormwater calculations within the City of Knoxville. It contains provisions for fees, bonds, necessary permits, right-of-entry, definitions, easements and penalties that will not be addressed here. A major purpose of the Stormwater and Street Ordinance is to improve water quality (erosion control, illicit storm drain connections, illegal dumping) as well as control stormwater quantity and flooding.

Major Points of Stormwater Detention

The following list summarizes the major points from the Stormwater and Street Ordinance that relate particularly to stormwater detention:

- Stormwater detention is required for any property containing one or more of the following conditions (according to Section 22.5-23):
    1. Road construction containing one-half acre or more of impervious surface.
    2. Commercial, industrial, educational, institutional or recreational developments containing one acre or more of disturbed area.
    3. Single-family or duplex residential development containing at least five acres of disturbed area or at least five lots.
    4. Any development containing one-half acre or more of additional impervious area.
    5. Any redevelopment that causes the improvement of 50% of the assessed value of the lot, building, or lot use.
- Stormwater detention is defined as limiting the peak discharge rate for the postdeveloped conditions to be no greater than the peak discharge rate for the predeveloped conditions. This must be accomplished for the 1-year, 2-year, 5-year, 10-year, 25-year and 100-year storms (with NRCS Type II 24-hour rainfall distribution) with AMC II curve numbers. (Sections 22.5-31 and 22.5-33)
- Stormwater detention requirements may be waived if a proposed development discharges directly to the Tennessee River, Holston River, or French Broad River by a small unnamed channel not adversely impacting downstream property owners. First flush volume treatment (or alternative) is still required.
- Stormwater detention requirements may be waived if hydrologic and

hydraulic computations, stamped and certified by a professional engineer, prove that detention is unwarranted in the judgment of the Engineering Director. First flush volume treatment (or alternative equal) is still required.

- The first flush volume is defined as the first ½-inch of direct runoff from the contributing drainage basin (with a minimum value of 4500 cubic feet). The first flush volume must be captured and then released over a minimum time of 24 hours and a maximum time of 72 hours in such a way as to maintain acceptable stormwater quality. (Sections 22.5-4 and 22.5-36)
- The stormwater detention facility must provide an additional measure of storage for the 100-year storm to allow for long-term loss of volume due to potential lack of maintenance. Additional storage volume must be provided by increasing the area for each computed contour by 15%. (Section 22.5-31)
- The top of berm for the stormwater detention facility must be higher than the computed 100-year water surface elevation by an amount equal to 1 foot plus the difference between the computed top of riser elevation and the adjusted top of riser elevation. (Section 22.5-31)
- A dry detention basin must have a minimum 2% slope in the bottom of the basin in order to drain properly without ponding water unnecessarily. The side slopes should generally be 3:1 (H:V) or flatter, unless a traversable access has been designed. (Section 22.5-31)
- All hydrologic and hydraulic computations for stormwater detention facilities must be prepared and stamped by a registered engineer (licensed in the state of Tennessee) and proficient in this field by education and experience. Plans must show sufficient information to enable the builder to construct the facility as required. (Sections 22.5-31 and 22.5-33)
- Discharge from a stormwater detention basin shall be analyzed downstream to either the second existing street crossing or a blue-line stream. Calculations shall show adequate capacity for 2-year and 10-year design storms, including demonstration of non-erosive channel velocities. (Section 22.5-31)

NRCS Methods – A Quick Review

In Section 22.5-33, hydrologic and hydraulic computations are required to be in accordance with National Resources Conservation Service (NRCS) methods. The NRCS Unit Hydrograph shall be used with average antecedent moisture conditions (AMC II) and Type II rainfall distribution, specified by Technical Release 55 (TR-55) publication from June 1986. The NRCS, part of the United States Department of Agriculture, was formerly known as the Soil Conservation Service (SCS). The TR-55 publication (Urban Hydrology for Small Watersheds, reference 175) is downloaded at:

http://www.info.usda.gov/CED/ftp/CED/tr55.pdf

Anyone who performs stormwater detention computations must be thoroughly familiar with the TR-55 publication, its uses, and its limitations. TR-55 is intended for urban watersheds which are relatively homogeneous. Each subwatershed is represented by a single curve number (CN) and time of concentration (Tc), used to generate a runoff hydrograph. Investigate project soils in the field and/or consult the Knox County Soils Survey. TR-55 Appendix A lists many soils (cross-referenced to the hydrologic soil group), or Table ST-10-1 has typical values for USDA soils textures. Typical CN values (varying from 30 to 98) are in Table ST-10-2. A subwatershed must have a principal path for determining Tc, or multiple paths with the same Tc. Tc values may range from 0.1 to 10 hours. Use a NRCS unit hydrograph shape factor of 484.

NRCS Computational Software

The NRCS website also contains the latest version of the TR-55 computer program, which can be used as a preliminary estimate of detention volumes. However, NRCS design storms must be hydraulically routed through a detention basin to verify that the detention basin design works. There are many commercially available software programs that use NRCS methods to generate hydrographs and then route them through a detention basin. Three possibilities are discussed below:

