Chapter 6. HYDRAULICS

6.1. Open Channel Flow

When dealing with the hydraulics of open channel flow, there are three basic relationships:

1. Continuity Equation
2. Energy Equation
3. Momentum Equation

6.1.1 Continuity Equation

The continuity equation may be written as:

Inflow – Outflow = Change in Storage

Where inflow represents the volume of flow into the system during a time interval and the outflow represents the volume of flow out of the system during the same time interval. The change in storage represents the change in volume of water stored in the system.

The continuity equation may also be expressed by:

Inflow Rate –Outflow Rate = Rate of Change in Storage

The flow rate, Q, is generally expressed in cubic feet per second (cfs), and may be written as:

$$Q = VA$$

Where:

V = Average flow velocity over a cross section (ft/sec); and,
A = Area of cross section (ft^2).

It should be noted that (V) in the flow equation is the average velocity of the flow perpendicular to the cross section. The actual pattern of flow velocity is complex, and the velocity varies greatly from the bottom of the channel to the top of the water surface. However, the velocity along the boundary of the channel bottom is considerably lower than average flow velocity for a particular cross section.

6.1.2 Energy Equation

In basic fluid mechanics, the energy equation is written in the form of Bernoulli's Equation:

$$\frac{v_1^{\ 2}}{2g} + y_1 + z_1 + \frac{\rho_1}{\gamma} = \frac{v_2^{\ 2}}{2g} + y_2 + z_2 + \frac{\rho_2}{\gamma} + h_L$$

Where:

v	=	Average flow velocity (ft/sec),
g	=	Gravitational constant,
y	=	Depth of flow,
z	=	Elevation of channel bottom,
γ	=	Unit weight of water, and
$\mathbf{h_L}$	=	Energy loss between the sections of interest.

The energy equation represents an energy balance between two points along a channel. Since the equation is an energy equation, the terms represent energy per unit width of flowing fluid. Since the units are a length, the terms are commonly known as "head". Therefore,

$\mathbf{v^2/2g}$	=	Velocity head
y + z	=	Elevation head
ρ/γ	=	Pressure head.

The sum of the velocity head, elevation head, and pressure head represents the total energy, also known as the energy grade line (EGL). The energy grade must be sloping downward in the direction of the flow unless external energy (pump) is added to the system.

The sum of the elevation head and pressure head is known as the hydraulic grade line (HGL).

The difference between open channel flow and pipe flow, is that the free water surface of open channel flow is exposed to the atmosphere and the pressure head is zero. Therefore, for open channel flow the pressure head is ignored and the HGL represents the water surface.

6.1.2.1. Critical Depth

Critical depth (yc) occurs when the Froude Number (F) is equal to 1.

$$F = \frac{v}{\sqrt{gd_h}} \quad where \quad d_h = \frac{A}{t}$$

The hydraulic depth dh is defined as being the flow area (A) divided by the flow top width (t).

Since the Froude number is independent on slope, critical depth (yc) depends only on discharge for a given channel. Channel roughness, velocity, discharge, and slope are interrelated. For a given discharge and roughness, the velocity can be increased and the depth of flow decreased by increasing the channel slope. When the channel slope is such that the flow depth resulting from uniform flow equals critical depth, the slope is called the critical slope. For subcritical flow, the slope is less than critical slope, and for supercritical flow, the channel slope is greater than critical slope. Critical depth, slope, and velocity for a given channel section change with discharge.

When designing channels for controlling and conveying runoff, it is desirable to design the channel for subcritical flow.

6.1.3 Momentum Equation

The momentum principle in open channel flow is defined by the basic relationship of mechanics:

$$\sum F_s = \Delta(mv_s)$$

This relationship states that the sum of forces in the s-direction equals the change in momentum in that direction. The depth corresponding to the minimum force plus momentum (M) is the critical depth (yc).

6.1.4 Uniform Flow

Open channel flow is classified with respect to changes in flow properties with time and location along a channel. If the flow characteristics are not changing with time, the flow is steady flow. If the flow properties are the same at every location along the channel, the flow is uniform. Flow with properties that change with channel location is classified as being non-uniform flow.

In natural flow conditions, the flow is usually non-steady and non-uniform. However, for channel design, steady, uniform flow is assumed based on a peak or maximum discharge.

6.1.5 Manning's Equation

Manning's equation is used to calculate average flow velocities for open channels where the factor related to channel roughness increases as the roughness increases.

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

Where:

V = Average flow velocity (ft/sec)
N = Manning roughness coefficient
R = Hydraulic radius (feet), Calculated to be A/P where:
A = flow cross sectional area (feet2)
P = wetted perimeter (feet) (length of boundary between water and channel)
S = Channel slope (ft/ft)

Manning's Roughness Coefficient (n) is influenced by many factors including:

- Physical roughness of the channel surface,
- The irregularity of the channel cross section,
- Channel alignment and bends,
- Vegetation,
- Silting and scouring, and
- Obstructions within the channel.

Manning's n is very important and critical in open channel flow computations. Variations in this variable can significantly affect discharge, depth, and velocity calculations. Sound engineering judgment must be exercised when selecting appropriate Manning's n values. Typical values for Manning's Roughness Coefficient (n) are located in Table 6-1.

Table 6-1 Uniform Flow Values for Manning's Roughness Coefficients

Type of Flow Media	Min	Normal	Max
Pipes			
Plastic (PVC and ABS)		0.009	
Cast iron, coated	0.011	0.013	0.014
Cast iron, uncoated	0.012		0.015
Clay or concrete drain tile	0.010	0.011	0.020
Concrete		0.013	
Corrugated metal	0.021	0.025	0.027
Steel, riveted and spiral	0.013	0.016	0.017
Brick		0.016	
Vitrified sewer pipe	0.010	0.014	0.017
Wrought iron, black	0.012		0.015
Wrought iron, galvanized	0.013	0.016	0.017
Excavated or Dredged Ditches and Channels			
Earth Straight and Uniform			
Clean recently completed	0.016	0.018	0.020
Clean after weathering	0.022	0.025	0.030
Gravel, uniform section, clean	0.022	0.027	0.033
Earth Winding and Sluggish			
No vegetation	0.023	0.025	0.030
Grass, some weeds	0.025	0.030	0.033
Dense weeds, plants in deep channels	0.030	0.035	0.040
Earth bottom and rubble sides	0.025	0.030	0.035
Stony bottom and weed sides	0.025	0.035	0.045
Cobble bottom and clean sides	0.030	0.040	0.050
Dragline Excavated or Dredged			
No vegetation	0.025	0.028	0.033
Light brush on banks	0.035	0.050	0.060
Rock Cuts			
Smooth and uniform	0.025	0.035	0.040
Jagged and irregular	0.035	0.040	0.050
Channels Not Maintained, Vegetation and Brush Uncut			
Dense vegetation in channel as high as flow depth	0.050	0.080	0.120
Clean bottom, vegetation and brush on sides	0.040	0.050	0.080
Clean bottom, brush and vegetation up to high stage	0.045	0.070	0.110
Clean bottom, dense brush and vegetation on overbanks	0.080	0.100	0.140

