City of Plymouth
Stormwater Technical Standards Manual

Version 1.2 (July 2026)



Developed by:
Lochmueller Group



Summary of Revisions

Version No.	Date	List of Major Revisions
1.0	June 2025	Original Version, Based on LTAP's Model Stormwater Technical Standards Manual (Updated October 2022)
1.1	July 2025	Corrected Table of Contents and Standard Forms
1.2	July 2026	Updated SWPPP and Drainage Checklists

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Chapter 1 | Introduction and Administration

A. Introduction

This Stormwater Technical Standards Manual ("Manual"), prepared by Lochmueller Group, contains the necessary technical standards for administering the requirements of IDEM MS4 General Permit (GP) and the City of Plymouth Stormwater Management Ordinance ("Ordinance"). This document should be considered as a companion document to the Ordinance. Whereas the Ordinance contains the majority of the regulatory authority and general requirements of comprehensive stormwater management, this document contains the necessary means and methods for achieving compliance with the Ordinance. It is not intended as a regulatory document, but rather guidance to assist plan reviewers, developers, and designers. In the event of conflicts between the requirements contained in this document and the Ordinance, the requirements of the Ordinance should prevail.

The Ordinance and this Technical Standards Manual provide the general requirements and recommended / acceptable methods for design and review of stormwater quantity and quality infrastructure within the City of Plymouth. For a comprehensive technical guidance on calculations and modeling of stormwater quantity and quality infrastructure in Indiana, refer to the latest edition of the "Indiana LTAP Stormwater Drainage Manual". For guidelines and specific stormwater quality measures for controlling soil erosion; controlling and treating the nonpoint source pollution associated with sediment-laden run-off; and the management and treatment of pollutants associated with post-construction land uses, refer to the Indiana Storm Water Quality Manual. In addition to a variety of storm water quality measures, the Indiana Storm Water Quality Manual also discusses the philosophy and planning procedures critical to developing an effective storm water pollution prevention plan.

B. Administration

1. Purpose

   Smaller streams and drainage channels serving the City of Plymouth may not have sufficient capacity to receive and convey stormwater runoff, resulting when land use changes from open or agricultural use to a more urbanized use; and the deposit of sediment from developments during and after construction can reduce capacities of storm sewers and drainage systems and result in damages to receiving lakes and streams; therefore, the City of Plymouth adopts this Stormwater Technical Standards Manual (and the companion Stormwater Management Ordinance) with the purpose of controlling urban runoff and minimizing urban flooding. This Manual and the companion City of Plymouth Stormwater Management Ordinance shall supersede any previous versions.

2. Stormwater Control Policy

   The storage and controlled release of excess stormwater runoff shall be required for all new business, institutional developments, commercial and industrial developments, residential subdivisions, planned development, rural estate

subdivisions, and any redevelopment or other new construction located within the City of Plymouth. All aforementioned projects shall be submitted to the City of Plymouth, through its Stormwater Management Permit process, for evaluation. The City of Plymouth, after thorough investigation and evaluation, may waive the requirement of controlled runoff for minor subdivisions and parcelization.

For the convenience of the user, the following Stormwater Management Permit information is provided below:

Stormwater Management Permit requirements for projects within the City of Plymouth:

i. 0 – 10,000 square feet (Ft^2) in size
    1. No permit needed.
ii. 10,000 square feet (Ft^2) - less than one (1) acre in size
    1. shall submit a Drainage Plan Application
iii. One (1) acre or more in size
    1. shall submit a Drainage Plan Application and a SWPPP Application
    2. Are regulated under the IDEM Construction Stormwater General Permit (CSGP, formerly "Rule 5"); Sections 4 and 5 of the Ordinance, as well as Chapters 6 and 7 of this Manual, are of particular note.

The City of Plymouth has the authority to modify, grant exemptions, and / or waive any and all the requirements of the Ordinance and this Manual. A pre-submittal meeting with the City of Plymouth may be requested by the applicant to discuss the applicability of various provisions of the Ordinance and this Manual.

3. When Effective
   This Manual shall become effective after its publication on the City of Plymouth's Stormwater Department's webpage.

4. Disclaimer of Liability
   The degree of protection required by this Manual is considered reasonable for regulatory purposes and is based on historical records, engineering, and scientific methods of study. Larger storms may occur, or storm water runoff depths may be increased by man-made or natural causes. This Manual does not imply that land uses permitted will be free from stormwater damage. This manual shall not create liability on the part of the City of Plymouth, the Plymouth Board of Works and Safety (BOWS), the City Sewer Superintendent, the City Engineer, or any officer or employee thereof for any damage which may result from reliance on this Manual or on any administrative decision lawfully made thereunder.

5. Corrective Action
   Nothing herein contained shall prevent the City of Plymouth, the Plymouth BOWS, the City Sewer Superintendent, or the City Engineer from taking such other lawful action as may be necessary, including referring the violator to the appropriate legal authority for criminal prosecution, to prevent or remedy any violation. Refer to the

Ordinance, and the Enforcement Matrix in **Appendix B**, for more information related to penalties for violations.

6. Connection to a County Regulated Drain
   I.C. 36-9-27-17 requires that a new drain connecting into a regulated drain under the jurisdiction of the Marshall County Drainage Board must have the approval of the Marshall County Surveyor.

C. Information Requirements

The following information and data are required to be provided by an Indiana licensed professional engineer, architect, or land surveyor engaged in storm drainage design.

1. Topographic and Soils Maps
   A soils map of the proposed development indicating soils names and their hydrologic classification must be provided when Soil Conservation Service (SCS) hydrologic methods are used. In addition, a topographic map of the land to be subdivided and such adjoining land whose topography may affect the layout or drainage of the development must be provided.

   The contour intervals shall be one (1) foot when slopes are less than 4% and shall be two (2) feet when the slope exceeds 2%. On this map, the following shall be shown:
   - The location of streams and other flood water runoff channels, the extent of the floodplains at the established 100-year flood elevation where available (regulatory floodway), and the limits of the floodway, all properly identified.
   - The normal shoreline of lakes, ponds, swamps and detention basins, their floodplains, and lines of inflow and outflow if any.
   - The location of regulated drains, farm drains, inlets and out-falls, if any of record.
   - Storm, sanitary, and combined sewers, and outfalls, if any of record.
   - Septic tank systems and outlets, if any of record.
   - Seeps, springs, flowing and other wells, that are visible or of record.

2. Preliminary Drainage Plan
   A comprehensive plan, in preliminary form (or in combined preliminary and final form), designed to handle safely the stormwater runoff and to retain / detain the increased stormwater runoff, must be provided.
   The plan shall provide or be accompanied by maps or other descriptive materials indicating the feasibility of the drainage plan and showing the following:
   - The extent and area of each watershed affecting the design of retention / detention facilities as shown on USGS Quadrangle Maps or other more detailed maps as required.
   - The preliminary layout and design of proposed storm sewers, the outfall and outlet locations and approximate elevations, the receiving stream or channel and its 100-year return period water elevation.

