CITY OF DELAWARE OHIO

Chapter 15: Storm Water Management

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PART 1 – GENERAL REQUIREMENTS

This chapter of the Infrastructure Design Manual is not a text of hydrology or hydraulic design. It assumes that the user has an understanding of hydrology and hydraulic engineering. This manual provides a uniform design procedure for summarizing and submitting plans in an acceptable and understandable manner. It does not provide uniform solution to all drainage problems. This section of the design manual provides sufficient information to develop drainage systems in accordance with city policy.

1. Summary of Stormwater Requirements

A. Submittal and General Planning Requirements

Requirements for submittal and approval of stormwater management plans can be found in Part 1 of this chapter.

B. Hydrology

Acceptable hydrologic methodologies and requirements can be found in Part 2 of this chapter.

C. Water Quantity

Requirements for water quantity control, also known as peak runoff control, can be found in Part 3 of this chapter. The City uses the Critical Storm method for determining allowable discharge rates and detention or retention volumes.

D. Water Quality

Requirements for water quality treatment are found in Chapter 16 of this manual. The City follows the Ohio Environmental Protection Agency (OEPA) construction stormwater permit requirements for post-construction water quality treatment. City-specific requirements that differ from the OEPA construction stormwater permit are also found in Chapter 16.

E. Sediment and Erosion Control During Construction

Requirements for sediment and erosion control during construction can be found in Chapter 16 and Chapter 17 of this manual. The City follows the Ohio Environmental Protection Agency (OEPA) construction stormwater permit requirements for erosion and sediment control during construction.

F. Design of Minor Stormwater Conveyances

Hydraulic design requirements for minor stormwater conveyances, such as storm sewers, culverts, open water courses, and flood routing can be found in Part 4 of this chapter.

2. Preliminary Stormwater Management Plans

A. General

The City of Delaware requires a preliminary stormwater management plan to be submitted for review showing basic hydrology prior to submittal of the final Stormwater Plan. This requirement may be waived at the discretion of the City upon written request by the design engineer for commercial outlots that are a part of a larger development and incorporated in an existing approved Stormwater Management Plan.

All Runoff Curve Numbers are to be per appropriate tables contained within this manual and using best engineering judgment and are subject to review by the Public Works Department.

It is highly recommended that the design engineer contact the Public Works Department as early in the design process as practicable. The purpose of this meeting is to come to an agreement on methodology, Runoff Curve Numbers, site specific constraints and other pertinent details before the Design Engineer invests a significant amount of time and resources in a design that will ultimately be un-approvable.

Centralized storm water detention is recommended and strongly encouraged for all developments over one (1) acre in size. This may include but is not limited to retention and detention basins, parking lot detention, bio-swales, underground detention, etc. All storm water best management practices (BMPs), layout, and location(s) shall also be considered with the preliminary stormwater management plan, in order to meet necessary requirements, specifically for basins.

B. Preliminary Stormwater Plan Content

The City of Delaware requires that a preliminary storm water management plan for a proposed development site be submitted to the Public Works Department for review and approval prior to proceeding with a full storm water management plan design. The preliminary review material must include:

1. Project Name & Location
2. Development Owner/Address/Telephone/email
3. Engineer/Address/Telephone/email
4. Topographic tributary area(s) map for both the entire site and offsite areas tributary to the property
5. Conceptual development site layout
6. Existing County and local field tile systems and drainage ditches as identified on current Delaware Water and Soil Conservation District office records.
7. Total property acreage & proposed pre-developed runoff coefficients
8. Proposed post development runoff coefficients
9. Identify proposed major flood routing, storm sewer outfall points, and off-site improvements
10. Identify downstream capacity concerns or limitations to the existing system
11. Critical storm calculation, pre-developed 1-year and 100-year flows
12. Required detention volumes for 1-year, critical, and 100-year events
13. Design methodology being implemented
14. Proposed detention and/or retention basin locations
15. Identify all EPA and Army Corp of Engineers permits applicable to the project.

3. Final Storm Water Management Plan

Final Stormwater Management Plans shall have the following content:

A. Existing Conditions Tributary Map

1. ANSI D bond paper
2. Existing contours
3. Delineated Existing Tributary Area
4. On site acreages for each tributary area
5. Offsite acreages for each tributary area
6. Flow line with sheet, shallow concentrated and channel flow areas labeled along with the slope of each section of the flow line and Time of Concentration for each tributary area
7. Label existing outfalls for each area

8. Existing storm structures and other infrastructure such as roads, buildings, sidewalks, parking lots, etc. that will impact the stormwater runoff.
9. RCN or C values for each area
10. The one year flow generated by each tributary area
11. Existing Field Tiles per SCS, ODNR, County Records

B. Proposed Conditions Tributary Map

1. ANSI D bond paper
2. Proposed and existing contours as appropriate
3. Delineated Proposed Tributary Area and offsite areas that are passing through the site
4. On site acreages for each tributary area
5. Offsite acreages for each tributary area
6. Flow line with sheet, shallow concentrated, pipe flow and channel flow sections labeled along with the slope of the flow line and time of concentration for each tributary area.
7. RCN or C values for each area
8. Detention/Retention Basin locations
9. Label proposed outfalls for each area
10. Existing and proposed storm structures as well as other infrastructure; such as roads, buildings, sidewalks, parking lots, etc.
11. Existing Field Tiles per SCS, ODNR, and/or County Records and details of the tie in points to new structures

C. Soils Map

1. ANSI D bond paper
2. Soil boundary line, soil type, and hydrological group
3. Site boundaries

D. Proposed Storm Structure Tributary Map

1. ANSI D bond paper
2. Proposed storm structures as well as other infrastructure such as roads, buildings, sidewalks, parking lots, etc.
3. Delineated tributary area to each structure
4. Structure tributary acreages
5. Structure number
6. Composite C values or C values from chart for each tributary area
7. Tc for each area
8. Use 10 minutes to inlet for paved areas and 15 minutes to inlet for grassy areas for Tc to structures taking in onsite flow.
9. Structures taking in offsite flow will use the Tc generated during the existing tributary analysis
10. Existing storm structures

E. Report Body

The design engineer should provide a written general summary of the stormwater management plan. The body of the report should have a minimum of seven (7) sections. More may be needed on a case by case basis.

1. Site Description
2. Hydrologic Methodologies used to determine the runoff
3. Existing Hydrologic conditions
4. Developed Hydrologic conditions
5. Storm water management plan – how the runoff is being moved through the site
6. Applicable permits
7. Calculation sheets and hydrograph printouts

F. Report Body Tables

1. The design engineer should provide a summary table for all onsite and offsite tributary areas for the pre-developed conditions in the Existing Conditions Section of the report. Use the following format:

Tributary Area Name	Area	CN	Tc	Q1	Q2	Q5	Q10	Q25	Q50	Q100

The design engineer should provide a summary table for all onsite and offsite tributary areas for the post-developed conditions in Developed Conditions Section of the report. Use the following format:

Tributary Area Name	Area	CN	Tc	Q1	QCrit	Q50	Q100

2. The design engineer should provide a summary table for all allowable release rates for each area. Use the following format:

Tributary Area Name	Q1-allowable	Q2-allowable	Q5-allowable	Q10-allowable	Q25-allowable	Q50-allowable	Q100-allowable

3. The design engineer should provide a table for each basin showing the required detention volumes and peak Water Surface Elevation for each named storm event in Developed Conditions Section of the report. Use the following format:

Storm Event	Water Surface Area (acre **or** sq ft)	Peak Water Surface Elevation (feet)	Detention Storage Volume (ac-ft **or** cubic feet)
1-Yr			
2-Yr			
5-Yr			
10-Yr			
25-Yr			
50-Yr			
100-Yr			

G. Calculations and Exhibits

The calculations and exhibits need to be arranged so an accurate comparison of the pre-developed and post-developed conditions can be accomplished. A partial list of calculations that may be required follows. All of these calculations may not apply to every project. Appropriate calculations to document and justify the engineering design of the project shall be submitted for review. The calculations shall be signed and sealed by the Design Engineer.

1. Composite C and/or RCN numbers - Documenting existing and design C and RCN values
2. MORPC Critical Storm Calculations
3. Time of Concentration Calculations- Documenting Existing and Design time of concentration
4. Storage Calculations indicating the amount of storage required and amount provided
5. Water Quality Volume Calculations per OEPA NPDES permits.
6. Downstream Capacity Calculations identifying any downstream “choke points” and verifying capacity of downstream systems as determined at the pre-design meeting for the Preliminary Stormwater Management Plan.
7. Storm Sewer Design Calculations indicating drainage to, and capacity of, any proposed storm sewers
8. Overland Flow and Flood-routing Calculations. The maximum elevation for the 100-yr flow shall be 1-ft below the Finished Floor elevation.
9. Culvert Design Data Sheet Calculations indicating:
10. Drainage area to the culvert
11. Inlet or outlet control
12. Headwater or tail-water (if applicable)
13. Capacity of the proposed culverts
14. Channel Calculations indicating drainage area, shape, slope, and capacity of all channels
15. Weir Calculations indicating sizing for all weirs including the emergency spillway
16. Orifice Calculations Orifice Calculations showing the size and number of orifices for the basin outlet structure
17. Spread Calculations-Indicating the gutter capacity of streets and amount of pavement flooded for the 5-yr design storm
18. Grate Calculations-Indicating the capacity of inlet grates to handle 5-yr design storm
19. Pump curves for stormwater lift stations (assumes pre-approval of a stormwater lift station by the City)
20. Construction Sedimentation Basin calculation
21. Temporary outlet structure calculations per OEPA NPDES & ODNR

4. General Design Considerations:

The City of Delaware has implemented MS4 community requirements in regards to considering Phase II storm water quality and design. All development activity within the City is subject to the regulations of OEPA's General Permit OHCO00002.

A. Discharge Point

Onsite stormwater systems must discharge to one of the following offsite stormwater systems:

1. A USGS "Blue Line" stream,
2. An open channel system (including roadside ditches)
3. A storm sewer system adequately sized for the intended flows

If none of the options above is feasible, then the Applicant must demonstrate that only sheet flow is being discharged with adequate quantity and quality controls in place, since concentrated flow may cause offsite erosion.

The Applicant shall use an accepted hydrologic method to demonstrate that the existing offsite stormwater system can convey existing offsite flows and projected onsite flows in a manner that does not increase downstream peak water surface elevations during the 1-year through the 100-year design storms and satisfies the various design criteria in this design manual.

B. Stormwater Diversions

The diversion of stormwater runoff from one watershed or receiving stormwater system to another is generally prohibited because such diversions have the potential to cause or exacerbate flooding, erosion, or water quality problems in receiving watercourses. For the purposes of this manual, stormwater diversions are defined as the relocation of stormwater discharges from original receiving streams or stormwater systems to other systems that did not receive such discharges prior to construction. While it is recognized that stormwater runoff from small, onsite, tributary areas must be conveyed between catch basin sub-catchments, the City of Delaware will not allow the diversion of stormwater runoff from one major storm sewer system or open watercourse to another without proper documentation that includes proof of benefit. Stormwater system diversions may be considered on a case-by-case basis under circumstances where it can be shown that flooding and erosion will not increase and benefits to each watercourse can be achieved as a result of diverted flows. The diversion of any stormwater runoff from one stormwater system or watercourse to another shall be at the sole discretion of the City Engineer or their designee.

C. Field Tiles

Agricultural field tiles are for agricultural drainage purposes only and may not be used as an outlet of any development or stormwater facility. Field tiles that are discovered or intercepted during construction and do not exhibit evidence of conveying septic effluent shall be reconnected or connected into the proposed stormwater system. Field tiles that exhibit evidence of conveying septic effluent shall not be used for stormwater conveyance and shall be reported to the Delaware County Health Department for resolution upon discovery.

Designers preparing plans for development on existing agricultural lands shall, at a minimum, contact the Delaware County Engineer's office and the Delaware Soil and Water Conservation District to confirm the existence and location of existing tile systems. All visible field tile outlets and locations shall be field located and shown on the stormwater management plans. Any plan information for field tile systems received from county agencies shall also be shown.

All field tiles discovered either through preliminary investigation or during construction shall be shown on as-built drawings.

PART 2 – HYDROLOGY

1. Methodologies for Determining Runoff Volume and Peak Flows

A. Rational Method

This method should only be used for drainage areas of 1 acres or less. This method is not acceptable for runoff attenuation. (Retention/Detention Basin Design) The most outstanding limitation of the Rational Method is that the only product of the method is peak discharge. The method provides only an estimate of a single point on the runoff hydrograph and, therefore, does not permit estimation of runoff volume - a computation that is mandatory for the design of stormwater storage facilities. As such, the rational method is more appropriately used in the sizing to stormwater conveyance systems.

Rational Method: $Q = CiA$

Where:

Q = the maximum rate of rainfall runoff in cubic feet per second (cfs)

C = the runoff coefficient which is the ratio between the maximum rate of runoff from the area and the average rate of rainfall on the area during the time of concentration.

i = the average rainfall intensity, in inches per hour, for a duration equal to the time required for the runoff to flow from the most remote part of the drainage area to the point under design.

A = the drainage area in acres tributary to the point under design.

Under the Rational Method, it is assumed that the rate of runoff to each point under design consideration is a function of the average rainfall rate during the time required for water to flow from the most distant part in the tributary area to the point. This time of flow is called the "time of concentration”. The rational method assumes that the peak rate of rainfall occurs within the time of concentration.

The runoff coefficients as shown in the table below shall be used to determine the values for use with the rational method.

Table 1: Runoff Coefficients for the Rational Formula

Cover type and hydrologic condition	Coefficients for hydrologic soil group:		
C Values for Urban Areas	**B**	**C**	**D**
Fully developed urban areas (vegetation established)			
Open space (lawns, parks, golf courses, cemeteries, etc.)			
Poor condition (grass cover < 50%)	0.45	0.61	0.69
Fair condition (grass cover 50% to 75%)	0.28	0.45	0.50
Good condition (grass cover > 75%)	0.16	0.36	0.47
Impervious areas:			
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)	0.96	0.96	0.96
Retention Basins, Ponds, River and Streams	0.96	0.96	0.96
Streets and roads:			
Paved; curbs and storm sewers (excluding right-of-way)	0.96	0.96	0.96
Paved; open ditches (including right-of-way)	0.69	0.77	0.80
Gravel (including right-of-way)	0.59	0.69	0.74
Dirt (including right-of-way)	0.52	0.64	0.69
Urban districts:			
Commercial and business (85% Impervious)	0.77	0.83	0.86
Industrial (72% Impervious)	0.66	0.74	0.80
Residential districts by average lot size:			
1/8 acre or less (town houses) (65% Impervious)	0.59	0.72	0.77
1/4 acre (38% Impervious)	0.37	0.54	0.64
1/3 acre (30% Impervious)	0.32	0.50	0.61
1/2 acre (25% Impervious)	0.29	0.47	0.59
1 acre (20% Impervious)	0.26	0.45	0.96
2 acres (12% Impervious)	0.21	0.41	0.52
C Values for Developing Urban Areas	**B**	**C**	**D**
Newly graded areas (pervious areas only, no vegetation)	0.61	0.74	0.83
C Values for Undeveloped or Agricultural Areas	**B**	**C**	**D**
Cultivated Land:			
With conservation treatment	0.37	0.47	0.59
Without conservation treatment	0.30	0.43	0.50
Pasture, grassland, or range - continuous forage for grazing			
Poor hydraulic condition (<50% ground cover)	0.39	0.52	0.61
Fair hydraulic condition (50 to 75% ground cover)	0.28	0.45	0.50
Good hydraulic condition (> 75% ground cover)	0.16	0.36	0.47
Meadow - continuous grass, protected from grazing, and mowed for hay	0.13	0.30	0.43
Brush - brush, weed, and grasses with brush the major element			
Poor hydraulic condition (<50% ground cover)	0.24	0.41	0.54
Fair hydraulic condition (50 to 75% ground cover)	0.11	0.29	0.41
Good hydraulic condition (> 75% ground cover)	0.05	0.39	0.34
Woods, Poor: forest litter, small trees & brush w/ regular grazing & burning	0.23	0.41	0.54
Fair: Woods are grazed but not burned, some forest litter covers the soil	0.15	0.34	0.45
Good: Woods are protected from grazing, litter & brush cover soil	0.10	0.29	0.41
Farmsteads - buildings, lawns, and driveways	0.36	0.52	0.61

B. NRCS (SCS) Methods

For drainage areas over 1 acre, the methods explained in "Urban Hydrology for Small Watersheds", Technical Release No. 55, latest edition, and/or Technical Release No. 20, latest edition shall be used to calculate runoff volumes and peak rates of flow. Technical Releases No. 55 & No. 20 can be obtained from the United States Department of Agriculture, Natural Resources Conservation Service.