Type of Flow Media	Min	Normal	Max
Natural Streams on Plain			
Clean straight, full stage, no rifts or pools	0.025	0.030	0.033
Stones, vegetation, straight, full stage	0.030	0.035	0.040
Clean, winding, some pools and shoals	0.033	0.040	0.045
Vegetation, stones, winding, some pools and shoals	0.035	0.045	0.050
Sluggish reaches, vegetation, deep pools	0.050	0.070	0.080
Much vegetation, deep pools, or floodways with timber and underbrush	0.075	0.100	0.150
Natural Mountain Streams with no Vegetation in Channel, Trees and Brush Along Banks are only Submerged at High Stages			
Bottom consists of gravel cobbles and few boulders	0.030	0.040	0.050
Bottom consists of cobbles with large boulders	0.040	0.050	0.070
Floodplains			
Pasture, no Brush			
Short grass	0.025	0.030	0.035
High grass	0.030	0.035	0.050
Cultivated Areas			
No crop	0.020	0.030	0.040
Mature row crop	0.025	0.035	0.045
Mature field crop	0.030	0.040	0.050
Brush			
Scattered brush, heavy weeds	0.035	0.050	0.070
Light brush and trees in winter	0.035	0.050	0.060
Light brush and trees in summer	0.040	0.060	0.080
Medium to dense brush in winter	0.045	0.070	0.110
Medium to dense brush in summer	0.070	0.100	0.160
Trees			
Dense willows, summer, straight	0.110	0.150	0.200
Cleared land, tree stumps, no sprouts	0.030	0.040	0.050
Cleared land, tree stumps, with heavy sprouts	0.050	0.060	0.080
Heavy stand of timber, floodstage below branches	0.080	0.100	0.120
Heavy stand of timber, floodstage above branches	0.100	0.120	0.160

Type of Flow Media	Min	Normal	Max
Lined Channels			
Asphaltic concrete machine placed		0.014	
Asphaltic exposed, prefabricated	0.015	0.016	0.018
Concrete		0.013	0.015
Concrete Rubble	0.016		0.029
Shotcrete	0.016		0.017
Grouted Riprap	0.028	0.030	0.040
Stone Masonry	0.030	0.032	0.040
Jute Net	0.019	0.022	0.028
Straw with net	0.025	0.033	0.065
Curled wood mat	0.028	0.035	0.066
Synthetic geotextile mat	0.021	0.025	0.036
Gravel Riprap			
1-inch D50	0.030	0.033	0.044
2-inch D50	0.034	0.041	0.066
Rock Riprap			
6-inch D50	0.035	0.069	0.104
12-inch D50	0.040	0.078	0.120

Sources: Design Hydrology and Sedimentology for Small Catchments, Hann et. al., 1995 and HEC-15

6.1.6 Trapezoidal Channels

The hydraulic radius (R) is defined as:

$$R = \frac{A}{P}$$

Where:

A = Cross sectional flow area (ft^2)
P = Wetted perimeter (ft)

The wetted perimeter is defined as being the length of the boundary between water and the channel sides and bottom at any cross section. The wetted perimeter is the distance around the flow cross section starting at one edge of the channel and traveling along the sides and bottom to the other channel edge.

The cross sectional area (A) for a trapezoidal channel can be determined from:

$$A = bd + Zd^2$$

Where:

A = Cross sectional flow area (ft^2)
b = Bottom width of channel (ft)
d = Flow depth of channel (ft)
Z = Channel side slopes (ZH:1V)

The hydraulic radius (R) for a trapezoidal channel can be calculated from:

$$R = \frac{bd + Zd^2}{b + 2d\sqrt{Z^2 + 1}}$$

The expression for the hydraulic radius for wide, shallow channels can be simplified for calculations. Consider a trapezoidal channel that is wide and shallow. The trapezoid can then be approximated by a rectangle.

$$R = \frac{A}{P} = \frac{bd}{b + 2d}$$

If b is much larger than d (b >> d), then the 2d in the denominator can be ignored leaving:

$$R = \frac{A}{P} = \frac{bd}{b} = d$$

6.1.7 Circular Channels (Pipes)

The maximum flow capacity of a circular pipe occurs at a depth equal to 0.938D.

The hydraulic radius of a pipe is defined by the flow depth and an angle (θ) that is measured in radians.

$$A = \frac{D^2}{8}(\theta - \sin\theta)$$

$$R = \frac{D}{4}\left(1 - \frac{\sin\theta}{\theta}\right)$$

6.1.7.1. Flow Depth 0 < y < D/2

For the flow depth (y) in a pipe and pipe diameter (D) where: **0 < y < D/2**

$$\theta = 2\ \tan^{-1}\left[\frac{\sqrt{\left(\frac{D}{2}\right)^2 - \left(\frac{D}{2} - Y\right)^2}}{\frac{D}{2} - Y}\right]$$

Flow Depth y = D/2

For the flow depth (y) in the pipe and pipe diameter (D) where: **y = D/2**

$$\theta = \pi$$

Flow Depth D/2 < y < D

For the flow depth (y) in the pipe and pipe diameter (D) where: **D/2 < y < D**

$$\theta = 2\pi \ + \ 2\tan^{-1}\left[\frac{\sqrt{\left(\frac{D}{2}\right)^2 - \left(Y - \frac{D}{2}\right)^2}}{\frac{D}{2} - Y}\right]$$

6.1.8 Normal Depth Calculation

Normal depth calculations can be found by using the following methods:

- Trial and Error
- Graphical Procedures
- Computer Models

Trial and Error

A trial and error procedure for solving Manning's equation can be used to calculate the normal depth of flow in a uniform channel when the channel shape, slope, roughness and design discharge known.

The flow rate, Q, is generally expressed in cubic feet per second (cfs), and may be written as

$$Q = VA$$

Where:

V = Average flow velocity over a cross section (ft/sec), calculated using Manning's equation
A = Area of cross section (ft^2)

Using Manning's Equation, the continuity equation can be solved as:

$$Q = \frac{1.49}{n} A\, R^{2/3} S^{1/2}$$

Rearrangement of the continuity equation results in the following ratio:

$$A\,R^{2/3} = \frac{n\,Q}{1.49\;S^{1/2}}$$

To calculate the normal depth of flow (dn) by the trial and error process, trial values of depth (dn) are selected to calculate a corresponding flow area (A), wetted perimeter (P), and hydraulic radius (R). For each trial depth selected, a corresponding AR2/3 value is calculated. Trial values of the depth are selected until the AR2/3 value equals the known ratio calculated by using the known roughness, design discharge, and channel slope.

Graphical Procedure

Graphical methods for simplifying the trial and error procedure have been created for trapezoidal channels to calculate the normal depth of flow. This method utilizes a known ratio based on the channel side slopes, channel bottom width, channel slope, Manning's roughness coefficient n, and design discharge.

The design ratio is expressed as:

$$d_n\ ratio = \frac{Q\ n}{b_o^{\;8/3} S^{1/2}}$$

Where:

Q	=	Peak flow rate (ft^3/sec)
n	=	Manning roughness coefficient
b_o	=	Channel bottom width (ft)
S	=	Channel slope (ft/ft)

Once the normal depth ratio is calculated, and the side slopes (ZH:1V) are determined, **Figure 6-1** may be used to determine the normal depth of flow.

- Locate the value for the normal depth ratio on the x-axis of **Figure 6-1**, and extend this value up to the appropriate side slope curve.
- From this intersection point, extend back to the y-axis to obtain the d_n/b ratio.
- Once the d_n/b_o ratio has been obtained, multiply the d_n/b_o ratio value by the channel bottom width (b_o) to calculate the normal depth of flow in the channel.

6.1.8.1. Graphical Procedure Example

Given: A channel is to be designed with the following parameters:

Peak flow rate Q	=	50 cfs

Manning's n	=	0.045
Channel bottom width	=	5 ft
Channel side slopes	=	3H:1V
Channel bed slope	=	0.01 ft/ft

Find: The normal depth (d_n) of flow in the channel.

