B. Stormwater Calculations

Stormwater programs in North Carolina require high density projects to treat the design storm depth in a stormwater control measure (SCM). To size the SCM, the designer must calculate the volume of runoff that will drain to it. Additional calculations are needed to design the outlet structures of SCMs.

The stormwater rules do not reference any calculation methods. This chapter provides technical guidance for stormwater calculation methods that are typically used in North Carolina.

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Rational Method for Peak Flow

$$Q_p = C * I * A$$

Where:	Qp	=	Peak flow for the storm of interest (cfs)
	C	=	Composite runoff coefficient (unitless)
	I	=	Rainfall intensity for the storm of interest (in/hr)
	A	=	Drainage area (ac)

The peak flow is often calculated using the Rational Method. Calculating the peak flow is an important design step in designing a flow-based device such as a level spreader-filter strip. It is also important to meet SA waters requirements per 15A NCAC 02H .1019.

The composite runoff coefficient, C, reflects the runoff potential of the drainage area. The range of runoff coefficients varies from 0.35 to 0.95, with higher values corresponding to greater runoff potential. The composite runoff coefficient is the weighted average of all of the land uses within the drainage area. Table 1 presents values of runoff coefficients for various surfaces. The Rational Method is most applicable to drainage areas that are 20 acres or less.

Table 1: Rational Runoff Coefficients by Land Use
(ASCE 1975, Viessman, et al. 1996, and Malcom 1999)

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

The rainfall intensity in inches per hour, I, can be obtained from the NOAA web site. From this web site, select from one of NOAA's numerous data stations throughout the state and select "precipitation intensity." This will open a table that displays precipitation intensity estimates for various annual return intervals (ARIs) (one year through 1,000 years) and various storm durations (5 minutes through 60 days). The requirements of the applicable stormwater program will determine the appropriate values for ARI and storm duration. If the design is for a level spreader that is receiving runoff directly from the drainage area, then the value for I should simply be one inch per hour (more information on level spreader design in Chapter 8).

Simple Method for Runoff Volume

$$R_V = 0.05 + 0.9 * I_A$$

Where:
R_V = Runoff coefficient (unitless)
I_A = Impervious fraction (unitless)

$$DV = 3630 * R_D * R_V * A$$

Where:
DV = Design volume (cu ft)
R_D = Design storm depth (in)
A = Drainage area (ac)

There are two methods that are often used to determine the volume of runoff from a given design storm: the Simple Method (Schueler 1987) and the discrete NRCS Curve Number Method (NRCS 1986).

The Simple Method was developed by measuring the runoff from many watersheds with known impervious areas and curve-fitting a relationship between percent imperviousness and the fraction of rainfall converted to runoff (the runoff coefficient). It uses a minimal amount of information to estimate the volume of runoff.

A couple of notes on the variables in the Simple Method:

- The ***runoff coefficient, R_V,*** is the runoff depth in inches divided by the rainfall depth in inches.
- The ***impervious fraction, I_A,*** is the impervious portion of drainage area in acres divided by the drainage area in acres
- The ***design volume, DV,*** is the volume of runoff that must be controlled for the design storm.
- The ***design storm depth, R_D,*** is 1.5" in Coastal Counties and 1.0" elsewhere. In SA waters, the DV is the difference between runoff volume pre- versus post-development for the 10-year storm.
- The ***drainage area, A,*** must include all of the surfaces that drain to the SCM regardless of whether they are on-site or off-site.

Discrete NRCS Curve Number Method for Runoff Depth

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

Where:	S	=	Maximum retention after rainfall begins (in)
	CN	=	Curve number (unitless)

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

Where:	Q*	=	Runoff depth (in)
	P	=	Rainfall depth (in)

The steps for using the Discrete NRCS Curve Number Method are as follows:

1. Divide the drainage area into land uses and assign an appropriate curve number, CN, to each one (see Table 4). The CN is a value between 30 and 98 that characterizes the amount of runoff generated by a drainage area based on its USGS Hydrologic Soil Group (HSG) and ground cover.
2. Determine the precipitation depth, P. P is 1.5" in Coastal Counties and 1.0" elsewhere. In SA waters, the DV is the difference between runoff volume pre- versus post-development for the 10-year storm (read value from the NOAA web site, using a storm duration equal to the time of concentration).

3. Compute runoff depth, Q*, for any built upon areas that are directly linked to surface waters via a swale or pipe using the formulas above (note that the CN for built-upon area is 98). Find the runoff volume from the directly connected impervious surfaces by multiplying Q* in inches times the area of the directly connected impervious surfaces in square feet and dividing by 12.
4. Compute runoff depth, Q*, for the remainder of the project. First, calculate a composite CN (a weighted average of the CNs from the land uses). If the CN is equal to or below 64, assume that there is no runoff resulting from either the 1.0" or 1.5" storm. If the composite CN is above 64, compute Q* for this area. Find the runoff volume from the remainder of the site by multiplying Q* in inches times the area of the remainder of the project in square feet and dividing by 12.
5. Find the design volume from the drainage area by adding the results of Steps 3 and 4.

