Chapter 11
Drainage Improvements

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11.1 Requirements for Storm Drainage Plans

11.1.1 General

The following criteria shall be utilized in the analysis of the drainage system:

a. Runoff analysis shall be based upon proposed land use, and shall take into consideration all contributing runoff from areas outside of the study areas.

   The analysis of storm runoff from existing developed areas lying outside of the study area shall be based upon present land use and topographic features.

   All undeveloped land lying outside of the study area shall be considered as fully developed based upon the Brandon Comprehensive Plan. Whenever the future land use of a specific undeveloped area cannot be accurately predicted, the average runoff coefficient to be used in said area shall not be less than 0.50 for the Rational Method runoff coefficient or an approved equivalent value for any other method, Table 11.1 (Appendix).

b. The probable future flow pattern in undeveloped areas shall be based on existing natural topographic features (existing slopes, drainage ways, etc.).

c. Average land slopes in both developed and undeveloped areas may be used in computing runoff. However, for areas in which drainage patterns and slopes are established, actual slopes and patterns shall be utilized.

d. Flows and velocities which may occur at a design point when the upstream area is fully developed shall be considered. Drainage facilities shall be designed to assure flows and velocities will not cause erosion damage.

e. The primary use of streets shall be for the conveyance of traffic. The computed amount of runoff in streets shall not exceed the requirements set forth in these Design Standards.

f. The use of onsite detention, detention within the development or detention in a drainage basin of which the development is part may be required. See the Subdivision Ordinances regarding drainageways and detention pond right-of-way dedication.

g. The changing of natural drainageway locations will not be approved unless such change is shown to protect against unreasonable hazard and liability, substantiated by thorough analysis.

h. The planning and design of drainage systems shall be such that problems are not transferred from one location to another. Outfall points shall be designed in such a manner that will not create flooding hazards.

i. Localized flooding information shall include the area inundated by the major storm runoff.

j. The flow routing for both the minor and major storm runoff shall conform to the master drainage plan. Drainage easements conforming to the City Master Drainage Plan will be required and shall be designated on all drainage drawings and subdivision plats.

k. Approval will not be made for any proposed building or construction of any type of structure including retaining walls, fences, etc., or the placement of any type of fill material, which will encroach on any utility or drainage easement or which will impair surface or subsurface drainage from surrounding areas.

11.1.2 Minor and Major Design Storms

1. Urban areas generally have two separate and distinct drainage systems. One is the minor system corresponding to the minor (or ordinary) storm recurring at regular intervals. The other is the major system corresponding to the major (or extraordinary storm) which has a one percent probability of occurring in any one year, called the 100-year storm event. Since the effects and routing of storm waters for the major storm may not be the same for the minor storm, all storm drainage plans submitted for approval shall be submitted in detail identifying the effects of both the minor storm and the major storm.

   a. Minor Storm Provisions.

      The minor storm drainage system shall be designed to provide protection against regularly recurring damage, to reduce street maintenance costs, to provide an orderly urban drainage system and to provide convenience to the urban residents. Storm sewer systems consisting of underground piping, natural drainageways, and other required appurtenances shall be considered as part of the minor storm drainage system.

   b. Major Storm Provisions.

      The major storm drainage system shall be designed to prevent major property damage or loss of life. The effects of the major storm on the minor drainage system shall be noted. The route of the major storm shall be noted to assure an outlet to a designated major drainageway is available.

11.1.3 Design Storm Calculations

1. Introduction

Presented in this section are the criteria and methodology for determining the storm runoff design peaks and volumes to be used in the City of Brandon for the preparation of storm drainage studies, plans, and facility design.

2. Design Frequencies

The residential and commercial design storm return frequency shall not be less than 5 years for the minor storm and 100 years for the major storm. The industrial design return frequency shall not be less than
5 years for the minor storm and 100 years for the major storm.

3. Design Rainfall

The design rainfall data to be used for the Brandon area was obtained from the National Weather Bureau. The intensity-duration-frequency chart in Figure 11.1A (Appendix) for storm durations less than 1 hour and the intensity-duration-frequency curves in Figure 11.1B (Appendix) for storm durations greater than one hour are presented for computations of rainfall intensities.

4. Rational Method

The Rational Method may be used in both the minor and major storm runoff computations for basins that are not complex and generally have less than 100 acres.

The Rational Method is based upon the following formula:

$$Q = CIA \qquad \text{(Equation 1)}$$

Where:

Q = Peak Discharge (cfs);
C = Runoff Coefficient;
I = Rainfall Intensity (inches/hour); and
A = Drainage Area (acres).

When using the Rational formula, an assumption is made that the maximum rate of flow is produced by a constant rainfall which is maintained for a time equal to the period of concentration of flow at the point under consideration. Theoretically, this is the time of concentration, which is the time required for the surface runoff from the most remote part of the drainage basin to reach the point being considered.

However in practice, the concentration time, Tc , is an empirical value that results in acceptable peak flow estimates.

Watershed and storm sewer modeling for City projects shall be completed using SWMM-based modeling software or other software capable of exporting to a SWMM format. Electronic versions of the storm sewer model shall be submitted to the City once construction as-built plans have been submitted. On private developments, other storm water modeling methods may be used. Design parameters used in the model shall be submitted with the drainage calculations.

5. Time of Concentration and Travel Time

As discussed in this Section, Tc, the time of concentration, is the time it requires for runoff to travel from the hydraulically most distant point of the watershed to the point of interest within the watershed.

Travel time is the time it takes water to travel from one location to another in a watershed.

In the application of the Rational Method, the time of concentration must be estimated so that the average rainfall rate of a corresponding duration can be determined from the intensity-duration-frequency chart in Figure 11.1A (Appendix) for storm durations less than one hour and the intensity-duration-frequency curves in Figure 11.1B (Appendix) for storm durations greater than one hour.

Water travels across a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection. The minimum time of concentration shall be 15 minutes.

a. Sheet Flow

Sheet flow is flow over plane surfaces. It usually occurs in the headwater of streams. 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, and rocks; and erosion and transportation of sediment. These n values are for very shallow flow depths of about 0.1 foot or so. Table 11.2 (Appendix) provides Manning's n values for sheet flow for various surface conditions.

For sheet flow of less than 300 feet, use Manning's kinematic solution (Overton and Meadows 1976) to compute T_t:

$$T_t = \frac{0.007\,(nL)^{0.8}}{(P_2)^{0.5}\,s^{0.4}} \qquad \text{(Equation 2)}$$

Where:

T_t = travel time (hr);
n = Manning's roughness coefficient, Table 11.2 (Appendix);
L = flow length (ft);
P_2 = Two-year, 24-hour rainfall (in) = 2.7 inch for our area; and
s = slope of hydraulic grade line (land slope, ft/ft).

This simplified form of the Manning's kinematic solution is based on the following: (1) shallow steady uniform flow; (2) constant intensity of rainfall excess (that part of a rain available for runoff); (3) rainfall duration of 24 hours; and (4) minor effect of infiltration on travel time.

i. Limitations

- Manning's kinematic solution should not be used for sheet flow longer than 300 feet. Equation 2 was developed for use with the four standard rainfall intensity-duration relationships.

- South Dakota is a Type II intensity-duration relationship, as defined by the Soil Conservation Service (SCS).
- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate T_C. 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 nonpressure flow.
- The minimum T_t used in Technical Release-55 (TR-55) Urban Hydrology for Small Watersheds is 0.1 hr (6 minutes).

b. Shallow Concentrated Flow

After a maximum of 300 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from Figure 11.2 (Appendix) in which average velocity is a function of watercourse slope and type of channel. Tillage can affect the direction of shallow concentrated flow.

After determining average velocity from Figure 11.2 (Appendix), use the following equation to estimate travel time for the shallow concentrated flow segment:

$$T_t = \frac{L}{3600\ V} \qquad \text{(Equation 3)}$$

Where:

T_t = travel time (hr);
L = flow length (ft);
V = average velocity (ft./sec.); and
3600 = conversion factor from seconds to hours.

c. Open Channel Flow

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. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full elevation.

Manning's equation is:

$$V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n} \qquad \text{(Equation 4)}$$

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); and
n = Manning's roughness coefficient for open channel flow.

Manning's n values for open channel flow can be obtained from standard hydraulic textbooks. After average velocity is computed using Equation 4, T_t for the channel segment can be estimated using Equation 3.

6. **Rainfall Intensity (I)**

The intensity (I) is the average rainfall rate in inches per hour for the period of maximum rainfall of a given frequency having a duration equal to the time of concentration. After the design storm frequency has been selected, the rainfall intensity shall be obtained from the intensity-duration-frequency chart in Figure 11.1A (Appendix) for storm durations less than one hour and the intensity-duration-frequency curves in Figure 11.1B (Appendix) for storm durations greater than one hour using the time of concentration calculated above.

7. **Runoff Coefficient (C)**

The runoff coefficient (C) represents the integrated effects of infiltration, evaporation, retention, flow routing, and interception, all of which effect the time distribution and peak rate of runoff. Table 11.1 (Appendix) presents the recommended values of C for the various recurrence frequency storms. The values are presented for different surface characteristics as well as for different aggregate land uses. The coefficient for various surface areas can be used to develop a composite value for a different land use.

11.1.4 Concept Drainage Plan

The Concept Drainage Plan may be submitted as part of the Concept Plan .

The purpose of the Concept Drainage Plan is to identify any proposed drainage concerns regarding the development. Approximate flow paths and existing conditions will be provided. The Concept Drainage Plan will provide information in accordance with the City Master Drainage Plan.

11.1.5 Developers Preliminary Drainage and Grading Plan

1. The developer shall submit a drainage plan which complies with the City Master Drainage Plan for the drainage basin(s) of which the development is included. Scales as small as 1 inch equals 500 may be used to show the entire development.

2. The following information shall be included in the submittal:

a. A route outlet map will be required. This map shall show how the drainage from the proposed development will be transmitted to the nearest major drainageway. The map shall show any existing structure(s) which may limit the flow en route to the major drainageway. The route outlet map shall show the drainage area upstream of the proposed development and the estimate of flow under current conditions presently draining onto and through the development.

b. Data for minor and major storm flows within the proposed development for all drainage basins and sub-basins, as identified per the City Master Drainage Plan.

c. Identification of drainage problems with proposed solutions to deal with the problems within the development.

d. Identification of downstream and upstream facilities as shown on the route outlet map in accordance with the City Master Drainage Plan.

e. Locations and size of proposed detention ponds as required by the City Master Drainage Plan and Best Management Practices Plan within the development shall be identified.

f. General locations and size of potential wetlands shall be identified. Include copy of correspondence with United States Army Corps of Engineers (USACOE) requesting wetland determination and any responses. Also note if any mitigated wetlands will be created.

g. Any and all existing 100-year floodplains must be identified, as shown by FEMA maps or the City Master Drainage Plan. The Assistant Director of Building Services will provide needed FEMA maps.

h. Existing contours.

i. Location and size of existing open channels, bridges, culverts, storm sewers, and ponding areas within the development.

j. Location of streets.

k. Identification of all drainage basins tributary to the development.

l. Drainage patterns within the proposed development.

m. Provide adequate information as to the effect of the drainage pattern on adjacent property. Provide survey data as required for adequate information. Identify the storm water path to the major drainway.

11.1.6 Development Engineering Final Drainage Plan

1. The Final Drainage Plan shall be a detailed plan of the proposed development phase, as defined per Subdivision Ordinance. It shall include detailed data for all runoff within the proposed development phase, and detailed data for the design of all drainage structures within the development phase.

2. Drawings and data (actual calculations may be required with submittal) comprising of the Final Drainage Plan shall comply with Chapter 2—Submittal Procedures, and shall include, but not be limited to the following information. Scale will be 1 inch equals 100 feet maximum.

   a. Proposed contours, and arrows indicating drainage paths.

   b. Location and elevations of bench marks.

   c. Property lines.

   d. Streets, names, and grades.

   e. Existing drainage facilities and structures, including existing roadside ditches, drainage ways, gutter flow directions, culverts, etc. All pertinent information such as size, shape, slope, location, etc., shall also be included to facilitate review and approval of drainage plans. Flow areas will be delineated.

   f. Proposed storm sewers and open drainage ways, easement, and right-of-way requirements, including proposed inlets, manholes, and culverts. General notes concerning erosion control and energy dissipation shall be provided.

   g. Proposed outfall point for runoff from the development phase.

   h. Routing and accumulative flows at various critical points for the minor and major storm runoff.

   i. 100-year flood level in all streets in which the curb is overtopped during the 100-year storm for sump condition or other critical points.

   j. Identify 100-year flood elevations for major and lateral drainageways.

   k. Inlet flow data. Figure 11.42 (Appendix).

   l. Pipe flow data. Figure 11.41 (Appendix).

   m. All flood plains, identified by FEMA maps, within the proposed development phase.

   n. Location and size of potential wetlands.

      1. Provide copy of correspondence with state and federal agencies related to the potential impact to wetlands or other cultural resources. This includes:

         a) Wetland determination for USACOE

         b) Wetland mitigation plan

c) Any restrictive covenants that would prevent the City from performing maintenance activities such as excavating within the wetlands.

o. Hydrological data for each drainage area.

1. Areas.
2. Watershed lengths, elevations, time of concentration.
3. Rainfall intensity.
4. Runoff coefficients.
5. Projected land uses and existing physical features of areas contributing runoff.
6. Storm duration.
7. Reference to City Master Drainage Plan for plan flows is acceptable.
8. Runoff (Q) (Note: This list of criteria assumes use of Rational Formula. If a different method is used, all relevant factors are to be enumerated.).
9. Maximum release rate of storm water generated by the development and post-developed upstream properties.

p. Major drainageways.

1. Alignments.
2. Profiles including existing and proposed.
3. "n" values (Manning).
4. Velocities.
5. Soils analysis with a discussion of the proposed channel erosion potential.
6. Shear stress.

q. Design recommendations.

1. Dikes.
2. Filling low areas.
3. Provision of easements.
4. Recommendations against building in certain areas.
5. Provisions for onsite retention and detention.
6. Other as appropriate for conditions.

11.1.7 Existing Floodplain Map—Revisions

All submittals for floodplain revision must be reviewed and approved by FEMA or their authorized agent. The City will not take responsibility for time, scheduling, or cost involved in floodplain map revisions or letters of map amendments.

The developer is responsible for submitting all information to FEMA. Copies of all information sent to, and correspondence with FEMA, must also be sent to the City Engineer.

11.1.8 Review by Other Agencies

All open channel construction and existing drainageway modifications will be reviewed by the City and other appropriate county, state, or federal agencies.

11.1.9 Hydrologic and Hydraulic Analysis Standards

The purpose of this standard is to define hydrologic and hydraulic (H&H) analysis to evaluate development for potential impacts to the elevation, storage, and conveyance of storm water in H&H Analysis Areas.

1. Hydrologic analysis

   a) Rainfall intensity-duration to utilize:

      (1) Engineering Design Standards.

      (2) Or approved method.

2. Hydraulic analysis

   a) The H&H Analysis shall utilize existing reports/studies with regard to hydraulic inundation in the study area. Deviations between an existing report/study and subsequent H&H analysis shall be justified and documented to the satisfaction of the reviewing authority.

   b) Hydraulic models shall be calibrated to existing conditions such as past hydraulic inundation elevations to determine the reasonableness of the model results. If historical data is not available, adequate justification shall be documented.

   c) H&H Analysis shall reflect anticipated conveyance from all projected future development based on best available information.

   d) H&H Analysis to generally determine the following potential impacts utilizing Developer-provided digital elevation models (existing/proposed) and other required information to support adequate analysis.

      (1) Elevation

(a) Determine impacts to hydraulic profile.