Solution:

1. Calculate the normal depth ratio (d_n ratio).

$$d_n\ ratio = \frac{(50)\ (0.045)}{5^{8/3}(0.01)^{1/2}}$$

$$d_n\ ratio = 0.31$$

2. Locate the value for the normal depth ratio on the x-axis of **Figure 6.1**, and extend this value up to the appropriate side slope curve.

    - Locate 0.31 on the x-axis and extend this value up to the side slope curve Z=3.

3. From this intersection point read back to y-axis to obtain the dn/bo ratio.

    - The ratio intersection reads to be 0.31

4. Multiply the dn/bo ratio by the bottom width (bo) to obtain the normal depth.

    - 0.31 * 5.0 ft = 1.55 ft

    - The normal depth (d_n) of the channel is calculated to be **1.55-feet.**

6.1.8.2. Computer Models

There are various computer models available that are capable of calculating the flow depth in a given channel reach. Many of these models are capable of handling a full network of channels, or just the computations for a single channel reach. These models are also capable of calculating water surface elevations for subcritical, supercritical, and mixed flow regimes. The effects of various obstructions such as bridges, culverts, weirs, and structures in the overbank areas may also be considered in the calculations. The actual models used for these calculations shall be at the discretion of the design professional with approval from the Greenville County Storm Water Management Director.

6.2. Outlet Hydraulics

Outlet structures provide the critical function of regulating flow volumes and peak flow rates from storm water management control structures. Flow control devices can operate as either open channel flow, in which the flow has a free water surface, or pipe flow in which the flow is in a closed conduit. In either situation, an increase in head on a structure increases the discharge flow rate through the structure.

There are numerous different types of outlet structures designed for specific types of flow control:

- Water quality and channel protection flows are typically controlled with smaller, more protected outlet structures such as:
  - Perforated plates or risers,
  - Reverse slope under-surface pipes,
  - Orifices located within screened pipes or risers,
  - V-notch weirs, and
  - Hooded orifices where the hood invert is located beneath the permanent pool elevation to protect the outlet structure from floatable debris
- Larger storm event flows are typically controlled by:
  - Risers with different sized and shaped openings,
  - Flow over the top of risers and/or drop inlet structures,
  - Flow over broad crested weirs or emergency spillways through embankments.

The basic stage discharge relationship for basin outlet structures is controlled by weir, orifice or pipe flow.

6.2.1 Weir Flow

6.2.1.1. Broad Crested Weirs

A weir in the form of a relatively long raised channel control crest section is a broad crested weir. The flow control section can have various shapes including circular, rectangular, and triangular. The general equation for a broad-crested weir is:

$$Q = C\,L\,H^{\frac{3}{2}}$$

Where:

Q = Discharge (cfs),
C = Weir coefficient (weir shape dependant, typically between **3.0 and 3.2 for risers**),
L = Weir length (feet), the total length over which flow crosses the weir (circumference of a pipe for circular drop inlets), and
H = Water head (feet).

6.2.1.2. V-Notch Weirs

A weir in the form of a V or a pie shaped cut in a vertical wall is classified as a V-notch weir.

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

Where:

Q = Discharge (cfs),
θ = Angle of v-notch (degrees),
H = Water head on apex of notch (feet).

6.2.2 Orifice Flow

An orifice is an opening of a designed size or shape. A typical orifice is circular or rectangular in shape.

The flow through an orifice is dependent of the height of water above the opening and the size and the edge treatment of the orifice.

$$Q = C' a \left(2gH\right)^{\frac{1}{2}}$$

Where:

Q = Discharge (cfs)
C' = Orifice coefficient
C' = 0.60 for sharp-edged orifices, where the material is thinner than the orifice diameter
C' = 0.80 where the material is thicker than the orifice diameter
C' = 0.92 where orifice edges are rounded.
A = Cross sectional area of the orifice (ft^2)
g = **32.2 ft/sec^2**
H = Head on the orifice (feet)

6.2.3 Pipe Flow

The outlet hydraulics for pipe flow can be expressed by the following equation based on Bernoulli's Equation and continuity principles:

$$Q = \frac{a\left(2gH'\right)^{\frac{1}{2}}}{\left(1 + K_e + K_b + K_c L\right)^{\frac{1}{2}}}$$

Where:

Q = Discharge (cfs),
A = Cross sectional area of the pipe (ft^2),
g = 32.2 ft/sec^2,
H' = Head (feet) defined as the distance from the water surface in the basin to a point **0.6 D** above the invert of the outlet barrel.
D = Outlet barrel diameter in feet,
K_e = Pipe entrance loss (typical value of **K_e = 1.0**),
K_b = Pipe bend loss if there is a bend (typical value of **K_b = 0.5**),
K_c = Head loss coefficient due to friction
K_c = $5087 * n^2 / D^{4/3}$
N = Manning's roughness coefficient of the barrel, and
D = barrel diameter in inches,
L = Total length of the pipe (feet).

6.2.4 Outlet Control Combinations

Any given spillway can have a variety of stage discharge relationships depending on the head. When the water level is just above a riser crest (a very low head on the riser), the riser crest acts like a weir, and flow is weir controlled. As the water level in the basin increases, water begins flowing in from all sides including directly above the inlet, and the inlet begins to act like an orifice. As the head continues to increase, the outlet eventually begins to flow full, and pipe flow dictates. To determine which of the three flow mechanisms is controlling at a particular water level in a storm water management control structure, all three equations should be utilized at each level. The minimum flow for a given stage indicates the actual discharge from the storm water management control structure and the flow mechanism that is controlling at that water level. **Figure 6-2** shows drop inlet control scenarios.

6.3. Hydraulics of Culverts

Culverts are conduits that are commonly used to pass drainage water through embankments.

- The 25-year 24-hour storm event shall be used in the design of all cross-drain culverts.
- The 10-year 24-hour storm event shall be used in the design of all interior culverts.
- The 100-year 24-hour storm event shall be used to check all systems for overtopping, flooding and surcharge.

6.3.1 Culvert Classes

Chow (1959) divided culvert flow into six different categories. Chow indicated that the entrance of an ordinary culvert will not be submerged if the outlet is not submerged unless the head water is greater than some critical value H*. The value of H* may vary from 1.2 to 1.5 time the culvert height as a result of entrance geometry barrel characteristics and approach conditions.

6.3.1.1. Type 1-Outlet Submerged.

The pipe will flow full and the discharge can be calculated from the pipe flow equation in Section 6.2.3.

6.3.1.2. Type 2-Outlet Not Submerged, H > H* Pipe flowing full.

This condition corresponds to a hydraulically long condition. The tailwater depth should be less than the height of the culvert. Discharge can again be calculated by using the pipe flow equation in Section 6.2.3.

6.3.1.3. Type 3-Outlet Not Submerged, H > H*, Pipe not flowing full.

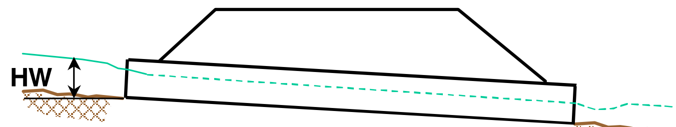
This condition corresponds to the hydraulically short condition. The tailwater depth should be less than the height of the culvert. Discharge is inlet controlled and can be determined form plots and nomographs.