C. USGS Regression Equations

The regression equations presented in USGS Report 93-135 is an accepted method for estimating design peak-discharge values for streams with drainage areas between 17 and 2,600 acres. **The application of this method is limited to the estimation of peak discharges for City funded culvert installation and replacement projects. The following equations shall be used for the various design storms:

$$Q_2 = 155\,(A)^{0.68}\,(P - 30)^{0.50}\,(13 - BDF)^{-0.5}$$

$$Q_5 = 200\,(A)^{0.71}\,(P - 30)^{0.63}\,(13 - BDF)^{-0.44}$$

$$Q_{10} = 228\,(A)^{0.74}\,(P - 30)^{0.68}\,(13 - BDF)^{-0.41}$$

$$Q_{25} = 265\,(A)^{0.76}\,(P - 30)^{0.72}\,(13 - BDF)^{-0.37}$$

$$Q_{50} = 293\,(A)^{0.78}\,(P - 30)^{0.74}\,(13 - BDF)^{-0.35}$$

$$Q_{100} = 321\,(A)^{0.79}\,(P - 30)^{0.76}\,(13 - BDF)^{-0.33}$$

Where:
Q_N = peak discharge rate in cfs,
A = the drainage area in square miles,
P = average annual precipitation in inches = 36.6* inches for Delaware, Ohio
BDF = the basin development factor.

* Data derived from The National Climatic Data Center (NCDC) Cooperative Stations. 58 complete years between 1931 and 1995.
** See USGS Report 93-135 pg. 22 for the statistical limitations for this method

The basin development factor (BDF) is determined by subdividing the drainage basin into thirds (lower, middle, and upper) with two lines drawn across the basin that are perpendicular to the main channel and principal tributaries. Four aspects of the drainage system are then evaluated within each third of the basin and assigned a value of 1 or 0:

1. Channel improvements include any straightening, enlarging, deepening, and clearing made in the main drainage channel and principal tributaries. If at least 50 percent of the upstream channels in the basin are improved, then a value of 1 is assigned.
2. Channel linings include any length of the main drainage channels and principal tributaries that have been lined with an impervious material such as concrete. A value of 1 is assigned if at least 50 percent of the upstream channels have been lined.

3. Storm drains or storm sewers are defined as enclosed drainage structures (usually pipes) frequently used on secondary tributaries where drainage is received directly from streets or parking lots. A value of 1 is then assigned when more than 50 percent of the upstream secondary tributaries consist of storm drains.
4. Curb and gutter streets frequently empty into storm drains. If more than 50 percent of the upstream basin is developed with streets and highways constructed with curbs and gutters, then a value of 1 will be assigned.

The following table provides an example for calculating the overall BDF for the entire basin that has channel improvements throughout, no channel linings, and storm drains with curb and gutter streets in the lower two-thirds (2/3) of the basin:

Table 2: Example Determination of the Basin Development Factor

Portion of Basin	Channel Improvements	Channel Linings	Storm Drains	Curb & Gutter Streets	Basin Development Factor
Lower 1/3	1	0	1	1	3
Middle 1/3	1	0	1	1	3
Upper 1/3	1	0	0	0	1
				Total:	7

D. Software

List any software used in the methodology section of the Stormwater Management Report. Common software programs that are acceptable to the Public Works Department are:

1. ODOT's CDSS, used primarily for culvert design, inlet spacing, and ditch design
2. Hydra-Flow, Computer program by "intelSOLVE", Program is based on the Rational Method and used primarily for storm sewer design
3. Haestad Methods POND 2 Software, which includes the U.S. SCS TR-55
4. Win-TR-55 - Hydrology design
5. HydroCAD Software Solutions LLC – Hydrology design
6. TR-20 - Hydrology design
7. HEC-RAS – The US Army Corps of Engineers river analysis program
8. Other software may be approved after review by the Public Works Department

2. Drainage Area Determination

A. The drainage area shall be determined from one or more of the following sources:

1. USGS maps
2. Photogrammetrics
3. Field investigations & Surveys
4. Soil Survey of Delaware County
5. Delaware County topographical maps and/or GIS contours.

B. Offsite Tributary Area

Stormwater runoff from offsite upstream tributary areas that discharge to or across a development site shall be accommodated within the stormwater facilities planned for the development site. No stormwater management plans will be approved until it is demonstrated that offsite runoff will be adequately conveyed through the development site in a manner that will not cause or contribute to hazardous or detrimental upstream and downstream flooding and erosion. The estimation of the offsite flows must be done separately from the estimation of onsite flows (i.e., separate hydrographs for offsite areas must be determined). Detention / Retention is not required for the offsite flows unless downstream condition warrant it. Offsite flows must be treated for water quality per Ohio EPA requirements.

The allowable release rate from a site that is "passing through" offsite runoff shall be the allowable release rate for the site plus the pre developed offsite flow for the design storm.

3. Time of Concentration (T_c)

Time of concentration (T_c) for a drainage area is defined as the time a drop of water takes to drain from the hydraulically most remote point in the watershed to the point of interest. T_c influences the shape and peak of the runoff hydrograph. Urbanization usually decreases T_c, thereby increasing the peak discharge rate. But T_c can be increased as a result of (a) ponding behind small or inadequate drainage systems, including storm drain inlets and road culverts, or (b) reduction of land slope through grading. In general, higher and faster peak discharge is associated with smaller T_c. The time of concentration determines the average rainfall rate on the rainfall intensity-duration-frequency curve.

Different methods are available for computing T_c for a drainage area. Acceptable methods in the City of Delaware are the TR-55 tabular method and SCS Lag method

A. Factors affecting Time of Concentration (T_c) and Travel Time (T_t)

1. Surface roughness

   One of the most significant effects of urban development on flow velocity is less retardance to flow. That is, undeveloped areas with very slow and shallow overland flow through vegetation become modified by urban development. The flow is then delivered to streets, gutters, and storm sewers that transport runoff downstream more rapidly. Travel time through the watershed is generally decreased.

2. Channel shape and flow patterns

   In small non-urban watersheds, much of the travel time results from overland flow in upstream areas. Typically, urbanization reduces overland flow lengths by conveying storm runoff into a channel as soon as possible. Since channel designs have efficient hydraulic characteristics, runoff flow velocity increases and travel time decreases.

3. Slope

   Slopes may be increased or decreased by urbanization, depending on the extent of site grading or the extent to which storm sewers and street ditches are used in the design of the water management system. Slope will tend to increase when channels are straightened and decrease when overland flow is directed through storm sewers, street gutters, and diversions.

B. SCS Lag Method for Computing T_c

This method was proposed by the Soil Conservation Services (SCS) and evaluates the drainage basin based on the average land slope, curve number (CN) and the hydraulic length.

From the known CN, the available storage (S) is computed using:

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

The basin lag is then estimated using:

$$Lag = \frac{L^{0.8} \times (S+1)^{0.7}}{1900 \times (s \times 100)^{0.5}} \; hours$$

Where:
Lag = basin lag in hours
L = hydraulic length in feet
S = available storage
s = average slope of the drainage area in ft/ft (It is not the slope of a specific flow path, but rather the average slope of the watershed area.)

The time of concentration (Tc) for the drainage basin is then computed using:

T_c = 1.67 x Lag (hours)
T_c = 1.67 x Lag x 60 (minutes)

C. Velocity Method for Computing T_c

The velocity method of computing T_c assumes that time of concentration is the sum of travel times for segments along the hydraulically most distant flow path. Travel time for each segment are computed and summed to arrive at the time of concentration. The segments used in the velocity method are of three types: (1) sheet flow, (2) shallow concentrated flow, and (3) open channel flow. Water moves through a watershed in combinations of these segments in the order given. The type that occurs is a function of the conveyance system and is best determined by field inspection.

Time of concentration (T_c) is the sum of time of travel values for the various consecutive flow segments:

$$T_c = t_{sheet} + t_{shallow} + t_{channel} + t_{conduit}$$

1. Sheet flow

Sheet flow is flow over plane surfaces and usually occurs at the upstream boundary of a watershed. Generally the depth of sheet flow is less than 0.1 feet and is a steady, uniform flow. With sheet flow, the friction value (Manning's n) is an effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges, rocks, erosion and transportation of sediment. The table below gives Manning's n values for sheet flow for various surface conditions. Typically sheet flow length is less than 100-feet for paved areas with a maximum of 300-feet in length for unpaved areas, and is less than 20 minutes in duration. In general, sheet flow will not exceed 1-inch in depth.

For Sheet flow of less than 300 feet, use Manning's Kinematic Solution (Overton and Meadows 1976) to compute T_{sheet}

$$t_{sheet} = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} s^{0.4}}$$

where:
t_{sheet} = travel time (hr),
n = Manning's roughness coefficient (table 3)
L = flow length (ft)
P2 = 2-year, 24-hour rainfall (in)
s = slope of hydraulic grade line (land slope, ft/ft)

This simplified form of the Manning's kinematic solution is based on the following assumptions:

a. Shallow steady uniform flow,
b. Constant intensity of rainfall excess (that part of a rain available for runoff),
c. Rainfall duration of 24 hours, and
d. Minor effect of infiltration on travel time.

Rainfall depth can be obtained from the rainfall runoff equations.

Table 3: Roughness coefficients (Manning's n) for Sheet Flow

Surface Description	n [1]
Smooth Surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short Grass Prairie	0.15
Dense Grasses [2]	0.24
Bermuda Grass	0.41
Range (natural)	0.13
Woods [3]	
Light Underbrush	0.40
Dense Underbrush	0.80
[1] The n values are a composite of information compiled by Engman (1986)	
[2] Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures	
[3] When selecting n, consider cover to a height of about 0.1 feet. This is the only part of the plant cover that will obstruct sheet flow.	

2. Shallow Concentrated Flow

Sheet flow is assumed to occur for no more than 300 feet after which water tends to concentrate in rills and then gullies of increasing proportion. This type of flow is classified as shallow concentrated flow. Shallow concentrated flow is assumed not to have a well-defined channel and has flow depths of 0.1 to 0.5 feet. To estimate shallow concentrated flow travel time, a velocity is determined based on the slope and surface type. For the purposes of this manual, only paved and unpaved surfaces will be considered. The following equations will provide the estimated velocity based on slope for paved and unpaved surfaces:

$V = 16.1345\ s^{0.5}$ (Unpaved)

$V = 20.3282\ s^{0.5}$ (Paved)

Where:

V = average velocity (ft/s), and

s = slope of hydraulic grade line (watercourse slope, ft/ft)

These two equations are based on a solution of the Manning equation with different assumptions for n (Manning roughness coefficient) and r (hydraulic radius, ft). For unpaved areas, n is 0.05 and r is 0.4; for paved areas, n is 0.025 and r is 0.2.

After determining average velocity, use the $t_{shallow}$ equation to estimate travel time for the shallow concentrated flow segment. The travel time for shallow concentrated flow is calculated as follows:

$$t_{shallow} = \frac{L}{3600 \cdot V}$$

where:
$t_{shallow}$ = Travel time for shallow concentrated flow in hours
L = Flow length in feet
V = Velocity in fps

3. Open Channel Flow

Open channel flow occurs beyond shallow concentrated flow when the flow depth exceed 0.5 feet in depth and enters into pipe systems, drainage ditches, or natural channels. The velocity of flow in an open channel or pipe can be estimated using the Manning's equation. Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. However, field observations should be made to determine the presence of defined channels or swales where depth of flow would exceed the 0.5 foot limitation of shallow concentrated flow. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for the bankfull condition.

Manning's equation is:

$$V = \frac{1.49 r^{2/3} s^{1/2}}{n}$$

where:
V = average velocity (ft/s)
r = hydraulic radius (ft) and is equal to a/ p_w
a = cross sectional flow area (ft^2)
p_w = wetted perimeter (ft)
s = slope of the hydraulic grade line (channel slope, ft/ft)
n = Manning's roughness coefficient for open channel flow. (See tables below)

After average velocity is computed using the above equation, $t_{channel}$ (for both channel and conduit flow) can be determined using the following equation:

$$t_{channel} = \frac{L}{3600 \cdot V}$$

where:
$t_{channel}$ = Travel time for shallow concentrated flow in hours
L = Flow length in feet
V = Velocity in fps

Given varying cross-sections of flow paths, and varying peak rates of flow and resulting varying depths of flow for different return periods, it is expected that each return period would result in a different t_c. In order to help simplify the modeling process the City will accept an average cross-section of the open channel in segments of 250-ft in length, with a depth of flow determined by the 2-year return period event. This simplification is for determining the t_c only, and is not to be used for the design of new open channels. New channels are to be designed using the design storm specified in the section on Open Watercourses.