(2) Storage

(a) Analyze development or grading for reduction of hydraulic storage.

(3) Conveyance: determine impacts to hydraulics

(a) Analyze development or grading for potential impacts of hydraulic conveyance.

e) Limits of the hydraulic model shall be based on detailed information for downstream structures (dam, bridge, culvert) to determine adequate starting hydraulic profile for the H&H Analysis.

f) Cross sections may be obtained by one of several methods, including surveying or LIDAR/photogrammetry. Cross section spacing shall be sufficient to accurately define a hydraulic profile and determine elevations at key locations such as roads, buildings, and property lines.

g) A survey of bridge and culvert openings and the top of road shall be required at each road crossing unless alternate methods are approved.

h) Parameters for such as flow, Manning's N valves, expansion and contraction coefficients, or effective flow limits to utilize:

(1) Engineering Design Standards

(2) Standard accepted engineering practices

i) The H&H Analysis must extend past the upstream limit of the difference in the existing and proposed flood profiles in order to provide a tie-in to existing analysis. The proposed hydraulic profile shall match the existing hydraulic profile at the tie-in locations. The hydraulic profile shall be calculated to the 0.01 foot.

3. H&H Analysis Areas are defined as the following:

a) Special Flood Hazard Areas (SFHA)

b) Reserved

4. Final report. A comprehensive final report shall be submitted to the City Engineer.

a) H&H Analysis shall be completed using approved H&H software with a final report including software file/s exported to a City-approved compatible format for all hydraulic conditions.

b) An exhibit of the reach analyzed shall be provided, showing all cross section locations, model centerline, preliminary and/or effective SFHA, major design event hydraulic inundation limits based on best available data, limits of the proposed development.

11.2 Storm Sewers

11.2.1 Design Flow

For areas smaller than 100 acres, the Rational formula is acceptable to compute runoff. For areas larger than 100 acres, the Soil Conservation Service method or other acceptable computer applications shall be used. Computations for storm sewer design and storm inlet designs shall be submitted on forms similar to those included in these standards.

11.2.2 Material and Installation

All construction shall be in accordance with the approved Standard Specifications and Standard Plates for drainage improvements.

11.2.3 Location of Storm Sewers

1. All public storm sewers shall be installed in the public easement or right-of-way. Storm sewer shall be placed as shown in Chapter 4. If storm sewer pipe is placed on back lot lines or otherwise placed across private property, a drainage easement is required provided the pipe is utilized to drain public storm water. If the storm sewer pipe is to be used for private storm water runoff, no easement is required.

a. Placement

Storm sewer must be extended to the far edge of the platted subdivision to be serviced, regardless of where the inlets are placed.

b. Easements

1. All easements must be mutually exclusive for the City of Brandon. Easements shall be identified as public utility and drainage easements. Final Drainage Plans shall identify the type of easement.
2. All drainage easements must be a minimum of 20 feet wide, additional width for access may be required. The pipe shall be placed only along the center of the easement, unless approved by the City Engineer.
3. No landscaping except grass may be placed in the easement.
4. No permanent structure may be placed in the easement.

11.2.4 Size

No public storm sewer shall be less than 12 inches in diameter. Trunk storm sewers must be a minimum of 18 inches in diameter.

All changes in pipe size must occur at a manhole, inlet, or junction box.

11.2.5 Depth

The minimum allowable sewer depth of cover shall be 18 inches.

11.2.6 Pipe

Pipes shall be sized to convey the minor design storm as referenced for the applicable land use classification. Hydraulics of the pipe shall be analyzed. The hydraulic gradient shall remain below the gutter or ground surface elevation to prevent overflow.

Storm sewer pipe shall be reinforced concrete unless otherwise approved by the City Engineer. In certain cases the designer may wish to specify one type of pipe for a certain purpose, in which case no alternate should be given. The "class" of reinforced concrete pipe shall be shown on the plans.

Storm sewer pipe made of other materials such as polyethylene may be approved by the City Engineer for private development storm sewer or storm sewer to be installed outside the public right-of-way.

Coefficients of roughness, "n," for use in the Manning formula as listed below shall normally be used:

Type of Pipe	"n"
Concrete	0.013
Polyethylene	0.010

11.2.7 Velocity

The minimum allowable velocity in a storm sewer shall be 3 feet per second (fps). The maximum velocity shall be 15 fps.

11.2.8 Pipe Strength

Pipe specified shall meet AASHTO HS-20 loadings.

11.2.9 Alignment

Sewer shall be installed with a straight alignment between structures with the following two exceptions: In locations where layouts are such that a straight alignment is not practical, sewers may be curved. The curvature must be concentric with the curvature of the street. The pipe manufacturer's recommended maximum deflection angle shall not be exceeded. Storm sewer bends will be shown as required. The City Engineer may require a structure instead of a bend.

11.2.10 Separation

1. Storm sewer crossings of the water main will be performed in accordance with the Standard Specifications. Water main will be installed at least ten 10 horizontally from any storm sewer. Exceptions will be allowed based on street width as per Chapter 4 of these standards.

   Crossings of water main and storm sewer will have a minimum of 18 inches clearance between the outside surface of the pipes.

2. Storm sewer crossings of sanitary sewer shall be performed in accordance with the Standard Specifications.

   Sanitary sewer shall be installed at least 2 feet horizontally from any storm sewer.

   Crossings of sanitary sewer and storm sewer will have a minimum of 6 inches clearance between the outside surface of the pipe. Crossings that have less than 18 inches of clearance will be structurally supported.

11.2.11 Ground Water Barriers

When there exists a possibility that ground water may be diverted and follow the path of the new sewer, ground water barriers shall be constructed in adequate numbers to prevent ground water migration along sewer trenches.

11.3 Storm Sewer Appurtenances

11.3.1 Junction Boxes

1. **Location**

   Trunk storm sewer is defined as any storm sewer 18 inches in diameter or larger that is used to convey storm water from two or more inlets.

   Lateral storm sewer is defined as the storm sewer that connects to the trunk sewer system. Minimum lateral storm sewer pipe shall be 12 inches in diameter.

   Structures shall be required when trunk line storm sewers intersect.

   Pipe Tee-Sections may be used to connect a lateral storm sewer to the trunk storm sewer when the lateral length between the Tee-Section and a structure is 75 feet or less.

   Field connections to connect a lateral system to the existing trunk storm sewer system, as described in the previous paragraph, will only be permitted if conditions prohibit the installation of a structure, as determined by the City Engineer.

   Bends may be used along the trunk system between structures when curvature alignment requires the bend and the maximum spacing between structures has not been exceeded. The City Engineer's Office may require a structure instead of a bend.

For 18-inch-diameter storm sewer, the maximum total bend or curvature allowed is 22.5 degrees. For 24-inch-diameter storm sewer and larger, the maximum single bend allowed is 45 degrees. If more than one bend is required due to alignment curvature, the maximum angle per bend is 7.5 degrees. Maximum total curvature is 90 degrees for 24-inch RCP and larger.

Structures shall be installed at the upper end of each line, at changes in grade, size, curvature or alignment, and at distances not greater than: 400 feet for sewers 15 inches in diameter or less; 450 feet for sewers
18 inches and 21 inches in diameter; and 500 feet for sewers 24 inches in diameter and larger.

Structures must be located in areas which allow direct access by maintenance vehicles.

2. Flow Channels

When there is an increase in sewer size of a smaller sewer connected with a larger one, the invert of the smaller sewer must be raised to maintain the same energy gradient. An approximate method of doing this is to place the 0.8 depth point of both sewers at the same elevation or to match the crown of the pipe. Structures that have a direction change of flow shall have a minimum 0.1-foot drop between the inverts.

Drop manholes shall be avoided whenever possible.

11.3.2 Outlets

1. Where a storm sewer discharges into a natural channel or irrigation ditch, an outlet structure shall be provided that will blend the storm sewer discharge into the natural channel flow in such a way as to prevent erosion of the bed or banks of the channel.

2. When the discharge velocity is low, or subcritical, the outlet structure may be one of the following:

   a. Flared end section
   b. Head wall
   c. Wing walls

3. If the discharge velocity is high, or supercritical, prevention of erosion of the natural channel bed or banks in the vicinity of the outlet may require an energy dissipating structure.

4. All outlets shall have an apron consisting of one of the following:

   a. Riprap with geotextile fabric base
   b. Concrete anchor mat
   c. Concrete slab
   d. Scour stop, or approved equal
   e. Other approved methods

11.3.3 Inlets

1. Introduction

A storm inlet is an opening into a storm sewer system for the entrance of surface storm runoff. There are three types of inlets: curb opening, grated, and combination. In addition, inlets may be further classified as being on a continuous grade or in a sump. The term "continuous grade" refers to an inlet so located that the grade of the street has a continuous slope past the inlet and therefore ponding does not occur at the inlet. The sump condition exists whenever water is restricted to the inlet area because the inlet is located at a low point. A sump condition can occur at a change in grade of the street from positive to negative or due to the crown slope of a cross street when the inlet is located at an intersection.

2. Inlet Standards

Acceptable inlets for public streets shall be Type I curb opening or Type II combination. Curb opening inlets shall be used at true sumps or at sumps formed by crown slope of cross section at the intersection. Either curb opening type or combination inlets may be used on continuous grade.

Grated inlets may be used for parking areas and open fields or other applications subject to approval by the City Engineer.

Reduction factors shall be applied to the theoretical calculated capacity of inlets based upon their type and function. The reduction factors compensate for effects which decrease the capacity of the inlet such as debris plugging, pavement overlaying, and in variations of design assumptions.

The allowable capacity of an inlet shall be determined by applying the applicable reduction factor from Table 11.3 (Appendix) to the theoretical capacity as presented in the following sections.

The size of outlet pipes from storm water inlets shall be based upon the theoretical capacity of the inlet, but shall not be less than 12 inches in diameter.

3. Curb Opening Inlet Hydraulics

A curb opening inlet may operate under two different conditions of flow: (1) free flow conditions under which a free water surface is continuous into the inlet; or (2) submerged conditions in which the inlet functions as an orifice. The continuous grade design procedures described herein assume that the inlets will be designed to operate under the free flow condition, since the gutter flow depth required to submerge the inlet is greater than the allowable street capacity.

The inlet dimensions evaluated herein are the standards used for Type I and II inlets.

4. Sump Condition

Presented in Figure 11.3 (Appendix) is a capacity nomograph for sump condition with a gutter depression at the inlet. This chart is an adaptation of a Bureau of Public Roads chart and is applicable to both the free flow and the submerged cases.

5. Continuous Grade

For the "continuous grade" condition, the capacity of the inlet is dependent upon many factors including gutter slope, depth of flow in the gutter, height and length of curb opening, street cross slope, and the amount of depression at the inlet. In addition, all of the gutter flow will not be intercepted and some flow will continue past the inlet area ("bypass"). The amount of bypass must be included in the downstream drainage facility evaluation as well as in the design of the inlet.

Inlet size and spacing is dependent upon the allowable use of streets for handling storm runoff. Section 11.6, of this chapter will address pavement encroachment and provide criteria for the maximum width of spread (W) as addressed below.

When the allowable pavement encroachment has been determined, the theoretical gutter capacity for a particular encroachment can be determined by the use of Figure 11.4 (Appendix). To further simplify computations, Figure 11.5 (Appendix) is provided to enable direct determinations for various street sections. Figure 11.4 (Appendix) as well as the charts for inlet capacity provided in the Appendix of these standards will assist the designer in solving for the capacity of an inlet on a continuous grade. The procedure for properly sizing and determining inlet spacing is as follows:

a. After the design has determined a total runoff discharge (Q) flowing upstream of the inlet, enter Figure 11.5 (Appendix) for design Q and extend a vertical line down to intersect with the longitudinal gutter slope (S_o). Extend a horizontal line from the point to the cross slope (S_x) of the street being studied and extend a vertical line down from this point to the width of spread (W). The depth of flow
(D) at the curb may also be determined if the vertical line intersecting the cross slope (S_x) on the lower portion of the graph is extended horizontally to intercept the depth at the curb.

b. Select the appropriate capacity chart from the Appendix for the type of inlet (Type I or II), street cross slope (S_x) and longitudinal gutter slope (S_o).

c. Type I inlets: Enter the chart for the inlet length selected. Extend a vertical line up to intersect the curve for the width of spread (W) determined in Step 1 and extend a horizontal line from this point to the inlet intercept ratio (Qi/Q).

   Type II inlets: Enter the chart for the width of spread (W) determined in Step 1. Extend a horizontal line across to intersect the line for the longitudinal gutter slope (S_o) and extend a vertical line from this point to the inlet intercept ratios (Qi/Q).

d. Multiply the inlet intercept ratio (Qi/Q) determined in Step 3 times the total discharge (Q) carried by the gutter, yields the quantity of water being intercepted by the inlet (Qi). For Type I inlets, the designer may want to repeat Steps 3 and 4 for other lengths of inlets.

   After the theoretical capacity has been determined as outlined above, capacity reduction factors must be applied as listed in Table 11.3 (Appendix). The designer will need to choose which type of inlet is most effective based upon both hydraulic and economic considerations.

6. Capacity of Grated Inlets in Sump

As previously noted, grated inlets may be used for parking areas and open fields or other areas subject to approval by the City Engineer. The design procedure presented in the following section is based upon the assumption that the grated inlet is clear from debris and is operating at its maximum efficiency.

For a grated inlet operating under sump conditions, the reduction factors of Table 11.3 (Appendix) shall be applied.

Under sump conditions a grated inlet acts essentially as a series of orifices. Design charts indicate that the application of the orifice formula to the clear opening of the inlets gives satisfactory capacities for a clean inlet. Figure 11.6 (Appendix) shows the results of the tests. The head used shall be determined by the allowable depth of ponding for the installation at the design storm frequency.

11.4 Culverts

11.4.1 General

Culverts may be of any shape and construction as required by existing topographic features; provided, however, the size, location, and type of construction of culverts shall be subject to acceptance by the City Engineer's Office.

Culverts within the major drainageways, as outlined in the Brandon Master Drainage Plan, that are under arterials or railroads shall have sufficient capacity to pass all of the runoff from the 100-year storm considering 20 percent of the inlet plugged, for pipes under 48-inch diameter.

For all other streets, culverts must be designed to convey a minimum of 10-year flow with no street overtopping and must be large enough so that the 100-year flow over the top of the road does not exceed 18 inches in depth above the invert of the gutter.

11.4.2 Design Criteria

1. The following design criteria shall be utilized for all culvert design:

   a. The culvert including inlet and outlet structures shall properly take care of storm water flow, bed-load, and debris at all stages of flow.

   b. **Inlets**. Culvert inlets shall be designed to minimize entrance and friction losses. Inlets shall be provided with either flared-end sections or head walls with wing walls. Projecting ends will not be acceptable. For large structures, provisions shall be made to resist possible structural failure due to hydrostatic uplift forces.

   c. **Outlets**. Culvert outlets shall be designed to avoid sedimentation, undermining of the culvert, or erosion of the downstream channel. Outlets shall be provided with either flared-end sections or headwalls, with wingwalls. Projecting outlets will not be acceptable. Additional outlet control in the form of rip rap, channel shaping, dissipation structure, etc., may be required where excessively high discharge velocities occur. All structural outlet velocity dissipaters shall be underlain with a suitable filter fabric to protect against scour.

d. **Slopes**. Culvert slopes shall be such that neither silting nor excessive velocities and scour occur. Generally, the minimum slope of culverts shall be limited to 0.50 percent.

e. **Hydraulic Design**. Culverts shall be analyzed to determine whether discharge is controlled by inlet or outlet conditions for both the initial storm discharge and the major storm discharge. The value of the roughness coefficient (n) used shall not be less than those specified by documentation of the culvert manufacturer. Computations for selected culvert sizes shall be submitted for review.

f. **Minimum Allowable Size**. The required size of the culvert shall be based upon adequate hydraulic design analysis. Approval will not be granted for round culverts with less than 18 inches inside diameter, or for arched or oval-shaped culverts with span-rise dimensions less than 24 inches x 18 inches nominal except that culverts 12 inches or greater in diameter may be used for single-family residential access drives.