6.3.1.4. Types 4-6 Outlet Not Submerged, H < H*.

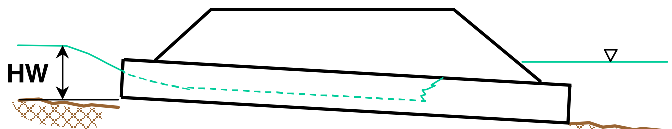
Under these conditions the pipe flows as an open channel. The discharge for a given head depends on the culvert slope entrance geometry culvert roughness and culvert size. A flow profile through the culvert must be developed to accurately predict the discharge. The exact shape of the profile will depend on the depth of the flow at the outlet. The factors that influence energy and hydraulic grade lines are used to determine the type of control for the discharge. The flows through the culvert are full flow partially full flow and free surface flow.

6.3.2 Inlet Control

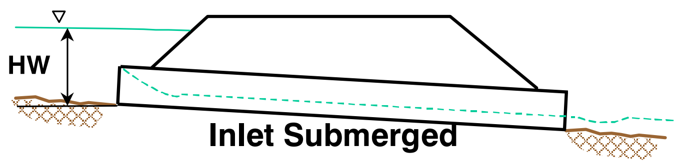
Inlet control occurs when the section that controls flow is located at or near the entrance of the culvert. Discharge is dependent only on the geometry of the inlet and the headwater depth for any particular culvert size and shape. The inlet will continue to control as long as water flowing through the culvert does not impede flow. If inlet control dictates downstream hydraulic factors such as slope culvert length and culvert roughness do not influence the flow capacity. If a culvert is operating under inlet control it will not flow full throughout the entire length of the culvert.



Outlet Unsubmerged



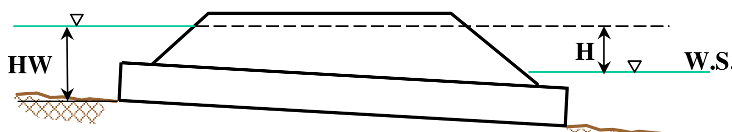
Outlet Submerged, Inlet Unsubmerged



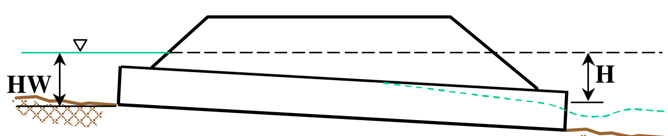
Inlet Submerged

6.3.3 Outlet Control

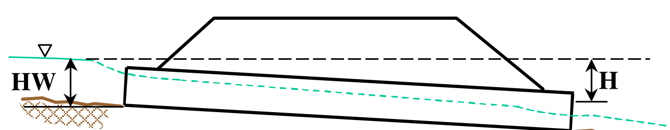
Outlet Control takes place when the control originates at or near the culvert outlet point. Outlet control discharge is dependent on all of the hydraulic factors upstream of the outlet and the tailwater depth.



Full Flow



Partially Full Flow



Free Surface Flow

6.3.4 Critical Depth in Culverts

When the sum of kinetic energy plus the potential energy for a specified discharge is at a minimum the maximum discharge through the culvert occurs with any specified total energy head. For a given flow rate critical flow occurs. During critical flow the depth of flow and slope associated with critical flow define the critical depth and critical slope. If a culvert has an unsubmerged outlet the maximum capacity of the culvert is established when critical flow occurs.

6.3.5 Culvert Selection and Design

Culvert selection techniques can range from solving empirical formulas, to using nomographs and charts, to comprehensive mathematical analysis for specific hydraulic conditions. The many hydraulic factors involved make precise evaluation time consuming and difficult without the help of computer programs and models. The actual models used for these calculations shall be at the discretion of the design professional with approval from the Greenville County Storm Water Plan Review Agency. Applicable computer models include, but are not limited to:

- HY8
- SEDCAD4
- Pond Pack
- HEC-RAS
- Culvert Master

The simple empirical and nomograph methods do not account for all of the factors that impact flow through culverts, but they can be easily used to estimate flow capacities for the conditions they represent.

6.3.6 Culvert Nomograph Procedure

The basic procedure for culvert design using nomographs based on a given flow rate involves selecting a trial culvert size and shape, determining if the culvert flow is classified as being inlet control or outlet control, and then finding the headwater (HW) required for controlling the condition. If the calculated headwater is unacceptable, another trial culvert size or shape may be selected and the process is repeated. The maximum headwater depth under inlet and outlet conditions shall be calculated for each trial culvert size and shape, and the larger of the two represent the controlling condition.

The nomographs referenced in this Design Manual are from the Federal Highway Administration (FHWA) Hydraulic Design Series Number 5 (HDS 5). Similar nomographs from other sources may be used if they are used in a similar manner as the techniques described for the FHWA nomographs.

6.3.6.1 Design Inputs

Culvert nomograph design procedure inputs include:

- Design discharge for design storm Q (cfs)
- Length of culvert L (ft)
- Slope of culvert S (ft/ft)
- Allowable headwater depth (HW_{al}) (= vertical distance from the culvert inlet invert to the maximum water elevation permissible on the upstream side of the culvert)
- Flow velocities or tailwater depth (TW)

- Culvert Shape:
  - Circular
  - Box, Rectangular
  - Elliptical
  - Pipe/Arch
  - Arch

- Culvert Material:
  - Corrugated Metal (CM)
  - Reinforced concrete (RC)

- Culvert entrance type
  - Headwall
  - Wingwalls
  - Projecting from fill
  - Square edge
  - Groove end
  - Chamfered Edge
  - Beveled Edge
  - Skewed

- Select a trial culvert size:

  - Suggest trial size to be a culvert with a diameter (circular) or height (rectangular) equal to 1/2 HW_{al}

6.3.6.2 Inlet Control

Given: Flow (Q), diameter (D) or culvert shape dimensions, entrance type, culvert material, and trial size, select the appropriate control nomograph from Appendix C.

On the appropriate nomograph, connect D or the culvert shape dimensions and Q with a straight line and continue that line to the first HW/D scale, indicated as (1).

Find HW/D scale that represents entrance.

- If required, extend the point of intersect at scale (1) to scales (2) and (3) with a horizontal line (do not follow the slope of the line connecting D and Q).

Multiply HW/D value read from the nomograph by D to find HW_{calc}.

If HW_{calc} < HW allowable, the trial culvert size is OK.

If HW_{calc} > HW allowable, select another trial size and repeat the process.

6.3.6.3 Outlet Control

Given: Flow (Q), culvert diameter or depth (D) or culvert shape dimensions, entrance type, estimated tailwater depth (TW) above outlet invert, and trial culvert size, select control nomograph.

Select entrance coefficient K_e from Table 6-2.

Connect K_e point on length scale to trial culvert size or shape using a straight line.

- Mark the point where the line crosses the “turning line.”

Form a straight line with point on the “turning line” and Q.

Project line to the head scale and read H from the nomograph.

If TW < top of culvert outlet (D),

$$h_o = \frac{d_c + D}{2}$$

Where d_c = critical depth read from nomographs in Appendix C, and D = depth of culvert

- h_o = TW, or
- Use the greatest h_o value calculated.

If TW ≥ top of culvert outlet,

- h_o = TW

Find HW using,

$$HW = H + h_o - S_o L$$

- H is read from applicable nomograph
- h_o dependent on TW
- S_o = culvert slope (ft/ft)
- L = culvert length (ft)

6.3.6.4 Compare Inlet and Outlet Control Headwaters

1. Compare the two headwaters determined for inlet control and outlet control.

2. The higher headwater of the two controls is the flow control existing under the design conditions for the trial culvert size and shape.