The four HSGs are described in Table 2 below.

Table 2: Rational Runoff Coefficients by Land Use
(NRCS 1986)

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

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

Table 3: HSGs for North Carolina Soil Types
(Malcom 1989)

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

Codurus	C	Kalmia	B	Plummer	D/B	
Colfax	C	Kenansville	A	Pocalla	A	
Comus	B	Kershaw	A	Pocomoke	D/B	
Congaree	B	Kinston	D/C	Pomello	C/A	
Cowarts	C	Lakeland	A	Ponzer	D/C	
Coxville	D/C	Leaf	D/C	Porters	B	
Craven	C	Lenoir	D/B	Portsmouth	D/C	
Davidson	B	Leon	C/B	Rabun	B	
Delanco	C	Liddell	D/C	Rains	D/B	
Dorovan	D	Lloyd	B	Ramsey	D	
Dothan	B	Lockhart	B	Ranger	C	

The type of ground cover at a given site greatly affects the volume of runoff. Undisturbed natural areas, such as woods and brush, have high infiltration potentials whereas impervious surfaces, such as parking lots and roofs, will not infiltrate runoff at all. The ground surface can vary extensively, particularly in urban areas, and Table 4 lists appropriate curve numbers for most urban land use types according to hydrologic soil group. Land use maps, site plans, and field reconnaissance are all effective methods for determining the ground cover.

Table 4: HSGs for North Carolina Soil Types

Urban areas runoff curve numbers for SCS method (SCS 1986)

Cover Description	Curve Number by HSG			
	A	B	C	D
Fully developed urban areas				
Open Space (lawns, parks, golf courses, etc.)				
Poor condition (< 50% grass cover)	68	79	86	89
Fair condition (50% to 75% grass cover)	49	69	79	84
Good condition (> 75% grass cover)	39	61	74	80
Impervious areas:				
Paved parking lots, roofs, driveways, etc.	98	98	98	98
Streets and roads:				
Paved; curbs and storm sewers	98	98	98	98
Paved; open ditches	83	89	98	98
Gravel	76	85	89	91
Dirt	72	82	85	88
Developing urban areas				
Newly graded areas	77	86	91	94
Pasture (< 50% ground cover or heavily grazed)	68	79	86	89
Pasture (50% to 75% ground cover or not heavily grazed)	49	69	79	84
Pasture (>75% ground cover or lightly grazed)	39	61	74	80

Meadow – continuous grass, protected from grazing and generally mowed for hay	30	58	71	78
Brush (< 50% ground cover)	48	67	77	83
Brush (50% to 75% ground cover)	35	56	70	77
Brush (>75% ground cover)	30	48	65	73
Woods (Forest litter, small trees, and brush destroyed by heavy grazing or regular burning)	45	66	77	83
Woods (Woods are grazed but not burned, and some forest litter covers the soil)	36	60	73	79
Woods (Woods are protected from grazing, and litter and brush adequately cover the soil)	30	55	70	77

Stage-Storage Table for Storage Volume of Ponds

Volume control is typically provided through detention measures like wet ponds with volume above the water operating level and below the required freeboard. Storage volume within a pond can be calculated using a stage-storage method. A table shall be provided showing incremental elevations of the pond with square footage values at the listed elevations. The elevation increments should be no greater than one foot. Columns show the incremental volume and cumulative volume of storage provided.

See Table 5 below for an example of a storage volume calculation. This method can be used for basin shapes as simple as a rectangle or as intricate as a curved, landscape designed wetland feature. It can also be used to calculate sediment storage volume and operating volume within ponds.