- The Engineering Department currently uses Haestad PondPack™ for the regulatory review of proposed detention basins and also for checking as-built conditions. PondPack™ allows many types of outlet structures with almost any combination and configuration of weirs, orifices or culverts. It is recognized that this program is relatively expensive, and would represent a sizable investment in software and training for any potential user.
- *(see BMP ST-11)* Complex spreadsheets are capable of handling the computations for designing a detention basin and then routing the specified design storms. The NRCS design storms and rainfall distributions are always the same, which simplifies part of the computation. Storage volumes and outlet discharges must be reprogrammed for each detention basin that is analyzed. This method generally requires very advanced hydrologic expertise.
- *(see BMP ST-12)* HEC-1 and HEC-HMS are freely available software programs that can be downloaded from the U.S. Army Corps of Engineers website. These programs can generate runoff hydrographs using several methods and then perform storage routing. The program user must generate stage-storage-discharge curves by manually computing and analyzing each type of outlet device http://www.hec.usace.army.mil/

  http://www.hec.usace.army.mil/software/legacysoftware/hec1/hec1.htm

Software computations submitted for review must include all necessary input data to reproduce the detention design, including details as needed to illustrate the outlet structure. Computations should be organized and neatly printed on standard 8.5” x 11” paper so that the results are easily referenced and located. The Engineering Department may require verification of software programs that are unproved or not well-known in the Knoxville area.

Example BMPs To Demonstrate NRCS Methods

Two BMPs (ST-11 and ST-12) contain the same worked example of a typical small development site of 1.5 acres. Both examples require the designer to make an initial volume estimate for preliminary design layout, prior to performing the actual routing computations. However, these detention volume estimates need to be adjusted upward to account for required first flush volume and also the 15% storage volume adjustment.

Particular attention should be given to Worksheet # 2 (page ST-11-11), an easily reproducible Excel spreadsheet. Worksheet #2 condenses the computations for peak discharge and initial detention storage volume estimate using the formulas directly from the NRCS TR-55 publication (only using a total of 6 inputs for predeveloped and postdeveloped areas, curve numbers and times of concentration).

Both BMPs (ST-11 and ST-12) require the stormwater designer to accurately compute contoured areas, storage volumes, flow rating curves and other necessary input data in order to perform detention routing computations.

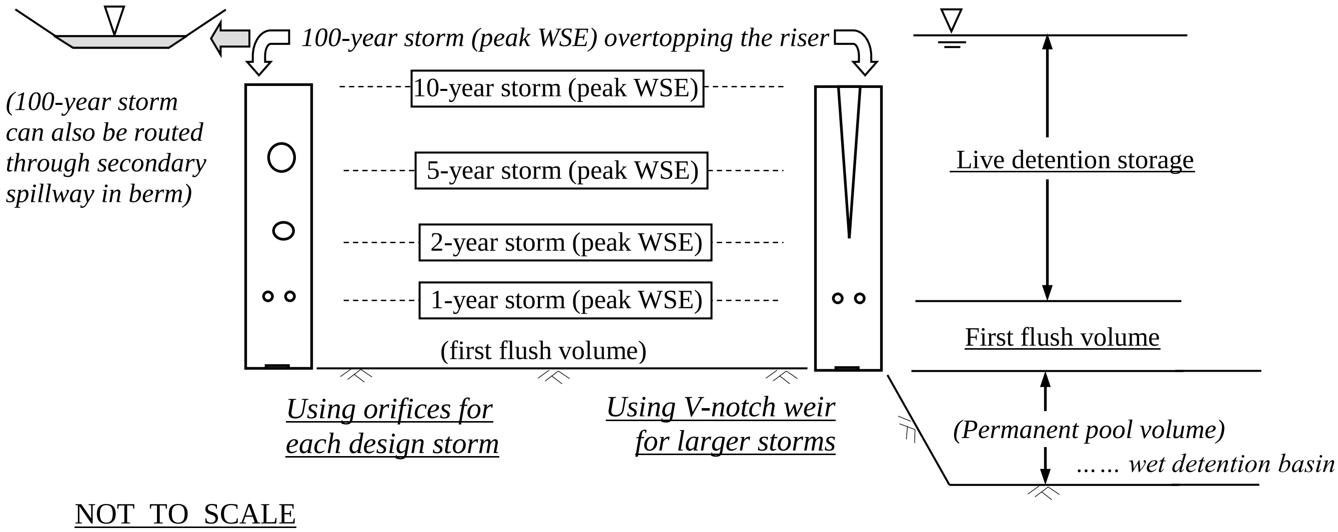


Figure ST-10-1
Working Profile of Detention Basins

First Flush Volume

The working volume of a dry detention basin or other detention structure can be separated into two elements (as shown in Figure ST-10-1):

- The live detention volume (which is the upper portion of a basin and represents the necessary detention capability)
- The first flush volume (which is the lower portion of a basin and represents the required stormwater quality treatment capability)

Wet systems (such as wetlands or wet detention basins) will also have a permanent pool volume, which is involved in supporting vegetation and animal life during non-rainfall periods. The dead pool will often be mixed with the other volumes during a rainfall event. However, the dead pool will not be drained by a controlled structure during the recovery time of a stormwater treatment BMP.

The first flush volume is specified in the City of Knoxville Stormwater and Street Ordinance (Section 22.5-36). Detention basins shall be sized to collect the first 0.5 inches of direct stormwater runoff from the entire developed site, or the first 4500 cubic feet of stormwater runoff, whichever is greater. The first flush volume must be released at a controlled rate over a period between 24 hours and 72 hours.

Section 4.5 of the Knoxville BMP Manual states that the pollutant removal rate shall be 75% of total suspended solids (based on a 24-hour drawdown time for detention basins). Figure 4-2 of the BMP Manual shows the measured pollution removal values for dry detention basins near metropolitan Washington, D.C. It is anticipated that this type of information will become more commonplace and more useful in the near future. It should be realized that future stormwater regulations are likely to be more stringent than current regulations. This is mostly driven by national and state standards that require municipalities and county governments to perform additional stormwater pollution reduction, with renewed efforts at water quality monitoring and enforcement.