   The minimum height of a reinforced box culvert should be 3 feet to facilitate cleanout and allow removal of forms during construction.

g. **Multiple Culvert Installation**. Where physical conditions dictate, multiple culvert installations will be acceptable, provided the minimum size of any culvert to be used shall not be less than the requirements set forth above.

h. **Structural Design**. The structural design of culverts shall conform to those methods and criteria recommended by the manufacturer of a specific type of culvert dependent upon the type of bedding, the method of installation, and the load.

i. **Trash and Debris Deflector**. When, in the opinion of the City Engineer, debris accumulation for a particular drainageway appears to pose a significant probability of culvert plugging, trash racks or debris deflectors will be required.

11.5 Open Channel Flow, Major Drainageway

11.5.1 General

Major drainageways and lateral drainageways will be classified by the City Engineer.

All drainageways to be dedicated to the City shall have a maintenance bench along both sides providing access to all channel areas. Drainageways adjacent to rights-of-way proposed with one maintenance bench will be reviewed by the City of a case-by-case basis for approval. The maintenance bench shall be 12 feet wide with a cross slope no greater than 10 percent. If the bench is located along a future recreational trail, the maximum cross slope may be reduced to meet trail requirements. Property delineation markers shall be installed upon dedication of the property in locations specified by City Engineer.

See Figure 11.7A, 11.7B, 11.7C (Appendices) for design standards for channel construction.

All channels will be designed with the five-year storm frequency and the 100-year storm frequency considered.

The property corner elevation abutting or property elevation adjacent to a drainageway shall be a minimum of 1 foot above the 100-year major design storm elevation.

Minimum ground elevations for structures abutting or potential to flood by drainageways or storm water facilities shall be 2 feet above the overflow elevation or 100-year major design storm elevation, whichever is higher. Where an overflow system is not available, minimum ground elevation shall be a minimum of 4 feet above the 100-year major design storm elevation or 150 percent of the 100-year major design storm elevation, whichever is higher.

Channels shall be designed in such a manner that flows at the critical depth and supercritical flows are avoided.

If increased flows are proposed for any channel, protection as required shall be provided for a natural channel. Channel protection will be designed to withstand forces that attempt to overtop the channel banks, deteriorate the channel lining, erode soils beneath the channel lining and erode unlined areas of the channel.

Open channels conveying storm water shall be designed using the Tractive Force Procedure*. The permissible shear stress, T_d, is the force required to initiate movement of channel lining material. Normal depths in the channel are calculated using Manning's equation. Manning's roughness coefficients for different ranges of depth are provided in Figure 11.7B. The coefficient of roughness generally decreases with increasing flow depth.

Shear stress, T, at normal depth, is computed for the lining by the following equation:

$$T = yds \qquad \text{(Equation 5a)}$$

Where:

T = shear stress in lb/ft$_2$
y = unit weight of water, 62.4 lbs/ft$_3$
d = flow depth in feet
s = channel gradient in ft/ft

If the permissible shear stress, T_d, given in Figure 11.7C, is greater than the computed shear stress, T, the chosen channel liner is considered acceptable. If the computed shear stress is too great, select a liner with a higher permissible shear stress and repeat the calculations for normal depth and shear stress. In some cases, it may be necessary to alter the channel dimensions to reduce the shear stress.

All channels shall be designed with proper and adequate erosion control features. When required, drops or check dams shall be installed to control water surface profile slope.

Grass-lined channels or side slopes of concrete-lined channels will be seeded with a mixture as set forth in these Design Standards.

Lateral drainageways without a low flow storm sewer will only be permitted with the acceptance of the City Engineer.

For channels that cross a roadway and overflow the street section within design standards, it is acceptable to provide an easement for the 12-foot access strip along the backwater area. The remaining channel dedication will be per ordinance.

11.6 Street Flow Capacity

11.6.1 General

The criteria set forth herein will be used in analyzing and approving the adequacy of streets as a function of the drainage system. Both the minor and 100-year storm runoff must be considered and calculations showing such runoff at critical sections shall be submitted. Street, curb and gutter, valley gutters, and curb cuts shall conform to the Standard Specifications.

11.6.2 Street Capacity for Minor Storms

Pavement encroachment for the minor design storm shall not exceed the limitations set forth in the following table:

Allowable Pavement Encroachment and
Depth of Flow for Minor Storm Runoff

Street Classification	Maximum Encroachment*
Local	No curb overtopping. Flow may spread to crown of street.
Collector	No curb overtopping. Flow spread must leave the equivalent of one 10-foot driving lane clear of water (one lane for two-lane street, two lanes for four-lane street).
Arterials	No curb overtopping. Flow spread must leave the equivalent of two 10-foot driving lanes clear of water; one lane in each direction.
Freeways	No encroachment is allowed on any traffic lane.

*Where no curbing exists, encroachment shall not extend past property lines.

The storm sewer system shall commence at the point where the maximum allowable encroachment occurs. All storm sewers systems shall be designed to convey the minor design storm as referenced for the applicable land use classifications.

When the allowable pavement encroachment has been determined, the theoretical gutter carrying capacity for a particular encroachment shall be computed using the modified Mannings formula for flow in a triangular channel as shown in Figure 11.4 (Appendix). To simplify computations, graphs for particular street shapes may be used as shown on Figure 11.5 (Appendix). An "n" value of 0.015 shall be used unless special considerations exist.

11.6.3 Street Capacity for Major Storms

The allowable depth of flow and inundated area for the major design storm shall not exceed the limitations set forth in the following table:

Allowable Depth of Flow and Inundated Area for 100-Year Storm Runoff

Street Classification	Allowable Depth and Inundated Areas
Local and Collector	Residential dwellings, public, commercial, and industrial buildings shall not be inundated at the ground line. The depth of water over the gutter flow line shall not exceed 18 inches.
Arterial and Freeway	Residential dwellings, public, commercial, and industrial buildings shall not be inundated at the ground line. Depth of water at the street crown shall not exceed 6 inches to allow operation of emergency vehicles. The depth of water over the gutter flow line shall not exceed 18 inches.

11.6.4 Cross Street Flow

Cross street flow can occur by two separate means. One is runoff which has been flowing in a gutter and then flows across the street to the opposite gutter or inlet. The second case is flow from some external source, such as a drainageway or conduit, which will flow across the crown of the street when the conduit capacity beneath the street is exceeded. The maximum allowable cross street flow depth based on the worst condition shall not exceed the limitation stipulated in the following table.

Allowable Cross Street Flow

Street Classification	Minor Storm Runoff	100-Year Design Storm Runoff
Local	6-inch depth at crown or in the valley gutter	18 inches of depth above gutter flowline
Collector	Depth of flow shall not exceed 6 inches above gutter flow line	18 inches of depth above gutter flowline
Arterial	None	6 inches or less over crown*
Freeway	None	6 inches or less over crown*

* Only in gutter flow crossing the street. External flows may not cross street.

11.6.5 Capacity Calculations

All theoretical flow capacities shall be reduced by the appropriate reduction factors as shown in Figure 11.8 (Appendix) to obtain allowable flow capacities.

11.6.6 Drainage Tract Requirements

All backward draining cul-de-sacs and sump streets are required to have a minimum 20-foot-wide drainage easement shown on the plat for the purpose of conveying drainage. The easement shall meet the applicable requirements for storm sewer easements.

11.6.7 Sump Pump Collection Systems

Sump pump collection system or underdrain systems shall be required for all proposed subdivisions.

a. It is acceptable for a sump pump collection system to be installed in the street right-of-way, within a front yard easement, or within a backyard easement. If within a backyard easement, the collection pipe shall be installed 1.5 feet south or east of the back property line. If within a front yard easement, the collection pipe shall be installed 2.5 feet from the street back of curb.

b. Services shall not cross the street section.

c. Storm sewer can serve as the sump pump collection pipe. RCP storm sewer pipe will be cored drilled for service connections. Polyethylene pipe or PVC pipe service connections will be installed in accordance with manufacturer specifications.

d. Service connections shall be installed and capped along the mainline pipe for each individual lot. For backyard installations, service stub-outs shall be centered on the back lot line.

e. A minimum 24-inch diameter structure shall be installed at a maximum distance of 600 feet. A 5-foot access easement from right-of-way to backyard easement is required for the backyard structures. Corner lots may not require a 5-foot access easement.

f. Structures will be installed at the farthest upstream and downstream end. For front yard installations, the structures shall be on the side lot line. For backyard installations, the structures shall be +/- 5 feet from the side lot line.

g. Minimum diameter of sump pump collection system pipe shall be 6 inches. Minimum velocity shall be 2 feet/sec.

h. Maximum number of homes connected to a sump pump collection system until connected to a trunk storm sewer system is 300 homes.

i. The connection of the sump pump collection pipe to the trunk storm sewer system shall be a minimum of 0.4 foot above the structure outfall storm sewer or flow line.

j. Depth of cover shall be a minimum of 4 feet. When the minimum cover cannot be established, the collection system shall be insulated per City Design Standard Plates. For those systems that are insulated, a minimum cover of 18 inches in grassed/ landscaped areas and 30 inches in asphalt/concrete areas shall be provided from finished grade to the top of the collection system pipe.

k. Bends will not be allowed. Minimum radius of curvature shall be 150 feet.

l. For backyard installations, if the swale along the back property line is less than 2 percent, the sump pump collection pipe may be a 6 inches min. perforated pipe with a filter fabric sock.

m. The minimum slope of collection pipes shall be 0.40 percent.

n. Animal guards shall be installed at the end of all collection pipes discharging directly into a public drainageway.

o. Sump pump collection pipe diameters less than 12 inches shall only convey groundwater.

2. A regional groundwater collection drain tile system can be installed instead of a sump pump collection system.

11.7 Detention Storage

11.7.1 General

Detention ponds shall be designed and constructed at those locations identified by the City Engineer. The use of onsite detention is permitted at those locations where the onsite drainage system cannot be tied into an existing drainage system.

Onsite detention may be used if the development cannot provide adequate storm sewer systems to achieve the required storm sewer standards.

Parking lots which serve as detention storage ponds must not have a storage depth of more than 1 foot. It is recommended that notification signs be installed in parking lots which serve as detention ponds. The signs shall be permanent and high quality, meeting the City's Specifications for Traffic Signs.

11.7.2 Design Storm

Detention ponds along major drainageways as identified in the City Master Drainage Plan shall be designed for a 100-year design flow.

Other detention ponds shall be designed such that the 5-year return storm is conveyed through the principal outlet assembly and the 100-year return storm is conveyed through the overflow assembly.

11.7.3 Release Methods

Intermittent ponds shall drain completely.

Careful consideration must be given to the discharge of the surface release as to the elimination of erosion potential, and the capacity of the downstream surface water course. The release structure shall be designed to withstand the forces caused by the structure being overtopped during a larger than design storm.

A stage (foot) versus release rate (CFS) curve must be provided for the release structure.

11.7.4 Maximum Release Rate

The detention pond volumes and release rate shall be designed to accommodate runoff generated by the development and post-developed upstream properties.

The release rate from the detention pond cannot exceed predevelopment rates for the 5-year and 100-year return storm when discharge is conveyed onto undeveloped property unless City-owned conveyance structures of adequate size are contiguous and downstream of the proposed discharge points.

If project size is over five acres and is zoned non-single-family residential, all discharge from site cannot exceed single-family discharge rates for the 5-and 100-year return storm.

11.7.5 Maintenance Requirements

Detention ponds and similar areas, not required as a necessary part of the major drainage system as outlined in the City Master Drainage Plan, may be accepted by the City for maintenance only if such land provides another useful public service such as a public park or wildlife area.

All detention areas shall have a 30-foot-wide access to a public right-of-way if they are not located adjacent to a public right-of-way.

Detention pond properties greater than two acres in size where discharge is generated from publicly maintained infrastructure may be considered for dedication to the City during preliminary plan approval. The ponds must have a maintenance bench providing access to all pond areas. The bench shall be 12 feet wide with a cross slope no greater than 10 percent. If the bench is located along a future recreational trail, the maximum cross slope may be reduced to meet trail requirements. Property delineation markers shall be installed upon dedication of the property in locations specified bythe City Engineer. . All other detention ponds shall be privately maintained.

11.7.6 Adjacent Property Elevations

The property corner elevation of properties abutting a detention pond shall be 1 foot above the 100-year design storm.

Recommended minimum ground elevations for homes abutting or affected by the detention pond shall be 2 feet above the overflow elevation. Recommended minimum ground elevation for homes abutting or affected by detention ponds will be a minimum of 4 feet above the 100-year pond high water elevation if an overflow system is not available or at an elevation that provides an additional 50 percent storage.

11.8 Best Management Practices

All projects have to meet the requirements of Section 11.8 Best Management Practices except those that are less than two acres and have less than one acre of new impervious area.

11.8.1 Two-Step Process of Best Management Practices

The following process is recommended for selecting structural BMPs in newly developing and redeveloping urban areas:

1. **Step 1. Employ Runoff Reduction Practices**

   To reduce runoff peaks and volumes from urbanizing areas, employ a practice generally called “minimizing directly connected impervious areas” (MDCIA). The principal behind MDCIA is two-fold: to reduce impervious areas and to route runoff from impervious surfaces over grassy areas to slow down runoff, promote infiltration, and reduce costs. The benefits are less runoff, less storm water pollution, and less cost for drainage infrastructure. There are several approaches to reduce the effective imperviousness of a development site:

   a. **Reduced Pavement Area**

      Using smaller roadway cross sections is encouraged. Creative site layout can reduce the extent of paved areas.

b. Porous Pavement

The use of modular block porous pavement or reinforced turf in low traffic zones such as parking areas and low use service drives such as fire lanes can significantly reduce site imperviousness.

c. Grass Buffers

Draining impervious areas over grass buffers slows down runoff and encourages infiltration, in effect reducing the impact of the impervious area.

d. Grass Swales

The use of grass swales instead of storm sewers slows down runoff and promotes infiltration, also reducing effective imperviousness and detention.

Two of the approaches for reducing imperviousness are structural BMPs and are described in detail in the following sections:

Section	Structural BMP
11.8.2	Grass Buffer
11.8.3	Grass Swale

e. Levels of Minimizing Directly Connected Impervious Areas

Minimizing directly connected impervious areas (DCIAs) can be implemented in varying degrees. Two general levels associated with minimizing DCIAs have been identified and are described below:

- **Level 1.** The primary intent is to direct all possible runoff generated by impervious surfaces to flow over grass-covered areas, and to provide sufficient travel time so as to encourage the removal of suspended solids before runoff leaves the site, enters a curb and gutter, or enters another storm water collection system.
- **Level 2.** This level replaces street curb and gutter systems with low-velocity grass-lined swales and pervious street shoulders. Conveyance systems and storm sewer inlets will still be needed to collect runoff at downstream intersections and crossings where storm water flow rates exceed the capacity of the swales. Small culverts will be needed at street crossings and at individual driveways until inlets are provided to convey the flow to a storm sewer. This level can only be used for City projects with approval of the City Engineer.