- The location and design of the proposed street system, especially including depressed pavements used to convey or temporarily store overflow from the heavier rainstorms, and the outlets for such overflow.
- The locations, cross sections, and profiles of existing streams and floodplains to be maintained, and new channels to be constructed.
- The materials, elevations, waterway openings, and the basis for design of proposed culverts and bridges.
- Existing ponds and basins to be maintained, enlarged, or otherwise altered and new ponds or basins to be built and the basis of their design.
- The estimated depth and amount of storage required in the new ponds or basins.
- The estimated location and percentage of impervious surfaces existing and expected to be constructed when the development is completed.
- Any interim plan which is to be incorporated into the development pending completion of the development and the final drainage plan.

3. Valley, Swale Cross Section
   One or more typical cross sections must be provided showing all existing and proposed channels or other open drainage facilities carried to a point above the 100-year high water elevation; showing the elevation of the existing land and the proposed changes thereto, together with the high water elevations expected from the 100-year storm under the controlled conditions called for by this Technical Standards Manual; and showing the relationship of structures, streets, and other facilities.

4. Site Plan
   A plan drawn to scale showing dimensions of the site with existing and proposed storm drainage facilities must be provided.

5. Final Drainage Plans
   The final plans shall provide or be accompanied by calculations, maps and / or other descriptive material showing the following:
   - The extent and area of each watershed tributary to the drainage channels in the development.
   - The street storm sewers and other storm drains to be built, the basis of their design, outfall and outlet locations and elevations, the receiving stream or channel and its high water elevation, and the functioning of the drains during high water conditions.
   - The parts of the proposed street system where pavements are planned to be depressed sufficiently to convey or temporarily store overflow from storm sewers and over the curb runoff resulting from the heavier rainstorms and the outlets for such overflow.
   - Existing streams and floodplains to be maintained, and new channels to be constructed, their locations, cross sections and profiles.
   - Proposed culverts and bridges to be built, their materials, elevations, waterway openings and basis of their design.

- Existing storage basins and ponds to be maintained, enlarged, or otherwise altered and new basins or ponds to be built and the basis of their design.
- The estimated location and percentage of impervious surfaces existing and expected to be constructed when the development is completed.
- The slope, type and size of all sewers and other waterways.
- For all storage basins, a plot or tabulation of storage volumes with corresponding water surface elevations and a plot or tabulation of the basin outflow rates for those water surface elevations.

6. Report

   A report shall be submitted with the preliminary or final drainage plan that includes the following information:
   - A description of the proposed development;
   - The current land use conditions;
   - The method of hydraulic and hydrologic analysis used, including any assumptions or special condition;
   - The results of the analysis; and
   - The recommended drainage control facilities.
   - Hydraulic and hydrologic calculations, including input and output files, shall be included as appendices to the report.

7. Permission to Connect to Legal Drains

   If plans include connection to a legal drain, written permission from the County Surveyor and / or the Marshall County Drainage Board shall be submitted with the preliminary or final drainage plan.

Chapter 2 | Methodology for Determination of Runoff Rates

Runoff rates shall be computed for the area of the parcel under development plus the area of the watershed flowing into the parcel under development (contributing drainage areas).

In addition, the release rate of stormwater from developed lands shall not exceed the release rate from the land area in its present land use. The owner, developer, or builder must submit detailed computations of runoff before and after development, redevelopment, or new construction which demonstrate that runoff will not be increased. These computations must show that the peak runoff rate after development for the 100-year return period storm of 24-hour duration must not exceed the 10-year return period pre-development peak runoff rate, calculated for a duration equal to the time of concentration (15 minutes minimum). (The critical duration storm is that storm duration that requires the greatest detention storage.)

Because topography and the availability and adequacy of outlets for storm runoff vary with almost every site, the requirements for storm drainage tend to be an individual matter for any project. It is recommended that each proposed project be discussed with the City Sewer Superintendent and the City Engineer, at the earliest practical time in the planning stage. The City of Plymouth Stormwater Management Permit application has a selection option for a pre-submittal meeting with the City of Plymouth's stormwater reviewer.

The rate of runoff which is generated as the result of a given rainfall intensity may be calculated as follows:

A. Development Sites, Including Contributing Drainage Areas, Less Than or Equal to 50 Acres In Size

The Rational Method may be used. In the Rational Method, the peak rate of runoff, Q, in cubic feet per second (cfs, Ft^3/s) is computed as:

$$\mathbf{Q = C\ I\ A}$$

Where:

C = Runoff coefficient, representing the characteristics of the drainage area and defined as the ratio of runoff to rainfall.

I = Average intensity of rainfall in inches per hour (in/hr) for a duration equal to the time of concentration (t_c) for a selected rainfall frequency.

A = Tributary drainage area in acres (ac).

Values for the runoff coefficient "C" are provided in **Tables 2-1** and **2-2**, which show values for different types of surfaces and local soil characteristics. The composite "C" value used for a given drainage area with various surface types shall be the weighted average value for the total area calculated from a breakdown of individual areas having different surface types. **Table 2-3** provides runoff coefficients and inlet times for different land use classifications.

For consistency amongst applicants, rainfall intensity shall be determined from the rainfall frequency data shown in **Table 2-4**.

In general, the time of concentration (t_c) methodology to be used for all stormwater management projects within the City of Plymouth shall be as outlined in the U.S. Department of Agriculture (USDA) - NRCS TR-55 Manual. In urban or developed areas, the methodology to be used shall be the sum of the inlet time and flow time in the stormwater facility from the most remote part of the drainage area to the point under consideration. The flow time in the storm sewers may be estimated by the distance in feet divided by velocity of flow in feet per second (fps, Ft/s). The velocity shall be determined by the Manning's Equation (see Chapter 4). Inlet time is the combined time required for the runoff to reach the inlet of the storm sewer. It includes overland flow time and flow time through established surface drainage channels such as swales, ditches, and sheet flow across such areas as lawns, fields, and other graded surfaces.

Table 2-1

Urban Runoff Coefficients				
Type of Surface	Runoff Coefficient "C" (by Storm Recurrence Interval)			
	< 25-year	25-year	50-year	100-year
Hard Surfaces				
Asphalt	0.82	0.90	1.00	1.00
Gravel	0.85	0.94	1.00	1.00
Concrete	0.85	0.94	1.00	1.00
Roof	0.85	0.94	1.00	1.00
Lawns (Sandy)				
Flat (0-2% Slope)	0.07	0.08	0.09	0.12
Rolling (2-7% Slope)	0.12	0.13	0.16	0.20
Steep (Greater than 7% Slope)	0.17	0.19	0.22	0.28
Lawns (Clay)				
Flat (0-2% Slope)	0.16	0.18	0.21	0.26
Rolling (2-7% Slope)	0.21	0.23	0.28	0.35
Steep (Greater than 7% Slope)	0.30	0.33	0.40	0.50

Source: HERPICC Stormwater Drainage Manual, July 1995, and other sources.