6.3.6.5 Outflow Velocity

1. If outlet control exists with tailwater, and the pipe is flowing full,

   - Outflow velocity = Q/A

2. If outlet control exists and the pipe is not flowing full

   - Flow area (A) is based on average flow depth (davg)
   - Average flow depth (davg) calculated by

$$d_{avg} = \frac{d_c + D}{2}$$

dc = critical depth (ft) read from nomographs in Appendix C
D = culvert diameter or depth (ft)

3. If inlet controls exists,

- Outflow velocity is approximated assuming open channel flow.
- Velocity calculated using Manning's open channel flow equation.

Table 6-2 Culvert Entrance Loss Coefficients $H_e = K_e [V^2/2g]$

Type of Structure and Design of Entrance	Coefficient K_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove end)	0.2
Square-edge	0.5
Rounded (radius = D/12)	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side or slope tapered inlet	0.2
Pipe, or Pipe-Arch. Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side or slope tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of D/12 or B/12 or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of D/12 or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	

Type of Structure and Design of Entrance	Coefficient K_e
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side or slope tapered inlet	0.2

Note: "End Sections conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

6.3.7 Culvert Nomograph Example Problem

Given: Design flow (Q) = 200 cfs
Culvert length (L) = 180 ft
Culvert Slope (So) = 0.02 ft/ft
Allowable head (HWal) = 10 ft
Tailwater depth (TW) = 4 ft
Culvert material = RCP
Entrance type = Projecting

Find: Diameter for a circular culvert.

Solution:

Trial Size:

$D = HW_{al} /2 = 10/2 = 5ft$

Inlet Control

1. Select control nomograph for circular concrete pipes with projecting entrance.

2. On the nomograph, connect D = 5-ft. (60-inches) and Q = 200 with a straight line and continue the line to the first HW/D scale, indicated as (1).

3. Find HW/D scale that represents a projecting entrance.

- Extend the point of intersect at scale (1) to scale (3) with a horizontal line.
- **HW/D = 1.37**

4. Multiply HW/D value read from the nomograph by D to find $HW_{calc.}$

- **HW/D * D = 1.37 * 5 = 6.85-feet.**

5. HW_{calc} < HW allowable, therefore the trial culvert size is OK.
   6.85 ft < 10 ft

Outlet Control

<u>**Given**</u>: Flow (Q) = 200, culvert diameter (D) = 5 ft, groove end projecting, and tailwater depth (TW) = 4 select outlet control nomograph for circular concrete pipes with projecting entrance.

Entrance coefficient from Table 6-2 for projecting from fill, socket end (groove-end) gives:

K_e = 0.2.

Connect K_e = 0.2 length scale L =180-feet, to trial culvert size D = 60-inches using a straight line and mark the point where the line crosses the "turning line."

Form a straight line with point on the "turning line" and Q.

Project line to the head scale and read H = 2.80.

TW (4 ft) < top of culvert outlet (5 ft)

- **h_o = 4 ft**, or

$$h_o = \frac{d_c + D}{2}$$

Where critical depth read from Appendix C is **d_c = 4.10 ft**

D = depth of culvert = 5-feet
h_o = (4.1 + 5) /2 = **4.55-feet**

- Use the greatest ho value calculated

<u>**h_o = 4.55 ft**</u>

If TW ≥ top of culvert outlet, ho = TW
Find HW using

$$HW = H + h_o - S_o L$$

- H = 2.80 ft read from graph
- ho = 4.55 ft
- So = 0.02 (ft/ft)
- L = 180-feet.

HW = 2.8+4.55 – 0.02(180) = **3.75-feet**

Compare Inlet Control and Outlet Control Headwaters

Compare the two headwaters determined for inlet control and outlet control.

Inlet Control	HW = 6.85-feet.
Outlet Control	HW = 3.75-feet.

The higher headwater of the two controls is the flow control existing under the design conditions for the trial culvert size.

Therefore, Inlet Control prevails and the maximum headwater located upstream of the culvert shall = **6.85-feet.**

6.4. Stage Discharge Equations for Rock Structures

Rock structures are commonly used as the outlet control structure of smaller sediment basins and sediment traps and in rock ditch checks. The flow through these structures is controlled by the following factors See **Figure 6-3**

- Static head drop as flow moves through the rockfill **(dh),**
- Upstream water depth **(h_1),**
- Downstream water depth **(h_2),**
- Flow length through the rockfill **(dl)** ,
- Average stone diameter of the rockfill **(d),**
- Porosity of the rockfill (**ξ**) (0.46 for graded rockfills constructed by dumping), and
- Reynolds Number (**R_e**) and friction factor (**f_k**), which are dictated by flow length through the rockfill, rock size, and porosity of the rockfill.

In the original equations proposed by Herrera (1989), porosity was included as a parameter. However, Herrera and Felton (1991) deleted porosity from the equations because it was found to have a constant value of approximately 0.46 in all of their laboratory tests. The Herrera and Felton equations require a trial and error computation process that requires six steps for each stage.

6.4.1 Calculating the Stage-Discharge Relationship for Rockfill Structures

The Herrera and Felton equations incorporate detailed computations requiring computers and spreadsheets that are capable of trial and error programming. However, when quick estimates are needed, graphical procedures are helpful. A graphical procedure for predicting the average gradient through rockfills **(dh /dl)** can be used to develop head loss as a power function of flow, which eliminates any trial and error procedures. The governing equation is:

$$\frac{dh}{dl} = aq^{b}$$

dh = Static head drop of water in meters (difference between upstream and downstream water surface elevations)
dl = Average flow path length through the rock in meters,
a = Dimensionless coefficient based on flow path length shown in **Figure 6-3**,
b = Dimensionless exponent based on average rock diameter (m) shown in **Figure 6-3**; and,
q = Flow per unit width of rockfill in cubic meters per second per meter (cms/m).

****All units must be converted to metric to use the graphical method.**

The equation can be rearranged so there is only one unknown, **q (csm/m)**.

$$q = \left[\frac{dh}{a(dl)}\right]^{\frac{1}{b}}$$

6.4.2 Flow Through a Rockfill Dam Example

Given: A rockfill dam is to be used as the principle spillway for a sediment trap. The average width of the dam is 10-feet (3 meters). The dam is 5-feet high with rock side slopes of 1:1. The flow length at the top of the dam is 3.3-feet, while the flow length at the bottom of the dam is 9.9-feet. The average stone diameter is 6-inches.

Find: Stage discharge relationship for the rock dam. Assume that the downstream depth is negligible so **dh** = upstream stage (Figure 6-3).

Solution:

Determine the number of desired stage elevations for computations.

- For this example calculations will be made every 1-foot.

Set up a table for each stage. (as shown below)

- Convert all units to **metric** before reading values from the graphs in Figure 6-4.

Calculate the discharge rate at each stage.