2. **Step 2. Provide Water Quality Capture Volume**

A fundamental requirement for any site addressing storm water quality is to provide water quality capture volume (WQCV). One or more of five types of water quality basins, each draining slowly to provide for long-term settling of sediment particles, may be selected to provide WQCV as shown in Figure 11.48. These five BMPs are described in detail in the following sections:

Section	Structural BMP
11.8.4	Porous Landscape Detention
11.8.6	Extended Detention Basin
11.8.7	Sand Filter Extended Detention Basin
11.8.8	Constructed Wetlands Basin
11.8.9	Retention Pond

The following BMP must be used with a BMP that meets the WQCV criteria. It does not provide WQCV by itself. It can, however, provide additional water quality treatment and aesthetic value:

Section	Structural BMP
11.8.10	Constructed Wetlands Channel

The following BMP does not meet the WQCV criteria. It is only intended for use in highly urbanized areas, such as redevelopment conditions, where existing development precludes the ability to meet the WQCV criteria. This BMP must be approved for use by the City Engineer. In determining BMP approval, preference will be given to structural BMPs providing WQCV as listed in this section.

Section	Structural BMP
11.8.11	Water Quality Catch Basins and Water Quality Catch Basin Inserts

The following BMP meets the WQCV criteria, but does not use Figure 11.48 in the design process. Provide runoff capture volume as indicated in Section 11.8.12.

Section	Structural BMP
11.8.12	Bioretention

11.8.2 Grass Buffer

1. **Description**

Grass buffer strips are an integral part of the MDCIA land development concept. They are uniformly graded and densely vegetated areas of turf grass. They require sheet flow to promote filtration, infiltration, and settling to reduce runoff pollutants. Grass buffers differ from grass swales as they are designed to accommodate overland sheet flow rather than concentrated or channelized flow. They can be used to remove larger sediment from runoff from impervious areas.

Whenever concentrated runoff occurs, it should be evenly distributed across the width of the buffer via a flow spreader. This may be a porous pavement strip or another type of structure used to achieve uniform sheet-flow conditions. Grass buffers can also be combined with riparian zones in treating sheet flows and in stabilizing channel banks adjacent to major drainageways and receiving waters. Grass buffers can be interspersed with shrubs and trees to improve their aesthetics and to provide shading.

2. General Application

A grass buffer can be used in residential and commercial areas. They are typically located adjacent to impervious areas. When used, they should be incorporated into site drainage, street drainage, and master drainage planning. Because their effectiveness depends on having an evenly distributed sheet flow over their surface, the size of the contributing area and the associated volume of runoff have to be limited. Flow can be directly accepted from an impervious area such as a parking lot or building roofs, provided the flow is distributed uniformly over the strip. Grass buffers provide only marginal pollutant removal and require that follow-up structural BMPs be provided. They do, however, help to reduce some of the runoff volume from small storms.

3. General Properties

a. General

The grass and other vegetation provide aesthetically pleasing green space, which can be incorporated into a development landscaping plan. Eventually, the grass strip next to the spreader or the pavement will accumulate sufficient sediment to block runoff. At that point, a portion of the grass buffer strip will need to be removed and replaced.

b. Physical Site Suitability

After final grading, the site should have a uniform slope and be capable of maintaining an even sheet flow throughout without concentrating runoff into shallow swales or rivulets. The allowable tributary area depends on the width, length, and the soils that lay under the grass buffer. Hydrologic Soil Groups A and B provide the best infiltration capacity, while Soil Groups C and D provide best site stability. The swelling potential of underlying soils should also be taken into account in how the soils may affect adjacent structures and pavement when water is delivered to the grassed areas.

c. Pollutant Removal

Pollutant removal depends on many factors, such as soil permeability; site slope; the flow path length along the buffer; the characteristics of drainage area; runoff volumes and velocities; and the type of vegetation. The general pollutant removal of both particulate and soluble pollutants is projected to be low to moderate. Grass buffers rely primarily upon the settling and interception of solids, and, to only a minor degree, on biological uptake and runoff infiltration. See Table 11.4 (Appendix) for an estimated range of pollutant removals. Maintenance requirements for this BMP are listed in Table 11.5 (Appendix).

4. Design Considerations

Design of grass buffers is based primarily on maintaining sheet-flow conditions across a uniformly graded, dense grass cover strip. When a grass buffer is used over unstable slopes, soils, or vegetation, rills and gullies will form that will disrupt sheet flow. The resultant short-circuiting will invalidate the intended water quality benefits. Grass buffers should be protected from excessive pedestrian or vehicular traffic that can damage the grass cover and affect even sheet-flow distribution. A mixture of grass and trees may offer benefits for slope stability and improved aesthetics.

5. Design Procedure and Criteria

The following steps outline the grass buffer design procedure and criteria. Figure 11.43 (Appendix) is a schematic of the facility and its components.

a. Step 1: Design Discharge

Determine the two-year peak flow rate of the area draining to the grass buffer. Also, determine the flow control type: sheet or concentrated.

b. Step 2: Minimum Length

Calculate the minimum length (normal to flow) of the grass buffer. The upstream flow needs to be uniformly distributed over this length. General guidance suggests that the hydraulic load should not exceed 0.05 cfs/linear foot of buffer during a two-year storm to maintain a sheet flow of less than 1 inch throughout dense grass that is at least 2 inches high. The minimum design length (normal to flow) is therefore calculated as:

$$L_G = \frac{Q_{2-year}}{0.05} \qquad \text{(Equation 6)}$$

In which: L_g = Minimum design length (feet)

Q_{2-year} = Peak discharge supplied to the grass buffers by a 2-year event (cfs)

Longer lengths may be used.

c. **Step 3: Minimum Width**

The minimum width (WG) (the distance along the sheet flow direction) of the grass buffer shall be determined by the following criteria for onsite and concentrated flow control conditions:

- Sheet Flow Control (use the larger value)

$W_G = 0.2L_I$ or 10 feet (Equation 7)

In which:

LI = The length of flow path of the sheet flow over the upstream impervious surface (feet)

- Concentrated Flow Control (use the larger value)

$W_G = 0.15(A_t/L_t)$ or 10 feet (Equation 8)

In which:

A_t = The tributary area (square feet)

L_t = The length of the tributary inflow normal to flow spreader (i.e., width of flow spreader (feet))

A generally rectangular-shaped strip is preferred and should be free of gullies or rills that concentrate the overland flow.

d. **Step 4: Maximum Slope**

Design slope in the direction of flow shall not exceed 4 percent.

e. **Step 5: Flow Distribution**

Incorporate a device on the upstream end of the buffer to evenly distribute flows along the design length. Slotted curbing, modular block porous pavement (MBP), or other spreader devices can be used to apply flows. Concentrated flow supplied to the grass buffer must use a level spreader (or a similar concept) to evenly distribute flow onto the buffer.

f. **Step 6: Vegetation**

Vegetate the grass buffer with dense turf to promote sedimentation and entrapment and to protect against erosion.

g. **Step 7: Outflow Collection**

Provide a means for outflow collection. Most of the runoff during significant events will not be infiltrated and will require a collection and conveyance system. In some cases, the use of under-drains can maintain better infiltration rates as the soils saturate and help dry out the buffer after storms or irrigation periods.

6. **Design Example**

Design forms that provide a means of documenting the design procedure are included in the Appendix. A completed form is shown in Figure 11.44 (Appendix) as a design example.

11.8.3 Grass Swale

1. **Description**

A grass swale sedimentation facility is an integral part of the MDCIA development concept. They are densely vegetated drainageways with low-pitched side slopes that collect and slowly convey runoff. Design of their longitudinal slope and cross section size forces the flow to be slow and shallow, thereby facilitating sedimentation while limiting erosion. Berms or check dams should be installed perpendicular to the flow as needed to slow it down and encourage settling and infiltration.

2. **General Application**

A grass swale can be located to collect overland flows from areas such as parking lots, buildings, residential yards, roadways, and grass buffer strips. They can be made a part of the plans to minimize a directly connected impervious area by using them as an alternative to a curb-and-gutter system if approved by the City Engineer. A grass swale is set below adjacent ground level, and runoff enters the swales over grassy banks. The potential exists for wetland vegetation to become established if the swale experiences standing water or if there is a base flow. A site with a base flow should be managed as either a swale with an unlined trickle channel, or as a wetland bottom channel, the latter providing an additional BMP to storm water runoff.

3. **General Properties**

a. **General**

A grass swale can be more aesthetically pleasing than concrete or rock-lined drainage systems. Although limited by the infiltration capacity of local soils, this BMP can also provide some reduction in runoff volumes from small storms. Dense grasses can reduce flow velocities and protect against erosion during larger storm events. Swales in residential and commercial settings can also be used to limit the extent of directly connected impervious areas.

b. **Physical Site Suitability**

A grass swale is practical only at sites with general ground slopes of less than 4 percent and are definitely not practical for sites steeper than 6 percent. The longitudinal slopes of a grass swale should be kept to less than 1.0 percent, which often necessitates the use of grade control checks or drop structures. Where the general terrain slope exceeds 4 percent, a grass swale is often practical only on the upslope side of the adjacent street.

When soils with high permeability (for example, Class A or B) are available, the swale will infiltrate a portion of the runoff into the ground, but such soils are not required for effective application of this BMP. When Class C and D soils are present, the use of a sand/gravel underdrain is recommended.

c. Pollutant Removal

Removal rates reported in literature vary and fall into the low to medium range. Under good soil conditions and low-flow velocities, moderate removal of suspended solids and associated other constituents can be expected. If soil conditions permit, infiltration can remove low to moderate loads of soluble pollutants when flow velocities are very low. As a result, small frequently occurring storms can benefit the most. See Table 11.4 (Appendix) for estimated ranges in pollutant removal rates by this BMP. Maintenance considerations for this BMP are listed in Table 11.6 (Appendix).

4. Design Considerations and Criteria

Figure 11.45 (Appendix) shows trapezoidal and triangular swale configurations. A grass swale is sized to maintain a low velocity during small storms and to collect and convey larger runoff events, all for the projected fully developed land use conditions.

A healthy turf grass cover must be developed to foster dense vegetation. Permanent irrigation in some cases may be necessary. Judicious use of grass swales can replace both the curb-and-gutter systems and greatly reduce the storm sewer systems in the upper portions of each watershed when designed to convey the "initial storm" (for example, a two- or a five-year storm) at slow velocities. However, if one or both sides of the grass swale are also to be used as a grass buffer, the design of the grass buffer has to follow the requirements of Section 11.8.2, Grass Buffer.

5. Design Procedure and Criteria

The following steps outline the grass swale design procedure and criteria:

a. Step 1: Design Discharge

Determine the two-year flow rate in the proposed grass swale for water quality. The swale shall also meet the conveyance requirements of Section 11.5.

b. **Step 2: Swale Geometry**

Select geometry for the grass swale. The cross section should be trapezoidal or triangular. The side slopes shall be flatter than 4:1 (horizontal/ vertical). The wider the wetted area of the swale, the slower the flow and the more effective it is in removing pollutants.

c. **Step 3: Longitudinal Slope**

Maintain a longitudinal slope for the grass swale between 0.2 and 1.0 percent. If the longitudinal slope requirements cannot be satisfied with available terrain, grade control checks or small drop structures must be incorporated to maintain the required longitudinal slope. If the slope of the swale exceeds 0.5 percent, the swale must be vegetated with irrigated turf grass to establish the vegetation.

d. **Step 4: Flow Velocity and Depth**

Calculate the velocity and depth of flow through the swale. Based on Manning's equation and a Manning's roughness coefficient of n=0.05, find the channel velocity and depth using the 2-year flow rate determined in Step 1.

e. **Step 5: Maximum Flow Velocity**

Maximum flow velocity of the channel shall not exceed 1.5 feet per second and the maximum flow depth shall not exceed 2 feet at the two-year peak flow rate. If these conditions are not attained, repeat Steps 2 through 4, each time altering the depth and bottom width or longitudinal slopes until these criteria are satisfied.

f. **Step 6: Vegetation**

Vegetate the grass swale with dense turf grass to promote sedimentation, filtration, and nutrient uptake, and to limit erosion through maintenance of low-flow velocities.

g. **Step 7: Street and Driveway Crossings**

If applicable, small culverts at each street crossing and/or driveway crossing may be used to provide onsite WQCV in a similar fashion to an extended detention basin (if adequate volume is available).

h. **Step 8: Drainage and Flood Control**

Check the water surface during larger storms such as the 5-year through the 100-year event to assure that drainage from these larger events is being controlled without flooding critical areas.

6. **Design Example**

Design forms to document the design procedure are included in the Appendix. A completed form is shown in Figure 11.46 (Appendix) as a design example.

11.8.4 Porous Landscape Detention

1. **Description**

Porous landscape detention consists of a low lying vegetated area underlain by a sand bed with an underdrain pipe. A shallow surcharge zone exists above the porous landscape detention for temporary storage of the WQCV. During a storm, accumulated runoff ponds in the vegetated zone and gradually infiltrates into the underlying sand bed, filling the void spaces of the sand. The underdrain gradually dewaters the sand bed and discharges the runoff to a nearby channel, swale, or storm sewer. This BMP allows WQCV to be provided on a site that has little open area available for storm water detention.

2. **General Application**

a. **Location**

A porous landscape detention can be located in just about any of the open areas of a site. It is ideally suited for small installations such as:

Parking lot islands.
Street medians.
Roadside swale features.
Site entrance or buffer features.

This BMP may also be implemented at a larger scale, serving as an infiltration basin for an entire site if desired, provided the water quality capture volume and average depth requirements contained in this section are met.

Vegetation may consist of irrigated bluegrass or natural grasses with shrub and tree plantings if desired.

3. **General Properties**

a. **General**

A primary advantage of porous landscape detention is making it possible to provide WQCV on a site while reducing the impact on developable land. It works well with irrigated bluegrass, whereas experience has shown that conditions in the bottom of extended detention basins become too wet for bluegrass. A porous landscape detention provides a natural moisture source for vegetation, enabling "green areas" to exist with reduced irrigation.

The primary drawback of porous landscape detention is a potential for clogging if a moderate to high level of silts and clays is allowed to flow into the facility. Also, this BMP needs to be avoided close to building foundations or other areas where expansive soils are present, although an underdrain and impermeable liner can reduce some of this concern.

b. Physical Site Suitability

If an underdrain system is incorporated into this BMP, porous landscape detention is suited for about any site regardless of in-situ soil type. If sandy soils are present, the facility can be installed without an underdrain (infiltration option); sandy subsoils are not a requirement. This BMP has a relatively flat surface area, and may be more difficult to incorporate into steeply sloping terrain.

c. Pollutant Removal

Although not tested to date in the Brandon area, the amount of pollutant removed by this BMP should be significant and should equal or exceed the removal rates provided by sand filters. In addition to settling, porous landscape detention provides for filtering, adsorption, and biological uptake of constituents in storm water. See Table 11.4 (Appendix) for estimated ranges in pollutant removals. See Table 11.7 (Appendix) for maintenance requirements for a porous landscape detention.

4. Design Considerations

Figure 11.47 (Appendix) shows a cross section for a porous landscape detention. When implemented using multiple small installations on a site, it is increasingly important to accurately account for each upstream drainage area tributary to each porous landscape detention site to make sure that each facility is properly sized, and that all portions of the development site are directed to a porous landscape detention.

5. Design Procedure

The following steps outline the porous landscape detention design procedure and criteria:

a. Step 1: Basin Storage Volume

Provide a storage volume based on a 12-hour drain time.

- Find the required storage volume (watershed inches of runoff). Using the tributary areas imperviousness, determine the required WQCV (watershed inches of runoff) using

Figure 11.48 (Appendix), based on the porous landscape detention 12-hour drain time.