Table 2-2

Rural Runoff Coefficients				
Type of Surface	Runoff Coefficient "C" (by Storm Recurrence Interval)			
	< 25-year	25-year	50-year	100-year
Woodland (Sandy)				
Flat (0-2% Slope)	0.10	0.11	0.13	0.17
Rolling (2-7% Slope)	0.25	0.28	0.33	0.41
Steep (Greater than 7% Slope)	0.30	0.33	0.40	0.50
Woodland (Clay)				
Flat (0-2% Slope)	0.30	0.33	0.40	0.50
Rolling (2-7% Slope)	0.35	0.39	0.46	0.58
Steep (Greater than 7% Slope)	0.50	0.55	0.66	0.83
Pasture (Sandy)				
Flat (0-2% Slope)	0.10	0.11	0.13	0.17
Rolling (2-7% Slope)	0.16	0.18	0.21	0.26
Steep (Greater than 7% Slope)	0.22	0.24	0.29	0.36
Pasture (Clay)				
Flat (0-2% Slope)	0.30	0.33	0.40	0.50
Rolling (2-7% Slope)	0.36	0.40	0.48	0.59
Steep (Greater than 7% Slope)	0.42	0.46	0.55	0.69
Cultivated (Sandy)				
Flat (0-2% Slope)	0.30	0.33	0.40	0.50
Rolling (2-7% Slope)	0.40	0.44	0.53	0.66
Steep (Greater than 7% Slope)	0.52	0.57	0.69	0.86
Cultivated (Clay)				
Flat (0-2% Slope)	0.50	0.55	0.66	0.83
Rolling (2-7% Slope)	0.60	0.66	0.79	0.99
Steep (Greater than 7% Slope)	0.72	0.79	0.95	1.00

Source: HERPICC Stormwater Drainage Manual, July 1995, and other sources.

Table 2-3

Runoff Coefficients by Land Use, Typical Inlet Times, and Storm Recurrence Interval													
Land Use	Runoff Coefficients "C" (by Storm Recurrence Interval)												Inlet Time (Minutes)[4]
	Flat[1]				Rolling[2]				Steep[3]				
	<25-yr	*25-yr*	*50-yr*	*100-yr*	*<25-yr*	*25-yr*	*50-yr*	*100-yr*	*<25-yr*	*25-yr*	*50-yr*	*100-yr*	
Commercial *(CBD)*	0.75	0.83	0.99	1.00	0.83	0.91	1.00	1.00	0.91	1.00	1.00	1.00	5
Commercial *(Neighborhood)*	0.54	0.59	0.71	0.89	0.60	0.66	0.79	0.99	0.66	0.73	0.87	1.00	5 - 10
Industrial	0.63	0.69	0.83	1.00	0.70	0.77	0.92	1.00	0.77	0.85	1.00	1.00	
Garden Apartments	0.54	0.59	0.71	0.89	0.60	0.66	0.79	0.99	0.66	0.73	0.87	1.00	
Churches	0.54	0.59	0.71	0.89	0.60	0.66	0.79	0.99	0.66	0.73	0.87	1.00	
Schools	0.31	0.34	0.41	0.51	0.35	0.39	0.46	0.58	0.39	0.43	0.51	0.64	10 - 15
Semi - Detached Residential	0.45	0.50	0.59	0.74	0.50	0.55	0.66	0.83	0.55	0.61	0.73	0.91	
Detached Residential	0.40	0.44	0.53	0.66	0.45	0.50	0.59	0.74	0.50	0.55	0.66	0.83	
Quarter Acre Lots	0.36	0.40	0.48	0.59	0.40	0.44	0.53	0.66	0.44	0.48	0.58	0.73	
Half Acre Lots	0.31	0.34	0.41	0.51	0.35	0.39	0.46	0.58	0.39	0.43	0.51	0.64	
Parkland	0.18	0.20	0.24	0.30	0.20	0.22	0.26	0.33	0.22	0.24	0.29	0.36	To Be Computed

Source: HERPICC Stormwater Drainage Manual, July 1995, and other sources.

1. Flat terrain involves slopes of 0-2%.
2. Rolling terrain involves slopes of 2-7%.
3. Steep terrain involves slopes greater than 7%.
4. Interpolation, extrapolation and adjustment for local conditions shall be based on engineering experience and judgment.

B. Development Sites, Including Contributing Drainage Area, Greater Than 50 Acres.

A computer model, such as TR-55 (NRCS), TR-20 (NRCS), HEC-HMS (COE), and HEC-1 (COE), that can generate hydrographs based on the NRCS TR-55 time of concentration and curve number (CN) calculation methodologies shall be used along with a 24-hour duration NRCS Type 2 storm. Note that for the purpose of determining the post-developed conditions CNs, due to significant disturbance to the upper soil layers during the construction activities, the initially determined hydrologic soil group for disturbed areas should be changed to the next less infiltrating capacity category (i.e., A to B, B to C, and C to D).

1. The 24-hour NRCS Type 2 storm distribution is used throughout this document as it is believed to be the most appropriate for design of infrastructure in preference to Huff and Atlas 2 rainfall distributions. The NRCS Type 2 distribution is a nested distribution that eliminates the need for critical duration analysis (thus helps in minimizing mistakes) and appropriate for larger events such as the 10% and 1% annual exceedance chance events at a time when our climate is changing, and distributions derived from historically observed events may not be relevant to future extreme events.

The 24-hour NRCS Type 2 Rainfall Distribution shall be utilized for runoff calculations; 24-hour Rainfall depth for various frequencies shall be taken from **Table 2-5**.

The NRCS Type 2 distribution ordinates are found in **Table 2-6**. This table should be used only when the rainfall distribution is not a default option in the computer program. Examples of computer models that can generate such hydrographs include TR-55 (NRCS), TR-20 (NRCS), and HEC-HMS (COE). These programs may be downloaded free of charge from the associated agencies' web sites. Other models may be acceptable on a case-by-case basis.

Note: There are user guides and manuals for the various computer models. Of note, the City of Plymouth recommends utilizing the Christopher B. Burke Stormwater Drainage Manual (2015)'s Chapter 8 – Computer Applications. It is available for download at the below link and provided within this Manual as **Appendix F**.

Christopher B. Burke Stormwater Drainage Manual (2015)
https://docs.lib.purdue.edu/inltappubs/100/

Table 2-4

Rainfall Intensities for Various Return Periods and Storm Durations						
Duration	Intensity (Inches / Hour), by Return Period (Years)					
	2 Years	*5 Years*	*10 Years*	*25 Years*	*50 Years*	*100 Years*
5 Minutes	5.33	6.41	7.25	8.33	9.16	9.98
10 Minutes	4.16	4.98	5.59	6.37	6.94	7.52
15 Minutes	3.39	4.08	4.59	5.24	5.73	6.22
30 Minutes	2.27	2.79	3.18	3.70	4.09	4.49
1 Hour	1.39	1.75	2.03	2.40	2.70	3.00
2 Hour	0.822	1.05	1.23	1.48	1.68	1.90
3 Hour	0.590	0.751	0.882	1.06	1.22	1.38
6 Hour	0.351	0.445	0.523	0.634	0.727	0.826
12 Hour	0.201	0.254	0.298	0.360	0.413	0.469
24 Hour	0.118	0.147	0.171	0.205	0.233	0.263

Source: NOAA, National Weather Service, "Precipitation-Frequency Atlas of the United States", NOAA Atlas 14, Volume 2, Version 3, rev 2006, for Plymouth, Indiana. Values for intermediate durations can be logarithmically interpolated. (All provided rainfall intensities are based on the default 50% values). Latitude: 41.3435°; Longitude: -86.3095°.