Table 4: Manning's n Values for Excavated Channels

Roughness Coefficient for Excavated Open Channels			Manning's n Range		
Type	**Characteristics**		**Minimum n**	**Normal n**	**Maximum n**
Earth, straight and uniform	clean, recently completed		0.016	0.018	0.020
	clean, after weathering		0.018	0.022	0.025
	gravel, uniform section		0.022	0.025	0.030
	clean with short grass, few weeds		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 or aquatic plants in deep channels		0.030	0.035	0.040
	earth bottom and ruble sides		0.028	0.030	0.035
	stony bottom and weed banks		0.025	0.035	0.040
	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	based on design section		0.035	0.045	0.050
	based on actual mean section	smooth and uniform	0.025	0.035	0.040
		jagged and irregular	0.035	0.040	0.050
Channels not maintained, weed and brush uncut	dense weed high as flow depth		0.050	0.080	0.120
	clean bottom, brush on sides		0.040	0.050	0.080
	clean bottom, brush on sides, highest stage of flow		0.045	0.070	0.110
	dense brush, high stage		0.080	0.100	0.140

Table 5: Manning's n Values for Roadside Ditches

Roadside channels and swales with maintained vegetation (Values shown are for velocities of 2 and 6 ft/sec)			Manning's n Range		
			Minimum n	**Normal n**	**Maximum n**
Depth of flow up to 0.7 ft	Bermuda grass, Kentucky bluegrass, buffalo grass	Mowed to 2 in	0.045	0.058	0.070
		Mowed to 6 in	0.050	0.070	0.090
	Good stand, any grass	Length about 12 inches	0.090	0.135	0.180
		Length about 24 inches	0.150	0.225	0.300
	Fair stand, any grass	Length about 12 inches	0.080	0.110	0.140
		Length about 24 inches	0.130	0.190	0.250
Depth of flow 0.7 - 1.5 ft	Bermuda grass, Kentucky bluegrass, buffalo grass	Mowed to 2 in	0.038	0.044	0.050
		Mowed to 6 in	0.040	0.050	0.060
	Good stand, any grass	Length about 12 inches	0.070	0.095	0.120
		Length about 24 inches	0.100	0.150	0.200
	Fair stand, any grass	Length about 12 inches	0.060	0.080	0.100
		Length about 24 inches	0.090	0.130	0.170

Table 6: Manning's n Values for Lined Channels

Roughness Coefficient for Lined Open Channels		Manning's n Range		
Type	**Characteristics**	**Minimum n**	**Normal n**	**Maximum n**
Cement	neat surface	0.010	0.011	0.013
	mortar	0.011	0.013	0.015
Concrete	trowel finish	0.011	0.013	0.015
	float finish	0.013	0.015	0.016
	finished, with gravel on bottom	0.015	0.017	0.020
	unfinished	0.014	0.017	0.020
	gunite, good section	0.016	0.019	0.023
	gunite, wavy section	0.018	0.022	0.025
	on good excavated rock	0.017	0.019	0.020
	on irregular excavated rock	0.022	0.025	0.027
Concrete bottom float finished with sides of :	dressed stone in mortar	0.015	0.017	0.020
	random stone in mortar	0.017	0.020	0.024
	cement rubble masonry, plastered	0.016	0.020	0.024
	cement rubble masonry	0.020	0.025	0.030
	dry rubble or riprap	0.020	0.030	0.035
Gravel bottom with sides of:	formed concrete	0.017	0.020	0.025
	random stone in mortar	0.020	0.023	0.026
	dry rubble or riprap	0.023	0.033	0.036
Brick	glazed	0.011	0.013	0.015
	in cement mortar	0.012	0.015	0.018
Masonry	cemented rubble	0.017	0.025	0.030
	dry rubble	0.023	0.032	0.035
Dressed ashlar	--	0.013	0.015	0.017
Asphalt	smooth	0.013	0.013	0.013
Vegetal lining	--	0.030	0.040	0.500

Table 7: Manning's n Values for Natural Channels

Natural Stream Channels			Manning's n Range		
Type	**Characteristics**		**Minimum n**	**Normal n**	**Maximum n**
Minor Streams (surface width at flood stage is less than 100 feet)	Fairly regular section	Some grass and weeds, little or no brush	0.030	0.033	0.035
		Dense growth of weed, depth of flow materially greater than weed height	0.035	0.043	0.050
		Some weeds, light brush on banks	0.040	0.045	0.050
		Some weeds, heavy brush on banks	0.050	0.060	0.070
		Some weed, dense willows on banks	0.060	0.070	0.080
	For trees within the channel, with branches submerged at high stage, increase all above values by:		0.010	0.015	0.020
	For irregular section, with pools, slight meander, increase value for fairly regular sections by about:		0.010	0.015	0.020
	Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage	Bottom of gravel, cobbles and few boulders	0.040	0.045	0.050
		Bottom of cobbles, with large boulders	0.050	0.060	0.070

4. Reservoirs or lakes

   Sometimes it is necessary to estimate the velocity of flow through a reservoir or lake at the outlet of a watershed. This travel time is normally very small and can be assumed as zero.

5. Limitations
   a. Manning's kinematic solution should not be used for sheet flow longer than 300 feet. Typically sheet flow length is less than 100-feet for paved areas with a maximum of 300-feet in length for unpaved areas. It takes approximately 20 minutes of rainfall at the 2 year intensity for measurable runoff to occur. In general, sheet flow will not exceed 1-inch in depth.
   b. Manning's Kinematic Equation was developed for use with the four standard SCS rainfall intensity-duration relationships.
   c. In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate Tc. Storm sewers generally handle only a small portion of a large event. The rest of the peak flow travels by streets, lawns, and so on, to the outlet. Consult a standard hydraulics textbook to determine average velocity in pipes for either pressure or non-pressure flow.
   d. The minimum T_c used in TR-55 is 0.1 hour.

e. A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. The procedures in TR-55 can be used to determine the peak flow upstream of the culvert. Detailed storage routing procedures should be used to determine the outflow through the culvert.

4. SCS Runoff Curve Numbers (CN)

The major factors that determine CN are the hydrologic soil group, cover type, treatment, hydrologic condition, and antecedent runoff condition. Another factor considered is whether impervious areas outlet directly to the drainage system (connected) or whether the flow spreads over pervious areas before entering the drainage system (unconnected).

Table 9: Runoff Curve Numbers (CN) for Typical Land Uses in the City of Delaware is provided to aid in selecting the appropriate figure or table for determining curve numbers for the City of Delaware. The CN values in table 9 represent average the runoff condition for urban, developing urban, and pre-developed agricultural land uses typical to area in and around the City of Delaware. The values in table 9 are to be used in all projects in the city and may be vary slightly from those found in other publications. In all cases the values found in table 9 supersede those found in other publications. Table 9 assumes impervious areas are directly connected to a suitable outlet.

5. Hydrologic Soil Groups (HSG)

Infiltration rates of soils vary widely and are affected by subsurface permeability as well as surface intake rates. Soils are classified into four HSG (A, B, C, and D) according to their minimum infiltration rate, which is obtained for bare soil after prolonged wetting. The vast majority of the soils in Delaware County fall into the C or D group. The soils in the area of interest may be identified from the Delaware County Soil Survey, which can be obtained from the Delaware Soil and Water Conservation office.

Most urban areas are only partially covered by impervious surfaces: the soil remains an important factor in runoff estimates. Any disturbance of a soil profile can significantly change its infiltration characteristics. With urbanization, native soil profiles may be mixed or removed or fill material from other areas may be introduced. In areas where soil has been previously disturbed by construction activities in the urban environment an HSG of D shall be used.

6. Ground Cover

Table 9 addresses most cover types typically found in the City of Delaware, such as common native and cultivated vegetation, bare soil, water surfaces and impervious surfaces. There are a number of methods for determining existing cover type for the proposed development. The most common are field reconnaissance, aerial photographs, and land use maps. Ponds are considered impervious and must be included at a CN of 98 or a C of 0.96.

7. SCS Type II Storm

Different rainfall distributions can be developed to emphasize the critical rainfall duration for the peak discharges. To avoid the use of a different set of rainfall intensities for each drainage area size, a set of synthetic rainfall distributions having “nested” rainfall intensities was developed. The set “maximizes” the rainfall intensities by incorporating selected short duration intensities within those needed for longer durations at the same probability level.

For the size of the drainage areas for which NRCS usually provides assistance, a storm period of 24 hours was chosen the synthetic rainfall distributions. The 24-hour storm, while longer than that needed to determine peaks for these drainage areas, is appropriate for determining runoff volumes. Therefore, a single storm duration and associated synthetic rainfall distribution can be used to represent not only the peak discharges but also the runoff volumes for a range of drainage area sizes. For the City of Delaware, the synthetic storm to be used in modeling rainfall events is the NRCS (SCS) Type II Storm.

Table 8: Runoff Curve Numbers (CN) for Typical Land Uses in the City of Delaware

Cover type and hydrologic condition	**Curve numbers for hydrologic soil group**		
Runoff Curve Numbers for Urban Areas	**B**	**C**	**D**
Fully developed urban areas (vegetation established)			
Open space (lawns, parks, golf courses, cemeteries, etc.)			
Poor condition (grass cover < 50%)	79	86	89
Fair condition (grass cover 50% to 75%)	69	79	84
Good condition (grass cover > 75%)	61	74	80
Impervious areas:			
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)	98	98	98
Retention Basins, Ponds, River and Streams	98	98	98
Streets and roads:			
Paved; curbs and storm sewers (excluding right-of-way)	98	98	98
Paved; open ditches (including right-of-way)	89	92	93
Gravel (including right-of-way)	85	89	91
Dirt (including right-of-way)	82	87	89
Urban districts:			
Commercial and business (85% Impervious)	92	94	95
Industrial (72% Impervious)	88	91	93
Residential districts by average lot size:			
1/8 acre or less (town houses) (65% Impervious)	85	90	92
1/4 acre (38% Impervious)	75	83	87
1/3 acre (30% Impervious)	72	81	86
1/2 acre (25% Impervious)	70	80	85
1 acre (20% Impervious)	68	79	84
2 acres (12% Impervious)	65	77	82
Runoff Curve Numbers for Developing Urban Areas	B	C	D
Newly graded areas (pervious areas only, no vegetation)	86	91	94
Runoff Curve Numbers for Undeveloped or Agricultural Areas	B	C	D
Cultivated Land:			
Without conservation treatment	75	80	85

With conservation treatment	71	78	81
Pasture, grassland, or range - continuous forage for grazing			
Poor hydraulic condition (<50% ground cover)	76	82	86
Fair hydraulic condition (50 to 75% ground cover)	69	79	84
Good hydraulic condition (> 75% ground cover)	61	74	80
Meadow - continuous grass, protected from grazing, and mowed for hay	58	71	78
Brush - brush, weed, and grasses with brush the major element			
Poor hydraulic condition (<50% ground cover)	67	77	83
Fair hydraulic condition (50 to 75% ground cover)	56	70	77
Good hydraulic condition (> 75% ground cover)	48	76	73
Woods , Poor: forest litter, small trees & brush w/ regular grazing & burning	66	77	83
Fair: Woods are grazed but not burned, some forest litter covers the soil	60	73	79
Good: Woods are protected from grazing, litter & brush cover soil	55	70	77
Farmsteads - buildings, lawns, and driveways	74	82	86

8. Rainfall Data

Acceptable rainfall data for use in culvert, storm system, and Hydrological design are those provided in the "Rainfall Frequency Atlas of the Midwest" by Floyd A. Huff and James R. Angel (Bulletin 71).

Total rainfall amounts for Delaware are presented in table 9 (below) for selected rainfall durations and recurrence intervals. The 24-hour rainfall amounts are to be used with the NRCS TR-55 methods.

Table 9: Bulletin 71 Sectional Mean Frequency Distributions for the City of Delaware

Duration	Rainfall (inches) for Given Recurrence Interval													
	5-min	10-min	15-min	30-min	1-hr	2-hr	3-hr	6-hr	12-hr	18-hr	24-hr	48-hr	72-hr	5-day	10-day
2-month	0.14	0.25	0.32	0.44	0.56	0.69	0.76	0.90	1.04	1.12	1.19	1.27	1.36	1.49	1.81
3-month	0.17	0.29	0.38	0.51	0.65	0.81	0.89	1.04	1.21	1.31	1.39	1.48	1.59	1.78	2.18
4-month	0.18	0.32	0.41	0.56	0.71	0.88	0.97	1.14	1.32	1.43	1.52	1.65	1.80	2.01	2.51
6-month	0.21	0.37	0.48	0.65	0.83	1.02	1.13	1.32	1.53	1.65	1.76	1.91	2.09	2.34	2.95
9-month	0.24	0.42	0.54	0.74	0.94	1.16	1.28	1.50	1.74	1.88	2.00	2.20	2.40	2.69	3.39
1-year	0.26	0.46	0.59	0.80	1.02	1.26	1.39	1.63	1.89	2.04	2.17	2.39	2.61	2.92	3.69
2-year	0.32	0.57	0.73	1.00	1.27	1.57	1.73	2.03	2.35	2.54	2.70	2.97	3.23	3.67	4.69
5-year	0.40	0.70	0.90	1.24	1.57	1.94	2.14	2.51	2.91	3.15	3.35	3.67	3.99	4.65	5.93
10-year	0.46	0.81	1.04	1.43	1.81	2.24	2.47	2.89	3.36	3.63	3.86	4.21	4.54	5.39	6.78
25-year	0.56	0.97	1.25	1.72	2.18	2.69	2.97	3.48	4.04	4.36	4.64	5.02	5.36	6.37	7.82
50-year	0.64	1.12	1.44	1.97	2.51	3.09	3.41	4.00	4.64	5.01	5.33	5.72	6.09	7.11	8.56
100-year	0.73	1.27	1.64	2.24	2.85	3.51	3.88	4.55	5.27	5.70	6.06	6.50	6.92	7.89	9.27

Source of data: Bulletin 71, Part 2, Table 8 for Climatic Section 5

A tabulation of rainfall intensities for the 1-, 2-, 5-, 10-, 25-, 50-. and 100-year recurrence intervals, and for duration ranging from 10 through 200 minutes, has been developed for Delaware and is shown in table 11. This data supersedes the rainfall data from Technical Paper No. 40. Intensities for durations not shown shall be calculated by the rainfall intensity equations provided.