Besides the pollutant removal rate, a second determining factor to the overall pollutant removal efficiency is how much stormwater runoff is actually being captured and then treated by various stormwater treatment BMPs. Figures ST-10-4 and ST-10-5 demonstrate how the runoff coefficient C and the volume of treatment storage can affect the percentage of average annual runoff capture. For a watershed with a runoff coefficient equal to 0.7 and using a treatment volume of 0.50 acre-inches per watershed acre, the average annual runoff capture would be 77% for a 36-hour drawdown time and 75% for a 42-hour drawdown time. The overall pollutant removal efficiency (when using values from Figure 4-2) for 3 different drawdown periods would then be:

	Capture	x	Removal	=	Overall Efficiency
24-hour drawdown	80%	x	75%	=	60%
36-hour drawdown	77%	x	83%	=	64%
42-hour drawdown	75%	x	87%	=	65%

Percentage of stormwater runoff captured (computed in Figures ST-10-4 and ST-10-5) is based upon extensive computer modeling by CDM using 44 years of hourly rainfall records for McGhee Tyson Airport and the US Army Corps of Engineers program STORM (reference 152). The key to translating the values in Table 4-2 to the values used by Figures ST-10-4 and ST-10-5 is by computing the percentage of directly impervious area (DCIA) into the weighted runoff coefficient C. For instance, an impervious area of 73% is computed to a weighted runoff coefficient of 0.70 by the following formula:

$$C = 0.15 + (0.90 - 0.15) \times DCIA$$

Overall pollutant removal efficiencies (in the range of 60% to 65%) may possibly underestimate or overestimate the actual field values. The 20% to 25% of untreated rainfall volume (that escapes capture and treatment) is generally the peak flow of a large storm. The untreated rainfall volume (at a high flow rate) may cause excessive erosion in ditches and channels, or perhaps carry away mulch and fertilizer into stormwater runoff, depending on measures taken to prevent stormwater pollution.

First Flush Volume Alternatives

Two alternative methods are shown here to illustrate how treatment volume size can be estimated for stormwater quality. In general, any development should use the stated first flush volume requirements as described in the Stormwater and Street Ordinance. Alternative method #1 is to size the treatment volume using Figure ST-10-4 (which has a 36-hour drawdown time). Using a goal of 90% average annual runoff capture and the actual runoff coefficient C, a treatment volume rate (acre-inches per watershed acre) can be selected that is likely to be more stringent than the 0.50 inches mandated by the Knoxville Stormwater and Street Ordinance. The first flush volume is then:

$$V_T = (A_T * V_{US})/12$$

V_T = Treatment volume (acre-feet), using average annual runoff capture

A_T = Total contributing drainage area (acres)

V_{US} = Unit basin storage volume (inches), from Figure ST-10-4

Alternative method #2 is to size the treatment volume based upon the maximized storm runoff capture volume and then drain over a 24-hour period. Use the following equation for volume capture ratio (with B = 1.582), which is more representative of long-term treatment rates:

$$V_{MC} = (B * C * A_T * P_M) / 12$$

V_{MC} = maximized stormwater runoff capture volume determined using either the event capture ratio or volume capture ratio as a basis (acre-feet)

B = regression constant (least-square analysis) for 85th percentile runoff:

Event capture ratio: 1.299 for 24-hour drawdown time

Volume capture ratio: 1.582 for 24-hour drawdown time

C = Weighted runoff coefficient

A_T = Total contributing drainage area (acres)

P_M = Mean storm precipitation depth (inches), use 0.53" for Knoxville

Outlet Structure – Orifice

The equation for a submerged orifice that is not subject to downstream controls:

$$Q = C_O * A * (2*g*H)^{0.5}$$

C_O = usually 0.6 (but can range as high as 0.8 in some references)
A = cross-sectional area of orifice (equal to πr^2 for circular shape)
g = 32.2 feet per second (gravitational constant)
H = Δ elevation from water surface to centroid of the orifice shape

An orifice coefficient of 0.6 is recommended in most instances. However, field tests in the Washington D.C. area (reference 50) have shown the following values can occur under ideal conditions:

C_O = 0.66 (thickness of riser is equal to or less than orifice diameter)

C_O = 0.80 (thickness of riser is greater than the orifice diameter)

Therefore, drilling an orifice into a concrete outlet structure can result in considerable impact on the coefficient, as does the beveling of the edge. The field test experimental values reported by reference 50 were conducted with sharp-edged orifices.

Outlet Structure – Weir

The equation for a rectangular weir that is not subject to downstream controls:

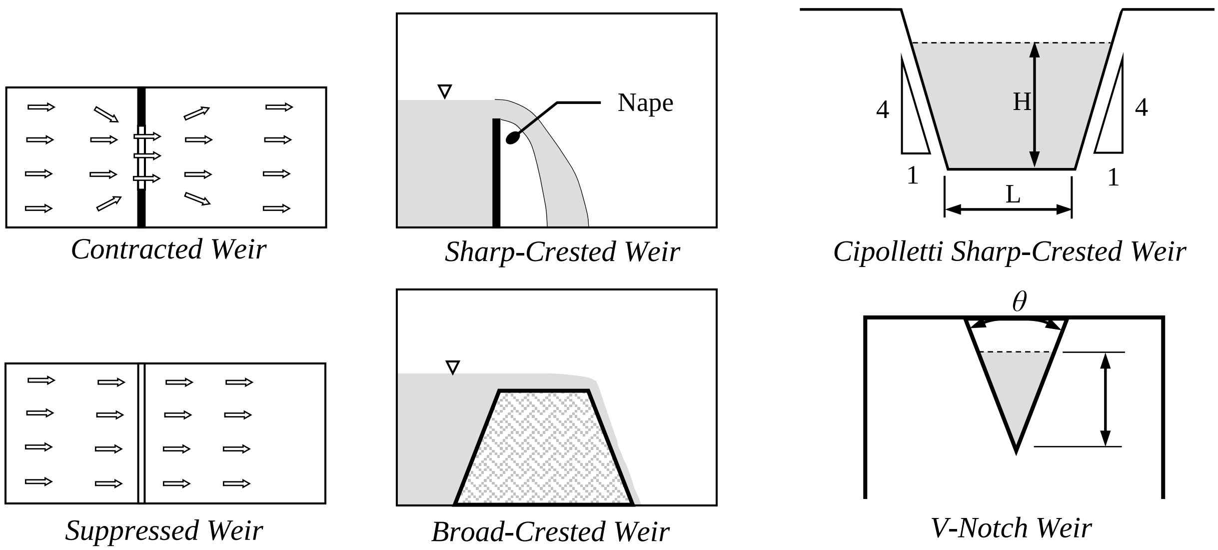
$Q = C_W * (L - 0.2H) * H^{1.5}$ "contracted" (the flow width gets narrower)
$Q = C_W * L * H^{1.5}$ "suppressed" (the flow width doesn't change)

C_W = typically 3.3 for sharp-crested weirs (but may also use 3.0 as typical)
C_W = usually 2.6 for broad-crested weirs 5' to 10' across (see Table ST-10-3)
L = length of weir
H = height of water surface above the weir invert = head

The basic differences in a sharp-crested weir and a broad-crested weir are shown in Figure ST-10-2. A sharp-crested weir allows a nape to form underneath the outlet discharge, and thus has more free-flow characteristics. A broad-crested weir (typical of an emergency spillway) has a lower weir coefficient value caused by friction losses.

A Cipolletti sharp-crested weir is shown in Figure ST-10-2. It is a modified rectangular weir that suppresses the contraction term. A Cipolletti sharp-crested weir is computed by the following formula:

$$Q = 3.367 * L * H^{1.5}$$



NOT TO SCALE

Figure ST-10-2
Types of Weirs

The equation for a V-notch sharp-crested weir that is not subject to downstream controls:

$$Q \;=\; C_V \;*\; H^{5/2} \;*\; \tan\left(\frac{\theta}{2}\right)$$

θ = notch angle
H = head or depth of water over invert of weir, ft
C_V = discharge coefficient for V-notch weir (see Table ST-10-4)

The notch angle should generally be selected at an angle from 22.5º to 90º. If the desired calculations show that a notch angle of less than 20º is appropriate, then the outlet should be designed as a narrow rectangular notch instead.

Outlet Structure – Culvert

The equations for a culvert flowing under inlet control (and not subject to downstream controls) are complex and not easily solved. In most cases, outlet flows in a detention basin are controlled by a concrete structure that discharges into an oversized concrete culvert. Culvert design is generally accomplished using FHWA Hydraulic Design Series No. 5 (reference 158) which can be downloaded from the following website: http://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm

This publication should be studied carefully for any purpose which involves the design and analysis of culverts (including the use of culverts as a control structure within a detention basin). As a quick reference for those who might wish to use equations rather than nomographs from reference 158, the two following equations apply to a circular culvert flowing under inlet control (and not subject to backwater conditions):

$$HW / D \;=\; H_c / D \;+\; K * (Q / AD^{0.5})^M \;-\; 0.5 \;*\; S \qquad \text{(Unsubmerged)}$$

$$HW / D = C * (Q / AD^{0.5})^2 + Y - 0.5 * S \qquad \text{(Submerged)}$$

HW = headwater depth above the culvert invert at inlet control section (feet)
D = inside diameter of culvert barrel (feet)
H_C = $d_C + V_C{}^2 / (2 * g)$ = specific head for critical depth at flow Q (feet)
Q = culvert discharge (cubic feet per second)
A = full cross-sectional area of culvert barrel (square feet)
S = culvert barrel slope (feet per foot)
K, M = coefficients for the unsubmerged equation
C, Y = coefficients for the submerged equation

Table ST-10-5 contains coefficients for concrete culverts (RCP) and for corrugated metal pipe (CMP) culverts with inlet control conditions. The unsubmerged equation applies for values of $Q/AD^{0.5}$ up to 3.5, and the submerged equation applies for values of $Q/AD^{0.5}$ of 4.0 and greater. For values of $Q/AD^{0.5}$ between 3.5 and 4.0, interpolate from the two boundary conditions to find values of HW_i (in transition from weir flow to orifice flow) for the culvert.

Increased Storage Volume (15%)

Section 22.5-31 of the Knoxville Stormwater and Street Ordinance requires that an additional 15% storage volume must be incorporated into the stormwater detention basin design. The extra 15% storage volume is based upon the 100-year design storm. The additional storage volume will compensate for the typical loss of volume over time that occurs for detention basins.

A detention basin is first designed in a manner that limits postdeveloped peak flow values to the corresponding predeveloped peak flow values (which is usually an iterative process). An outlet structure (usually a concrete riser) is selected to control the stormwater, with weirs or orifices of known size and elevation. Then the design storms are routed hydraulically through the detention basin, using detention software or other means, in order to verify that the outlet structure design works. After the computations are satisfied, each of the computed contours used in the detention basin computations shall be adjusted upwards to provide an additional 15% storage area. The 15% size adjustment shall not create steep embankments, flat drainage slopes or other potential problems.