Table 2-5

Rainfall Depths for Various Return Periods and Storm Durations							
Duration	Depth (Inches), by Return Period (Years)						
	1 Year	*2 Years*	*5 Years*	*10 Years*	*25 Years*	*50 Years*	*100 Years*
5 Minutes	0.372	0.444	0.534	0.604	0.694	0.763	0.832
10 Minutes	0.578	0.694	0.830	0.932	1.06	1.16	1.25
15 Minutes	0.708	0.848	1.02	1.15	1.31	1.43	1.56
30 Minutes	0.937	1.14	1.40	1.59	1.85	2.05	2.25
1 Hour	1.14	1.39	1.75	2.03	2.40	2.70	3.00
2 Hour	1.35	1.64	2.10	2.46	2.96	3.36	3.80
3 Hour	1.46	1.77	2.26	2.65	3.20	3.66	4.14
6 Hour	1.73	2.10	2.67	3.13	3.80	4.35	4.95
12 Hour	2.00	2.43	3.07	3.59	4.35	4.98	5.66
24 Hour	2.35	2.83	3.55	4.12	4.94	5.61	6.31

Source: NOAA, National Weather Service, "Precipitation-Frequency Atlas of the United States", NOAA Atlas 14, Volume 2, Version 2, rev 2006, for Plymouth, Indiana. (Rainfall depths provided for 2- through 100-year storm are based on the default 50% values. 1-year rainfall depth is based on partial duration series). Latitude: 41.3435°; Longitude: -86.3095°.

Table 2-6

NRCS, Type 2 Rainfall Distribution Ordinates					
Cumulative Storm Time (Hour)	Cumulative Percent (%) of Storm Depth	Cumulative Storm Time (Hour)	Cumulative Percent (%) of Storm Depth	Cumulative Storm Time (Hour)	Cumulative Percent (%) of Storm Depth
0.00	0	8.25	12.6	16.50	89.3
0.25	0.2	8.50	13.3	16.75	89.8
0.50	0.5	8.75	14	17.00	90.3
0.75	0.8	9.00	14.7	17.25	90.8
1.00	1.1	9.25	15.5	17.50	91.3
1.25	1.4	9.50	16.3	17.75	91.8
1.50	1.7	9.75	17.2	18.00	92.2
1.75	2	10.00	18.1	18.25	92.6
2.00	2.3	10.25	19.1	18.50	93
2.25	2.6	10.50	20.3	18.75	93.4
2.50	2.9	10.75	21.8	19.00	93.8
2.75	3.2	11.00	23.6	19.25	94.2
3.00	3.5	11.25	25.7	19.50	94.6
3.25	3.8	11.50	28.3	19.75	95
3.50	4.1	11.75	38.7	20.00	95.3
3.75	4.4	12.00	66.3	20.25	95.6
4.00	4.8	12.25	70.7	20.50	95.9
4.25	5.2	12.50	73.5	20.75	96.2
4.50	5.6	12.75	75.8	21.00	96.5
4.75	6	13.00	77.6	21.25	96.8
5.00	6.4	13.25	79.1	21.50	97.1
5.25	6.8	13.50	80.4	21.75	97.4
5.50	7.2	13.75	81.5	22.00	97.7
5.75	7.6	14.00	82.5	22.25	98
6.00	8	14.25	83.4	22.50	98.3
6.25	8.5	14.50	84.2	22.75	98.6
6.50	9	14.75	84.9	23.00	98.9
6.75	9.5	15.00	85.6	23.25	99.2
7.00	10	15.25	86.3	23.50	99.5
7.25	10.5	15.50	86.9	23.75	99.8
7.50	11	15.75	87.5	24.00	100
7.75	11.5	16.00	88.1	-	-
8.00	12	16.25	88.7	-	-

Source: National Resources Conservation Service (NRCS), "TR-20 Computer Program for Project Formulation Hydrology", page F9, May 1982.
NOTE: For use only when SCS Type 2 rainfall distribution is not a default option in the computer program.

Chapter 3 | Methodology for Determination of Retention / Detention Storage Volumes & Detention Design Standards

Section 1 (Methodology for Determination of Retention / Detention Storage Volumes)

A. Development Sites, Including Contributing Drainage Area, Less Than 50 Acres.

The required volume of stormwater storage may be calculated using the Modified Rational Method, unless the City of Plymouth stormwater reviewer finds the site conditions warrant another method.

Modified Rational Method:

Calculate the required detention volume by iterating through different storm durations to find the peak storage volume (which may or may not be the time of concentration).

The following eight (8)-step procedure, based on the Rational Method, may be used to determine the required volume of storage:

1. Determine the parcel area (in acres, ac) "A" tributary to each outlet.

2. Compute the 10-year frequency allowable release rate ("Q_u") to each outlet based on methodologies provided in Section 2 (Stormwater Detention Design Standards for Peak Flow Control) of this Chapter.
    a. Determine composite runoff coefficient ("C_u") based on existing land use (undeveloped).
    b. Determine time of concentration ("t_c") in minutes based on existing conditions. For "t_c" determined to be less than 15 minutes, 15 minutes shall be used.
    c. Determine rainfall intensity "I_u" in inches per hour (in/hr), based on time of concentration from data given in **Table 2-4** for the ten (10) year-return period.
    d. Compute runoff based on existing land use (undeveloped) and ten (10) year return period:

$$Q_u = (C_u)(I_u)(A_u)$$

3. Determine composite runoff coefficient ("C_d") based on developed conditions and a 100-year return period.

4. Determine 100-year return rainfall intensity ("I_d") for various storm durations ("t_d") up through 24 hours for the developed area using **Table 2-4**.

5. Determine developed inflow rates ("Q_d") for various storm durations ("t_d"), measured in hours.

$$Q_d = (C_d)(I_d)(A_d)$$

6. Compute a storage rate ("$S(t_d)$") for various storm durations ("t_d") up through 24 hours for the developed area.

$$S(t_d) = (Q_d) - (Q_u)$$

7. Compute required storage volume ("S_R") in acre-feet for each storm duration ("t_d"). This assumes a triangular hydrograph of duration ($2t_d$) hours with a peak flow of $S(t_d)$ at t_d hours.

$$S_R = S(t_d)\left(\frac{t_d}{12}\right)$$

8. Select largest storage volume computed in Step 7 for any storm duration "t_d" for detention basin design.

B. Development Sites, Including Contributing Drainage Area, Greater Than 50 Acres.

The required volume of stormwater storage for all development sites shall be computed at the discretion of the City of Plymouth stormwater reviewer.

With the approval of the City of Plymouth stormwater reviewer, the required volume of stormwater storage for all development sites may be computed using the (modified) rational method or a computer model that can generate hydrographs (SCS) based on the NRCS TR-55 time of concentration and curve number calculation methodologies.