A. Rainfall Intensity Equations incorporating Bulletin 71 Data

Federal Highway Administration Hydraulic Engineering Circular No. 12 Appendix A offers a methodology for converting I-D-F data points to an equation of the general form. The Intensity can be found using the following equation:

$$i = \frac{\mathrm{x}}{(\mathrm{y} + \mathrm{t})^{z}}$$

Where:
i = Rainfall Intensity in inches per hour
t = time of concentration (minutes)
Constants x, y, and z given for each of the design year storms given in the table below:

Table 10 - Constants for I-D-F Curves

Frequency (Years)	Constant "x"	Constant "y"	Constant "z"
2	90.36	18.49	0.98
5	61.39	12.49	0.86
10	68.67	12.08	0.85
25	101.85	14.86	0.89
50	96.25	12.89	0.85
100	334.68	23.51	1.08

Table 11: Bulletin 71 – Selected Design Rainfall Intensities

	Rainfall Intensity (in/hr) for Recurrence Interval of:					
Duration (min)	**2 years**	**5 years**	**10 years**	**25 years**	**50 years**	**100 years**
10	3.42	4.27	4.95	5.82	6.72	7.62
10.5	3.36	4.20	4.85	5.72	6.60	7.50
11	3.31	4.12	4.77	5.62	6.48	7.38
11.5	3.25	4.05	4.68	5.52	6.37	7.27
12	3.20	3.97	4.60	5.43	6.26	7.16
12.5	3.15	3.91	4.52	5.34	6.15	7.05
13	3.10	3.84	4.44	5.26	6.05	6.95
13.5	3.05	3.78	4.37	5.18	5.95	6.85
14	3.01	3.72	4.30	5.10	5.86	6.75
14.5	2.96	3.66	4.23	5.02	5.77	6.65
15	2.92	3.60	4.16	4.94	5.68	6.56
15.5	2.88	3.54	4.10	4.87	5.60	6.47
16	2.84	3.49	4.03	4.80	5.51	6.38
16.5	2.80	3.44	3.97	4.73	5.43	6.30
17	2.76	3.39	3.92	4.67	5.36	6.21
17.5	2.72	3.34	3.86	4.60	5.28	6.13
18	2.69	3.29	3.80	4.54	5.21	6.05
18.5	2.65	3.25	3.75	4.48	5.14	5.97
19	2.62	3.20	3.70	4.42	5.07	5.90
19.5	2.58	3.16	3.65	4.36	5.00	5.82
20	2.55	3.12	3.60	4.31	4.94	5.75
21	2.49	3.04	3.51	4.20	4.81	5.61
22	2.43	2.96	3.42	4.10	4.70	5.48
23	2.37	2.89	3.34	4.00	4.58	5.35
24	2.31	2.83	3.26	3.91	4.48	5.23
25	2.26	2.76	3.18	3.82	4.38	5.12
26	2.21	2.70	3.11	3.74	4.28	5.00
27	2.16	2.64	3.05	3.66	4.19	4.90
28	2.12	2.58	2.98	3.58	4.10	4.80
29	2.08	2.53	2.92	3.51	4.02	4.70
30	2.03	2.48	2.86	3.44	3.94	4.60
32	1.95	2.38	2.75	3.31	3.79	4.42
34	1.88	2.30	2.65	3.19	3.65	4.26
36	1.81	2.21	2.55	3.08	3.52	4.11
38	1.75	2.14	2.47	2.97	3.41	3.96
40	1.69	2.07	2.39	2.88	3.30	3.83
42	1.64	2.00	2.31	2.79	3.19	3.70
44	1.59	1.94	2.24	2.70	3.10	3.58
46	1.54	1.89	2.17	2.62	3.01	3.47
48	1.49	1.83	2.11	2.55	2.92	3.37
50	1.45	1.78	2.06	2.48	2.85	3.27
52	1.41	1.74	2.00	2.41	2.77	3.18
54	1.37	1.69	1.95	2.35	2.70	3.09
56	1.34	1.65	1.90	2.29	2.63	3.00
58	1.30	1.61	1.85	2.23	2.57	2.93
60	1.27	1.57	1.81	2.18	2.51	2.85

PART 3 – STORMWATER CONTROLS

1. Peak Runoff Control (Critical Storm Method)

Stormwater runoff controls shall be designed in order to control the peak rate of runoff from small, moderate, and large storm events before it is discharged offsite. Peak runoff shall be controlled in accordance with the Critical Storm Method as presented in this chapter for all land development and redevelopment projects. The requirement for peak runoff control on developments and redevelopments less than one acre in size may be modified or waived on a case-by-case basis if approved by the Public Works Department after submission of a Design Exception Form. The project size shall be based on area of disturbance associated with the development or redevelopment

The peak rate of runoff from an area after development shall not exceed the peak rate of runoff from the same area before development for all storms up to a 100-year frequency, 24-hour storm. In addition, if it is found a proposed development will increase the volume of runoff from an area, the peak rate of runoff from certain more frequent storms must be controlled further in accordance with the Critical Storm Method as described below

There are two reasons why increases in volume of runoff require a control standard more restrictive than controlling to the predevelopment condition. First, increases in volume mean runoff will be flowing for a longer period of time. When routed through a watershed, these longer flows may join at some point or points downstream thereby creating new peak flows and the problems associated with peak flow (flooding). This is known as the "Routing Problem." Second, longer flow periods of large runoff quantities place a highly erosive stress on natural channels. Reducing the rate of discharge can minimize this stress. The permissible peak rate shall be determined as follows:

Determine the total volume of runoff from a 1-year frequency 24-hour storm, occurring over the area before and after development. The following are the equations used in the TR-55 method for determining runoff volume. Alternative methods can be used if they (a) give a runoff volume, and (b) are pre-approved by the Public Works Department. The Rational Method does not produce a runoff volume and cannot be used to determine either the critical storm or detention/retention requirements.

$$q = \frac{[I - (0.2 \times S)]^2}{I + (0.8 \times S)}$$

Where,
q = Direct surface runoff in inches
I = Storm rainfall in inches
S = Maximum potential difference between rainfall and runoff in inches

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

$$Q = \frac{q}{12} \times A$$

Where,

Q = volume of flow in acre-feet
A = area in acres

Determine the percent of increase in volume due to development

$$Increase = \left(\frac{Q_{post}}{Q_{ex}} - 1 \right) \times 100\%$$

Using this percentage, pick the critical storm from the following table:

Table 12: Critical Storm

If the percentage of increase in the volume of runoff is:				The Critical Storm for discharge limitation will be:
Equal to or greater than		and less than		
0%	~	10%		1 year critical storm
10%	~	20%		2 year critical storm
20%	~	50%		5 year critical storm
50%	~	100%		10 year critical storm
100%	~	250%		25 year critical storm
250%	~	500%		50 year critical storm
500%	~	-		100 year critical storm

The peak rate of runoff from the critical storm occurring over the development shall not exceed the peak rate of runoff from a 1-year frequency storm occurring over the same area under predevelopment conditions. Storms of less frequent occurrence (longer return period) than the critical storm shall have peak rate of runoff not greater than for the same storm under predevelopment conditions. As an example, if the total volume is shown to be increased by 35%, the critical storm is a 5-year storm. The peak rate of runoff for all storms up to this intensity shall be controlled so as not to exceed the peak rate of runoff from a 1-year frequency storm under predevelopment conditions for the area. The runoff from a more intense storm need only be controlled so as not to exceed the predevelopment peak rate from the same frequency of storm. The critical storm shall be computed for each sub-catchment of a parcel. The allowable release rates are to be based on the individual sub-catchment. If the development has multiple outlets it may have differing critical storms for each outlet.

Storage volume does not have to be provided for off-site upstream flows. Flow from such areas will be routed through the drainage system in the development under consideration at a rate determined in the same manner as the on-site system. Off-site land uses prevailing at the time of development shall be considered as the predevelopment condition for the purpose of calculating changes in runoff.

2. Retention/Detention Basins

This section provides design criteria for facilities that control stormwater discharges to control peak rates of runoff, prevent flooding, stream bank erosion, and water quality impairment downstream of development / redevelopment projects. Separate design criteria are provided for stormwater quantity and quality control facilities; however, in many cases quantity and quality controls are integrated into a single facility. This section provides criteria in three major sections: (A) General Criteria, (B) Stormwater Quantity Controls, and (C) Post Construction Stormwater Quality Controls.

A. General Criteria

1. Centralized storm water detention is recommended for all developments wherever possible.
2. Rear yard or enlarged drainage swale detention basins are not recommended unless no other option is available to the property.
3. Stormwater control facilities shall not be located within the Stream Corridor Protection Zone defined using OEPA Criteria
4. Stormwater control facilities shall not be located within designated Federal Emergency Management Agency (FEMA) floodplain boundaries.
5. Discharges from stormwater control facilities shall be directed into an approved discharge point as previously defined in this design guide.
6. Stormwater runoff shall not be diverted from an existing naturally occurring wetland that is not approved for filling and/or removing (as necessary) via an approved Section 404 permit issued by the U.S. Army Corps of Engineers. Wetland hydrology shall be sustained to the extent possible. The quantity and quality of this runoff shall be controlled prior to its release to the wetland system.

B. Stormwater Quantity Controls

Detention structures can be categorized as dry basins, permanent (wet) basins (A.K.A. ponds), underground storage chambers, stormwater wetlands, vegetated swales, bioretention facilities, multi-use storage areas such as rooftops, parking lots, and other shallow holding areas. The objective of control structures is to reduce the peak rate of discharge by storage and controlled release.

1. General Design Criteria for Retention/Detention Basins

   All proposed dry and wet detention basins shall be designed according to the general criteria in this section, as well as the specific criteria for stormwater quantity basins stormwater quality basins or both.

   NOTE: There are special considerations for basins in the vicinity of Delaware Airport; therefore the developer, owner, and/or design engineer shall contact the Public Works Department to discuss these considerations and requirements should the development be within 5000-feet of the Delaware Airport. See Appendices for additional information.

2. Layout and Geometry Requirements

   Modern day water quality requirements have created a proliferation of stormwater basins commonly constructed near roadways. There is a growing concern in the City over proximity of ponds to roadways as more incidents of vehicles entering ponds are reported. The Engineer is required to implement a pond buffer of some type between roadways and retention ponds – whether it be a large physical separation, mounding, rail, or combination thereof. The purpose of this section is to provide guidance to the Engineer in order to design a buffer that is approvable by the City.

Proposed stormwater retention basins shall be located as far as practicable from existing and proposed roadways. Most developments cannot accommodate large enough separations that would eliminate the concern of vehicles entering the pond, therefore, the following guidance is provided so the Engineer can determine an appropriate and approvable buffer to propose for approval by the City.

The Clear Zone, per the OMUTCD, shall be clear and unobstructed by a stormwater basin; otherwise the stormwater basin shall be considered an obstruction of the clear zone as a non-traversable hazard and the designer shall provide the appropriate corrective treatment per the OMUTCD. Again, the preference is for the pond to be as far as practicable from the roadway.

If a stormwater basin has to be located near a roadway, the following design parameters are to be met, in order of preference as listed:

a. The stormwater basin shall be designed as a dry detention basin. Under most circumstances, a buffer will not be required for safety purposes around dry detention basins if the side slopes do not exceed 3:1.
b. Landscaped mounding shall be constructed six (6) feet in height not to exceed 3:1 side slopes between the roadway and the basin in combination with the largest practicable separation between the roadway and the pond.
c. If the proposed roadside grading is such that the ground slopes continuously down from the edge of pavement to the pond, then guardrail shall be required adjacent to the roadway – regardless of the separation distance between the roadway and the pond. The Engineer shall attempt to avoid this condition whenever possible.
d. If mounding is not practicable, the pond is out of the clear zone, and the grading is relatively flat between the road and the basin, the Engineer shall incorporate a wood rail barrier - located near the top of slope down to the normal water surface. Reference the RDWD series of the standard drawings for an example of an acceptable single wood rail barrier.

The Engineer must incorporate a more robust stormwater basin buffer when any of the following conditions are present:

- High ADT (i.e. collector or arterial roadway)
- Speeds greater than or equal to 35 mph
- Curved roadway alignment
- Pond located across from T-intersection
- Parking stalls fronting a pond

Even if mounding or rail is not needed because the physical separation between the roadway and retention basin is so large, the wet retention pond must be delineated somehow (ex. Plantings, fencing, etc.) so the pond is clearly delineated and identifiable when frozen and/or covered with snow.

Guardrail may be wood timber guardrail, reference USDOT FHWA Federal Lands Highway Standard Drawings Section 617-Guardrail.

Additionally, the following criteria shall be used to define the layout and geometry of all stormwater basins in the City:

a. Detention basins shall not be located on un-compacted fill, on slopes greater than 3:1, or where infiltrating groundwater could adversely impact slope stability.
b. Where berms constructed of fill which will be over 6 feet high are proposed, calculations supporting the stability of the fill berms are to be submitted by a Geotechnical Engineer.
c. Detention basins shall be designed such that they readily accommodate flow from a site's major flood routing path(s) Overland flow from a site shall be directed to a site's detention basin(s), to ensure that all site runoff is controlled.
d. The basin shall be designed with an emergency spillway for storms that exceed the 100-year, 24-hour storm event. The emergency spillway shall be designed to direct the flow to a suitable downstream flood routing path without erosion, scouring, or soil undermining, and to meet other pertinent Ohio Dam Safety requirements. A cut section detailing the emergency spillway showing elevations and cross sections is required as a part of the plan set.
e. The basin shall be designed so that the peak water surface elevation in the basin does not overtop the basin embankment or flood structures around the basin. One foot (1-ft) of freeboard is required on all earthen basins, measured from the peak 100-yr 24-hour storm event to the top of embankment.
f. Side slopes within and adjacent to the basin must not exceed 3:1, with a 4:1 slope or flatter preferred at all locations to prevent bank erosion and minimize safety risks when the basin is full.
g. All basins shall have a 10-foot wide vehicle access way or berm adjacent to the entire perimeter of the basin to provide access for maintenance vehicles. The maximum cross slope for the vehicle access way shall be 10:1.
h. Basins shall be designed to limit the migration of groundwater from the basin towards sanitary sewers and building basements. No sanitary lines may pass under the basin or within the 100-year ponding limits of the basin.
i. All inflow pipes to the detention basin shall be designed with full height headwalls.
j. Rock channel protection shall be used to minimize erosion around the headwall, as well as along the side slopes of the basin under each inflow pipe or open channel.
k. Woody vegetation may not be planted or allowed to grow on the embankment, within 15 feet of the toe of the embankment, and within 25 feet from the principal spillway structure. The establishment of woody vegetation in other areas around the basin is encouraged to provide shade and moderate surface water temperatures.

l. Permanent stormwater quantity control basins, as defined herein, may be used as temporary sedimentation basins designed to control sedimentation during construction as long as collected sediments are removed, the design grade of the facility is restored, permanent vegetation is established, the temporary outlet is removed, and permanent outlet structure is constructed as designed.

3. Debris Control Requirements

   Debris control structures (trash racks) for both wet and dry basins may be required at the basin outlet if the potential exists for large debris to enter the detention basin through an open watercourse or large diameter inlet pipe. Debris control structures shall be designed using Hydraulic Engineering Circular No. 9, available from the U.S. Department of Transportation, Federal Highway Administration.

4. Outlet Control Structure and Outfall Protection Requirements
   a. The basin shall be designed with an outlet control structure sized to meet the stormwater quantity and quality control requirements
   b. The outlet structure shall be designed to minimize the transport of floating debris, oil, and grease through the detention facility. The design of the facility shall also include adequate provisions to minimize erosion in the vicinity of the inlet and outlet, and on the side slopes of surface facilities. Open channels receiving discharges from the facility shall be protected with rock channel protection.
   c. Seepage along any conduit that extends through the basin embankment to the downstream slope shall be controlled using an approved, engineered anti-seep collar or clay dam.
   d. The minimum orifice size is 4 inches in diameter. Required orifice openings of less than 4-inches in diameter must conform to current control structure specifications per ODOT STD DWG WQ-1.1 & W-Q1.2. Alternative outlet designs (e.g., V-notch weir, perforated riser pipes, etc.) may be permitted upon the approval of the Public Works Department and if acceptable design practice is proven for site conditions.