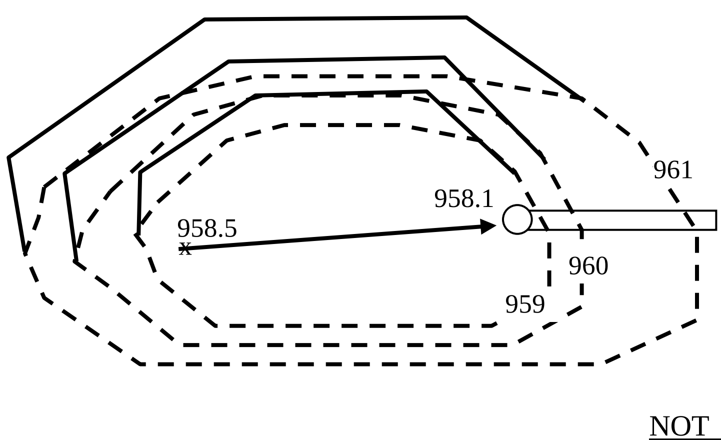


Figure ST-10-3
Storage Volume Adjustment

First Flush Release Rate

The first flush volume shall be released slowly over a minimum period of 24 hours, with a maximum drain time of 72 hours to allow the detention volume to be recovered prior to the next rainfall. For detention basins with gradual slopes and no irregular features, this can be accomplished by sizing an orifice to release at the average flow rate using the average hydraulic head. Consider a first flush volume of 4500 cubic feet, which is contained between elevations 955.0 and 958.2 (part of the example in ST-11 and ST-12). What size orifice is necessary to control the first flush volume?

An orifice can usually be sized from average head and flow.

Average flow = 4500 cubic feet / (24 * 3600 seconds) = 0.0521 cfs

Average head = 956.6 – 955.0 – radius ~ 1.55 feet (if head >> orifice size)

$$Q = C_O * A * (2*g*H)^{0.5}$$

$$0.0521 = 0.6 * \pi r^2 * (2*32.2*1.55)^{0.5}$$

$$r = \text{radius} = 0.0526 \text{ feet} = 0.63 \text{ inches}$$

Therefore, use an orifice diameter of 1.25" to drain the first flush volume. For detention basins without a regular volume-elevation relationship, compute the drawdown time using incremental volumes with a spreadsheet. The following example is a spreadsheet using a 1.25" diameter orifice and the same volumes as ST-11/ST-12:

A method to check the overall drain time of first flush volume if there are any unusual conditions.

Time needed to drain the first flush region for example in ST-11/ST-12

Dia = 1.25 inches   Invert = 955.00
Center = 955.05

Cumul Volume (ft^3)	Interval From ---	Interval To ---	Interval Avg H (feet)	Interval Q (cfs)	Increm Volume (ft^3)	Increm Time (hours)	Cumul Time (hours)
4455	958.1	958.2	3.10	0.072	227	0.87	0.87
4228	958.0	958.1	3.00	0.071	222	0.87	1.74
4006	957.9	958.0	2.90	0.070	215	0.85	2.60
3791	957.8	957.9	2.80	0.069	209	0.85	3.44
3582	957.7	957.8	2.70	0.067	203	0.84	4.28
3379	957.6	957.7	2.60	0.066	196	0.82	5.10
3183	957.5	957.6	2.50	0.065	191	0.82	5.92
2992	957.4	957.5	2.40	0.064	184	0.80	6.72
2808	957.3	957.4	2.30	0.062	179	0.80	7.52
2629	957.2	957.3	2.20	0.061	173	0.79	8.31
2456	957.1	957.2	2.10	0.059	166	0.78	9.09
2290	957.0	957.1	2.00	0.058	162	0.78	9.87
2128	956.9	957.0	1.90	0.057	155	0.76	10.63
1973	956.8	956.9	1.80	0.055	150	0.76	11.38
1823	956.7	956.8	1.70	0.053	144	0.75	12.13
1679	956.6	956.7	1.60	0.052	139	0.74	12.88
1540	956.5	956.6	1.50	0.050	133	0.74	13.61
1407	956.4	956.5	1.40	0.048	128	0.73	14.34
1279	956.3	956.4	1.30	0.047	123	0.73	15.08
1156	956.2	956.3	1.20	0.045	118	0.73	15.81
1038	956.1	956.2	1.10	0.043	113	0.73	16.54
925	956.0	956.1	1.00	0.041	108	0.73	17.27
817	955.9	956.0	0.90	0.039	103	0.74	18.00
714	955.8	955.9	0.80	0.037	98	0.74	18.75
616	955.7	955.8	0.70	0.034	93	0.75	19.50
523	955.6	955.7	0.60	0.032	88	0.77	20.27
435	955.5	955.6	0.50	0.029	83	0.80	21.07
352	955.4	955.5	0.40	0.026	79	0.85	21.91
273	955.3	955.4	0.30	0.022	75	0.93	22.84
198	955.2	955.3	0.20	0.018	70	1.07	23.91
128	955.1	955.2	0.10	0.013	66	1.43	25.34
62	955.0	955.1	0.00	-----	62	----	-----

In general, the release mechanism for a first flush volume should be a controlled structure with a known discharge equation, such as an orifice. Gravel filters are very effective at removing pollutants and to prevent clogging for dry detention basins. An underdrain system (perforated pipe with sand bedding) can also be used. In these instances, the number and size of perforated openings or slots should greatly exceed the design opening area that is needed for the first flush release rate.