- Calculate the design volume in cubic feet as follows: (Equation 9)

$$Design\ Volume = \left(\frac{Water\ Quality\ Capture\ Volume}{12}\right) * Area$$

In which:

Area = The watershed area tributary to the porous landscape detention basin (square feet)

b. Step 2: Surface Area

- Calculate the minimum required surface area as follows: (Equation 10)

$$Surface\ Area = \left[\frac{Design\ Volume\ (ft^3)}{d_{av}}\right]$$

In which:

d_{av} = average depth (feet) of the porous landscape detention basin.

c. Step 3: Sand-Peat Media

Provide, as a minimum, an 18-inch-thick layer of well mixed sand and peat (2/3 sand and 1/3 peat) for plant growth as shown in Figure 11.47. Keep the top surface as flat as possible, while avoiding side slopes steeper than 4:1.

When installing in type NRCS Type D or expansive soils and no subdrain outlet is possible, use a total sand-peat mixed layer thickness of 36-inches and no granular subbase.

d. Step 4: Granular Subbase

Whenever an under-drain is used or when the soils are not expansive (i.e., soils are NRCS Type A, B, or C) and an under-drain is not used, use an 8-inch layer of granular subbase with all fractured faces meeting the requirements of AASHTO #67 coarse aggregate.

e. Step 5: Membrane Liner

If expansive or NRCS Type D soils are present, install an impermeable 15-mil-thick, or heavier, liner on the bottom and sides of the basin.

If soils are not expansive (i.e., NRCS Type A, B, or C), use porous geotextile fabric to line the entire basin bottom and sides. Porous membrane liner shall be of woven monofilament as manufactured by Carthage Mills-Carthage 15 percent (or equal) having an open surface area of 12–15 percent, with openings equivalent to AOS U.S. Std. Sieve size of 40 to 50.

f. **Step 6: WQCV Depth**

Maintain an average WQCV depth between 6 inches and 12 inches. Average depth is defined as water volume divided by the water surface area.

6. **Design Example**

Design forms that document the design procedure are included in the Appendix. A completed form is shown in Figure 11.49 (Appendix) as a design example.

11.8.5 Incorporating Water Quality Capture Volume into Storm Water Quantity Detention Basins

Wherever possible, it is recommended that WQCV facilities be incorporated into storm water quantity detention facilities. This is relatively straightforward for an extended detention basin, constructed wetland basin, and a retention pond. The 100-year detention level is provided above the WQCV and the outlet structure is designed to control two or more different releases. Figures 11.50a and 11.50b (Appendix) show examples of combined quality/quantity outlet structures. Figure 11.50c (Appendix) contains typical outlet structure notes applicable to the design of outlet structures.

Storm water quantity detention could be provided above the WQCV for porous pavement and landscape detention, provided the drain times for the larger events are kept short. The following approach is suggested:

- **Water Quality:** The full WQCV is to be provided according to the design procedures documented for the Structural BMP.
- **100-Year Storm:** The WQCV plus the full 100-year detention volume is to be provided.

11.8.6 Extended Detention Basin

1. **Description**

An extended detention basin is a sedimentation basin designed to totally drain dry over an extended time after storm water runoff ends. It is an adaptation of a detention basin used for flood control. The primary difference is in the outlet design. The extended detention basin uses a much smaller outlet that extends the draining time of the more frequently occurring runoff events to facilitate pollutant removal. The extended detention basin's drain time for the brim-full water quality capture volume (i.e., time to fully evacuate the design capture volume) of 40 hours is recommended to remove a significant portion of fine particulate pollutants found in urban storm water runoff. Soluble pollutant removal can be somewhat enhanced by providing a small wetland marsh or ponding area in the basin's bottom to promote biological uptake. The basins are considered to be "dry" because they are designed not to have a significant permanent pool of water remaining between storm water runoff events. However, an extended detention basin may develop wetland vegetation and sometimes shallow pools in the bottom portions of the facilities.

2. General Application

An extended detention basin can be used to enhance storm water runoff quality and reduce peak storm water runoff rates. If these basins are constructed early in the development cycle, they can also be used to trap sediment from construction activities within the tributary drainage area. The accumulated sediment, however, will need to be removed after upstream land disturbances cease and before the basin is placed into final long-term use. Also, an extended detention basin can sometimes be retrofitted into existing flood control detention basins.

Extended detention basins can be used to improve the quality of urban runoff from roads, parking lots, residential neighborhoods, commercial areas, and industrial sites and are generally used for regional or follow-up treatment. They can also be used as an onsite BMP and work well in conjunction with other BMPs, such as upstream onsite source controls and downstream infiltration/filtration basins or wetland channels. If desired, a flood routing detention volume can be provided above the WQCV of the basin.

3. General Properties

a. General

An extended detention basin can be designed to provide other benefits such as recreation and open space opportunities in addition to reducing peak runoff rates and improving water quality. They are effective in removing particulate matter and associated heavy metals and other pollutants. As with other BMPs, safety issues need to be addressed through proper design.

b. Physical Site Suitability

Normally, the land required for an extended detention basin is about 0.5 to 2.0 percent of the total tributary development area. In high groundwater areas, instead consider the use of retention ponds in order to avoid many of the problems that can occur when the extended detention basin's bottom is located below the seasonal high water table. Soil maps should be consulted, and soil borings may be needed to establish design geotechnical parameters.

c. Pollutant Removal

The pollutant removal range of an extended detention basin is presented in Table 11.4 (Appendix). Removal of suspended solids and metals can be moderate to high, and removal of nutrients is low to moderate. The removal of nutrients can be improved when a small shallow pool or wetland is included as part of the basin's bottom or the basin is followed by BMPs more efficient at removing soluble pollutants, such as a filtration system, constructed wetlands, or wetland channels.

The major factor controlling the degree of pollutant removal is the emptying time provided by the outlet. The rate and degree of removal will also depend on influent particle sizes. Metals, oil and grease, and some nutrients have a close affinity for suspended sediment and will be removed partially through sedimentation.

d. Aesthetics and Multiple Uses

Since an extended detention basin is designed to drain very slowly, its bottom and lower portions will be inundated frequently for extended periods of time. Grasses in this frequently inundated zone will tend to die off, with only the species that can survive the specific environment at each site eventually prevailing. In addition, the bottom will be the depository of all the sediment that settles out in the basin. As a result, the bottom can be muddy and may have an undesirable appearance. To reduce this problem and to improve the basin's availability for other uses (such as open space, habitat, and passive recreation), the designer should provide a lower-stage basin as suggested in the Two-Stage Design procedure. As an alternative, a retention pond could be used, in which the settling occurs primarily within the permanent pool.

e. Design Considerations

Whenever desirable and feasible, incorporate the extended detention basin within a larger flood control basin. Whenever possible, try to provide for other urban uses such as passive recreation and wildlife habitat. If multiple uses are being contemplated, consider the multiple-stage detention basin to limit inundation of passive recreational areas to one or two occurrences a year. Generally, the area within the WQCV is not well suited for active recreation facilities such as ballparks, playing fields, and picnic areas. These are best located above the WQCV pool level.

Figure 11.51 (Appendix) shows a representative layout of an extended detention basin.

Perforated outlet and trash rack configurations are illustrated in Figures 11.50a, 11.50b, and 11.52 through 11.56 (Appendix). Figure 11.52 (Appendix) equates the WQCV that needs to be emptied over 40 hours to the total required area of perforations per row for the standard configurations shown in that section. The chart is based on the rows being equally spaced vertically at 4-inch centers. The total area of perforations per row is then used to determine the number of uniformly sized holes per row as shown in Figures 11.53 and 11.54 (Appendix). One or more perforated columns on a perforated orifice plate integrated into the front of the outlet can be used. Other types of outlets may also be used, provided they control the release of the WQCV in a manner consistent with the drain time requirements.

Although the soil types beneath the pond seldom prevent the use of this BMP, they should be considered during design. Any potential exfiltration capacity should be considered a short-term characteristic and ignored in the design of the WQCV because exfiltration will decrease over time as the soils clog with fine sediment and as the groundwater beneath the basin develops a mound that surfaces into the basin.

High groundwater should not preclude the use of an extended detention basin. Groundwater, however, should be considered during design and construction, and the outlet design must account for any upstream base flows that enter the basin or that may result from groundwater surfacing within the basin itself.

Stable, all weather access to critical elements of the pond, such as the inflow area, outlet, spillway, and sediment collection areas, must be provided for maintenance purposes. Maintenance requirements for the extended detention basin are provided in Table 11.8 (Appendix).

4. Design Procedure and Criteria

The following steps outline the design procedure and criteria for an extended detention basin.

a. Step 1: Detention Pond Storage Volume

Provide a storage volume equal to 120 percent of the WQCV based on a 40-hour drain time, above the lowest outlet (i.e., perforation) in the basin. The additional 20 percent of storage volume provides for sediment accumulation and the resultant loss in storage volume.

- Determine the WQCV tributary catchment's percent imperviousness. Account for the effects of DCIAs, if any, on effective imperviousness. Using Figure 11.55 (Appendix), determine the reduction in impervious area to use with WQCV calculations.

- Find the required storage volume (watershed inches of runoff). Determine the required WQCV (watershed inches of runoff) using Figure 11.48 (Appendix), based on the extended detention basin's 40-hour drain time. Calculate the design volume in acre-feet as follows: (Equation 11)

$$Design\,Volume = \left[\frac{Water\;Quality\;Capture\;Volume}{12}\right] * Area * 1.2$$

In which:
Area = The watershed area tributary to the extended detention basin (acres)
1.2 Factor = Multiplier of 1.2 to account for the additional 20 percent of required storage for sediment accumulation

b. Step 2: Outlet Control

The outlet controls are to be designed to release the WQCV (i.e., not the "design volume") over a 40-hour period, with no more than 50 percent of the WQCV being released in 12 hours. Refer to the Appendix for schematics pertaining to structure geometry; grates, trash racks, and screens; outlet type (orifice plate or perforated riser pipe); cutoff collar size and location; and all other necessary components.

For a perforated outlet, use Figure 11.52 (Appendix) to calculate the required area per row based on WQCV and the depth of perforations at the outlet. See Figures 11.53 and 11.54 (Appendix) to determine the appropriate perforation geometry and number of rows (the lowest perforations should be set at the water surface elevation of the outlet micropool). The total outlet area can then be calculated by multiplying the area per row by the number of rows. Figure 11.50c contains typical outlet structure notes applicable to the design of outlet structures.

c. Step 3: Trash Rack

Provide a trash rack of sufficient size to prevent clogging of the primary water quality outlet. Size the rack so as not to interfere with the hydraulic capacity of the outlet. Using the total outlet area and

the selected perforation diameter (or height), Figures 11.50a, 11.50b, 11.56, or 11.57 (Appendix) will help to determine the minimum open area required for the trash rack. Use one half of the perforated plate's total outlet area to calculate the trash rack's size. This accounts for the variable inundation of the outlet orifices.
Figures 11.50a, 11.50b, and 11.56 (Appendix) were developed as suggested standardized outlet designs for smaller sites.

d. **Step 4: Basin Shape**

Shape the pond whenever possible with a gradual expansion from the inflow area and a gradual contraction toward the outlet, thereby minimizing short circuiting. It is best to have a basin length to width ratio between 2:1 and 3:1. It may be necessary to modify the inflow and outlet points through the use of pipes, swales, or channels to accomplish this. Always maximize the distance between the inlet and the outlet.

e. **Step 5: Two-Stage Design**

A two-stage design with a pool that fills often with frequently occurring runoff minimizes standing water and sediment deposition in the remainder of the basin. The two stages are as follows:

- **Top Stage:** The top stage should be 2 or more feet deep with its bottom sloped at 2 percent toward the low-flow channel.
- **Bottom Stage:** The active surcharge storage basin of the bottom stage should be 1.0 to 2.0 feet deep below the bottom of the top stage and store no less than 3 percent of the WQCV. Provide a micropool below the bottom active storage volume of the lower stage at the outlet point. The pool should be one half the depth of the upper WQCV depth or 2.5 feet, whichever is larger.

f. **Step 6: Low-Flow Channel**

Conveys low flows from the forebay to the bottom stage. Erosion protection should be provided where the low-flow channel enters the bottom stage. Lining the low-flow channel with riprap is recommended. Make it at least 9 inches deep if buried riprap is used. At a minimum provide capacity equal to twice the release capacity at the upstream forebay outlet.

g. **Step 7: Basin Side Slopes**

Basin side slopes should be stable and gentle to facilitate maintenance and access. Side slopes shall be no steeper than 4:1.

h. Step 8: Dam Embankment

The embankment should be designed not to fail during a 100-year or larger storm. Embankment slopes should be no steeper than 3:1, and planted with turf-forming grasses. Poorly compacted native soils should be excavated and replaced. Embankment soils should be compacted to at least 95 percent of their maximum density according to ASTM D 698-70 (Modified Proctor).

i. Step 9: Vegetation

Bottom vegetation provides erosion control and sediment entrapment. Pond bottom, berms, and side sloping areas may be planted with native grasses or with irrigated turf, depending on the local setting.

j. Step 10: Maintenance Access

All weather stable access to the bottom, forebay, and outlet controls area shall be provided for maintenance vehicles. Maximum grades should not exceed 10 percent, and a stable driving surface capable for use by maintenance equipment. If conditions warrant, a gravel or hard surface shall be provided.

k. Step 11: Inflow Point

Dissipate flow energy at the pond's inflow point(s) to limit erosion and promote particle sedimentation.

l. Step 12: Forebay Design

Provide the opportunity for larger particles to settle out in the inflow area, the area that has a solid surface bottom, to facilitate mechanical sediment removal. A rock berm should be constructed between the forebay and the main extended detention basin. The forebay volume of the permanent pool should be about 5 percent of the design water quality capture volume. A pipe throughout the berm to convey water to the main body of the extended detention basin should be offset from the inflow streamline to prevent short circuiting and should be sized to drain the forebay volume in
15 minutes. Presedimentation forebays shall only be utilized when the extended detention basin water quality capture volume exceeds 4,000 cubic feet.

m. Step 13: Flood Storage

Combining the water quality facility with a flood control facility is recommended. The 100-year or other floods may be detained above the WQCV. See Section 11.8.5, Incorporating WQCV into Storm Water Quantity Detention Basins, for further guidance.

n. Step 14: Multiple Uses

Whenever desirable and feasible, incorporate the extended detention basin within a larger flood control basin. Also, whenever possible, try to provide for other urban uses such as active or passive recreation and wildlife habitat. If multiple uses are being contemplated, use the multiple-stage detention basin to limit inundation of passive recreational areas to one or two occurrences a year. Generally, the area within the WQCV is not well suited for active recreation facilities such as ballparks, playing fields, and picnic areas. These are best located above the WQCV level.

5. Design Example

Design forms that document the design procedure are included in the Appendix. A completed form is shown in Figure 11.58 (Appendix) as a design example.

11.8.7 Sand Filter Extended Detention Basin

1. Description

A sand filter extended detention basin is a storm water filter consisting of a runoff storage zone underlain by a sand bed with an underdrain system. During a storm, accumulated runoff ponds in the surcharge zone and gradually infiltrates into the underlying sand bed, filling the void spaces of the sand. The underdrain gradually dewaters the sand bed and discharges the runoff to a nearby channel, swale, or storm sewer.

2. General Application

A sand filter extended detention basin is generally suited to off-line, onsite configurations where there is no base flow and the sediment load is relatively low.