5. Useful Equations for Basin Routing
   a. Weir Equation ~ Broad-crested Weirs and Spillways

      Typically a basin emergency spillway is a broad-crested weir or an ogee spillway. A weir is considered broad-crested if the weir thickness is greater than half of the head (H). An ogee spillway is one that is designed with an ogee shaped bottom to minimize cavitation of the overflow. The flow through both of these weirs can be approximated by the following equation:

$$Q = \frac{2}{3} \cdot C_s \cdot b \cdot \sqrt{2 \cdot g} \cdot H^{3/2}$$

      Where,
      C_s = 0.50 to 0.57 (broad-crested)
      C_s = 0.60 to 0.75 (ogee spillway)

b = bottom width
H = head
g = 32.2 ft/sec^2

b. Orifice Equation

$$Q = C_d \cdot A \cdot \sqrt{2 \cdot g \cdot h}$$

Where,
C_d = discharge coefficient
A = cross-sectional area of the orifice
h = elevation of the water surface above the center of the orifice
g = gravitational constant (32.2 ft/sec^2)

6. Additional Layout Requirements for Dry Detention Basins

   The following additional requirements shall apply to the design of dry detention basins for stormwater quantity control:

   a. Dry detention basins shall be designed to drain toward the outlet or micro-pool in order to minimize standing water and saturated soil conditions that impede maintenance and mowing of the facility.
   b. The detention basin bottom shall be sloped to drain, and such slopes shall be sufficient to militate against "flat spots" developing due to construction errors and soil conditions. The minimum transverse slope for the bottoms of such facilities shall be 2%. Basins with less fall across the bottom shall include a paved low flow channel from each inlet pipe or open channel to the basin’s outfall.
   c. Low flow channels shall be designed per the following requirements:
      i) Bottom width – minimum width shall be 4 feet
      ii) Side slopes – shall not be steeper than 4:1,
      iii) Channel slope – minimum slope toward the basin outlet shall be 0.5 percent for channels with paved bottoms,
      iv) Channel depth – minimum depth of channel shall be 0.5 feet
      v) The bottom and side slopes of the paved channel shall be Class C concrete reinforced with steel mesh a minimum of 6 inches thick,
      vi) Weep holes shall be designed in the concrete side walls.
   d. The minimum bottom width for dry detention basins, other than the low flow channel, shall be 12 feet.
   e. Dry detention basins shall be provided with topsoil, and shall be seeded and mulched to prevent erosion. Grasses seeded within the basin should be able to survive 48 hours under water. Jute and Excelsior matting shall be used as required to stabilize slopes and prevent erosion.

7. Additional Layout Requirements for Wet Detention Basins

   The following additional requirements shall apply to the design of wet basins for stormwater quantity control:

   a. The permanent pool must have adequate depth to during the dry season. The minimum depth of from the normal pool elevation to the bottom of the basin shall be 8-feet. The depth of wet detention basins shall be no more than 12 feet below the basin's normal water elevation. The minimum bottom width of wet basins shall be 12 feet.

b. The perimeter of all permanent pool areas deeper than 4 feet shall be surrounded by an aquatic bench that extends at least 8 feet and no more than 15 feet outward from the normal water edge. The aquatic bench shall have an average depth of 24 inches below the permanent pool to promote the growth of aquatic vegetation. The maximum slope of the aquatic bench shall be 10:1

c. The City prohibits the construction of wet detention basins and stormwater wetlands 5,000-ft from the municipal airport, as recommended by the Federal Aviation Administration (FAA), Advisory Circular Number 150/5200-33. As an alternative stormwater best management practice that does not maintain a permanent pool of water and is not as likely to attract large numbers of waterfowl is required in the vicinity of the airport. An access easement that extends 12 feet from the top of bank outward for any BMP in this zone shall be dedicated to the City of Delaware to ensure that proper maintenance is performed. See standard details for additional specifications and details.

d. Submerged Outlets

   i) Submerged outlets may be permitted provided a structure is constructed between the outlet at the retention basin and the main storm system. The pipe can be submerged to this first structure only. This structure must also be after the last pavement crossing. The structure shall have a grated cover

   ii) The pipe from the pond outlet to the first manhole shall have watertight joints.

   iii) The invert elevation of the pipe entering into this first structure will be at least 0.5-feet above the normal pool elevation.

   iv) The slope of the basin at the outlet shall be no flatter than 2:1 to avoid siltation at the outlet.

8. Parking Lot Storage

Parking lot storage is a stormwater quantity control method allowing shallow ponding within paved portions of the parking lot during the design storm event. Parking lot storage is a convenient multi-use structural control method where impervious parking lots are planned. The following criteria shall apply to parking lot storage facilities:

a. Ponding in parking or traffic areas shall be designed for a maximum ponding depth of twelve (12) inches for all storms up to and including the 100-year event. Flood routing or overflow to a designed conveyance system must occur after the maximum depth is reached.

b. Runoff from specific graded areas within a parking lot shall be controlled by orifices. The release rate of the flow from a parking lot storage facility shall meet the allowable post-development runoff criteria. The minimum size outlet device shall be a 4-inch single orifice for water quantity control. Alternative outlet designs of smaller diameter may be permitted with Public Works Department approval.

c. A site with a parking lot storage facility shall employ a separate water quality treatment This BMP may be located either downstream of the parking lot or integrated into the medians, landscaping, or other pervious areas of the parking lot.

d. Ponding areas must also be designed so that a minimum of 8 feet of traveled way is provided during a critical-year storm if a storm structure is located in a drive aisle or along an inverted crown.

9. Underground Storage Units

Underground storage is a stormwater quantity control method that employs a series of underground pipes or chambers and has a designed release feature to control stormwater discharge. This method is most applicable where land is valuable or the site is constrained, such as in industrial, commercial, and redevelopment areas. Construction costs and operation costs, which may include pumps, make this method relatively expensive.

a. If underground storage is to be used for a site, a plan for long term maintenance of the facility shall be provided to the Public Works Department, including a health and safety plan for confined space entry.
b. The release rate of the flow from an underground storage facility shall meet the calculated allowable post-development runoff rate.
c. A site with an underground storage facility shall employ a separate water quality BMP that meets all of the water quality treatment criteria.

C. Stormwater Quality Controls

Post Construction Storm Water Best Management Practices (BMPs) are provided for perpetual management of runoff quality and quantity so that a receiving stream's physical, chemical and biological characteristics are protected and stream functions are maintained.

For stormwater quality requirements, Chapter 16 should be referenced. This section only provides design requirements for specific BMPs that provide stormwater quality treatment. In addition to this section, requirements of the current edition of the Ohio Rainwater and Land Development Manual shall be followed in the design of stormwater BMPs.

For all projects with one or more acre of development or projects that are a part of a larger common plan of development which will have one or more acre of development, BMPs shall be incorporated into the permanent drainage system for the site.

1. Stormwater Wetlands

Stormwater wetlands are similar in design to wet basins. Constructed wetlands treat stormwater by providing an extended detention zone above shallow permanent pools that are sized to capture and release the calculated WQv over a minimum time of 24 hours. Stormwater wetlands are heavily planted, depressed areas that are designed to maintain flow during dry periods in order to support aquatic vegetation. The amount of surface area required for a stormwater wetland is typically larger than that of a wet basin due to the limited allowable depths required for wetland design. The following criteria shall apply to the design of stormwater wetlands.

a. Owners of wetland systems must agree to provide a mosquito monitoring and control plan within the maintenance plan.
b. The recommended minimum drainage area of 20 acres is proposed to avoid wetlands where the permanent pool completely evaporates during dry weather conditions.
c. The permanent pool of any proposed stormwater wetland shall be at least two times the volume of evapotranspiration during a thirty day drought at summer evaporation rates or 0.75 WQv, whichever is greater.
d. An extended detention volume equal to the WQv shall be provided above the permanent pool of the stormwater wetland. The outlet structure of the stormwater wetland shall be designed with a WQv draw-down time of not less than 24 hours.
e. The minimum length-to-width ratio for a constructed wetland shall be 2:1. Where site conditions allow, basins should be wedge-shaped, narrowest at the inlet and widest at the outlet, to achieve the required length-to-width ratio. Where site conditions do not allow this configuration, the length-to-width ratio shall be increased by relocating the basin inlet or outlet where possible, or by installing berms or baffles within the basin to the full depth of the WQv to avoid short-circuiting and to increase travel time to the outlet.
f. Approximately 50 percent of the permanent pool volume, plus a sediment storage volume equal to at least 20 percent of the WQv, shall be placed in deep water zones (areas with depths between 4- and 12-feet) to sustain fish communities and provide wave action to control mosquito populations. At a minimum, deep water zones shall be placed (1) within the forebay and (2) around the primary outlet.
g. The remainder of the facility shall consist of shallow water zones. Dry weather depths in shallow water zones (i.e., areas less than 18 inches deep) should vary depending on the vegetation selected. Permanent pool depths shall be 6 inches or less within at least 35 percent of the shallow water zone.
h. The bottom of the permanent pool between the deep and shallow water zones shall be sloped no steeper than 3:1.
i. The maximum depth of the extended detention zone above the permanent pool shall not exceed 2 feet to reduce stress on herbaceous wetland plants.
j. Permanent pool areas of wetlands that are deeper than 4 feet shall be provided with an aquatic safety bench. Wetland plants shall be placed along the aquatic bench and other shallow pool areas (less than 4 feet).

Due to the sensitivity of wetland vegetation to sedimentation, a forebay, or other pretreatment feature, shall be provided at the inlets of all stormwater wetlands. Wetland forebays shall be three feet to six feet in depth and shall meet the following minimum requirements:

a. Wetland forebays shall be sized to provide at least 10 percent of the WQv. The storage volume provided within the forebay will count toward the total WQv requirement.
b. The forebay shall consist of a separate cell, formed by an acceptable barrier such as a rock weir.

c. Direct maintenance access shall be provided to the forebay at a slope no steeper than 10:1.
d. Forebay side-slopes shall not exceed 3:1.
e. A fixed vertical sediment depth marker shall be installed in the forebay to measure sediment deposition over time. The maintenance schedule must address the dredging of accumulated sediment from the forebay.
f. A landscaping plan shall be provided that indicates the methods used to establish and maintain wetland coverage. Minimum elements of a plan include: delineation of landscaping zones, selection of appropriate plant species, planting plan, sequence for preparing the wetland bed (including soil amendments, if needed), and sources of plant material. The width of the vegetation zones and amount of emergent vegetation shall be limited to no more than 50 percent of the wetland area in order to control mosquitoes. If a minimum vegetative coverage of 50 percent is not achieved in the planted wetland zones after the second growing season, a reinforcement planting will be required. All landscaping and reinforcement plans shall be signed by a landscape architect, with direction provided by an experienced wetland scientist.

2. Vegetated Swales

Vegetated swales are open, shallow channels with vegetation covering the side slopes and bottom that collect and slowly convey runoff flow to downstream discharge points. They are designed to treat runoff through filtering by the vegetation in the channel, filtering through a subsoil matrix, and/or infiltration into the underlying soils. Swales can be natural or manmade.

Vegetated swales are designed to treat the entire WQv from the tributary area. Since treatment occurs as stormwater “flows through” the swale, it is necessary to develop a design peak flow for the WQv representative of flow conditions that maximize treatment and retard flow. The following procedure shall be used to develop the design peak flow:

a. Determine the time of concentration for the area draining into the swale.
b. Define the peak intensity of the design storm producing the WQv using the intensity-duration-frequency curve
c. Use the rational method to determine the peak flow through the swale.

The topography of the site shall be used or re-graded as needed to design a channel with appropriate slope and cross-sectional area. Site topography may also dictate a need for additional structural controls.

Swales are generally recommended for drainage areas less than 5 acres, with a total drainage area of 1 to 2 acres preferred.

Trapezoidal channels are normally recommended, but other configurations, such as parabolic, can also provide substantial water quality improvement and may be easier to mow than designs with sharp breaks in slope.

If the design peak for the WQv is larger than 1 cfs, then more than one swale shall be provided in order to maintain sheet flow and shallow flow depths within the swale. Flow shall be distributed among the swales to keep peak flows during the WQv design event less than 1 cfs in each.

Use Manning's Equation to design the swale under peak WQv design storm conditions, using criteria within this section to establish design limits for the swale.

The swale shall be designed so that the water level does not exceed 3 inches at the design peak flow for the WQv.

It is recommended that longitudinal slopes along the swale shall not exceed 2.5 percent, and shall be milder if necessary to keep the peak velocity within the swale less than 0.9 feet/second. However, longitudinal slopes may range between 2 and 6 percent. Flatter slopes can be used, if sufficient to provide adequate conveyance. Steep slopes increase flow velocity, decrease detention time, and may require energy dissipating and grade check. Steep slopes also can be managed using a series of check dams to terrace the swale and reduce the slope to within acceptable limits. The use of check dams with swales also promotes infiltration.
The width of the swale should be determined using Manning's Equation, at the peak flow during the water quality design storm, using a Manning's "n" of 0.25. The maximum bottom width shall not exceed 10 feet unless a dividing berm is provided.

The swale shall have a length that provides a minimum hydraulic residence time, or the time it takes for the water to pass through the swale, of at least ten minutes. Regardless of the hydraulic residence time, the length of the swale shall not be less than 100 feet.

Swales may be designed to safely convey storms generating more than the WQv. The peak velocity of the 10-year design storm through the swale shall be non-erosive for the soil and vegetative cover provided Three inches of freeboard should be provided. The side slopes of the swale shall be no steeper than 3:1.

Swales must be vegetated in order to provide adequate treatment of runoff. It is important to maximize water contact with vegetation and the soil surface. For general purposes, select fine, close-growing, water-resistant grasses. If possible, divert runoff (other than necessary irrigation) during the period of vegetation establishment. Where runoff diversion is not possible, cover graded and seeded areas with suitable erosion control materials.

The surface shall be graded flat prior to placement of vegetation. Initial establishment of vegetation shall receive attentive care, including appropriate watering, fertilization, and prevention of excessive flow, until vegetation completely covers the area and is well established.

Runoff shall enter swales as sheet flow. Use of a level spreading device (vegetated berm, sawtooth concrete border, rock trench, etc.) to facilitate overland sheet flow may be allowed, but is not normally recommended because of maintenance considerations and the potential for standing water.

3. Bioretention Facilities

A bioretention system consists of a soil bed planted with native vegetation located above an underdrained gravel layer. Stormwater runoff entering the bioretention system is filtered first through the vegetation and then the soil bed before being conveyed downstream through the underdrain system, slowing the runoff velocity and treating stormwater runoff by absorption, decomposition, and filtration. Bioretention facilities are often sited adjacent to and used to treat runoff from paved surfaces such as parking lots. Sites utilizing bioretention facilities for water quality control must also meet the stormwater quantity control requirements. Stormwater quantity controls may either be integrated into the bioretention system or provided in a separate downstream facility.

Bioretention facilities shall be designed to capture and store the WQv prior to filtration and shall provide a filtration time of no less than 24 hours (when the filter media is new) and no more than 40 hours (when the filter media is clogged and requires maintenance). A drawdown time of 40 hours shall be used for facility design. The following criteria shall apply to the design of bioretention facilities.

The recommended maximum total drainage area of a bioretention facility is 5 acres, with drainage areas of one acre or less preferred in order to maximize sheet flow into the facility and to minimize ponding depth.

Bioretention systems may be constructed on-line or off-line. On-line systems receive runoff from all storms, providing treatment of the WQv, with runoff from larger storms conveyed and/or stored within the bioretention system and discharged through an overflow. In off-line bioretention systems, most of the runoff from storms larger than the
WQv bypasses the system through an upstream diversion and is directed toward a stormwater quantity control device.

Bioretention facilities shall not be allowed in areas where the water table or bedrock is above the invert of the underdrain system.

In order to maximize treatment effectiveness, the site must be graded in such a way that minimizes erosion as sheet flow is conveyed to the treatment area.