- At a stage = 1 foot

dh = 0.31 meters
dl = 3.0 meters
stone diameter of 6-inches = 0.15 meters
a = 1.80 (from Figure 6-4)
b = 0.6657 (from Figure 6-4)

$$q = \left[\frac{dh}{a\ (dl)}\right]^{\frac{1}{b}} = \left[\frac{0.31}{1.80 * (3.0)}\right]^{\frac{1.0}{0.6657}} = 0.0137 \quad cms\ /\ m$$

- Convert cms/m to cms by multiplying by the average flow width at the stage

(0.0137 cms/m) * (3 m) = **0.041 cms**

- Convert cms to cfs

(0.041 cms) * (35.315 cfs/cms) = 1.447 cfs

Stage (ft)	Flow Length (ft)	Flow Width (ft)	dh Stage (m)	dl Flow Length (m)	a	b	Flow cms/m	Flow Width (m)	Flow cms	Flow cfs
0	9.9	10	0.00	3.00	3.12	0.6657	0.0000	3.0	0.000	0.0
1	9.9	10	0.31	3.00	3.12	0.6657	0.0137	3.0	0.041	1.4
2	8.3	10	0.61	2.52	2.06	0.6657	0.0407	3.0	0.122	4.4
3	6.6	10	0.91	2.00	2.31	0.6657	0.0870	3.0	0.261	9.2
4	5.0	10	1.22	1.52	2.97	0.6657	0.1400	3.0	0.420	14.8
5	3.3	10	1.52	1.00	3.63	0.6657	0.2704	3.0	0.811	28.7

6.4.3 Flow Through a Rock Ditch Check Example

Given: A Rock Ditch Check with the following characteristics:

Dimension	Standard Units	Metric Conversion
Bottom Width	3-feet	0.91 meters
Side Slopes	3:1	3:1
Depth	2-feet	0.61 meters
Top Width	15-feet	4.57 meters
Top Flow Length	3-feet	0.91 meters
Bottom Flow Length	7-feet	2.13 meters
Rock Fill Diameter	6-inches	0.15 meters

Find: Stage discharge relationship for Rock Ditch Check.
Solution:
1. To properly apply the rock fill flow equation all values must be converted to **metric units.**

2. Determine the number of desired stage elevations for computations.
- For this example calculations will be made every 0.5-feet.

Based on the rock size and the flow lengths, an appropriate value for the exponent *b* must be selected from Table 6-3.

- Linear interpolation can be used to find *b* when the rock diameter = 0.15 m.

b = 0.6651 + [(.15 - .10) / (.20-.10)] * (0.6662 – 0.6651)
b = 0.6657

Based on a rock size of 0.15 meters and the flow lengths at different stages, the appropriate values for the coefficient a can be selected from Table 6-3 by using linear interpolation.

Stage (ft)	Stage (m)	Flow Length (m)	Coefficient a
0.0	0.00	2.13	2.26
0.5	0.15	1.83	2.55
1.0	0.31	1.52	3.00
1.5	0.46	1.22	3.37
2.0	0.61	0.91	3.67

Table 6-3. Values for Rock Check Flow Coefficient and Exponent

Stone Diameter(m)	Exponent b	Coefficient a dl= 1m	dl = 2m	dl = 3m
0.01	0.6371	9.40	6.05	4.60
0.02	0.6540	7.40	4.65	3.55
0.03	0.6589	6.40	4.08	3.08
0.04	0.6609	5.85	3.65	2.80
0.05	0.6624	5.40	3.35	2.60
0.06	0.6635	5.05	3.15	2.40
0.08	0.6644	4.50	2.85	2.20
0.09	0.6648	4.28	2.70	2.10
0.10	0.6651	4.13	2.60	2.05
0.20	0.6662	3.20	2.05	1.57
0.30	0.6664	2.80	1.75	1.30
0.40	0.6665	2.50	1.55	1.16
0.50	0.6666	2.30	1.40	1.08

Source: Design Hydrology and Sedimentology for Small Catchments, Hann et. al., 1995

1. Determine the total flows for each staging using the values determined above. The total flow is computed by multiplying the unit flow by the flow width.

$$q = \left[\frac{dh}{a\,(dl)}\right]^{\frac{1}{b}} = \left[\frac{0.61}{3.67 * (0.91)}\right]^{\frac{1.0}{0.6657}} = 0.0778\ cms/m$$

- At the stage of 2-feet (0.61 meters) the flow is calculated to be:

- Convert cms/m to cms by multiplying by the average flow width at the stage
  (0.0778cms/m) * (4.57 m) = **<u>0.355 cms</u>**

- Convert cms to cfs
  (0.355 cms) * (35.315 cfs/cms) = **<u>12.5 cfs</u>**

Stage (m)	Flow Length (m)	Unit Flow (cms/m)	Flow Width (m)	Total Flow (cms)	Stage (ft)	Total Flow (cfs)
0.00	2.13	0.000	0.91	0.000	0.0	0.0
0.15	1.83	0.006	1.83	0.011	0.5	0.4
0.31	1.52	0.018	2.74	0.048	1.0	1.7
0.46	1.22	0.037	3.66	0.136	1.5	4.8
0.61	0.91	0.078	4.57	0.355	2.0	12.5

6.5 Storm Drainage Design Requirements

This section provides the design requirements for various storm water drainage system components including:

- Design storms,
- Design velocities; and,
- Design pipe sizes.

6.5.1 Storm Drainage Systems

Storm drainage systems shall include all storm drainage structures and pipes that do not convey runoff under public roadways. These systems are commonly referred to as lateral closed systems.

6.5.1.1 Design Storm Requirements

These storm drainage systems shall be designed based upon the following criteria:

- 10- to 25- year 24-hour design storm event capacity for pipe design.
- 10- to 25- year 24-hour design storm event capacity for inlet structure design.
- 25- year 24-hour design storm event capacity for drainage channels.
- 50-year 24-hour design storm event capacity for sump inlets, unless overflow facilities are designed.
- 100-year 24-hour storm event shall be used to check all drainage designs using for local flooding, and possible flood hazards to adjacent structures and/or property.
- The rational method and SCS method for peak runoff flow rates are acceptable under the conditions outlined in Sections 5.2.1 and 5.2.2.

6.5.1.2 Manning's Equation to Determine Flow Capacity

When a storm drainage system has less than 20 connections, Manning's Equation shall be acceptable for sizing the capacity of drain pipes for non submerged conditions where the free water surface elevation is below the crown of the pipes.

6.5.1.3 Hydraulic Grade Line

6.5.1.3.1 Requirements

All head losses in a storm drainage system shall be considered when computing the hydraulic grade line to determine water surface elevations under design conditions.

Any system that contains 20 or more pipe connections shall have the hydraulic grade line computed, along with all head losses through the entire system.

If the outlet is submerged in a backwater condition, a more sophisticated design methodology than Manning's Equation shall be required. Individual head losses in the pipe systems shall be calculated. These head losses are added to a known downstream water surface elevation to give a design water surface elevation for a given flow at a desired upstream location. Various accepted computer models are available for analysis of storm drain systems under backwater and/or pipe flow surcharge conditions.

6.5.1.3.2 Head Loss

Total head losses in a pipe system shall be determined as follows:

$$H_T = H_f + H_{MH} + H_V + H_J$$

Where:

$\mathbf{H_T}$	=	Total head loss (ft.)
$\mathbf{H_f}$	=	Friction head loss (ft.)
$\mathbf{H_{MH}}$	=	Manhole head loss (ft.)
$\mathbf{H_V}$	=	Velocity head loss (ft.)
$\mathbf{H_J}$	=	Junction head loss (ft.)

Frictional losses may be computed from Manning's Equation expressed as:

$$H_f = L \frac{(nV)^2}{2.221\, R^{\frac{4}{3}}}$$

Where:

$\mathbf{H_f}$	=	Friction head loss (ft.)
L	=	Length of pipe section (ft.)
n	=	Manning roughness coefficient
V	=	Average flow velocity (ft/sec)
R	=	Hydraulic radius (ft.), Calculated to be A/P where: A = flow cross sectional area (ft.^{2}) P = wetted perimeter (ft.) (length of boundary between water and channel)

The remaining components of the total head loss (H_T) may be computed using standard equations, charts, tables or may estimated by using graphical procedures.