Limitations Available space for detention is often limited on smaller project sites. Detention volumes and stormwater quality must be considered in the conceptual design phase of any development or redevelopment project. Designing a detention basin and outlet structure is an iterative process which requires the engineer to check the results using detention computations. Revised computations may be needed as the project proceeds.

References **50, 152, 153, 154, 158, 175, 180, 181, Knoxville Stormwater and Street Ordinance** (see BMP Manual Chapter 10 for list)

Summary of detention computations:

- NRCS Type II rainfall distribution for 24-hour storm events.
- AMC II (antecedent moisture conditions are average).
- Limit the postdevelopment peak flows to predevelopment levels.
- Provide first flush volume with slow release (for stormwater quality).
- Analyze the 1-year, 2-year, 5-year, 10-year, 25-year and 100-year storm events.
- Perform detention routing computations to verify adequacy of design.

<u>Design storms</u>		
	1-year, 24-hour precipitation	= 2.5 inches total
	2-year, 24-hour precipitation	= 3.3 inches total
	5-year, 24-hour precipitation	= 4.1 inches total
	10-year, 24-hour precipitation	= 4.8 inches total
	25-year, 24-hour precipitation	= 5.5 inches total
	50-year, 24-hour precipitation	= 6.1 inches total
	100-year, 24-hour precipitation	= 6.5 inches total
	500-year, 24-hour precipitation	= 7.6 inches total

Here is an equation to compute CN from precipitation P and runoff volume Q, which may be useful in determining a maximum developed CN allowed for a given detention basin size. P and Q are in units of inches.

$$CN = 100 \ / \ (1 + 0.5 * (P + 2Q - (4Q^2 + 5QP)^{0.5}))$$

<u>Table ST-10-1</u>
<u>Hydrologic Soil Groups</u>

Hydrologic Soil Group	USDA Soil Texture	Comments
A	**Sand, Loamy Sand, Sandy Loam**	1. See Figure ST-03-1 for USDA Soils Triangle, which classifies soils according to the percentages of sand, silt and clay.
B	**Silt Loam, Loam**	
C	**Sandy Clay Loam**	2. These classifications may be superseded by more specific information for the soil types and geologic conditions.
D	**Clay Loam, Silty Clay Loam, Silt, Sandy Clay, Silty Clay, Clay**	

Table ST-10-2
Runoff Curve Numbers for AMC II Conditions

….. from TR-55 publication (reference 175) …..

FULLY DEVELOPED URBAN AREAS (vegetation established)

Land Cover	Hydrologic Soil Group A	B	C	D
Open space (lawns, parks, golf courses, cemeteries)				
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	98	98	98	98
Streets and roads:				
Paved; curbs and storm sewers (not including right-of-way)	98	98	98	98
Paved; open ditches (with right-of-way)	83	89	92	93
Gravel (with right-of-way)	76	85	89	91
Dirt (with right-of-way)	72	82	87	89
Newly graded area (pervious only, no vegetation)	77	86	91	94

Land Use	% Impervious (typical)	Hydrologic Soil Group A	B	C	D
Urban districts					
Commercial & business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size					
1/8 acre (townhouses)	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

AGRICULTURAL AREAS (See TR-55 publication for cultivated lands)

Land Cover	Condition	Hydrologic Soil Group A	B	C	D
Pasture or grassland or range (Note 2) (continuous forage for grazing)	poor	68	79	86	89
	fair	49	69	79	84
	good	39	61	74	80
Meadow: continuous grass, protected from grazing and generally mowed for hay	----	30	58	71	78
Brush: (Note 3) brush-weed-grass mixture with mostly brush	poor	48	67	77	83
	fair	35	56	70	77
	good	30	48	65	73

Table ST-10-2
Runoff Curve Numbers for AMC II Conditions

….. from TR-55 publication (reference 175) …..

FULLY DEVELOPED URBAN AREAS (vegetation established)

Land Cover	Hydrologic Soil Group A	B	C	D
Open space (lawns, parks, golf courses, cemeteries)				
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	98	98	98	98
Streets and roads:				
Paved; curbs and storm sewers (not including right-of-way)	98	98	98	98
Paved; open ditches (with right-of-way)	83	89	92	93
Gravel (with right-of-way)	76	85	89	91
Dirt (with right-of-way)	72	82	87	89
Newly graded area (pervious only, no vegetation)	77	86	91	94

Land Use	% Impervious (typical)	Hydrologic Soil Group A	B	C	D
Urban districts					
Commercial & business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size					
1/8 acre (townhouses)	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

AGRICULTURAL AREAS (See TR-55 publication for cultivated lands)

Land Cover	Condition	Hydrologic Soil Group A	B	C	D
Pasture or grassland or range (Note 2)	poor	68	79	86	89
(continuous forage for grazing)	fair	49	69	79	84
	good	39	61	74	80
Meadow: continuous grass, protected from grazing and generally mowed for hay	----	30	58	71	78
Brush: (Note 3) brush-weed-grass mixture	poor	48	67	77	83
with mostly brush	fair	35	56	70	77
	good	30	48	65	73

Table ST-10-2 (continued)
Runoff Curve Numbers for AMC II Conditions

Land Cover	Condition	Hydrologic Soil Group A	B	C	D
Woods-grass combination (orchard, tree farm) Assumes 50% woods and 50% pasture.	poor	57	73	82	86
	fair	43	65	76	82
	good	32	58	72	79
Woods: (Note 4)	poor	45	66	77	83
	fair	36	60	73	79
	good	30	55	70	77
Farmsteads (buildings, lanes, driveways & surrounding lots)	----	59	74	82	86

Notes:

1. All CN values represent average runoff conditions and Ia = 0.2 * S.
2. Poor: < 50% ground cover or heavily grazed with no mulch
   Fair: 50% to 75% ground cover and not heavily grazed
   Good: > 75% ground cover and lightly or only occasionally grazed
3. Poor: < 50% ground cover
   Fair: 50% to 75% ground cover
   Good: 75% ground cover
4. Poor: Forest litter, small trees and brush are destroyed by heavy grazing or regular burning.
   Fair: Woods are grazed but not burned, and some forest litter covers the soil.
   Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

Adjusting curve numbers for other antecedent moisture conditions:

AMC I	Dry conditions	Less than 0.5” rainfall during the preceding 5 days.
AMC II	Average conditions	From 0.5” to 1.5” rainfall during the preceding 5 days.
AMC III	Wet conditions	More than 1.5” rainfall during the preceding days. (or freezing temperatures with icy conditions)

CN (AMC II)	CN (AMC I)	CN (AMC III)
100	100	100
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
60	40	78
55	35	74
50	31	70
45	26	65
40	22	60
35	18	55
30	15	50

The City of Knoxville requires the use of average moisture conditions (AMC II) for all detention basin computations.

AMC I and AMC III are provided to allow easy conversion for other types of analysis involving different moisture conditions.

Table ST-10-3
Coefficients for Broad-Crested Weirs (C_W)

Head (feet)	Breadth of weir crest (feet) 0.5	1.0	1.5	2.0	2.5	3.0	4.0	5.0	10.0	15.0
0.2	2.80	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.5	3.32	3.24	3.00	2.83	2.72	2.66	2.66	2.66	2.66	2.63
2.0	3.32	3.30	3.03	2.85	2.76	2.72	2.72	2.65	2.64	2.63
2.5	3.32	3.31	3.28	3.07	2.89	2.81	2.73	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.20	3.05	2.92	2.76	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.19	2.97	2.79	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.07	2.88	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.79	2.64	2.63

Table ST-10-4
Coefficients for V-Notch Weirs (C_V)

Head (feet)	Angle of V-notch opening 22.5º	30º	45º	60º	90º
0.2	2.78	2.72	2.66	2.61	2.58
0.5	2.62	2.59	2.55	2.53	2.50
1.0	2.54	2.53	2.50	2.48	2.47
1.5	2.51	2.50	2.47	2.46	2.45
2.0	2.50	2.48	2.47	2.45	2.44
2.5	2.48	2.47	2.46	2.45	2.44
3.0	2.48	2.47	2.46	2.44	2.44

Table ST-10-5
Coefficients for Circular Culverts Under Inlet Control

(from reference 158, FHWA Hydraulic Design Series No. 5)

FHWA Nomographs	Scale #	Inlet edge description	Unsubmerged K	M	Submerged C	Y
Chart 1 Concrete	1	Square edge with headwall	0.0098	2.0	0.0398	0.67
	2	Groove end with headwall	0.0018	2.0	0.0292	0.74
	3	Groove end projecting	0.0045	2.0	0.0317	0.69
Chart 2 CMP	1	Headwall	0.0078	2.0	0.0379	0.69
	2	Mitered to slope	0.0210	1.33	0.0463	0.75
	3	Projecting	0.0340	1.5	0.0553	0.54