Runoff from the tributary area of the bioretention facility shall be directed into a swale or other storage area sized to contain the entire WQv. This swale shall partially or completely overlay the bioretention facility, as long as the maximum depth of water over the filtering media is no more than 12 inches. Water shall not cover the media longer than 72 hours after a precipitation event.

The surface area of the bioretention soil bed shall be determined based on the following equation:

$$A = \frac{WQv \cdot d}{\left[3600 \cdot K \cdot T \cdot (h + d)\right]}$$

where:
A = surface area of the bioretention planting soil bed (acre)
WQv = water quality volume (acre-ft)
d = depth of the planting soil bed (ft)
T = drawdown time (hours) = 40 hours
K = permeability of the planting soil within the soil bed (feet/sec) = 1.2×10^{-5} feet/sec (minimum), which is equivalent to 0.5 inches/hr
h = average depth of water above filter bed (ft) = half the maximum depth of water (maximum depth = 12 inches)

The minimum dimensions of the bioretention facility shall be 15 feet wide by 40 feet long, with facility widths of 25 feet preferred. These width criteria are established to allow enough space for a dense, randomly-distributed area of trees and shrubs to become established, enabling the bioretention area to tolerate the effects of heat stress, acid rain, runoff pollutants, and insect and disease infestations.

The side slopes for bioretention facilities shall not exceed 3:1.

The function of a bioretention facility is largely dependent on the characteristics of the planting soil (filter media) through which the runoff passes. The following criteria shall be used:

a. The planting soil for bioretention facilities shall consist of a mixture of sand, topsoil, and compost with a pH range of 5.5 and 6.5, a range where pollutants such as organic nitrogen and phosphorus can be absorbed by the soil and microbial activity can flourish. In addition, the soil shall have infiltration rates no less than 0.5 inches per hour, achieved through the following standards:
b. 4 parts sand
c. 2 parts topsoil, and
d. 2 parts compost.
e. Additional recommendations for the planting soil include a 1.5 to 3 percent organic content and a maximum 500 ppm concentration of soluble salts.
f. If the existing soil does not meet the above characteristics, then it shall either be adjusted to meet the criteria or removed and replaced with an acceptable planting soil. Soil tests shall be performed for every 500 cubic yards of planting soil, with the exception of pH and organic content tests, which are required only once per bioretention area.
g. To prevent root intrusion into the underdrain system, the depth of the planting soil shall be no shallower than the root zone of the vegetation planted in the bioretention cell. The minimum depth shall be 2.5 feet. In addition, the depth shall be at least 4 inches below the largest root ball.
h. Planting soil depths of greater than 4 feet may require additional construction practices such as shoring measures. Planting soil shall be placed in 18 inches or greater lifts and lightly compacted until the desired depth is reached.
i. An 8-inch thick sand layer shall be placed along the sides of the trench, starting 12 inches below the final grade level and extending to the gravel

layer in the bottom of the trench. The sand shall comply with CMS 703.06, and shall be surrounded with a filter fabric.

Bioretention facilities shall be planted with a mixture of grass and other hardy, vegetation that can withstand prolonged periods in a wet environment, and be tolerant to road salts if receiving runoff from areas to be treated with deicing materials. Vegetation may include a mix of grasses and woody species, or may include woody species only with bare ground covered with mulch. Approximately one tree or shrub per 50-ft2 (or 1000 per acre) of bioretention area should be included. It is recommended that three species each of both trees and shrubs are planted. Trees with high branching or open habits of growth are recommended to avoid shading and loss of grass cover. Since high canopy trees may be destroyed during maintenance the bioretention area should be vegetated to resemble a terrestrial forest community ecosystem that is dominated by understory trees. The shrub-to-tree ratio shall be 2:1 to 3:1.

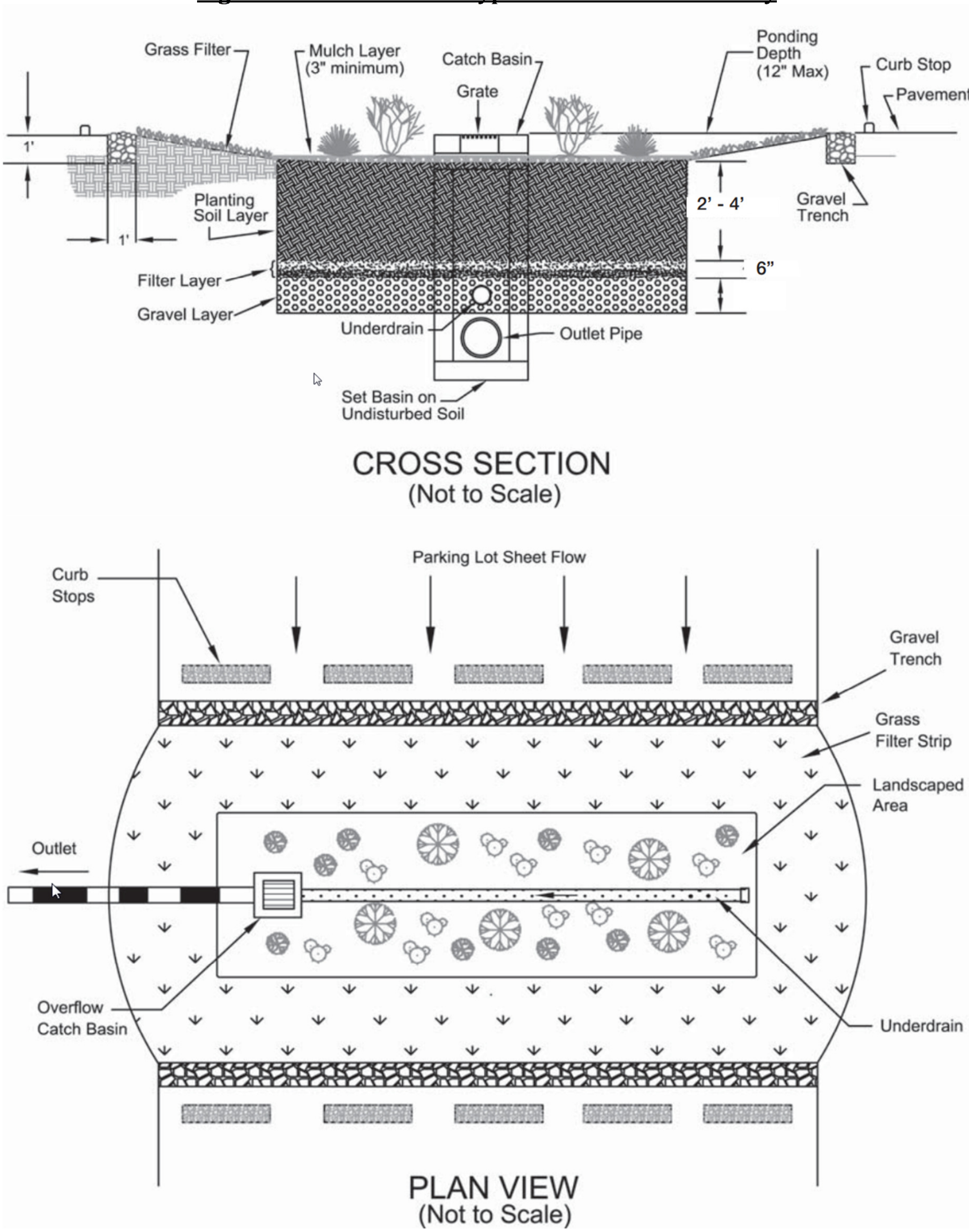
A mulching layer 2 to 3 inches thick shall be provided above the planting soil when grass is not used, and shall be composed of shredded hardwood material to avoid floating. Mulch plays an important role in the bioretention facility. It helps maintain soil moisture, prevents erosion and helps to trap finer sediments.

A perforated pipe underdrain shall be provided beneath the planting soil. The underdrain shall have a minimum grade of 0.5 percent. The perforated pipe shall have a diameter of 4 or 6 inches. A granular backfill of durable No. 57 aggregate shall be provided up to a minimum of 4 inches above the outside diameter of the pipe.

An overflow designed to convey all storms up to and including the 100-year event shall be provided. Use of a vertical stand pipe or catch basin is recommended. For on-line facilities, this overflow may be designed to achieve the water quantity control criteria.

Flow entering the bioretention facility shall be limited to sheet flow to prevent eroding the side slopes of the facility. If flow has been concentrated prior to entering the bioretention facility, it shall be converted to sheet flow using a level spreader. In addition, the frequency of maintenance for bioretention facilities may be reduced by providing vegetated swales or filter strips around the facility. The purpose of the filter strip is to trap course sediments before they reach filter media, thereby reducing maintenance and increasing media longevity. Vegetated swales and filter strips can also be used as holding areas for the WQv prior to filtration.

Figure 1: Schematic of a Typical Bioretention Facility



PART 4 – DESIGN OF MINOR STORMWATER CONVEYANCES

1. Methodology

The design of minor stormwater conveyances shall be performed utilizing the Rational Method (Q = CiA).

2. Time of Concentration

One of the basic assumptions underlying the Rational Method is that runoff is a function of the average rainfall rate during the time required for water to flow from the most remote part of the drainage area under consideration to the design point. For urban storm sewers, the time of concentration consists of an inlet time, or time required for runoff to flow over the surface to the nearest inlet, and time of flow in the sewer to the point under consideration. Inlet time will vary with surface slope, depression storage, surface cover, antecedent rainfall and infiltration capacity of soil, as well as distance of surface flow. Pipe flow time can be calculated by Manning's Formula.

$$V = \frac{1.49 r^{2/3} s^{1/2}}{n}$$

Where,
V = average velocity (ft/s)
r = hydraulic radius (ft) and is equal to a/pw
A = cross sectional flow area (ft2)
pw = wetted perimeter (ft)
s = slope of the hydraulic grade line (channel slope, ft/ft)
n = Manning's roughness coefficient for open channel flow.

$$Q = V \times A$$

Where,
Q = flow in cfs (ft3/sec)
V = velocity (ft/sec)
A = Cross sectional flow area (ft2)

3. General Requirements

A. All drainage calculations, drainage area outlines, and contributing areas used in drainage design shall be furnished.
B. All stormwater calculations to be signed, stamped and dated
C. Easements are required for all drainage ways that pass from one parcel to another or which contain public storm systems, either surface or subsurface, on private property. Where an open watercourse is used, an easement shall be provided at least equal to the area required for the 100-year rainfall. The easement shall be shown on the final plat as a "Drainage and Utility Easement." Restrictions as to the use of this easement shall be shown on the final plat. If a plat is not recorded, a recorded easement is necessary.

4. Flood Routing Path

A. The routing path for the major drainage system is that part of the storm drainage system which carries the runoff which exceeds the capacity of the designed drainage facilities. The major drainage system shall have the capacity to carry runoff from a storm with a return period of not less than 100 years without causing significant threat to property or public safety. The peak water surface elevation of the flow shall be at least one-foot below the first floor elevation of the structures within and adjacent to the development. The major storm routing path shall begin along swales located between structures that drain individual properties, be directed to either roads, other public rights-of-way , or constructed open watercourses through the development, to the stormwater detention facility serving the development. This detention facility shall be designed to control the 100-year event without overtopping its embankment.

B. Generally, it is not economically feasible to size a storm sewer system to collect and convey more than the frequent storm runoff. Essentially, the complete drainage system of an urban area contains two separate drainage elements. While the storm sewers belong to the initial system, surface drainageways must be provided for the major flow from more intense storms. The intent of planning for the major drainage element is to ensure stormwater runoff which exceeds the capacity of the initial drainage system has a route to follow which will not cause a major loss of property or any loss of life. It should be remembered that the major drainage system exists even when it is not planned and whether or not development exists with respect to it. To get in the way of the major storm is careless and costly.

C. A hydraulic analysis shall be required to verify that the peak water surface elevation during the 100- year design storm meets the design criteria cited in this section. For preliminary design purposes, the flow in the major drainage system during the 100-year design storm event shall equal the design capacity of the major system.

D. The use of normal flow depths derived using the Manning's Equation will suffice for estimating inundation limits along streets. At culverts, the major storm shall be designed to flow across streets at low areas or in sags of vertical curves. Street elevations shall be set to permit the major storm to flow across the street and to prevent damage to any existing or proposed building structure. Backwater calculations shall be performed along USGS Blue Line streams where a roadway crossing over these streams is proposed as part of the development. The backwater analysis shall proceed upstream from the roadway crossing to the boundary of the development site.

E. Where a major drainage way is located outside of a street right-of-way, easements shall be provided. The 100-year flood routing path shall be shown on the master drainage plan that is to be submitted with the Stormwater Management Report. Routing path illustrations shall include elevations along the routing path and other elevations necessary to show that the major storm is contained within the planned area and dedicated easements.

F. Street rights-of-way are a common choice for conveying major drainage flows. Again, such use must be anticipated when the street layout is established. Side and rear lot lines offer one alternative to the street. The problem with this alternative is the possibility that individual property owners may encroach on the major drainage easement. Rarely is the problem recognized until the infrequent rainstorm occurs and the major system fails to operate properly. Where the street is designated as the major drainage way, the depth of flow shall not exceed 12 inches at the gutter line for local and collector streets and shall not exceed 6 inches of depth at the crown for arterial streets. The same maximum depth criteria will apply where a major drainageway crosses the street. Where a major drainageway is located outside of a street right-of-way, easements should be provided and a grading plan should be submitted with detailed engineering plan submission. The grading plan will include elevations along the routing path and other elevations necessary to show the major storm is contained within the planned area.

G. In order to protect the integrity of non-street drainage rights-of-way, it is effective to design routing paths for multi-purpose functions. Pedestrian, bicycle paths, and utility easements lend themselves naturally to this application. Linear parks aligned along the major drainage corridor are also very effective, but usually require greater width than would normally be necessary for drainage purposes.

H. The major storm runoff is routed through the drainage system to determine if the combined capacity of the routing path and storm sewer system is sufficient. The capacity of the conduit at any given point is assumed to be the same for the major storm as for the initial storm. (the flood route capacity shall equal the 100-yr flow minus the 2-year flow) Routing of the major storm at culvert locations shall be at low areas or sags of vertical curves of streets. Elevations for the design of the street shall be such to permit the major storm to flow across the street and to prevent damage to any existing or proposed building structure.

I. Certain limited areas of the City lie within the floodplain of the Olentangy River or its tributaries. The City shall maintain on file for public examination current maps delineating the boundaries within the City of all lands designated "100-year Floodplain" by the Federal Emergency Management Agency (FEMA). All improvements in a floodplain shall be in compliance with the City of Delaware regulations pertaining to floodway districts.

5. Culvert Design

A. General

A culvert is a relatively short segment of conduit that is typically used to transport water underneath a roadway or other type of earthen embankment. Two types of flow may occur in culverts: flows with inlet control and flow with outlet control. The design engineer shall determine the appropriate flow conditions for each culvert in the project. Culvert Design should be according to the principles outlined in the Ohio Department of Transportation, Location and Design Manual, Volume 2, Drainage Design, current edition. The publication, Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5, Federal Highway Administration 1985 by J. M. Normann, R. J. Houghtalen, and W. J. Johnston, also outline an acceptable design procedure.

B. Design Procedure

1. The design must account for the ability of the downstream channel to handle the flow satisfactorily. Additionally, in order to minimize property damage, the 100-year frequency head water shall not exceed any existing or proposed building first floor elevation.