6.5.1.4 Pipe Size

The minimum pipe size to be used in storm drainage systems is 15-inches in diameter

6.5.1.5 Flow Velocity and Pipe Slope

- The minimum design velocity for pipe flow shall be 2.0-feet/sec at the design flow or 2.5-feet/sec at full flow, whichever requires the greater slope.
- The maximum design velocity shall be 20-feet/sec.
- The minimum slope of storm drain systems shall be 0.5 percent.
- Storm drainage systems shall be designed to convey storm water runoff by gravity flow unless otherwise approved.

For very flat flow lines, flow velocities shall increase progressively throughout the system. Upper reaches of the pipe system may have flatter slopes than the lower end of the system. Progressively increasing slopes keep solids moving toward the outlet and inhibit the settling of particles.

The minimum required slope shall be calculated by a modified form of Manning's Equation.

$$S = \frac{(nV)^2}{2.208\ R^{\frac{4}{3}}}$$

Where:

S = Minimum slope of the hydraulic grade line (ft/ft)
n = Manning's roughness coefficient
V = Average flow velocity (ft/sec)
R = Hydraulic radius (ft.), Calculated to be A/P where:
A = flow cross sectional area (ft.^{2})
P = wetted perimeter (ft.) (length of boundary between water and channel)

6.5.1.6 Fill Requirements

The minimum fill cover on all pipes shall be 1 foot. The maximum cover shall be based on the design loads which are calculated from pipe shape, pipe size, pipe material and location.

6.5.1.7 Catch Basin and Inlet Design

The design methodology utilized to compute the capacity of storm drain inlets and grates shall apply the weir, orifice and pipe flow characteristics as outlined in Section 6.2. The following design requirements shall be followed:

- Inlets shall be designed to convey the 10-year 24-hour storm event.
- The maximum depth in which the water may pond above or around an inlet must not threaten surrounding permanent structures or public facilities including vehicular or pedestrian traffic.

- Inlets placed in sump conditions shall have emergency overflow points designed.

- Inlets placed in roadway gutter lines must be spaced to prevent flow from entering public road intersections.

    - Maximum spread of 6-feet in the travel lane.
    - Valley gutter shall have a maximum allowable spread of 7-feet.
    - Standard 2-feet 6-inch curb and gutter is allowed a total maximum spread of 8-feet from the face of the curb.

- In depth design procedures for inlet design may be referenced in the American Association of State Highway and Transportation Officials (AASHTO) Model Drainage Manual, 1991.

6.5.2 Roadway Culvert Design

Roadway culvert design shall include all cross drainage facilities that transport storm water runoff under roadways. These systems shall be designed based upon SCDOT requirements where applicable. For non-SCDOT roads, the following criteria shall be followed:

- All cross-drain culverts shall be designed to pass the 25–year 24-hour design storm event without overtopping the road.

- All interior culverts shall be designed to pass the 10–year 24-hour design storm event without overtopping the road.

- Additional hydraulic capacity shall be required as necessary to prevent backwater effects that may adversely impact upstream property or structures.

- Refer to Section 6.3 for further details on culvert design.

6.6 Open Channel Design

6.6.1 General Requirements

Open channels shall include all permanent storm drainage channels including swales and diversions. These storm drainage systems shall be designed based upon the following criteria:

- Channels shall be designed to carry the 25-year 24-hour design storm event.

- Major channels may be designed for greater storm frequencies if directed by Greenville County.

- The minimum channel slope shall be 0.5 percent, unless supporting calculations show that there will be no pools or standing water areas formed in the channels at smaller slopes.

- Except for roadside ditches, the side slopes of grassed lined channels without Erosion Control Blankets or Turf Reinforcement Matting shall be no steeper than 3H to 1V.

- Manning's Equation (as described in Section 6.1.5) may be used to design open channels and swales where backwater effects created from obstructions and/or tailwater is not present.

- Channels may be designed with multiple stage levels with a low flow section to carry the 2-year storm event and a high flow section to carry storms of larger frequencies.

- Maximum flow velocities shall be determined based on the channel bottom material and bank slope material. Table 6-5 contains an expanded list of permissible velocities for various different types of channel vegetation and slopes.

6.6.2 Vegetated Channel Design

6.6.2.1 Background

The allowable velocities and tractive forces for non-vegetated (erodible) channels are relatively small and the design requires wide, shallow channels to carry the design flow rates. Vegetation protects the channel material from the erosive action of design flows and binds the channel material together. Vegetated channels can be used to carry storm water runoff but are generally not recommended to carry sustained base flows because most vegetation cannot survive continual submergence or saturation of the root zone.

The design of vegetated channels is more complex than a basic earth lined, or structurally lined channel. The additional design consideration for vegetated channels involves a variation in roughness (Manning's n) with the height and type of vegetation. Generally, a tall grass provides much resistance when flow in the channel is shallow. As the flow depth increases, the resistance of some vegetation may decrease. In many cases, the vegetation will lay over in the direction of the flow when the flow reaches a sufficient depth. When vegetation lies over, the resistance produced by the vegetation is considerably less than it is during shallow flow conditions.

The design of vegetated channels shall be performed for the following two design conditions:

<u>Stability/Permissible Velocity</u>: This design process involves evaluating how the channel will respond under low vegetation retardance conditions. This condition is defined when vegetation is cut low or lies down, producing a lower Manning's n value, lower flow depths, and higher flow velocities. The limiting factor for stability design is the permissible velocity of the flow in the vegetated channel.

<u>Capacity</u>: This design process involves evaluating how the channel will respond under high vegetation retardance conditions. This condition is defined when vegetation is not maintained or is very long and rigid, producing a higher Manning's n value, higher flow depths, and lower flow velocities. The limiting factor for capacity design is the cross sectional area of the vegetated channel.

The design of vegetated channels may be done using the techniques discussed in this section, or by using computer software that is capable of designing for stability and capacity.

6.6.2.2 Vegetation Retardance Classes

Vegetation used for channel design has been divided into five retardance classes designated as being A, B, C, D, and E. Tables 6-6 and 6-7 lists each vegetation retardance class with corresponding species and stand heights. If a particular vegetation type is not listed in Table 6-4, similar vegetation shall be used in determining the retardance class. If the vegetation will be mowed part of the time and long at others, both conditions and retardance classes must be considered.

6.6.2.3 Manning's n Relation to Vegetation

Manning's n value can be related to the product of the flow velocity and the hydraulic radius of the channel, **vR**. Different types of vegetation have different Manning's n to **vR** relationships. These relationships are shown in Figure 6-5. This relationship can be expressed using the following equation:

$$n = \exp\left[I\left(0.01319\ \ln(vR)^2 - 0.09543\ \ln(vR) + 0.2971\right) - 4.16\right]$$

Where:

- **n** = Manning's roughness coefficient.
- **I** = Coefficient based on Retardance Class as shown in Table 6-4.
- **vR** = Calculated value of **vR**.
- **v** = velocity (ft./sec)
- **R** = hydraulic radius (ft.)

Table 6-4. Retardance Class Coefficient I

Retardance Class	Coefficient I
A	10.000
B	7.643
C	5.601
D	4.436
E	2.876

Source: Design Hydrology and Sedimentology for Small Catchments, Hann et. al., 1995

6.6.2.4 Stability/Permissible Velocity Design

The following design parameters are required when designing a vegetated channel based on stability:

- Calculate the required discharge (Q) for the design storm using the procedures in Section 5.2.
- Determine the channel bottom slope (S).
- Select channel cross section dimensions including bottom width, depth, side slopes, and top width.
- Select the type of vegetation to be placed in the channel.
- Determine the retardance class of the vegetation.
  - When designing based on stability, the lowest applicable retardance class should be used.
  - Retardance class D is recommended for maintained permanent vegetation.
  - Retardance class E is recommended for temporary vegetation.
- Determine the permissible velocity (Vp) based on soil type, vegetation and slope from Table 6-5.