The 24-hour NRCS Type 2 Rainfall Distribution shall be utilized to determine the required storage volume. The allowable release rates shall be determined based on the methodologies provided in Section 2 (Stormwater Detention Design Standards for Peak Flow Control) of this Chapter. Examples of computer models that can generate such hydrographs include XPSWMM, TR-55 (NRCS), TR-20 (NRCS), HEC-HMS (COE), and HEC-1 (COE). Many of these programs may be downloaded free of charge from the associated agencies' web sites. Other models may be acceptable and should be approved by the City of Plymouth City Engineer / stormwater reviewer prior to their utilization.

The following steps can be used to determine the required storage volume of the detention basin(s):

1. Calibrate the hydrologic / hydraulic model that is to be used for prediction of runoff and routing of stormwater.
2. Determine the 10-year, undeveloped peak flow. Denote this flow by Q_u10.
    a. This is the allowable release rate.
3. Determine the 100-year runoff hydrograph (H_d100) for developed conditions.
4. Route the on-site 10-year and 100-year inflow hydrographs through the pond (by temporarily ignoring off-site flows) and size the main outlet to limit peak outflows to the allowable release rates. If an orifice will need to be used to limit the outflow, use the actual orifice size needed to meet the release rate.

Note: There are user guides and manuals for the various computer models. Of note, the City of Plymouth recommends utilizing the Christopher B. Burke Stormwater Drainage Manual

(2015)'s Chapter 8 – Computer Applications. It is available for download at the below link and provided within this Manual as **Appendix F**.

Christopher B. Burke Stormwater Drainage Manual (2015)
https://docs.lib.purdue.edu/inltappubs/100/

C. Post-Development Hydrologic Parameters

Note that for the purpose of determining the post-developed conditions curve numbers, due to significant disturbance to the upper soil layers during the construction activities, the initially determined hydrologic soil group for disturbed areas should be changed to the next less infiltrating capacity category (i.e., A to B, B to C, and C to D).

Further, for the TR-20 (NRCS) method, an antecedent moisture condition of 2, reflective of normal soil moisture, shall be used with the curve number method.

D. Design Storm and Allowable Release Rates for Development Sites with On-Site Detention Facilities

The 24-hour NRCS Type 2 Rainfall Distribution shall be utilized to determine the required storage volume. The allowable release rates shall be determined based on methodologies provided in Section 2 (Stormwater Detention Design Standards for Peak Flow Control) of this Chapter.

E. Design Storm and Allowable Release Rates for Development Sites within a Designated Drainage Area Serviced By a Regional Detention Facility

Development sites within Designated Drainage Areas making use of regional detention ponds are required to provide detention for amounts greater than the lot impervious percent (%) used to calculate the regional detention volume.

The 24-hour NRCS Type 2 Rainfall Distribution shall be utilized to determine the required storage volume. The stormwater model should not include an on-site detention facility. The outflow hydrograph at the downstream-most point in the site's stormwater system shall be compared to the allowable release rate for the Designated Drainage Area to determine the required storage volume. The allowable release rate for a Designated Drainage Area shall be as defined by the resolution forming the Designated Drainage Area.

The storage volume shall be determined by calculating the volume of outflow from the site that exceeds the given allowable release rate.

Due to predominant soil conditions in the City of Plymouth, retention facilities are generally not feasible and will only be allowed at the discretion of the City Engineer.

Section 2 (Stormwater Detention Design Standards for Peak Flow Control)

Basins must be constructed to retain and / or temporarily detain the stormwater runoff that exceeds the maximum peak release rate authorized by the Ordinance and these technical

standards. The required volume of storage provided in these basins, shall be sufficient to control excess runoff from the 10-year or 100-year storm as explained below in Section B. Also, basins shall be constructed to provide adequate capacity to allow for sediment accumulation resulting from development and to permit the pond to function for reasonable periods between cleanings.

F. Suggested Calculation Sequences for Designing a Detention Pond for Peak Flow Control

In addition to the requirement for peak flow control through retention and / or detention, the Stormwater Management Ordinance and Technical Standards require the developer to address Water Quality Control requirements discussed in Chapter 7. The proper way to accommodate the water quality and peak flow rate control of a site is to first consider addressing the water quality volume requirements through the conventional approach (as described in Chapter 7) and then determine the size and dimensions of the required retention or detention storage for peak flow rate control.

Chapter 7 provides several BMPs and options to address the water quality requirements. However, in many cases, providing some level of extended detention may become necessary to meet those requirements. While such extended detention is best provided in a separate facility, many developers choose to combine the needed extended detention feature with the detention pond needed for peak runoff rate control of the site into one facility.

A combined facility must accommodate the water quality volume and design storm detention to meet allowable release rate requirements while also meeting water quality detention time requirements.

These requirements can be challenging to meet, especially with additional considerations needed for bypassing runoff from off-site areas. The following are suggested calculation sequences for designing a detention pond for peak flow control only, and for combining extended detention with peak flow control. However, every site is different and depending on the site conditions and the layout of the pond, there may be other ways to design the pond such that all the pond's objectives are met.

<u>Peak Flow Control Only</u>

1. Determine the main outlet's control elevation.
2. Route the on-site 100-year inflow hydrographs through the pond (by temporarily ignoring off-site flows) and size the main outlet to limit peak outflows to the allowable release rates. If an orifice will need to be used to limit the outflow, use at minimum a 4-inch diameter orifice; coordinate with the City of Plymouth as needed for clarification. The rate of release for detention storage shall be controlled by an orifice plate or other devices, subject to approval of the City Engineer, where the 12-inch pipe will not limit the rate of release as required. The resulting maximum water surface elevation is the top of peak flow control storage for on-site flows. If no off-site flows are routed through the detention facility, then this will be the pond's 100-year elevation and the emergency spillway invert elevation is also set at this elevation.

3. Off-site flows that are being routed through the proposed site detention pond that do not require additional detention shall be brought to the attention of the City Engineer and shall be accounted for in a manner approved by the City Engineer.

Combined Peak Flow Control and Extended Detention

1. Calculate the extended detention storage volume as needed to address the requirements noted in Chapter 7. Also, note that there may be no need to provide extended detention, depending on the approach used to address water quality requirements.

2. Determine the outlet control elevation. This will be the bottom of the extended detention storage volume, and the permanent pool elevation if a wet-bottom pond is used.

3. Design the pond to provide the extended detention storage volume determined in Step 1, and assuming no outflow through the pond outlet. The top of this storage volume will be the invert elevation of the main outlet.

4. Route the on-site 10-year and 100-year inflow hydrographs through the pond (with the initial elevation at normal pool, i.e., the bottom of extended detention space) and size the main outlet to limit peak outflows to the allowable release rates. If an orifice will need to be used to limit the outflow, use at minimum a 4-inch diameter orifice; coordinate with the City of Plymouth as needed for clarification. The rate of release for storage shall be controlled by an orifice plate or other devices, subject to approval of the BOWS, where the 12-inch pipe will not limit the rate of release as required. The resulting maximum water surface elevation is the top of peak flow control storage for on-site flows. If no off-site flows are routed through the detention facility, then this will be the pond's 100-year elevation and the emergency spillway invert elevation is also set at this elevation.