3. General Properties

a. General

Primary advantages of sand filter extended detention basins include effective water quality enhancement through settling and filtering. The primary drawback is a potential for clogging if a moderate to high level of silts and clays is allowed to flow into the facility. For this reason, it should not be put into operation while construction activities are taking place in the tributary catchment. Also, this BMP should not be located close to building foundations or other areas where expansive soils are a concern, although an underdrain and impermeable liner can reduce some of this concern.

b. **Physical Site Suitability**

Since an underdrain system is incorporated into this BMP, a sand filter extended detention basin is suited for about any site; the presence of sandy subsoils is not a requirement. This BMP has a relatively flat surface area, so it may be more challenging to incorporate it into steeply sloping terrain.

c. **Pollutant Removal**

Although not fully tested to date in the Sioux Falls area, the tests on filter vaults throughout the United States show that the amount of pollutant removed by this BMP should be significant and should at least equal the removal rates by sand filters tested elsewhere. See Table 11.4 (Appendix) for estimated ranges in pollutant removals.

d. **Maintenance Needs**

Before selecting this BMP, be sure that the maintenance specified in Table 11.9 (Appendix) will be provided by the owner with responsibilities negotiated with the City. This BMP's performance is dependent on having regular maintenance provided.

4. **Design Procedure and Criteria**

The layout of a sand filter extended detention basin is shown in Figure 11.59 (Appendix). The following steps outline the design procedure and criteria for a sand filter extended detention basin.

a. **Step 1: Basin Storage Volume**

Provide a storage volume equal to 100 percent of the WQCV based on a 40-hour drain time, above the sand bed of the basin.

- Determine the WQCV tributary catchment's percent imperviousness. Account for the effects of DCIA, if any, on effective imperviousness. Using Figure 11.55 (Appendix), determine the reduction in impervious area to use with WQCV calculations.
- Find the required storage volume (watershed inches of runoff).
- Determine the required WQCV (watershed inches of runoff) using Figure 11.48 (Appendix), based on the sand filter extended detention basin's 40-hour drain time.

- Calculate the design volume in acre-feet as follows: (Equation 12)

$$Design\ Volume = \left[\frac{Water\ Quality\ Capture\ Volume}{12}\right] * Area$$

In which:

Area = The watershed area tributary to the sand filter extended detention basin (acres)

b. Step 2: Basin Depth

Maximum design volume depth shall be 3 feet.

c. Step 3: Filter's Surface Area

Calculate the minimum sand filter area (A_S) at the basin's bottom with the following equation:

(Equation 13)

A_s = *Design Volume / 3* *43,560 (square feet)

d. Step 4: Outlet Controls

An 18-inch layer of sand (ASTM C-33) over an 8-inch gravel layer (AASHTO No. 8) shall line the entire sand filter extended detention basin for purposes of draining the WQCV.

If expansive soils are a concern or if the tributary catchment has chemical or petroleum products handled or stored, install a 15-mil-thick impermeable membrane below the gravel layer.

In addition, an overflow shall be provided to convey flows in excess of the WQCV out of the basin.

5. Design Example

Design forms that document the design procedure are included in the Appendix. A completed form is shown in Figure 11.60 (Appendix) as a design example.

11.8.8 Constructed Wetlands Basin

1. Description

A constructed wetlands basin is a shallow retention pond that requires a perennial base flow to permit the growth of rushes, willows, cattails, and reeds to slow down runoff and allow time for sedimentation, filtering, and biological uptake.

Constructed wetlands basins differ from "natural" wetlands as they are totally human artifacts that are built to enhance storm water quality. Sometimes small wetlands that exist along ephemeral drainageways could be enlarged and incorporated into the constructed wetland system. Such action, however, requires the approval of federal and state regulators.

2. General Application

A constructed wetlands basin can be used as a follow-up structural BMP in a watershed, or as a stand-alone onsite facility if the owner provides sufficient water to sustain the wetland. Flood control storage can be provided above the constructed wetlands basin's WQCV pool to act as a multiuse facility.

A constructed wetlands basin requires a net influx of water to maintain its vegetation and microorganisms. A complete water budget analysis is necessary to assure the adequacy of the base flow.

The basic formula for the water budget is as follows:

(Equation 14)

$$\frac{\Delta S}{\Delta t} = Q_i - Q_o$$

Where:

$\frac{\Delta S}{\Delta t}$ = the change in storage volume per change in time Q_i = the flow rate of water entering the wetland, vol/time Q_O = the flow rate of water leaving the wetland, vol/time

Equation 15 translates into the following equations where all values are given in consistent units of volume per unit time unless otherwise specified:

(Equation 15)

For water entering a wetland, the formula is:

$Q_i = P + R_i + B_i + G_I$

Where:

P = Direct precipitation on impoundment area
R_I = Storm water runoff from contributing drainage area B_I
= Base flow entering the wetlands
G_I = Seepage and springs from ground water sources

For water leaving, the formula is:

(Equation 16)

$Q_O = E + T + R_O + B_O + G_O$

Where:

E = Evaporation from surface
T = Transpiration from plants
R_O = Storm water outflow

B_O = Base flow leaving the wetlands
G_O = Deep percolation below the root zone of the substrate

To ensure adequate base flow using the water budget analysis, the value of all variables should be determined and the net influx of water ($Q_I - Q_O$) must be greater than the change in storage volume divided by change in storage time.

3. General Properties

a. General

A constructed wetlands basin offers several potential advantages, such as natural aesthetic qualities, wildlife habitat, erosion control, and pollutant removal. It can also provide an effective follow-up treatment to onsite and source control BMPs that rely upon settling of larger sediment particles. In other words, it offers yet another effective structural BMP for larger tributary catchments.

The primary drawback of the constructed wetlands basin is the need for a continuous base flow to assure viable wetland growth. In addition, silt and scum can accumulate, and unless properly designed and built, can be flushed out during larger storms. In addition, in order to maintain a healthy wetland growth, the surcharge depth for WQCV above the permanent water surface cannot exceed 2 feet.

Along with routine good housekeeping maintenance, occasional cleaning will be required when sediment accumulations become too large and affect performance. Periodic sediment removal is also needed for proper distribution of growth zones and of water movement within the wetland.

b. Physical Site Suitability

A perennial base flow is needed to sustain a wetland, and should be determined using a water budget analysis. Loamy soils are needed in a wetland bottom to allow plants to take root. Exfiltration through a wetland bottom cannot be relied upon because the bottom is either covered by soils of low permeability or because the groundwater is higher than the wetland's bottom. Also, wetland basins require a near-zero longitudinal slope, which can be provided using embankments.

c. Pollutant Removal

See Table 11.4 (Appendix) for estimated ranges in pollutant removals. Reported removal efficiencies of constructed wetlands vary significantly. Primary variables influencing removal efficiencies include design, influent concentrations, hydrology, soils, climate, and maintenance. With periodic sediment removal and routine maintenance, removal efficiencies for sediments, organic matter, and metals can be moderate to high; for phosphorous, low to high; and for nitrogen, zero to moderate. Pollutants are removed primarily through sedimentation and entrapment, with some of the removal occurring through biological uptake by vegetation and microorganisms. Without a continuous dry weather base flow, salts and algae can concentrate in the water column and can be released into the receiving water in higher levels at the beginning of a storm event as they are washed out.

d. Design Considerations

Figure 11.61 (Appendix) illustrates an idealized constructed wetlands basin. An analysis of the water budget is needed to show the net inflow of water is sufficient to meet all the projected losses (such as evaporation, evapotranspiration, and seepage for each season of operation). Insufficient inflow can cause the wetland to become saline or to die off. Typical maintenance requirements for wetland BMPs include the items listed in Table 11.11 (Appendix).

4. Design Procedure and Criteria

The following steps outline the design procedure for a constructed wetlands basin.

a. Step 1: Basin Surcharge Storage Volume

Provide a surcharge storage volume equal to the WQCV based on a 24-hour drain time, above the lowest outlet (i.e., perforation) in the basin.

- Determine the WQCV using the tributary catchments percent imperviousness. Account for the effects of DCIA, if any, on effective imperviousness. Using Figure 11.55 (Appendix), determine the reduction in impervious area to use with WQCV calculations.

- Find the required storage surcharge volume (watershed inches of runoff) above the permanent pool level. Determine the required storage (watershed inches of runoff) using
Figure 11.48 (Appendix), based on the constructed wetland basin 24-hour drain time. Calculate the surcharge volume in acre-feet as follows: (Equation 17)

$$Design\ Surcharge\ Volume = \left[\frac{Water\ Quality\ Capture\ Volume}{12}\right] * Area$$

In which:

$Area$ = The drainage area tributary to the constructed wetlands basin (acres).

b. Step 2: Wetland Pond Depth and Volume

The volume of the permanent wetland pool shall be no less than 75 percent of the WQCV found in Step 1.

Proper distribution of wetland habitat is needed to establish a diverse ecology. Distribute pond area in accordance with Table 11.10 (Appendix).

c. Step 3: Depth of Surcharge

The surcharge depth of the WQCV above the permanent pool's WQCV water surface shall not exceed 2.0 feet.

d. Step 4: Outlet Control

Provide outlet controls that limit WQCV depth to 2 feet or less. Use a water quality outlet that is capable of releasing the WQCV in no less than a 24-hour period. Refer to Figures 11.62, 11.63, 11.64, and 11.65 (Appendix) for schematics pertaining to structure geometry; grates, trash racks, and screens; outlet type (orifice plate or perforated riser pipe); cutoff collar size and location; and all other necessary components.

Use Figure 11.66 (Appendix) to calculate the required area per row based on WQCV and the depth of perforations at the outlet. See Figures 11.53 and 11.54 (Appendix) for the appropriate perforation geometry and number of rows (the lowest perforations should be set at the water surface elevation of the outlet pool). The total outlet area can then be calculated by multiplying the area per row by the number of rows. Minimize the number of columns and maximize the perforation hole diameter when designing the outlet to reduce chances of clogging. Figure 11.50c contains typical outlet structure notes applicable to the design of outlet structures.

e. Step 5: Trash Rack

Provide a trash rack of sufficient size to prevent clogging of the primary water quality outlet. Size the rack so as not to interfere with the hydraulic capacity of the outlet. Using the total outlet area and the selected perforation diameter (or height), Figures 11.50a, 11.50b, 11.56, or 11.57 (Appendix) will help to determine the minimum open area required for the trash rack. If a perforated vertical plate or riser is used, use one half of the total outlet area to calculate the trash rack's size. This accounts for the variable inundation of the outlet orifices. Figures 11.50a, 11.50b, and 11.56 (Appendix) were developed as suggested standardized outlet designs for smaller sites.

f. Step 6: Basin Use

Determine if flood storage or other uses will be provided for above the wetland surcharge storage or in a separate facility. Design for combined uses when they are to be provided.

g. Step 7: Basin Shape

Shape the pond with a gradual expansion from the inflow and a gradual contraction to the outlet, thereby limiting short circuiting. The basin length to width ratio between the inflow area and outlet should be 2:1 to 4:1, with 3:1 recommended. It may be necessary to modify the inflow area and outlet point through the use of pipes, swales, or channels to accomplish this. Always maximize the distance between the inlet and outlet.

h. Step 8: Basin Side Slopes

Basin side slopes are to be stable and gentle to facilitate maintenance and access needs. Side slopes should be no steeper than 4:1.

i. Step 9: Base Flow

A net influx of water that exceeds all of the losses must be available throughout the year. The following equation and parameters can be used to estimate the net quantity of base flow available at a site: (Equation 18)

$$Q_{net} = Q_{Inflow} - Q_{Evap} - Q_{Seepage} - Q_{E.T.}$$

Where:

Q_{Net}	=	Net quantity of base flow (acre-feet/year)
Q_{Inflow}	=	Estimated base flow (acre-feet/year) (estimate by seasonal measurements and/or comparison to similar watersheds)
Q_{Evap}	=	Loss attributed to evaporation less the precipitation (acre-feet/year) (computed for average water surface)
$Q_{Seepage}$	=	Loss (or gain) attributed to seepage to groundwater (acre-feet/year)
$Q_{E.T.}$	=	Loss attributed to plant evapotranspiration (computed for average plant area above water surface, not including the water surface)

j. Step 10: Inflow Area and Outlet Protection

Provide a means to dissipate flow energy entering the basin to limit sediment resuspension. Outlets should be placed in an outlet bay that is at least 3 feet deep. The outlet should be protected from clogging by a skimmer shield that starts at the bottom of the permanent pool and extends above the maximum capture volume depth. Also provide for a trash rack.

k. Step 11: Forebay Design

Provide the opportunity for larger particles to settle out in an area that has a solid driving surface bottom for vehicles to facilitate sediment removal. The forebay volume of the permanent pool should be 5 to 10 percent of the design water quality capture volume.

l. Step 12: Vegetation

Cattails, sedges, reeds, and wetland grasses should be planted in the wetland bottom. Berms and side-sloping areas should be planted with native or irrigated turf-forming grasses. Initial establishment of the wetlands requires control of the water depth. After planting wetland species, the permanent pool should be kept at 3 to 4 inches to allow growth and to help establish the plants, after which the pool should be raised to its final operating level.

m. Step 13: Maintenance Access

Vehicle access to the forebay and outlet area must be provided for maintenance and removal of bottom sediments. Maximum grades should not exceed 10 percent, and a stabilized, all-weather driving surface capable for use by maintenance equipment shall be provided. If conditions warrant, a gravel or hard surface shall be provided.

5. Design Example

Design forms that document the design procedure are included in the Appendix. A completed form is shown in Figure 11.67 (Appendix) as a design example.

11.8.9 Retention Pond

1. Description

A retention pond is a sedimentation facility that has a permanent pool of water that is replaced with storm water, in part or in total, during storm water runoff events. In addition, a temporary detention volume is provided above this permanent pool to capture storm water runoff and enhance sedimentation. Retention ponds are similar to extended detention basins because they are designed to capture in total, as a surcharge to the pond, runoff from frequently occurring storms. However, retention ponds differ from extended detention basins because the influent water mixes with the permanent pool water as it rises above the permanent pool level. The surcharge captured volume above the permanent pool is then released over 12 hours.

Retention ponds require a dry-weather base flow to maintain the permanent pool. They can be very effective in removing pollutants, and, under the proper conditions, can satisfy multiple objectives.

2. General Application

A retention pond can be used to improve the quality of urban runoff from roads, parking lots, residential neighborhoods, commercial areas, and industrial sites and is generally used as regional or follow-up treatment because of the base flow requirements. It can be used as an onsite BMP if the owner provides sufficient water to keep the pond full between storms. A retention pond works well in conjunction with other BMPs, such as upstream onsite source controls and downstream filter basins or wetland channels.

3. General Properties

a. General

A retention pond provides the following benefits:

- Moderate to high removal rates of many urban pollutants.
- Wildlife habitat opportunities.
- Recreation, aesthetics, and open space opportunities.
- Part of a larger flood control basin.

Their primary drawbacks include safety concerns; more difficult maintenance and sediment removal than for extended detention basins; floating litter, scum, and algal blooms; possible nuisance odors; and possible mosquito problems. Aquatic plant growth can be a factor in clogging outlet controls. The permanent pool can attract water fowl, which can add to the nutrient load entering and leaving the pond.

b. Physical Site Suitability

Although site suitability concerns are similar to those stated for an extended detention basin, a retention pond has one primary difference—it requires sufficient continuous base flow to maintain the pool. A complete water budget under the projected urbanized watershed conditions should be performed to assure that the base flow will exceed evaporation, evapotranspiration, and seepage losses.

c. Pollutant Removal

See Table 11.4 (Appendix) for pollutant removal ranges. A retention pond achieves moderate to high removal rates for particulate matter through sedimentation during and shortly after the runoff event. During a storm event, part or all of the permanent pool water is displaced and the pool becomes a mixture of the former pool water and new runoff. The period between storms allows biological uptake of soluble nutrients and metals from the water column in the permanent pool while also providing time for quiescent settling of fine sediment particles that remain in the pool after a storm. Some of the sediment can resuspend and soluble compounds can remobilize if a large storm event causes intense mixing or when unfavorable chemical conditions exist in the pool (such as low dissolved oxygen [DO] or pH). Also, algal growth and other biological activity can produce suspended solids and increased concentrations of certain forms of phosphates and nitrogen compounds in dry-weather base flow discharges from the pond.