Once the design parameters have been determined, design the vegetated channel by using the following steps.

1) Assume a trial depth (d_t).

2) Knowing the channel dimensions, calculate the corresponding trial hydraulic radius (Rtrial) for this assumed depth using:

$$R_{trial} = \frac{bd_t + Zd_t^{\,2}}{b + 2d_t\sqrt{Z^2 + 1}}$$

Where:

R_{trial} = Hydraulic radius (ft.)
b = Known bottom width of channel (ft.)
Z = Known side slopes of channel
d_t = Trial depth of flow in channel (ft.)

3) Divide the known design flow rate (Q) by the vegetated channel cross sectional area to obtain a velocity (V).

4) Multiply the velocity (V), calculated in Step 3, by the trial hydraulic radius (Rtrial) calculated in Step 2 to obtain a value for vR.

5) Using the calculated vR value in Step 4, and the known Retardance Class of the vegetation, read the corresponding Manning's n value from **Figure 6-5** or calculated from Equation 6.6.2.3.

6) Using Manning's Equation, calculate the flow velocity in the vegetated channel by using the trial hydraulic radius (Rtrial) calculated in Step 2, the known channel slope, and the Manning's n value calculated in Step 5.

$$V_{calc} = \frac{1.49}{n} R_{trial}^{\frac{2}{3}} \quad S^{\frac{1}{2}}$$

Where:

V_{calc} = Calculated velocity (ft/sec),
R_{trial} = Trial hydraulic radius (ft.) from Step 2,
n = Manning's n determined from Step 4, and
S = Slope of channel (ft/ft).

7) Compare the velocity values calculated in Step 3 and Step 6. If the values do not match within ± 0.1, return to Step 1 and repeat the process. When the values do match, the matching value must be less than the permissible velocity (Vp) to be acceptable. If the matching velocity value is greater than the permissible velocity (Vp), then the channel bottom width and/or side slopes must be adjusted accordingly.

If the final trial depth is greater than the allowable depth of the channel, the channel dimension must be altered so the vegetated channel has enough capacity to handle the peak flow rate (**Q**).

6.6.2.5 Capacity Design

Once the design for stability has been completed, the channel must be checked to see if it has enough capacity to handle flows when the vegetation moves into a different Retardance Class. The following steps shall be executed:

1. Assume a trial depth (d_t) that is greater than the final flow depth calculated from the Stability Design in Section 6.6.2.4.

2. Knowing the channel dimensions, calculate the corresponding trial hydraulic radius (Rtrial) for this assumed depth using:

$$R_{trial} = \frac{bd_t + Zd_t^{\,2}}{b + 2d_t\sqrt{Z^2 + 1}}$$

Where:

R_{trial} = Hydraulic radius (ft.)
b = Known bottom width of channel (ft.)
Z = Known side slopes of channel
d_t = Trial depth of flow in channel (ft.)

3. Divide the known design flow rate (Q) by the vegetated channel cross sectional area to obtain a velocity (v).

4. Multiply the velocity (v), calculated in Step 3, by the trial hydraulic radius (Rtrial) calculated in Step 2 to obtain a value for vR.

5. Select the desired Retardance Class for the vegetation from Tables 6-6 and 6-7. It is recommended that the minimum Retardance Class utilized for capacity design be Retardance class C.

6. Using the calculated vR value in Step 4, and the known Retardance Class of the vegetation, read the corresponding Manning's n value from **Figure 6-5** or calculate it using Equation 6.6.2.3.

7. Using Manning's Equation, calculate the flow velocity in the vegetated channel by using the trial hydraulic radius (Rtrial) calculated in Step 2, the known channel slope, and the Manning's n value from Step 6.

8. Compare the velocity values calculated in Step 3 and Step 7. If the values do not match, return to Step 1 and repeat the process. If the trial depth is determined to be greater than the depth of the channel, the channel dimension must be altered so the vegetated channel has enough capacity to handle the peak flow rate (Q).

Table 6-5. Maximum Permissible Velocities for Vegetated Channels

	Permissible Velocity (ft./sec.)*					
	Erosion Resistant Soils % Slope			Easily Eroded Soils % Slope		
Cover	0-5	5-10	> 10	0-5	5-10	> 10
Bermuda Grass	8	7	6	6	5	4
Bahia Buffalo Grass Blue Gamma Centipede Grass Tall Fescue Kentucky Bluegrass Red Canary Grass	7	6	5	5	4	3
Grass-legume Mixture	5	4	NR	4	3	NR
Lespedeza Sericea Weeping Lovegrass Kudzu Alfalfa Small Grains Temporary Vegetation	3.5	NR	NR	2.5	NR	NR

* Allow velocities over 5 ft/sec only where good cover and maintenance will be provided. If poor vegetation exists due to shade, climate, soils or other factors, the permissible velocity shall be reduced by 50 percent

NR = Not Recommended

Sources: Elementary Soil and Water Engineering, Shwab et. al.
Design Hydrology and Sedimentology for Small Catchments, Hann et. al., 1995

Table 6-6. Vegetated Retardance Classes based on Vegetation

Retardance Class	Vegetation	Condition
A	Red Canary Grass Weeping Lovegrass Yellow Bluestem Ischaetum	Excellent stand, tall (average 36-inches) Excellent stand, tall (average 30-inches) Excellent stand, tall (average 36-inches)
B	Bermuda Grass Native grass mixtures Tall fescues Lespedeza Sericea Grass-legume Mixture Red Canary Grass Alfalfa Weeping Lovegrass Kudzu Blue Gamma	Good stand, tall (average 12-inches) Good stand, uncut Good stand, uncut (average 18-inches) Good stand, not woody, tall (average 19-inches) Good stand, uncut (average 20-inches) Good stand, mowed (average 12- to 15-inches) Good stand, uncut (average 11-inches) Good stand, uncut (average 13-inches) Dense growth, uncut Good stand, uncut (average 13-inches)
C	Bahia Crabgrass Bermuda Grass Common Lespedeza Grass-legume Mixture Centipede Grass Kentucky Bluegrass	Good stand, uncut (6- to 8-inches) Fair stand, uncut (10- to 48-inches) Good stand, mowed (average 6-inches) Good stand, uncut (average 11-inches) Good stand, uncut (6- to 8-inches) Very dense cover (average 6-inches) Good stand, headed (6- to 12-inches)
D	Red Fescue Bermuda Grass Common Lespedeza Buffalo Grass Grass-legume Mixture Lespedeza Sericea	Good stand, uncut (3- to 6-inches) Good stand, cut to 2.5-inches Excellent stand, uncut (average 4.5-inches) Good stand, headed (12- to 18-inches) Good stand uncut (4- to 5-inches) Very good stand, mowed (2-inches)
E	Bermuda Grass Bermuda Grass	Good stand, cut to 1.5-inches Burned Stubble

Table 6-7. Vegetated Retardance Classes based on Stand and Vegetation Height

Stand	Height of Vegetation (inches)	Retardance Class
Good	> 30 11-24 6-10 2-6 < 2	A B C D E
Fair	> 30 11-24 6-10 2-6 < 2	B C D D E

Source: Soil Conservation Service Engineering Field Manual, 1979