2. Culverts are to be designed to transport the flows from the design year storm listed in the following table.

Table 13: Design Year Flows

Roadway Type	Design Storm Event
Interstate Highways, Other Freeways and Expressways	50-Year
Major Arterials (ADT>10,000)	25-Year
Minor Arterial and Collectors (ADT 3,501 – 10,000)	25-Year
Local Streets and Roads (ADT ≤ 3,500) and Parking Lots	10-Year

3. In addition to conveying the design year flows, the design of the culvert must take into account the maximum allowable headwater for the 100-year event. The maximum headwater on all public streets and bike paths is 1-foot below the near, low edge of pavement or 18-inches below the top of curb elevation. The maximum allowable headwater is at or below the near edge of pavement for driveway culverts conveying runoff along roadside ditches.

4. The maximum allowable outlet velocity for culverts is as follows:
   a. Bare Earth Channel 6 fps
   b. Engineered Rock Protection 18 fps
   c. Stilling Basin Over 18 fps

5. Culverts shall have full height headwall with flared wings at upstream and downstream ends. Other special type headwalls must be approved before use.

6. The structural design criteria for culverts will be as required by the City of Delaware together with the City of Columbus, Ohio, Construction and Material Specifications, including all specifications thereto.

6. Storm Sewer System Design Requirements

A. Methodology

The method outlined herein will provide a general guide as to the criteria and procedures to be used for storm sewer design.

1. Storm sewer pipe sizes are to be determined by using Manning's Formula with a minimum coefficient of roughness n = 0.013 for all pipe laid to proper line and grade.
2. The Rational Method shall be used for all storm sewer design (closed system.)
3. The following rainfall frequency curves shall be used for design:
   a. Storm Sewer Capacity - 2-year rainfall
   b. Strom Sewer Hydraulic Grade Line – 5-year event below the top of casting
4. The minimum post-development time of concentration for storm inlets is 15 minutes in grassy areas and 10 minutes in paved areas.
   a. All future and offsite flows must be accounted for.
   b. Match crowns where possible for all storm systems. If matching crowns is not feasible due to site constraints match 0.80 diameters.

B. General Requirements

1. A chart with State Plane Coordinates Including blank columns for as-builts must be provided for all public and private storm structures.
2. Minimum cover for storm sewer pipe shall be 30-inches from the top of pavement to the outside top of the pipe except as approved by the Public Works Department. If minimum cover is not met under any private or public street the conduit shall be concrete encased from structure to structure per City of Delaware standards. Under no circumstances will less than 9-inches of cover to the bottom of the aggregate base be permitted.
3. A minimum of 1.5-feet of cover from the finished ground surface to the outside top of the pipe for any storm sewer system located beyond the limits of street right-of-way must be maintained.
4. There must be 1.5-ft vertical and 10-ft horizontal separation (outside wall to outside wall) from all sanitary and water lines
5. Standard full height headwalls are to be constructed at the inlet and outfall of all daylighted storm sewers, and shall be shown on the plan and profile. Half height headwall may be permitted for large (36-inch and greater) pipes that are submerged outlets to a retention basin.
6. Erosion control shall be provided at the outfall of all storm sewers, and shown on the plan and profile. In addition, measures approved by the City shall be taken to control erosion during construction in accordance with the current Water Pollution and Sediment Runoff Control Regulations.
7. The invert of the first downstream storm sewer outlet shall be above the computed floodplain elevation.
8. Pipe for all storm sewers (public and private) shall not be less than 12-inches in diameter unless solely serving as roof drain collector.
9. All structures are to be; City of Delaware, ODOT, or City of Columbus standard structures with the exception of combined curb and gutter inlets, which must be City of Delaware standard structures only. The inverts of all curb inlets, manholes, yard inlets, and other appurtenances shall be formed to reduce turbulence to a minimum.
10. Blind taps are not permitted.
11. The maximum distance for overland flow shall be 300 feet before entering a surface yard inlet or 425 feet before entering a curb inlet. Except that the maximum overland drainage area tributary to any yard inlet or curb inlet shall not exceed 1 acre. At the discretion of the City and on a case by case basis, an exception may be made where there is phased construction or for structures that are receiving large amounts of offsite flow.
12. The maximum spacing of storm access structures is 300-feet for pipes 12-inches to 60-inches in diameter and 500-feet for pipes greater than 60-inches in diameter.
13. Rear yard swales must be designed at a a minimum 1.5% slope and have storm structures constructed on every other property line. The preferred slope on rear yard swales is 2.0%. All rear yard swales with a slope less than 2% shall have an underdrain for the full length of the swale that outlets to a catch basin.
14. The minimum allowable velocity shall be 3 feet per second (fps) in all storm pipes in the just full condition and the minimum Manning's "n" value shall be 0.013. (See table below) The maximum allowable velocity shall be 15 fps.

Table 14: Minimum Slope for all Storm Conduits

Pipe Size (inches)	Pipe Size (feet)	Minimum Slope for a Velocity of 3 FPS	Capacity of Pipe At Minimum Slope (cfs)	Elliptical Equivalent (inches)
4	0.34	1.89%	0.26	---
6	0.50	1.10%	0.59	---
8	0.67	0.75%	1.05	---
10	0.83	0.56%	1.64	---
12	1.00	0.44%	2.36	---
15	1.25	0.32%	3.68	---
18	1.50	0.25%	5.30	14 x 23
21	1.75	0.21%	7.22	---
24	2.00	0.17%	9.42	19 x 30
27	2.25	0.15%	11.93	23 x 34
30	2.50	0.13%	14.73	24 x 38
36	3.00	0.10%	21.21	29 x 45
42	3.50	0.08%	28.86	34 x 53
48	4.00	0.07%	37.70	38 x 60
54	4.50	0.06%	47.71	43 x 68
60	5.00	0.05%	58.90	48 x 76
66	5.50	0.05%	71.27	53 x 83
72	6.00	0.04%	84.82	58 x 91
78	6.50	0.04%	99.55	63 x 98

B. Profiles

1. Profiles are required for all Public and Private storm sewers
2. The storm conduit material and class is to be listed on both in the quantities and on the profiles.
3. The 5-year hydraulic grade line shall be shown on the profile sheet. The hydraulic grade line shall be calculated based on an observed or calculated tailwater depth in the receiving channel or basin determined through downstream analysis or the following equation, whichever is greater:

   $T_w = (d_c + D)/2$

   where:
   Tw = Tailwater depth (feet)
   d_c = Critical depth in the pipe (feet)
   D = Inside pipe diameter (feet)

C. Backfill Requirements

1. Compacted granular backfill, COC Item 912, is required beneath all paved areas, including public and private roadways and parking lots.

2. COC Item 911 is required outside of the pavement influence, but inside the Right-of-Way. A minimum of 95% compaction for all fill material is to be maintained.
   Storm Sewers between homes shall be reinforced concrete pipe (RCP) with watertight rubber gasket joints (ASTM C-443) and have no granular bedding or backfill.

D. Curb and Gutter Inlets

The satisfactory removal of surface water from curbed pavement is as important as any other phase of stormwater control. Gutter flow in streets serves to transport runoff from surface areas to storm inlets or to open drainage channels. The primary function of the street inlet is to provide stormwater runoff flow from the gutter into the storm sewer. Spread of water on the pavement for the design storm is considered as the best control for pavement drainage. Standard City of Delaware curb and gutter structures and frames are to be used for all land development projects.

1. <u>Maximum spread of flow:</u> Spread calculations are required for all public curb and gutter inlets. The design storm is the 5-yr event for local and collector roads and the 10-yr event for Arterials.
   a. Local streets with a width of 28-foot from back of curb to back of curb or less, the flow may spread to a width of 8 feet from face of curb.
   b. Local streets with a width greater than 28-foot and up to 38-foot back of curb to back of curb, the flow may spread to a width of 9 feet from face of curb.
   c. Collector streets, a minimum of one lane in each direction, to remain passable to traffic.
   d. Arterials are per current edition of ODOT's Location and Design Manual, Volume II.

2. <u>Design Criteria for Curb and Gutter inlets:</u> The maximum spacing for curb inlets shall not exceed 400 feet unless approved by the City.

3. <u>Sag or Sump at Adverse Slopes:</u> Curb and gutter inlets are required at all sag locations. On combined runs of over 300 feet contributing to a sag curve, an additional inlet may be required near the low point, 0.2 + or - feet above the inlet at the sag.

4. <u>Continuous Grade:</u> Curb inlets shall be located at all points where the maximum pavement encroachment is reached and at the low points of street intersections. No cross street flow is permitted. The City may require additional inlets at intermediary points if the flow in the gutter at design conditions might create a hazard to vehicular traffic, public safety, or property flooding. The rational method and a minimum time of concentration of 10 minutes shall be used to determine the amount of runoff that will be collected by the proposed inlet structures. Hydraulic analyses used to size and space inlets and catch basins shall be based on the methods presented in (FHWA) Hydraulic Engineering Circular No. 12: "Drainage of Highway Pavements" and Hydraulic Engineering Circular No. 22: "Urban Drainage Design Manual." The following table

summarizes the dimensions of the inlets and catch basins that are provided in the City of Delaware's standard drawings. These dimensions may be used with the design aids (i.e., charts, graphs, nomographs, etc.) provided in the references cited above to assist in determining the capacity and spacing of the inlets and catch basins under different pavement and flow conditions.

Table 15: Selected COD & COC Basin Grate & Curb Inlet Dimensions

Standard Drawing	Description	Clear Opening Area, A (ft^2)	Grate Length, L (ft)	Grate Width, W (ft)	Inlet Length (ft)	Inlet Height (ft)**
SEWD 09*	Combination Curb & Gutter Inlet*	3.27	2.95	1.48	-	-
SEWD 11	Square Catch Basin	2.5	1.9	1.9	-	-
AA-S115	Round Catch Basin	2.9	3	3	-	-
AA-S116	Round Catch Basin	2.7	3	3	-	-
AA-S123	42 inch Curb Inlet	-	-	-	3.5	>4
AA-S123	60-inch Curb Inlet	-	-	-	5	>4
AA-S126	Standard Curb Inlet	-	-	-	2.33	6.25
AA-S133	Square Catch Basin	2.5	3.9	2.1	-	-
AA-S138	Round Catch Basin	0.9	1.9	1.9	-	-
AA-S139	Square Catch Basin	1.9	13.6	2	-	-
AA-S140	Rectangular Catch Basin	1.1	1.8	1.2	-	-
AA-S141	Square Catch Basin	1.9	1.9	2	-	-
* The capacity of combination curb & gutters shall be calculated as a grate inlet. The additional capacity for the curb inlet is to serve as overflow when the grate becomes blocked with debris.						
** Depth of inlet opening can vary depending on the height of curb and capacity needs. A depression should be provided to achieve an inlet height of at least 4-inches.						

5. The projected gutter flow approaching each basin/inlet, the flow projected to enter each basin/inlet, and the flow projected to bypass each basin/inlet shall be provided in the Stormwater Management Report.

7. Open Watercourses

The requirements in this section are applicable to newly constructed open watercourses. A constructed channel shall be shaped or graded to the required dimensions and established with a suitable lining as necessary to convey stormwater runoff without allowing channel erosion.

A. All open watercourses having less than 3% longitudinal (running) slope shall have a 4-inch (minimum) diameter perforated underdrain. Underdrains are to be outleted to storm structure if practicable, or if this is unfeasible the underdrain is to be daylighted before the termination of the channel. The crown of the underdrain shall be 12-inches below the bottom of the watercourse. The underdrain ditch shall be a minimum of 16-inches by 16-inches, backfilled with #57 stone, and lined with filter-fabric. Watercourses that are engineered to be used as bio-swales to help treat water quality should follow the design criteria outlined in the **Vegetated Swales** section of this manual

B. Constructed open watercourses shall be designed to convey the 10-year design storm without causing erosion, sedimentation, or overbank flooding within and along the channel. Criteria outlined in the Flood Routing Section shall be used if the channel will also serve as a flood routing channel for the 100-year design storm. Open watercourses may also be designed for stormwater quality control.

C. A ditch computation sheet shall be used to present open channel calculations.

D. Parabolic and trapezoidal channel shapes shall be used for open watercourses within development projects. Side slopes shall be 3:1 or milder, with a minimum 2-foot bottom width for trapezoidal channels, unless alternative dimensions are approved by the City due to specific project conditions. Channel cross sections shall be designed such that erosion and sediment deposition is minimized.

E. An open channel is categorized by its lining. There are three main types of channel linings:

   1. Vegetated - A vegetative lining, such as grass with mulch and sod and lapped sod, is required where site constraints and flow velocity conditions allow.
   2. Flexible - Flexible linings include rock channel protection and cellular soil retaining mats and are typically less expensive than a rigid lining. The use of flexible linings, however, may require the installation of a filter fabric or other means to protect the underlying soil, prevent washout, and prevent soil piping through the rock when using channel protection.
   3. Rigid - Rigid linings include concrete and rigid block and are usually used where high velocities are unavoidable.

F. Final design of constructed open channels should be consistent with velocity limitations for the selected channel lining. Maximum velocity values for selected vegetated and non-vegetated lining categories are presented in the table below. The Manning's Equation shall be used to design an open channel that satisfies the maximum velocity criteria in the previous sections:

G. Open Watercourse Profile

   All open watercourses that are to be constructed or modified with the project shall be profiled. The profile shall include:

   1. Profile of the top of bank on both sides where difference in elevation exists
   2. Profile of flow line (invert) of streambed.
   3. Profile of the computed water elevation.
   4. Where land included is subject to flood, cross-sections, topography, and spot elevations are required.

Table 16: Maximum Velocities for Channel Lining Materials

Channel Lining Material	Maximum Allowable Velocity (ft/s)
Vegetated Channels	
Seed mixtures for urban areas	2.5
Other seed mixtures	2.5
Crown vetch	2.5
Established Seed or Sodded Channels	6.0
Flexible Linings	
Slope Erosion Protection	Follow manufacturer's criteria
Erosion Control Matting	
Rock Channel Protection	Use shear stress analysis
Rigid linings	
Concrete	18
Concrete block mat	18

H. Open channels shall be designed to flow under subcritical flow conditions at all times. A subcritical flow regime is characterized by a Froude Number less than 1:

$$F = \frac{V}{(g \cdot D)^{0.5}} < 1$$

Where,
F = Froude Number
D = hydraulic depth (ft) = A / T
A = cross-sectional area of flow (ft^2)
T = top width of water surface (ft)
V = flow velocity (ft/sec)
g = acceleration due to gravity = 32.2 feet/sec^2

The Stormwater Management Report shall demonstrate that the calculated Froude Number is less than 1 over the anticipated range of flow conditions within the channel.