Without a sufficient continuous base flow, a wet pond can concentrate levels of salts and algae between storm events through evaporation. Besides contributing to nuisance problems, the water quality of the pool is very important. A storm event will displace any concentrated pond water, and in some instances, can result in discharges of water with pollutant concentrations exceeding the inflow—exactly the opposite of the intent for providing this BMP.

d. Aesthetics and Multiple Uses

A retention pond offers improved aesthetics and multiple uses beyond those typically found at an extended detention basin. The bulk of the capture volume occurs as a surcharge above the permanent pool, with some of it occurring above the dry-weather bank areas. As a result, most of the sediment deposits are left behind within the permanent pool zone, where they are not seen by the public. Also, the permanent pool offers some aquatic habitat and is a habitat for water fowl. However, water fowl can be a nuisance because of the fecal matter they deposit on the banks and in the pool. Maintenance requirements for the retention pond are similar to those for the extended detention basin found in Table 11.8 (Appendix).

4. **Design Considerations**

The required total basin design volume of a retention pond facility includes the volume needed for a permanent pool (greater than or equal to water quality capture volume) plus a water quality capture volume as a surcharge above the permanent pool. If desired, a flood routing detention volume can be provided above the water quality capture volume.

Whenever desirable and feasible, incorporate the retention pond within a larger flood control basin. Also, whenever possible, try to provide for other urban uses such as active or passive recreation and wildlife habitat. Try to locate recreational areas to limit the frequency of inundation to one or two occurrences a year. Generally, the area within the water quality capture volume is not well suited for active recreation facilities such as ballparks, playing fields, and picnic areas. These should be located above this pool level.

Land requirements are typically 0.5 to 2 percent of the tributary watershed's area. High exfiltration rates can initially make it difficult to maintain a permanent pool in a new retention pond, but the bottom can eventually seal with fine sediment and become relatively impermeable over time. It is best, however, to seal the bottom and the sides of a permanent pool if it is located on permeable soils and to leave the areas above the permanent pool unsealed to promote exfiltration of the storm water detained in the surcharge water quality capture volume.

There are two primary differences in design between a retention pond and an extended detention basin:

- The retention pond requires a base flow to maintain and to flush a permanent pool.
- A retention pond is designed to empty the surcharge water quality capture volume over a 12-hour period, instead of the longer
    - 40 hours needed for an extended detention basin because the sediment removal process is more efficient when the outflow occurs above the bottom of the basin. Sediments become trapped below the outlet and sedimentation continues in the pool after the captured surcharge volume is emptied.
    - Figure 11.68 (Appendix) shows a representative layout for a retention pond. Although flood storage has not been addressed in these recommendations for the same reasons mentioned under

extended detention basins, it can be easily provided for above the surcharge water quality capture volume. Embankment and safety design considerations for a retention pond are identical to those discussed for an extended detention basin, except more attention should be given to cutoff collars on the outlet pipe to safeguard against piping along the outlet.

The amount of construction activity within a basin, the erosion control measures implemented, and the size of the basin will influence the frequency of sediment removal from the pond. It is estimated that accumulated sediment will need to be removed at 5- to 20-year intervals if there are no construction activities within the tributary catchment.

5. Design Procedure and Criteria

The following steps outline the design procedure and criteria for a retention pond.

a. Step 1: Basin Surcharge Storage Volume

Provide a storage volume equal to the WQCV based on a 12-hour drain time, above the lowest outlet (i.e., perforation) in the basin.

- Determine the WQCV using the tributary catchment's percent imperviousness. Account for the effects of DCIA, if any, on effective imperviousness. Using Figure 11.55 (Appendix), determine the reduction in impervious area to use with WQCV calculations.
- Find the required storage surcharge volume (watershed inches of runoff). Determine the required water quality capture volume in watershed inches of runoff using Figure 11.48 (Appendix), based on the retention pond 12-hour drain time. The water quality capture volume is the surcharge volume above the permanent pool. Calculate the design surcharge volume in acre-feet as follows:

(Equation 19)

$$Design\ Surcharge\ Volume = \left[\frac{Water\ Quality\ Capture\ Volume_I}{12}\right] * Area$$

In which:

Water Quality Capture Volume = Water quality capture volume from Figure 11.48 in watershed inches

Area = The tributary catchment drainage area (acres).

b. **Step 2: Permanent Pool**

The permanent pool provides storm water quality enhancement between storm water runoff events through biochemical processes and continuing sedimentation.

- Volume of the permanent pool:

Permanent Pool 1.0 to 1.5 (WaterQualityCaptureVolume)

c. **Step 3: Depth Zones**

The permanent pool shall have two depth zones:

- A littoral zone 6 to 12 inches deep that is between 25 to
- 40 percent of the permanent pool surface area for aquatic plant growth along the perimeter of the permanent pool.
- A deeper zone of 4 to 8 feet average depth in the remaining pond area to promote sedimentation and nutrient uptake by phytoplankton. The maximum depth in the pond shall not exceed 12 feet.

d. **Step 4: Base Flow**

A net influx of water must be available through a perennial base flow and must exceed the losses. The following equation and parameters can be used to estimate the net quantity of base flow available at a site:

(Equation 20)

$$Q_{Net} = Q_{Inflow} - Q_{Evap} - Q_{Seepage} - Q_{E.T.}$$

In which:

Q_{net}	=	Net quantity of base flow (acre-feet/year)
Q_{inflow}	=	Estimated base flow (acre-feet/year) (estimate by seasonal measurements and/or comparison to similar watersheds)
Q_{evap}	=	Loss because of evaporation less the precipitation (acre-feet/year) (computed for average water surface)
$Q_{Seepage}$	=	Loss (or gain) because of seepage to groundwater (acre-feet/year)
$Q_{E.T.}$	=	Loss because of plant evapotranspiration (additional loss through plant area above water surface not including the water surface)

e. **Step 5: Outlet Control**

Provide outlet controls that limit WQCV depth to depths identified in Cahpter 11 Appendix. . Use a water quality outlet that is capable of releasing the WQCV in no less than a 12-hour period. Refer to Figures 11.62, 11.63, 11.64, and 11.65 (Appendix) for schematics pertaining to structure geometry; grates, trash racks, and screens; outlet type; orifice plate or perforated riser pipe; cutoff collar size and location; and all other necessary components.

Use Figure 11.69 (Appendix) to calculate the required area per row based on WQCV and the depth of perforations at the outlet. See Figures 11.53 and 11.54 (Appendix) to determine the appropriate perforation geometry and number of rows (the lowest perforations should be set at the water surface elevation of the outlet pool). The total outlet area can then be calculated by multiplying the area per row by the number of rows. Minimize the number of columns of perforations and maximize the diameter of perforation holes to reduce clogging potential. Figure 11.50c contains typical outlet structure notes applicable to the design of outlet structures.

f. **Step 6: Trash Rack**

Provide a trash rack of sufficient size to prevent clogging of the primary water quality outlet. Size the rack so as not to interfere with the hydraulic capacity of the outlet. Using the total outlet area and the selected perforation diameter (or height), Figures 11.50a, 11.50b, 11.56, or 11.57 (Appendix) will help to determine the minimum open area required for the trash rack. Use one half of the total perforated plate outlet area to calculate the trash rack's size. This accounts for the variable inundation of the outlet orifices. Figures 11.50a, 11.50b, and 11.56 (Appendix) were developed as suggested standardized outlet designs for smaller sites.

g. **Step 7: Basin Shape**

Shape the pond with a gradual expansion from the inflow area and a gradual contraction toward the outlet, thereby limiting short circuiting. The basin length to width ratio between the inflow area and outlet should be between 2:1 and 3:1. It may be necessary to modify the inflow area and outlet point through the use of pipes, swales, or channels to accomplish this. Always maximize the distance between the inlet and the outlet.

h. **Step 8: Basin Side Slopes**

Side slopes should be stable and sufficiently gentle to limit channel erosion and to facilitate maintenance. Side slopes above the permanent pool should be no steeper than 4:1. The littoral zone should be very flat (that is, 40:1 or flatter) with the depth ranging from 6 inches near the shore and extending to no more than 12 inches at the furthest point from the shore. The side slope below the littoral zone shall be 3:1 or flatter.

i. **Step 9: Dam Embankment**

The embankment should be designed not to fail during a 100-year or larger storm. Embankment slopes should be no steeper than 3:1, but preferably 4:1 or flatter, and covered with turf-forming grasses to limit erosion. Poorly compacted native soils should be removed and replaced. Embankment soils should be compacted to
95 percent of their maximum density according to ASTM D 698-70 (modified proctor).

j. **Step 10: Vegetation**

Vegetation provides erosion control and enhances site stability. Berms and side-sloping areas should be planted with native turf-forming grasses or irrigated turf, depending on the local setting and proposed uses for the pond area. The shallow littoral bench should have a 4- to 6-inch organic topsoil layer and be vegetated with aquatic species.

k. **Step 11: Maintenance Access**

Vehicle access to the basin bottom, forebay, and outlet area must be provided for maintenance and removal of bottom sediments. Maximum grades should not exceed 10 percent, and a stabilized, all-weather driving surface capable for use by maintenance equipment shall be provided. If conditions warrant, a gravel or hard surface shall be provided.

l. **Step 12: Inflow Area**

Dissipate flow energy at the inflow area to limit erosion and to diffuse the inflow plume where it enters the pond.

m. **Step 13: Forebay Design**

To provide an opportunity for larger particles to settle out, install an area that has a solid driving surface bottom to facilitate sediment removal. A berm consisting of a rock and topsoil mixture should be part of the littoral bench to create the forebay and have a minimum top width of 8 feet and side slopes no steeper than 4:1. The forebay volume of the permanent pool should be 5 to 10 percent of the design water quality capture volume.

n. **Step 14: Underdrains**

Provide underdrain trenches near the edge of the pond. The trenches should be no less than 12 inches wide and filled with ASTM C-33 sand to within 2 feet of the pond’s permanent pool water surface, and with an underdrain pipe connected through a valve to the outlet. These underdrains will permit the pond to be dried out when it has to be cleaned out to restore volume lost due to sediment deposition.

6. **Design Example**

Design forms that document the design procedure are included in the Appendix. A completed form is shown in Figure 11.70 (Appendix) as a design example.

11.8.10 Constructed Wetlands Channel

1. **Description**

Constructed wetland-bottomed channels take advantage of dense natural vegetation (rushes, willows, cattails, and reeds) to slow down runoff and allow time for settling out sediment and biological uptake.

Constructed wetlands differ from “natural” wetlands as they are artificial and are built to enhance storm water quality. Sometimes small wetlands that exist along ephemeral drainageways may be enlarged and incorporated into the constructed wetland system. Such action, however, requires the approval of federal and state regulators.

2. **General Application**

Wetland bottom channels can be used in the following two ways:

- A wetland can be established in a totally man-made channel and can act as a conveyance system and water quality enhancement facility. This design can be used along wide and gently sloping channels.
- A wetland bottom channel can be located downstream of a storm water detention facility (water quality and/or flood control) where a large portion of the sediment load can be removed. The wetland channel then receives storm water and base flows as they drain from the detention facility, provides water quality enhancement, and at the same time conveys it downstream. The application of a wetland channel is recommended upstream of receiving waters and within lesser (i.e., ephemeral) receiving waters, thereby delivering better quality water to the more significant receiving water system.

A constructed wetland channel requires a net influx of water to maintain its vegetation and microorganisms. A complete water budget analysis is necessary to assure the adequacy of the base flow.

3. General Properties

a. General

Constructed wetlands offer several potential advantages, such as natural aesthetic qualities, wildlife habitat, erosion control, and pollutant removal. Constructed wetlands provide an effective follow-up treatment to onsite and source control BMPs that rely upon settling of larger sediment particles.

The primary drawback to wetlands is the need for a continuous base flow to assure their presence. In addition, salts and scum can accumulate, and unless properly designed and built, can be flushed out during larger storms.

Other drawbacks include the need for regular maintenance to provide nutrient removal. Regular harvesting and removal of aquatic plants, cattails, and willows are required if the removal of nutrients in significant amounts has to be assured. Even with that, recent data puts into question the net effectiveness of wetlands in removing nitrogen compounds and some form of phosphates. Periodic sediment removal is also necessary to maintain the proper distribution of growth zones and of water movement within the wetland.

b. Physical Site Suitability

A perennial base flow is needed to sustain a wetland, and should be determined using a water budget analysis. Loamy soils are needed in wetland bottoms to allow plants to take root. Infiltration through a wetland bottom cannot be relied upon because the bottom is either covered by soils of low permeability or because the groundwater is higher than the wetland's bottom. Wetland bottom channels also require a near-zero longitudinal slope; drop structures are used to create and maintain a flat grade.

c. Pollutant Removal

Removal efficiencies of constructed wetlands vary significantly. Primary variables influencing removal efficiencies include design, influent concentrations, hydrology, soils, climate, and maintenance. With periodic sediment removal and plant harvesting, expected removal efficiencies for sediments, organic matter, and metals can be moderate to high; for phosphorous, low to moderate; and for nitrogen, zero to low. Pollutants are removed primarily through sedimentation and entrapment, with some of the removal occurring through biological uptake by vegetation and microorganisms. Without a continuous dry-weather base flow, salts and algae can concentrate in the water column and can be released into the receiving water in higher levels at the beginning of a storm event as they are washed out.

4. **Design Considerations**

Wetlands can be set into a drainageway to form a wetland bottom channel as shown in Figure 11.71 (Appendix). An analysis of the water budget is needed so that the inflow of water throughout the year is sufficient to meet all the projected losses (such as evaporation, evapotranspiration, and seepage) for satisfactory functioning of the wetland. An insufficient base flow could cause the wetland bottom channel to dry out and die. Maintenance requirements for wetland BMPs are shown in Table 11.11 (Appendix).

5. **Design Procedure and Criteria**

The following steps outline the constructed wetlands channel design procedure. Refer to Figure 11.71 (Appendix) for its design components.

a. **Step 1: Design Discharge**

Determine the two-year peak flow rate in the wetland channel without reducing it for any upstream ponding or flood routing effects. The channel shall also meet the conveyance requirements of Section 11.5.

b. **Step 2: Channel Geometry**

Define the newly-built channel's geometry to pass the design two-year flow rate at 2.0 feet per second with a channel depth between 2.0 and 4.0 feet. The channel cross section should be trapezoidal with side slopes of 4:1 (horizontal/vertical) or flatter. Bottom width shall be no less than 8.0 feet.

c. **Step 3: Longitudinal Slope**

Set the longitudinal slope using Manning's equation and a Manning's roughness coefficient of n=0.03, for the two-year flow rate. If the desired longitudinal slope cannot be satisfied with existing terrain, grade control checks or small drop structures must be incorporated to provide desired slope.

d. **Step 4: Final Channel Capacity**

Calculate the final (or mature) channel capacity during a two-year flood using a Manning's roughness coefficient of n = 0.08 and the same geometry and slope used when initially designing the channel with n = 0.03. The channel shall also provide enough capacity to contain the flow during a 100-year flood while maintaining 1 foot of free-board. Adjustment of the channel capacity may be done by increasing the bottom width of the channel. Minimum bottom width shall be 8 feet.

e. **Step 5: Drop Structures**

Drop structures should be designed considering low and high flow hydraulic conditions using standard engineering practices.

f. **Step 6: Vegetation**

Vegetate the channel bottom and side slopes to provide solid entrapment and biological nutrient uptake. Cover the channel bottom with loamy soils, upon which cattails, sedges, and reeds should be established. Side slopes should be planted with native or irrigated turf grasses.

g. **Step 7: Maintenance Access**

Vehicle access along the channel length must be provided for maintenance. Maximum grades should not exceed 10 percent, and a stabilized, all-weather driving surface capable for use by maintenance equipment shall be provided.