5. Off-site flows that are being routed through the proposed site detention pond that do not require additional detention shall be brought to the attention of the City Engineer and shall be accounted for in a manner approved by the City Engineer.

6. Determine the size and design the retention / extended detention storage drain in a manner to meet the extended detention minimum and maximum emptying time requirements discussed in Chapter 7, using both on-site and, if applicable, off-site runoff. The rate of release for storage shall be controlled by an orifice plate or other devices, subject to approval of the BOWS, where the 12-inch pipe will not limit the rate of release as required.

7. To make sure that the addition of the release through the drain will not cause the on-site only allowable release rate to be exceeded, reroute the on-site 10-year and 100-year inflow hydrographs through the pond (with the initial elevation at normal pool, i.e., the bottom of extended detention space), this time allowing water to also leave through the extended detention storage drain as the pond fills up. If the total peak outflow discharge exceeds the on-site only allowable release rate, the size of the main outlet orifice may need to be reduced or the storage volume increased.

Note that in some instances such as relatively small development sites less than 10 acres or sites with highly restrictive site-specific maximum allowable release rates, when the required outlet orifice size and / or the required size of the extended detention drain will be small, the calculated drain time may extend beyond the maximum required 48-hour emptying time. Often times, the situation can be addressed through enlarging the pond volume or reconfiguring the pond's shape. When the situation cannot be resolved in a reasonable manner despite those attempts, the City of Plymouth or designee may, on a case-by-case basis, allow deviation from the required orifice size, maximum allowable release, or emptying time after considering reasonable options and examining the potential impacts on downstream or upstream areas. Economic factors shall not be considered for this determination.

The following shall govern the design of any improvement with respect to the retention / detention of stormwater runoff for peak flow control.

G. Acceptable Detention Facilities

The increased stormwater runoff resulting from a proposed development should be retained (if possible) and detained on-site (if retention is not feasible due to limited infiltration capacity) by the provisions of appropriate above- or below-ground wet bottom or dry bottom detention facilities, parking lots, or other acceptable techniques. Measures that hinder the rate of overland flow and the velocity in runoff channels shall also be used to partially control runoff rates. Infiltration for outlet rate only allowed for detention calculations if soil percolation rate is shown to exceed 6 inches / hour.

H. Allowable Release Rates

1. General Release Rates

Control devices shall limit the discharge to a rate such that the post-developed release rate from the site is no greater than the 10-year return interval storm discharge on the undeveloped site. The above fixed general release rates may be set at a lower value by the City of Plymouth for geographical areas determined as Impact Drainage Area or for certain watersheds if more detailed data becomes available as a result of comprehensive watershed studies conducted and / or formally approved and adopted by the City of Plymouth, or Marshall County. The applicant shall confirm the applicable release rates with the City of Plymouth or designee prior to initiating the design calculations to determine whether a basin-specific release rate has been established for the watershed of their interest or whether the site is located within a designated or an Impact Drainage Area.

For sites where the pre-developed area has more than one (1) outlet, the release rate should be computed based on pre-developed discharge to each outlet point. The computed release rate for each outlet point shall not be exceeded at the respective outlet point even if the post developed conditions would involve a different arrangement of outlet points.

2. Management of Off-Site Runoff

Runoff from all upstream tributary areas (off-site land areas) may be bypassed around the retention / detention facility without attenuation. Such runoff may also be routed through the detention / retention facility, provided that a separate outlet system or channel is incorporated for the safe passage of such flows, i.e., not through the primary outlet of a detention facility.

Note that the efficiency of the retention / detention facility in controlling the on-site runoff may be severely affected if the off-site area is considerably larger than the on-site area. As a general guidance, on-line detention may not be effective in controlling on-site runoff where the ratio of off-site area to on-site area is larger than 5 to 1 (5:1). Additional detention (above and beyond that required for on-site area) may be required by the City of Plymouth when the ratio of off-site area to on-site area is larger than 5:1.

3. Downstream Restrictions

In the event the downstream receiving channel or storm sewer system is inadequate to accommodate the post-developed release rate provided above, then the allowable release rate shall be reduced to that rate permitted by the capacity of the receiving downstream channel or storm sewer system. Additional detention, as determined by the City of Plymouth, shall be required to store that portion of the runoff exceeding the capacity of the receiving sewers or waterways. When such downstream restrictions are suspected, the City of Plymouth may require additional analysis to determine the receiving system's limiting downstream capacity.

If the proposed development makes up only a portion of the undeveloped watershed upstream of the limiting restriction, the allowable release rate for the development shall be in direct proportion to the ratio of its drainage area to the drainage area of the entire watershed upstream of the restriction.

As an alternative to reduction of release rates, the City of Plymouth may require the applicant to pursue alleviating downstream restrictions. The applicant would be responsible for obtaining all permits and consents required and for incurring all expenses involved in such undertaking.

4. Documentation of Results

The results of the allowable release rate determinations (if retaining the entire required peak flow rate volume is not feasible) as well as the modeling simulation results must be summarized in a table that shall be included in the Stormwater Drainage Technical Report and on the Drainage Plan. The table must include, for each eventual site outlet, the pre-developed acreage tributary to each eventual site outlet, the unit discharge allowable release rate used, the resulting allowable release rate in Ft^3/s (cfs) for the post-developed 10-year and 100-year events, as well as pre- and post-developed flow rates for 10- and 100-year events. The worksheet provided as **Table 3-1** should be filled for each final site outlet.

Table 3-1

Documentation of Results Worksheet									
Site Outlet Number	Item	Pre-Development				Post-Development			
		Drainage Area (Acres)	Depressed Storage? (Yes or No)	10-yr	100-yr	Drainage Area (Acres)	Depressed Storage? (Yes or No)	10-yr	100-yr
1	Default Unit Discharge Allowable Release Rate (cfs / acre)								
1	Basin-Specific Unit Discharge Allowable Release Rate, if any (cfs / acre)								
1	Unit Discharge Allowable Release Rate Based on Downstream Restrictions, if any (cfs / acre)								
1	Adopted Unit Discharge Allowable Release Rate (cfs/acre)								
1	Contributing Area of Development Site (ac) and Allowable Release Rate (cfs)								
1	Total Contributing DA (ac) and Modeling Results (cfs)								

I. General Detention Basin Design Requirements

1. The retention / detention facility shall be designed in such a manner that a minimum of 90% of the maximum volume of water stored and subsequently released at the design release rate (if not retaining the entire peak flow rate volume) shall not result in a storage duration in excess of 48 hours from the start of the storm unless additional storms occur within the period. In other words, the design must ensure that a minimum 90% of the original retention / detention capacity is restored within 48 hours from the start of the design 100-year storm.

2. The 100-year elevation of stormwater retention / detention facilities must be separated by not less than 25 feet from any building or structure to be occupied. The Lowest Adjacent Grade (including walkout basement floor elevation) for all residential, commercial, or industrial buildings must be set a minimum of 2 feet above the 100-year pond elevation or 2 feet above the emergency overflow weir elevation, whichever is higher. In addition to the Lowest Adjacent Grade requirements, any basement floor must be at least a foot above the normal water level of any wet-bottom pond or the local groundwater table, whichever is higher, to avoid the overuse of sump pumps and frequent flooding of the basement.