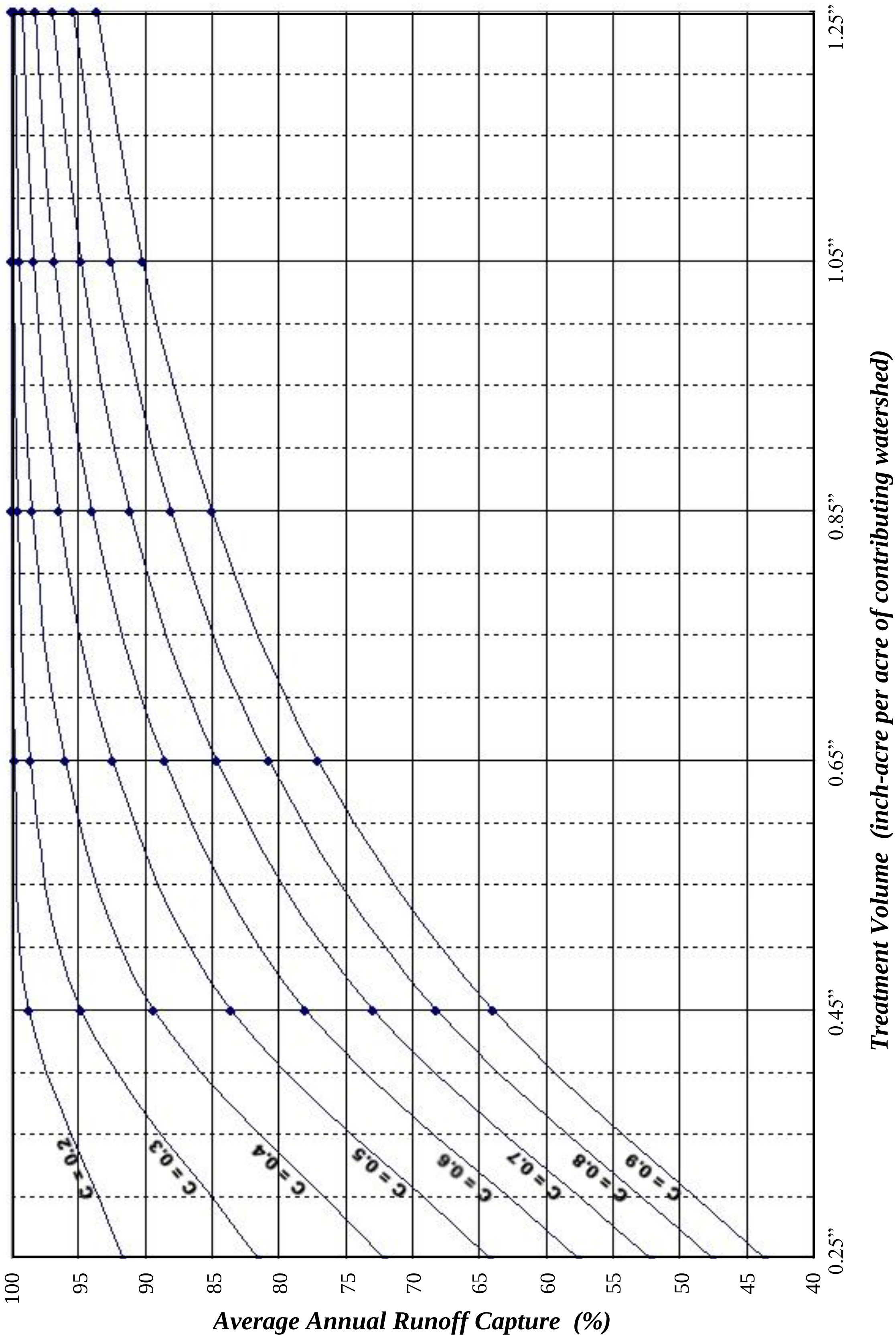


Figure ST-10-4
Average Annual Runoff Volume Capture for 36-Hour Drawdown

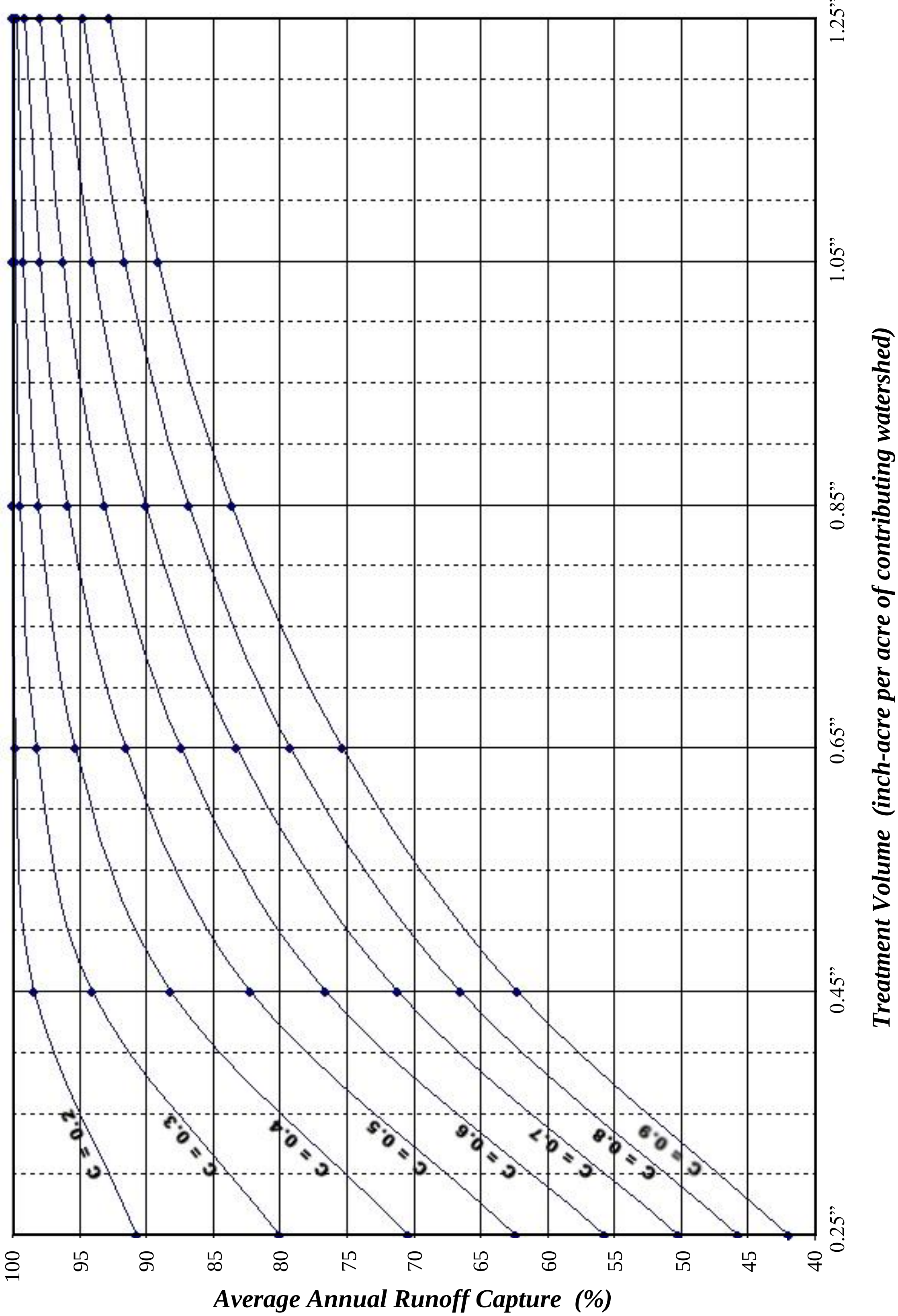


Figure ST-10-5
Average Annual Runoff Volume Capture for 42-Hour Drawdown