6. **Design Example**

Design forms that provide a means of documenting the design procedure are included in the Appendix. A completed form is shown in Figure 11.72 (Appendix) as a design example.

11.8.11 Water Quality Catch Basins and Water Quality Catch Basin Inserts

1. **Description**

A catch basin is an inlet to the storm drain system that typically includes a grate or curb inlet and a sump to capture sediment, debris, and associated pollutants. Catch basins act as pretreatment for other treatment practices by capturing large sediments. The performance of catch basins at removing sediment and other pollutants depends on the design of the catch basin (e.g., the size of the sump) and maintenance procedures to retain the storage available in the sump to capture sediment. Catch basin efficiency can be improved using inserts, which can be designed to remove oil and grease, trash, debris, and sediment. There are various manufacturers of water quality catch basins and the efficiency may vary with the manufacturer.

2. **Applicability**

Catch basins are used in drainage systems throughout the United States. Ideal application of catch basins is as pretreatment to another storm water management practice. Catch basin inserts for both new development and retrofits at existing sites may be preferred when available land is limited, as in urbanized areas.

3. Siting and Design Considerations

The performance of catch basins is related to the volume in the sump (i.e., the storage in the catch basin below the outlet). Lager et al. (1997) described an "optimal" catch basin sizing criterion, which relates all catch basin dimensions to the diameter of the outlet pipe (D):

- The diameter of the catch basin should be equal to 4D.
- The sump depth should be at least 4D. This depth should be increased if cleaning is infrequent or if the area draining to the catch basin has high sediment loads.
- The top of the outlet pipe should be 1.5D from the bottom of the inlet to the catch basin.

Catch basins can also be sized to accommodate the volume of sediment that enters the system. Pitt et al. (1997) proposed a sizing criterion based on the concentration of sediment in storm water runoff. The catch basin is sized, with a factor of safety, to accommodate the annual sediment load in the catch basin sump. This method is preferable where high sediment loads are anticipated, and where the optimal design described above is suspected to provide little treatment.

4. References

Aronson, G., D. Watson, and W. Pisaro. *Evaluation of Catch Basin Performance for Urban Storm Water Pollution Control*. U.S. Environmental Protection Agency, Washington, DC.

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Lager, J., W. Smith, R. Finn, and E. Finnemore. *Urban Storm Water Management and Technology: Update and Users' Guide*. Prepared for U.S. Environmental Protection Agency. EPA-600/8-77-014. 313 pp. 1977.

Mineart, P., and S. Singh. *Storm Inlet Pilot Study.* Alameda County Urban Runoff Clean Water Program, Oakland, CA. 1994.

Pitt, R., and P. Bissonnette. *Bellevue Urban Runoff Program Summary Report.* U.S. Environmental Protection Agency, Water Planning Division, Washington, DC. 1984.

Pitt, R., M. Lilburn, S. Nix, S.R. Durrans, S. Burian, J. Voorhees, and J. Martinson. *Guidance Manual for Integrated Wet Weather Flow (WWF) Collection and Treatment Systems for Newly Urbanized Areas (New WWF Systems)*. U.S. Environmental Protection Agency, Office of Research and Development, Cincinnati, OH. 2000.

11.8.12 Bioretention

1. **Definition**

   A typical bioretention area is shown in Figure 11.73 (Appendix). Two general types of bioretention facilities exist: off-line and on-line areas. Off-line bioretention areas consist of sand and soil mixtures planted with native plants, which receive runoff from overland flow or from a diversion structure in a traditional drainage system. On-line bioretention areas have the same composition as off-line areas, but are located in grass swales or other conveyance systems that have been modified to enhance pollutant removal by quiescent settling and biofiltration.

2. **Purpose**

   Bioretention is an efficient method for removing a wide variety of pollutants, such as suspended solids and nutrients. It can also be an effective means of reducing peak runoff rates and recharging groundwater by infiltrating runoff. However, not all bioretention facilities will necessarily be optimized for all of these functions.

3. **Application**

   Bioretention areas consisting of sand and soil mixtures planted with native plants, which filter urban runoff, can be used in residential and nonresidential developments. Sources of runoff can be overland flow from impervious areas or discharge diverted from a drainage pipe. Also, on-line bioretention facilities use check dams or other barriers to retain flow in grass swales.

   Bioretention facilities are most effective if they receive runoff as close as possible to the source. A site designer needs to look for opportunities to incorporate bioretention facilities throughout the site and minimize the use of inlets, pipes, and downstream controls.

   Bioretention should not be used in areas with the following characteristics:

- The water table is within 6 feet of the land surface (the use of collector pipes may reduce this limitation).
- Mature trees would be removed for constructing the bioretention area.
- Slopes are 20 percent or greater.
- An unstable soil stratum is in the area.

   a. **Off-Line**

      Off-line bioretention facilities can be applied to most development situations. They are particularly applicable in urban areas where the opportunities and the land available for controlling storm water reliably are scarce. Bioretention facilities may be installed in median strips, parking lot islands, or lawn areas of commercial developments. They also can be used in residential subdivisions with open drainage systems or in easements located around lots. Figure 11.74 (Appendix) shows a bioretention area receiving runoff diverted from a storm sewer.

b. On-Line

On-line bioretention facilities use check dams to "collect" the water in the bioretention area, as shown in Figure 11.75 (Appendix). Adding a bioretention area, as shown in Figure 11.75 (Appendix). Add a bioretention area behind the check dam allows filtering and sedimentation to occur. Check dams should only be used in small open channels or in filter strips that drain five acres or less. Runoff from storms larger than water quality design storm should safely flow over or bypass the bioretention area.

4. Recommended Design Criteria

a. Performance-Based Guidelines

Bioretention facilities should be optimized to treat the runoff generated by the water quality design storm. The peak discharge from larger storms should be bypassed, if possible.

A homogenous soil mix of 50 percent construction sand; 20 to 30 percent topsoil with less than 5 percent clay content, and 20 to 30 percent organic compost containing no animal waste provides a planting medium with adequate infiltration capacity. Soil amendments can be added according to the plant species selected. This soil guidance is taken from the North Carolina BMP manual.

In areas where clay contents are higher and the soil is not conducive to infiltration, the bioretention facility can be modified with a collector pipe system installed beneath the basin to form a bioretention filter. The city of Alexandria, Virginia, has developed design guidelines for bioretention filters (city of Alexandria, 1995) and collector pipes for areas of clay soil. As a standard practice, a collector pipe system is now used on all bioretention applications.

Bioretention areas can be used successfully in a wide range of drainage areas. Median strips, ramp loops, and parking lot islands are examples of small drainage areas (less than one acre). In large drainage areas (less than ten acres), diversion structures and energy dissipation devices need to be incorporated into the design to preserve the integrity of the bioretention area.

It is recommended that the size of the bioretention area be 5 to 7 percent of the drainage area multiplied by the *c* coefficient of the Rational Formula (Prince George's County, 1993). However, both smaller and larger ranges are allowed. Ongoing monitoring data will provide better guidance on the design of these facilities. The land required for bioretention facilities can be reduced by partially substituting vertical-extended detention storage for horizontal storage.

Check dams, as shown in Figure 11.75 (Appendix), reduce the velocity of concentrated storm water flows, promoting sedimentation behind the dam. If properly anchored, railroad ties, gabions, or rock filter berms may be used as check dams. The use of railroad ties is shown in Figure 11.76 (Appendix). The use of gabions as a drop structure is shown in Figure 11.77 (Appendix). These types of structures can be used in swales with moderate slopes.

Check dams must be sized and constructed correctly and maintained properly, or they will be either washed out or contribute to flooding. The relationship between ponding depth and discharge rate can be computed by using the critical-depth formula, which accounts for a generalized weir profile. The relevant equation is:

(Equation 21)

$$Q = \left((A^3 \text{ x } g) / T \right)^{1/2}$$

Where:

Q	=discharge rate
A	=area subtended by top of check dam and ponding elevation
T	=width of check dam
g	=gravitational constant

Check dams can be constructed of either rock or logs. The use of other natural materials available on the site that can withstand the storm water flow velocities is acceptable. Check dams should not be constructed from straw bales or silt fences because concentrated flows quickly wash out these materials.

Maximum velocity reduction is achieved if the toe of the upstream check dam is at the same elevation as the top of the downstream dam. The center section of the dam should be lower than the edge sections to minimize the potential for erosion of the abutments during frequently occurring storm events.

b. Operation and Maintenance

Monthly inspections are recommended until the plants are established. Annual inspections should then be performed.

Accumulated sediment behind check dams should be removed when it reaches one half of the sump depth.

c. Considerations

If used, collector pipe systems in bioretention areas can become clogged by underlying clay soil. Pipe cleanouts are recommended to facilitate unclogging of the pipes without disturbing the bioretention areas.

5. Specifications and Methodology

a. Planting Plan

Using plants in bioretention areas is intended to replicate a terrestrial forest community ecosystem. The components of this community include trees, a shrub layer, and a herbaceous layer. Native plants should be able to tolerate typical storm water pollutant loads, variable soil moisture, and ponding fluctuations (Prince George's County, 1993). Designers are encouraged to check other sources, such as The Agronomy Guide, the Field Office Technical Guide, and local nurseries, to identify plants that can adapt to specific site conditions.

The plant material layout should resemble a random and natural placement of plants rather than a standard landscaped approach with trees and shrubs in rows or other orderly fashion. The location of the plant material should provide optimal conditions for plant establishment and growth (Prince George's County, 1993).

b. Off-Line Bioretention Areas

There are six major components to the bioretention area:

- Grass buffer strip or energy dissipation area
- Ponding or treatment area
- Planting soil
- Sand bed (optional)
- Organic layer
- Plant material

The grass buffer strip or energy-dissipation area filters particles from the runoff and reduces its velocity. The sand bed further slows the velocity of the runoff, spreads the runoff over the basin, filters part of the water, provides positive drainage to prevent anaerobic conditions in the planting soil, and enhances exfiltration from the basin.

The ponding area functions as storage area for runoff awaiting treatment and as presettling basin for particulates that have not been filtered out by the grass buffer. The organic or mulch layer acts as a filter for pollutants, protects the soil from eroding, and is an environment for microorganisms to degrade petroleum-based compounds and other pollutants.

The planting soil layer nurtures the plants with stored water and nutrients. Clay particles in the soil adsorb heavy metals, nutrients, hydrocarbons, and other pollutants. The plant species are selected on the basis of their documented ability to cycle and assimilate nutrients, pollutants, and metals through the interaction among plants, soil, and organic layers (Bitter and Bowers, 1994). The minimum depth of the planting soil layer should be 3 to 4 feet.

The number of tree and shrub plantings may vary, especially in areas where aesthetics and visibility are vital to site development, and the density should be determined on an individual site basis. The minimum and maximum number of individual plants and spacing recommended are shown in Table 11.12 (Appendix). A minimum of three species of trees and three species of shrubs should be selected to assure diversity.

As with any BMP, sizing rules are continually changing. Although the site requirements will determine the actual dimensions, the following dimensions are recommended for bioretention areas:

- Minimum width is 10 to 15 feet.
- Minimum length is 30 to 40 feet.
- The ponded area should have a maximum depth of 6 inches. If collector pipes are used, the maximum pond depth can be increased to 12 inches.
- The planting soil should have a minimum depth of 4 feet.

Figures 11.78 and 11.79 (Appendix) show a profile and plan of a typical bioretention area. A curb diversion structure that can be installed to divert gutter flow to a bioretention area is shown in Figure 11.80 (Appendix).

c. On-Line Bioretention Areas

A bioretention area upstream of a check dam is constructed with similar specifications as the off-line bioretention areas. The depth of the planting soil zone can be reduced (1 to 2 feet) if the drainage area is small (less than two acres).

Rock check dams usually are constructed of approximately 8- to 12-inch rock. The rock is placed either by hand or mechanically, but never just dumped into the swale. The dam must completely span the ditch or swale to prevent being washed out. The rock used must be large enough to stay in place, given the expected design flow through the channel.

Railroad tie check dams are illustrated in Figure 11.76 (Appendix). The railroad ties should be embedded into the soil at least 18 inches. Gabion applications are illustrated in Figures 11.75 and 11.77 (Appendix).

6. Design Methodology for Controlling Runoff Volume

The runoff capture volume is the minimum volume of rainfall that must be retained and completely infiltrated onsite during every storm. It is also equal to the rainfall quantity associated with the runoff capture design storm.

The runoff capture volume is conveniently stated as a rainfall depth, in inches, over the area of the site. To achieve a suitable level of groundwater recharge, a minimum of 0.73 inch of rainfall from every storm should be detained and infiltrated. All rainfall events with less than 0.73 inch of rainfall should be completely infiltrated.

Analysis of the site will establish the total runoff capture storage that must be provided by infiltration BMPs at a particular site. In general, the retention volume of appropriately located bioretention areas can be applied to satisfy the runoff capture storage requirement for the site.

Bioretention facilities are effective measures for increasing the runoff capture capability of the site. Other methods that can be used to improve runoff capture and infiltration include:

- Installing permeable pavement.
- Installing infiltration trenches or dry wells.
- Modifying the site design to decrease imperviousness.

7. Design Methodology for Runoff Peak Attenuation

Only bioretention facilities with large retention storage capacities will be effective in controlling runoff peak discharge rates. To predict a change in peak runoff, the Natural Resources (formerly Soil) Conservation Service's (NRCS) methodology can be used. This methodology includes the so-called soil cover complex and nondimensionalized unit hydrograph techniques and is implemented in a variety of computer simulation packages. Alternative methodologies, including kinematic wave runoff routing and synthetic unit hydrograph generation, also are available in various computer software packages.

By retaining runoff during the initial stages of a storm, bioretention facilities can significantly reduce peak runoff rates. With these measures implemented, runoff from the site will be delayed until the storage capacity of the facilities is exceeded. When using the NRCS methodology, this effect can be accounted for as an increase in the initial abstraction, *Ia*, for the drainage subarea in which the facility is located. The relationship can be expressed as follows:

$$R = \frac{(V - Ia)^2}{(V - Ia) + S} \qquad \text{(Equation 22)}$$

$$S = \frac{1{,}000}{CN} - 10 \qquad \text{(Equation 23)}$$

Where:

V = Rainfall volume (inches, over the drainage area)
R = Runoff volume (inches, over the drainage area)
Ia = Initial abstraction (inches, over the drainage area)
S = Potential maximum retention after runoff begins (inches, over the drainage area)
CN = NRCS runoff curve number

Ia can be approximated as the combined runoff capture storage divided by the surface area of the drainage subarea. The effect will be more important for small runoff peak attenuation design storms.

Some bioretention facilities may also include peak attenuation storage. The impacts of large flows and velocities on the plant material need to be carefully evaluated before using bioretention facilities as peak attenuation facilities. Bioretention facilities with small drainage areas (i.e., less than 0.25 acre) may be effective for peak attenuation if they are installed throughout a subdivision or nonresidential development.

8. References

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11.8.13 Other BMPs

Use of BMPs other than those listed in this manual may be allowed when approved by the City Engineer on a case by case basis.

11.8.14 Acknowledgement

The descriptions of the BMPs contained in this chapter were adapted from descriptions of BMPs found in the Urban Drainage and Flood Control District (Colorado) Drainage Criteria Manual Volume 3 and the Pennsylvania Handbook of Best Management Practices for Developing Areas.

Original spreadsheets and design forms for Best Management Practices were developed for the Urban Drainage and Flood Control District of Denver, Colorado, and were modified by the City of Sioux Falls to fit local conditions and policies.

11.9 Reference Material

Hastead Quick TR-55.