3. No detention facility or other water storage area, permanent or temporary, can be constructed under or within 20 feet of any pole or high voltage electric line. Likewise, poles or high voltage electric lines must not be placed within 20 feet of any detention facility or other water storage area.

4. Detention facilities shall be separated from parking lots and roadways by an appropriately selected and designed method of safety barrier, such as guard rails, bollards, or other physical barriers capable of deterring the passage of a vehicle into the pond.

5. Slopes no steeper than 3 horizontal to 1 vertical (3:1) for safety, erosion control, stability, and ease of maintenance shall be permitted.

6. Safety screens having a maximum opening of 6 inches must be provided for any pipe or opening end sections 12 inches in diameter or larger. Storm drainpipes outleting into the pond must not be submerged.

7. Use of fences around all retention / detention ponds is encouraged to assure safety. Unless specifically required by the City of Plymouth, the decision to use fencing around retention / detention ponds are left to the owner or the developer. Recommendations contained within this document do not relieve the applicant and owner / developer from the responsibility of taking all necessary steps to ensure public safety with regards to such facilities. Fencing is required on all retention / detention ponds deeper than four (4) feet.

8. Outlet control structures must be designed to operate as simply as possible and must require little or no maintenance and / or attention for proper operation. For maintenance purposes, the outlet from the pond (if any) must be a minimum of 0.5 feet above the normal water level of the receiving water body. They must limit discharges into existing or planned downstream channels or conduits so as not to exceed the predetermined maximum authorized peak flow rate. If an orifice will need to be used to limit the outflow, use at minimum a 4-inch diameter orifice; coordinate with the City of Plymouth as needed for clarification. Other devices are subject to approval of the City of Plymouth Engineer.

9. Emergency overflow spillways must be provided for the release of exceptional storm runoff or in emergency conditions should the normal discharge devices become totally or partially inoperative. The overflow facility shall be of such design that its operation is automatic and does not require manual attention.

Emergency overflow spillways must be designed to convey, without overtopping the detention facility banks, the peak inflow discharge resulting from the 100-year design storm event runoff from the entire contributing watershed draining to the detention / retention facility, assuming post-development condition on-site and existing condition off-site. The overland overflow spillway shall be determined by using the weir equation with the maximum head being the difference between the weir control elevation and the top of bank.

The emergency overflow routing from the emergency overflow facility to an adequate receiving system must be positive (by gravity) and shown on the construction plans. It must be sized to accommodate the design flow of the pond's emergency overflow weir. The Lowest Adjacent Grade of all residential, commercial, or industrial buildings along this emergency overflow route must be set a minimum of two (2) feet above the flood elevation along the route, calculated based on the pond's emergency overflow weir design discharge.

10. Grass or other suitable vegetative cover shall be provided along the banks of the retention / detention storage basin. Vegetative cover around detention facilities must be maintained as appropriate.

11. Debris and trash removal and other necessary maintenance must be performed on a regular basis to ensure continued operation in conformance to design.

12. For new subdivisions, no residential lots or any part thereof can be used for any part of a detention basin assumed full to the 100-year water surface elevation or the emergency overflow weir elevation, whichever is higher. Detention basins assumed full to the 100-year water surface elevation or the emergency overflow weir elevation, whichever is higher, shall be placed within a common area either platted or legally described and recorded as a perpetual stormwater easement.

13. Dry bottom detention ponds are required to maintain positive slope along the pond bottom and may not be paved.

J. Additional Requirements for Wet-Bottom Facility Design

Where part of a detention facility will contain a permanent pool of water, all the items required for detention storage apply. Also, a controlled positive outlet is required to maintain the design water level in the wet bottom facility and provide required detention storage above the design water level. The following additional conditions apply:

1. Facilities designed with permanent pools or containing permanent lakes must have a water area of at least one-half (0.5) acre with a minimum depth of eight (8) feet over the majority of pond area. If fish are to be used to keep the pond clean, a minimum depth of approximately ten (10) feet must be maintained over at least 25% of the pond area. The remaining pond area must have no extensive shallow areas, except as required to install the

safety ramp, safety ledge, and BMPs as required below. Construction trash or debris must not be placed within the permanent pool.

2. All wet detention / retention ponds must be constructed in as natural a shape (footprint) as possible and have a vegetated safety ledge (approximately six (6) inches below normal pool) and / or have native vegetation planted on the pond banks to create a riparian buffer (minimum ten (10) feet wide). Native vegetation can be installed as container grown plants or as seed at the time of construction. The vegetation must be planted in a manner so as not to hide or disguise the ponds edge. Maintenance of the vegetated barrier is the responsibility of the owner or the homeowners' association. All pond slopes must be 3 to 1 (3:1) or flatter.

3. For wet-bottom facilities without a security fence, a maintenance ledge ten (10) feet in width is required and must be installed approximately twelve (12) inches above the permanent water level and a safety ledge ten (10) feet in width is required and must be installed approximately six (6) inches below the permanent water level. If a non-vegetated safety ledge is installed, the depth of the safety ledge must be approximately eighteen (18) inches below normal pool. The slope between the two (2) ledges must be stable and protected from erosion with hard armoring or bioengineered techniques.

For wet-bottom facilities with a security fence, a maintenance ledge six (6) feet in width is required and must be installed approximately twelve (12) inches above the permanent water level and a safety ledge six (6) feet in width is required and shall be installed approximately six (6) inches below the permanent water level. If a non-vegetated safety ledge is installed, the depth of the safety ledge shall be approximately eighteen (18) inches below normal pool. The slope between the two (2) ledges shall be stable and protected from erosion with hard armoring or bioengineered techniques.

The maintenance ledge may be waived if pond side slopes above normal water are 6 to 1 (6:1) or flatter.

4. Prior to final acceptance of construction and release of bonds, danger signs warning of deep water, possible flood conditions during storm periods, or other dangers, must be installed at an interval of 400 feet around the perimeter of wet-bottom facilities without a vegetated safety ledge, riparian buffer, or a security fence. Maintenance of the warning signs shall be the responsibility of the owner or the homeowners' association.

5. If a retaining wall is used below the normal pool of wet detention pond, the wall must have either steps or a ladder incorporated into the construction at the center of the wall span.

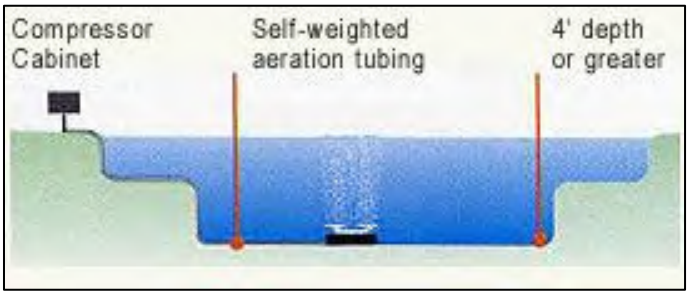
6. A safety ramp exit from the lake is required in all cases and must have a minimum width of twenty (20) feet and exit slope of 6 to 1 (6:1). The safety ramp must be constructed of suitable material to prevent structural instability due to vehicles or wave action. Adequate access to the safety ramp must be provided by locating it adjacent to public ROW or by providing a clear route recorded within an access easement or a common area.