Table 5: Stage-Storage Volume Calculation Example

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

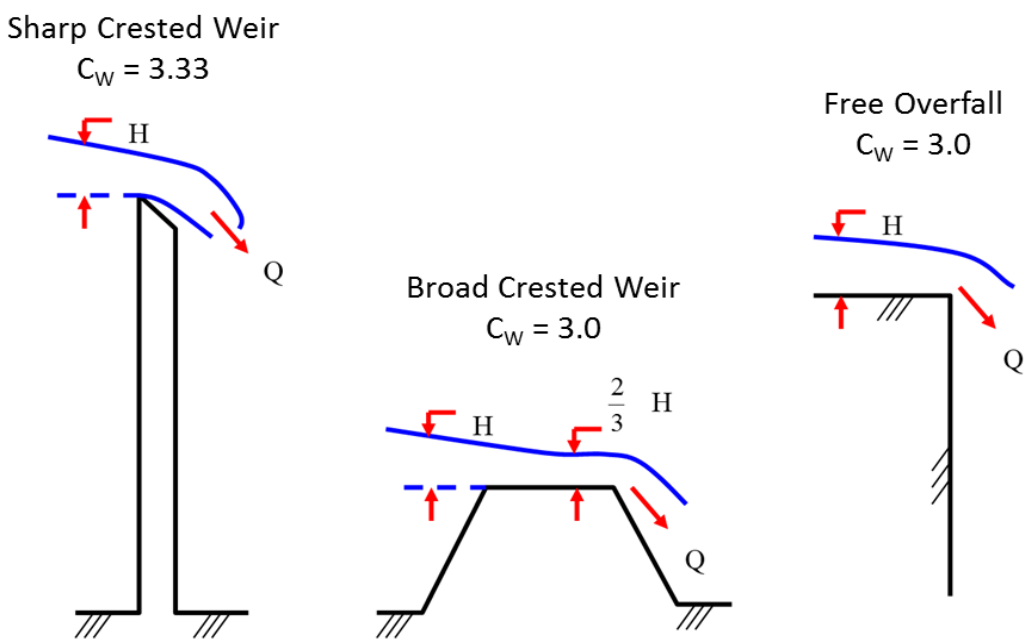
Weir Equations for Outlet Flow

Weir equations are useful in designing a number of SCM components, including:

- Outlet devices that are weirs or function as weirs until fully submerged,
- Flow splitter devices, and
- Check dams in treatment swales.

Weirs are typically categorized as either ***sharp-crested, broad-crested*** or ***v-notch***. See Figure 1 for schematic sections through weirs. The equations for flow over the weir depend on its configuration.

Figure 1: Schematic Sections through Sharp-Crested and Broad-Crested Weirs
(Malcom 1989)

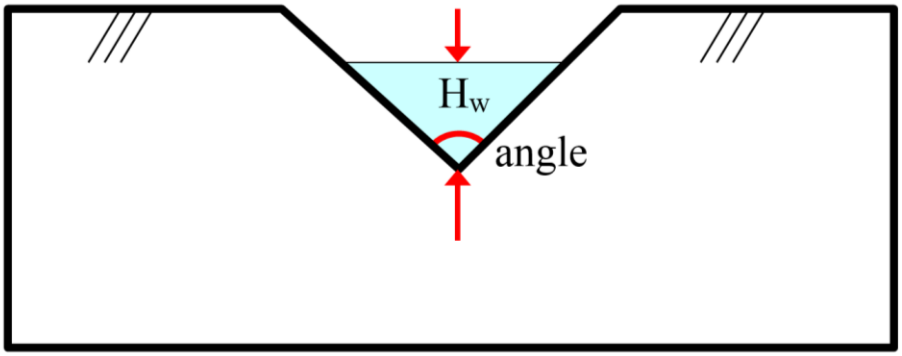


For sharp-crested and broad-crested weirs, the basic equation is:

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

Where:	Q	=	Discharge (cfs)
	C_W	=	Weir coefficient per Figure 1 (unitless)
	L	=	Length of weir (ft), measured along the crest
	H	=	Driving head (ft), measured vertically from the crest of the weir to the water surface at a point far enough upstream to be essentially level

Figure 2: Schematic Sections through V-Notch Weir
(Malcom 1989)



For V-notch weirs, the basic equation is:

$$Q = C_V H_W^{2.5}$$

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

Orifice Equation for Outlet Flow

An orifice is simply a hole. In the design of SCMs, orifices are often used for drawing down the design volume. It is important to determine the size an orifice correctly so that the appropriate detention time provided.

The basic equation for discharge from an orifice is:

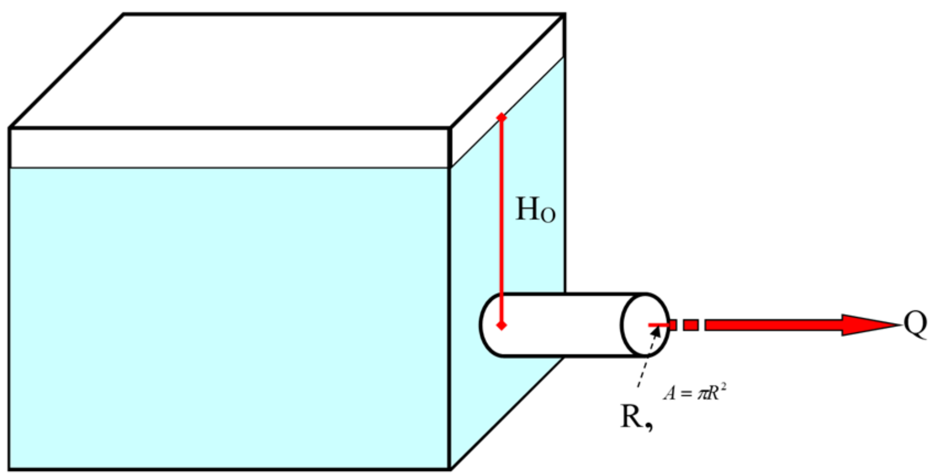
$$Q = C_D A (2 g H_o)^{0.5}$$

Where:	Q	=	Discharge (cfs)
	C_D	=	Coefficient of discharge per Table 6 (unitless)
	A	=	Cross-sectional area of orifice entrance (sq ft)
	g	=	Acceleration of gravity (32.2 ft/sec^2)
	H_o	=	Driving head from water surface to centroid orifice (ft) **usually use **H_o/3** to compute drawdown through an orifice to reflect the fact that head is decreasing as drawdown occurs**

Table 6: Values of Coefficient of Discharge for Orifices, C_D
(Malcom 1989)

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

Figure 6: Schematic of Orifice Flow
(Malcom 1989)



Chainsaw Routing for Stage-Storage-Discharge

Creating a stage-storage-discharge model is crucial for wet ponds and stormwater wetlands. These SCMs provide volume control for the specified storm (for example, the 1.0 or 1.5-inch storm depth) in a temporary pool that is above the permanent pool.

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

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

The application of the Chainsaw Routing method is described in detail in Elements of Urban Stormwater Design (Dr. H. Rooney Malcom, P.E. 1989).

Other Models for Stage-Storage-Discharge

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

Table 7: Models for Stage-Storage-Discharge

Model & Link	Description of Model
HEC-HMS	Developed by the U.S. Army Corps of Engineers, provides a variety of options for simulating precipitation-runoff processes. This model can simulate unit hydrograph and hydrologic routing options. The latest version also has capabilities for continuous soil moisture accounting and reservoir routing operations.
WinTR-55	Developed by the NRCS, can be used to analyze the hydrology of small watersheds. A final version (including programs, sample data, and documentation) is now complete.
SWMM	Developed by the EPA, can be used to analyze stormwater quantity and quality associated with runoff from urban areas. Both single-event and continuous simulation can be performed on catchments having storm sewers, or combined sewers and natural drainage, for prediction of flows, stages and pollutant concentrations.

Manning Equation for Channel Flow

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

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

For V-notch weirs, the basic equation is:

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

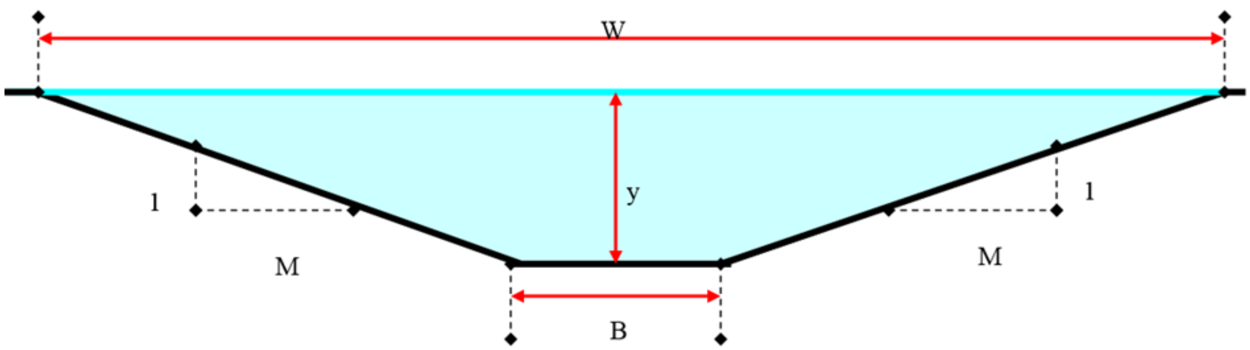
Where:

	Q	=	Peak discharge to the channel (cfs)
	n	=	Manning roughness coefficient, see Table 8 (unitless)
	A	=	Cross-sectional area of flow, see below (sq ft)
	R	=	Hydraulic radius, see below (ft)
	S	=	Longitudinal slope of channel invert (ft fall/ft run).

$$A = By + My^2 \qquad P = B + 2y\,(1 + M^2)^{0.5} \qquad R = A / P$$

Where:	A	=	Cross-sectional area of flow (sq ft)
	B	=	Bottom width of the channel (ft)
	M	=	Side slope ratio (ft horizontal/ft vertical)
	P	=	Wetted perimeter, the distance along the cross-section against which water is flowing (ft)
	R	=	Hydraulic radius (ft)
	y	=	Depth of flow (ft)

Figure 7: Schematic of Channel Cross-Section
(Malcom 1989)



Note: M is often determined by channel side slope requirements, typically 3:1 (M=3).

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

Table 8: Rational Runoff Coefficients
(adopted from Munson et al. 1990 and Chow et al. 1988)

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