I. Rock Channel Protection Shear Stress Analysis

1. Type B, C or D rock channel protection shall be provided in accordance with CMS Section 601.08. Type B, C or D rock channel protection shall only be placed outside of guardrails, barriers or other unobstructed areas provided outside of the traveled way for vehicles to stop safely or regain control. The actual shear stress (Γ_{ac}) must be less than or equal to the allowable sheer stress (Γ_a) listed in the following table for the rock channel protection type used. The actual shear stress shall be determined for the channel slope and the depth of flow during a 10-year design storm. The following equation is valid for discharges less than 50 cfs and with slopes less than 10%:

$$\tau_{ac} = 62.4 \cdot D \cdot S$$

Where,
D = depth of flow (feet)
S = channel slope (feet/feet)
Γ_{ac} = actual shear stress (lbs/feet^2)

Table 17: Allowable Shear Stress for Rock Channel Protection

Type of Rock Channel Protection	Γ_a (lbs/feet^2)
B	6
C	4
D	2

2. In extreme site conditions, Type B or C rock channel protection shall be utilized for lining channels with steep grades (slopes 10%-25%) that carry flow from the end of a cut section down to the lowest elevation on the bottom of the channel. Federal Highway Administration's HEC-15 procedures for steep gradient channels shall be used with a safety factor of 1.5. The Public Works Department shall be consulted if rock channel protection is proposed in instances where the peak flow during the 10-year design storm is greater than or equal to 50 cfs.
3. All constructed open watercourses shall have a structurally sound and stable outlet with adequate capacity to prevent ponding or flooding damage.

8. Rock Channel Protection

A. Description: A rock or riprap apron is typically needed at the outlet of storm drains, culverts, or open channels. Rock Outlet Protection provides an erosion resistant transition area where concentrated or high velocity flows enters less modified channels or natural streams.

B. Conditions Where Practice Applies: This practice applies where discharge velocities from channels, storm drains or culverts are high enough to erode receiving streams or areas. Suggested areas of application are:

1. Outfalls of stormwater detention facilities or sediment traps or basins.
2. Constructed channel outlets
3. Culvert outlets

This practice is not intended for use on slopes greater than 10%, or at the top of cut or fill slopes. Caution should be used when design flows exceed 100 cubic feet per second (cfs) from a 10-yr.-frequency storm.

C. Planning Considerations: Rock Outlet Protection may be used in conjunction with other practices, such as level spreaders. Rock Outlet Protection and Level Spreaders can both be used at the end of pipe outlets. This practice should be used alone where flow will continue as concentrated flow. Level Spreaders can be used with Rock Outlet Protection only when flow can be converted to and continue as sheet flow.

D. Permits: The U.S. Army Corps of Engineers and the Ohio Environmental Protection Agency, through Sections 404 and 401, respectively, of the Clean Water Act, may require a permit for an outlet protection that is located adjacent to a stream.

E. Water Quality: Rock outlet protection may also provide water quality benefits by providing for channel stability, prevention of excessive erosion, and limiting subsequent downstream sedimentation.

F. Design Criteria

1. Runoff: Runoff computation will be based upon the most severe soil and cover conditions that will exist in the area draining into the channel during the planned life of the structure. NRCS Technical Release 55 (TR 55) or other approved method shall be used to determine peak rate of runoff.

2. Velocity: Outlet protection shall be designed to be stable for discharge velocity expected from a 10-year frequency storm. Where high-hazard conditions exist, higher frequency storms should be chosen to provide protection compatible with conditions. Outlet protection shall meet the following criteria:

3. Design Velocity: Outlet protection shall be designed to be stable for the velocity of flow expected from a 10-year frequency storm. Outlet protection shall be designed to meet the criteria below or by other accepted engineering methods.

4. Width: The width of the outlet protection shall be the greater of either the width of the headwall or 4 feet wider than the pipe diameter (2 feet on each side of the edge of pipe).

5. Bottom Grade: The outlet protection should be constructed with no slope along its length. The elevation on the downstream end of the outlet protection shall be equal to the elevation of the receiving stream or channel.

6. Length of Rock Outlet Protection and Rock Size: Use the velocity calculated at the pipe outlet, the pipe diameter, and Figure 4. Outlet Protection Length, to find the length of outlet protection needed and rock size to use.

G. Rock Lining: The outlet protection shall consist of the rock riprap layer and an underlying filter or bedding. Minimum thickness of the rock riprap layer shall be the maximum stone size. Stone used for riprap shall be dense and hard enough to withstand exposure to air, water, freezing and thawing. Rock riprap must have a well-graded distribution and be placed to obtain a solid, compact layer of riprap. This may require some hand placing and tamping with construction equipment. Spreading gravel over top of the placed riprap surface will fill the voids by interlocking the riprap together.

Table 18: Rock Riprap Size

Type of Rock or Riprap (ODOT)	"n" Value	Size of Rock	
		50% by weight	95% by weight
Type D	.036	> 6 inches	3 – 12 inches
Type C	.040	> 12 inches	6 – 18 inches
Type B	.043	> 18 inches	12 – 24 inches
Type A	.045	> 24 inches	18 – 30 inches

1. Filter or Granular Bedding: Filter or granular bedding must be placed beneath all riprap to prevent the underlying soil from eroding and undermining the riprap, and to collect seepage and base flow. Minimum bedding thickness shall be 4 inches. Use of large size riprap may necessitate the use of a thicker bedding layer or 2 differently sized bedding layers. Care should be taken to select granular bedding that that is suitable with the subgrade material.

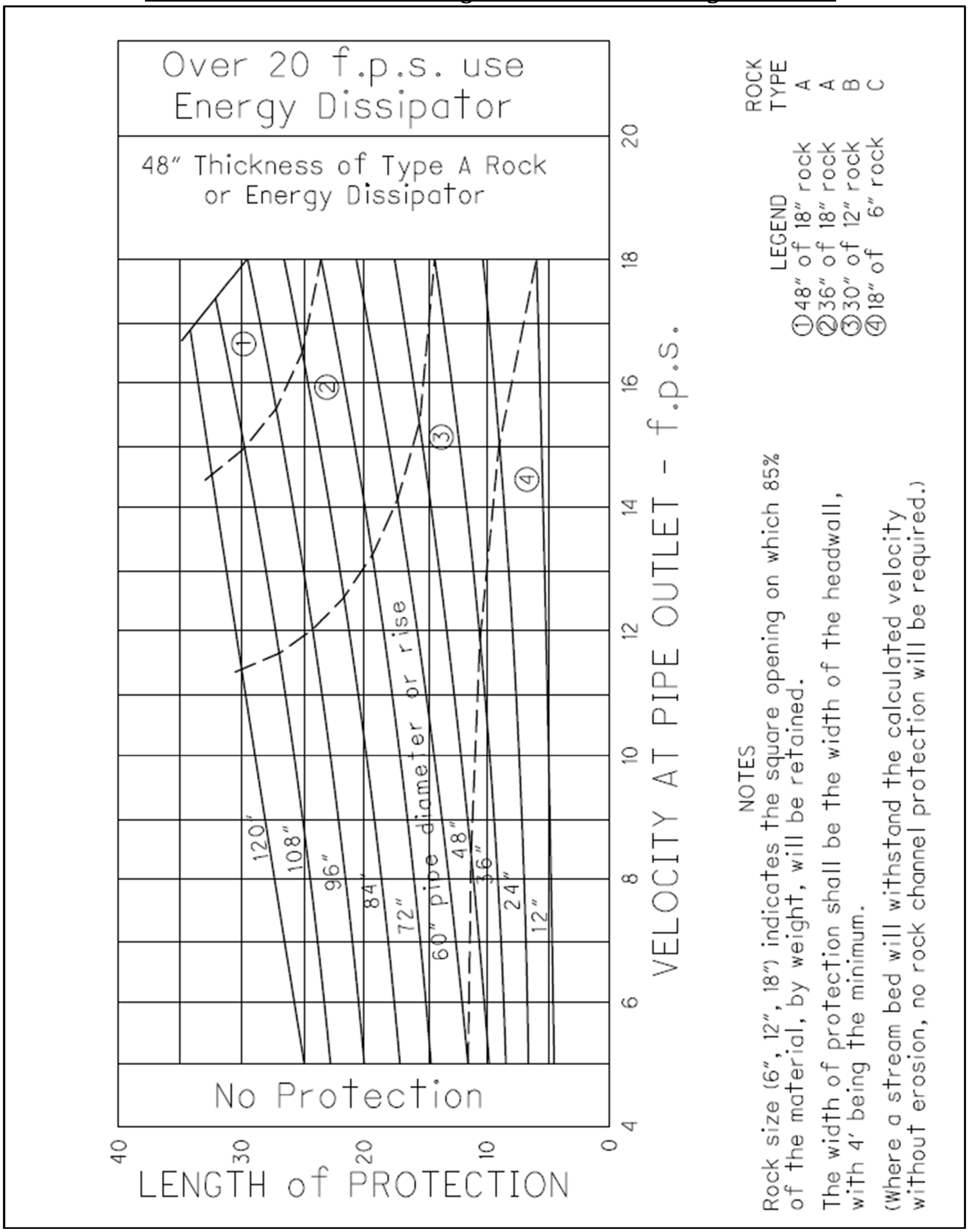
2. Geotextile: Geotextile may be used as a filter to be placed beneath the riprap to prevent piping of the soil where wetness, seepage, or prolonged base flow is the reason for lining the channel with riprap. If design of the outlet protection results in high velocities and steep grades, granular bedding should be used instead of geotextile. Care should be taken to properly anchor the geotextile to prevent unraveling under flowing water. Geotextile shall be woven or nonwoven monofilament yarn and shall meet Class I criteria in the attached table "Requirements for Geotextile".

Table 19: Requirements for Geotextiles

Property	Test Method	Woven – Class 1	Nonwoven – Class 1
Tensile strength (pounds) 1/	ASTM D 4632 grab test	200 minimum in any direction	180 minimum
Elongation at failure (percent) 1/	ASTM D 4632 grab test	< 50	≥ 50
Puncture (pounds) 1/	ASTM D 4833	90 minimum	80 minimum
Ultraviolet light (% residual tensile strength)	ASTM D 4355 150-hr exposure	70 minimum	70 minimum
Apparent opening size (ADS)	ASTM D 4751	As specified, but no smaller than 0.212 mm (#70) 2/	As specified max. #40 2/
Percent open area (percent)	CWO-02215-86	4.0 minimum	---
Permittivity sec-1	ASTM D 4491	0.10 minimum	0.70 minimum
1/ Minimum average roll value (weakest principle direction)			
2/ U.S. standard sieve size			
Note: CWO is a USACE reference			

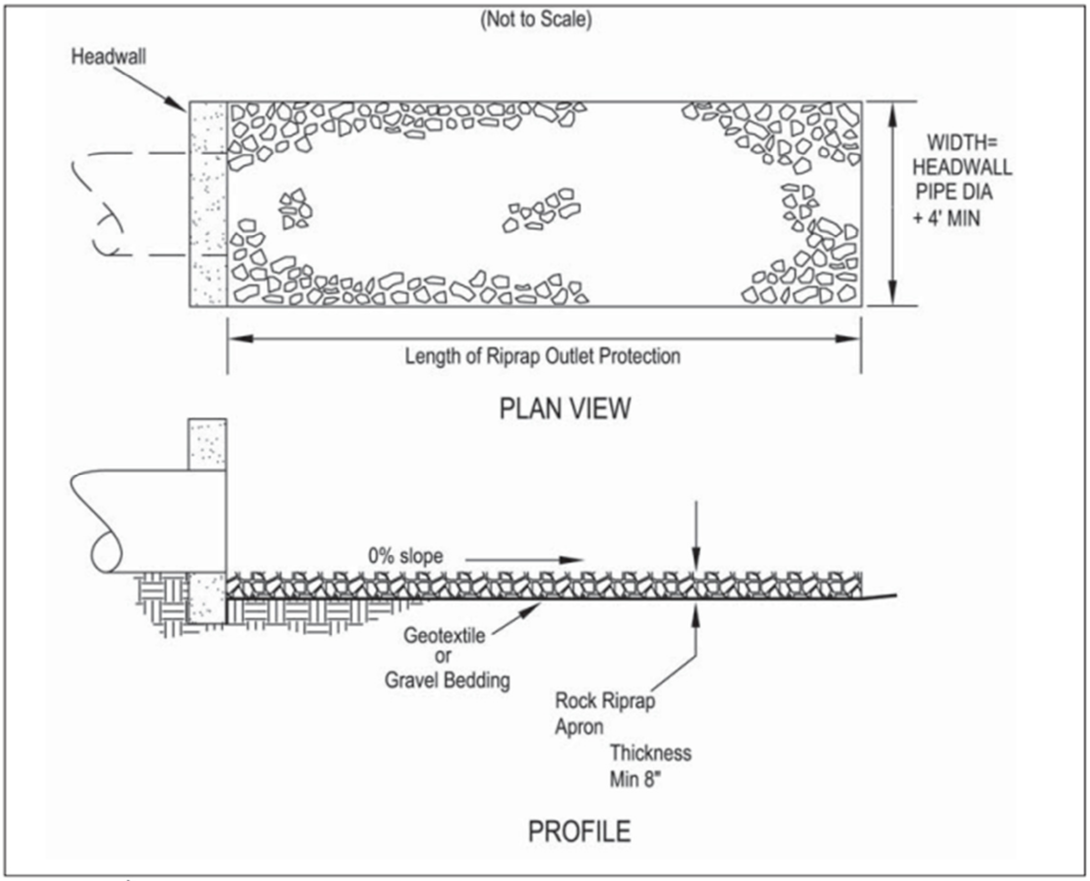
H. Maintenance: A maintenance program shall be established to maintain riprap, vegetative cover above the riprap, and associated structural components such as pipe outlets, and tile lines. Items to consider in the maintenance program include:
   1. Determine responsible party to inspect and maintain the outlet protection after construction
   2. Missing riprap should be replaced as soon as possible.
   3. Protect the outlet protection from damage by equipment and traffic
   4. Fertilize the vegetated area annually to and maintain a vigorous stand of grass
   5. Mow the vegetated area to maintain a healthy and vigorous stand of grass.
   6. Seed and mulch any bare areas that develop.
   7. Remove sediment and debris that have accumulated.
   8. Easements, or other means, should be obtained to ensure the channel is maintained as constructed

Figure 2: Rock Channel Protection
Source: ODOT Location & Design Manual Volume II, Figure 1107-2



I. Specifications for Rock Outlet Protection: The figure below provides the specifications required.

Figure 3: Plan and Profile for Typical Rock Outlet Protection



1. Subgrade for the filter or bedding and riprap shall be prepared to the required lines and grades as shown on the plan. The subgrade shall be cleared of all trees, stumps, roots, sod, loose rock, or other material.
2. Riprap shall conform to the grading limits as shown on the plan.
3. Geotextile shall be securely anchored according to manufacturers' recommendations.
4. Geotextile shall be laid with the long dimension parallel to the direction of flow and shall be laid loosely but without wrinkles and creases. Where joints are necessary, strips shall be placed to provide a 12-in. minimum overlap, with the upstream strip overlapping the downstream strip.
5. Gravel bedding shall be ODOT No. 67's or 57's unless shown differently on the drawings.
6. Riprap may be placed by equipment but shall be placed in a manner to prevent slippage or damage to the geotextile.
7. Riprap shall be placed by a method that does not cause segregation of sizes. Extensive pushing with a dozer causes segregation and shall be avoided by delivering riprap near its final location within the channel.
8. Construction shall be sequenced so that outlet protection is placed and functional when the storm drain, culvert, or open channel above it becomes operational.
9. All disturbed areas will be vegetated as soon as practical.

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