7. In addition to provisions specified in Section H.3 of this Chapter (if applicable), parks, playgrounds, and athletic fields must be separated from all stormwater detention facilities by no less than 100 feet, measured from the 100-year pool of the pond. Trails and sidewalks must be separated from all stormwater detention facilities by no less than twenty-five (25) feet, measured from the 100-year pool of the pond.

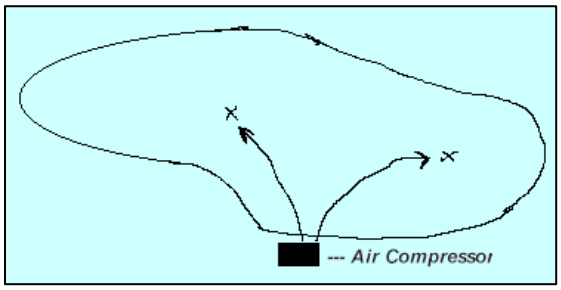
8. Periodic maintenance is required in lakes to control weed and larval growth. The facility must also be designed to provide for the easy removal of sediment that will accumulate during periods of reservoir operation. A means of maintaining the designed water level of the lake during prolonged periods of dry weather may also be required.

9. For maintenance purposes, the outlet of storm sewers entering the pond must be a minimum of one-half (0.5) feet above the normal pool level.

10. Methods to prevent pond stagnation, including but not limited to surface or sub-surface aeration or destratification facilities that can, at the minimum, achieve one (1) complete pond volume turnover per day should be considered. The figure below shows a typical diffuser aeration system that consists of a quiet air compressor at the shore, aeration tubing, and one or more sets of diffuser head(s).



Irregularly shaped ponds should be treated as two or three separate ponds. Likewise, larger ponds will also need multiple aeration units.



Design calculations to substantiate the effectiveness of proposed aeration facilities must be submitted with final engineering plans. Design calculations must, at a minimum, indicate that the device and / or series of devices are providing a minimum volume turnover of once (1x) per 24 hours over the majority of the pond volume (i.e. pump radius of influence calculations, etc.). Agreements for the perpetual operation and maintenance of aeration facilities by the property owner or the homeowners' association must be prepared similar to that noted for post-construction BMPs outlined in Chapter 7 of these Technical Standards.

If the detention pond is also proposed to be used as a post-construction stormwater BMP, alternative means of aeration (such as diffuser aeration systems) must be used that would not result in re-suspension of sediment particles and would not prevent the efficient settling of sediment particles.

11. If the facility is being located near an airport, a minimum horizontal separation distance between the airport property and the pond will need to be provided in accordance to Federal Aviation Administration (FAA) advisory Circular 150/5200-33, titled "Hazardous Wildlife Attractions On or Near Airports" and dated 8/28/2007, or the current version.

K. Additional Requirements for Dry-Bottom Facility Design

In addition to general design requirements, retention / detention facilities that will not contain a permanent pool of water must comply with the following requirements:

1. Provisions must be incorporated into facilities for complete interior drainage of dry bottom facilities, including the provisions of natural grades to outlet structures (if any), longitudinal and transverse grades to perimeter drainage facility. Unless designed as a retention facility, a six (6)-inch diameter underdrain, with a minimum of one and one-half (1.5) feet of cover, is recommended within all dry-bottom ponds.

2. For residential developments, unless the facility is fully fenced in and gated, the maximum planned depth of stormwater stored must not exceed four (4) feet.

3. In excavated retention / detention facilities, a minimum side slope of 3 to 1 (3:1) must be provided for stability. In the case of valley storage, natural slopes may be considered, pending stability.

L. Detention Facilities in Floodplains

No detention facilities are allowed within FEMA regulatory floodplains.

M. Joint Development of Control Systems

Stormwater control systems may be planned and constructed jointly by two (2) or more developers as long as compliance with the Ordinance and this Technical Standards Manual is maintained.

N. Diffused Outlets

When the allowable runoff is released in an area that is susceptible to flooding or erosion, the developer may be required to construct appropriate storm drains through such area to prevent increased flood hazard caused by the concentration of allowable runoff at one point instead of the natural overland distribution. The requirement of diffused outlet drains shall be at the discretion of the City of Plymouth.

O. IDNR Requirements

Any construction in the floodway of a stream with a drainage area of one (1) square mile or more must satisfy IDNR permit requirements.

P. Allowance for Sedimentation

Retention / detention basins must be designed with an additional 6% of available capacity to allow for sediment accumulation resulting from development and to permit the pond to function for reasonable periods between cleanings. Basins should be designed to collect sediment and debris in specific locations, such as a forebay, so that removal costs are kept to a minimum. For wet-bottom ponds, the sediment allowance may be provided below the permanent pool elevation. No construction trash or debris are allowed to be placed within the permanent pool. If the pond is used as a sediment control measure during active construction, the performance sureties / financial guarantees will not be released until sediment has been cleaned out of the pond, elevations and grades have been reestablished as noted in the

accepted plans, and the volume is checked by a licensed professional for compliance with the accepted plans.

Q. Maintenance

The routine maintenance of stormwater detention facilities (i.e., trash pickup, aeration, weed control, sediment removal, etc.) is the responsibility of the Homeowners' Association or property owner.

Chapter 4 | Storm Sewer Design Standards and Specifications

All storm sewers, whether private or public, and whether constructed on private or public property shall conform to the design standards and other requirements contained herein. Proposed storm sewer systems must be sized and designed to convey at least the 10-year frequency on-site stormwater runoff, as well as the anticipated 10-year frequency tributary off-site stormwater runoff based on the future developed condition (using **Table 3-1** of the Technical Standards). An analysis of the emergency routing of stormwater runoff through the subject development must be provided to confirm that the development will not obstruct the free flow of floodwaters from the tributary off-site property in its current condition and after development.

A. Design Storm Frequencies

1. All storm sewers, inlets, catch basins, and street gutters shall accommodate (subject to the "allowable spread" provisions discussed later in this Section), as a minimum, peak runoff from a 10-year return frequency storm calculated based on methodology described in Chapter 2. Additional discharges to storm sewer systems allowed in Section L below of this Section must be considered in all design calculations.

2. Culvert capacities for conveyance under interior local, collector, or arterial streets without roadway overtopping shall be the runoff resulting from the 25-year, 50-year, and 100-year frequency storms, respectively, for off-site areas under existing condition and on-site areas under post-developed conditions. Driveway culvert capacities shall be capacities required for the street classification to which the driveway connects. Greater culvert capacity shall be required to protect the finished floor elevation of buildings from the post-developed 100-year frequency storm when, in the opinion of the design engineer or the City of Plymouth, the finished floor elevation is threatened. If the street or road provides the only access to or from any portion of any commercial or residential development, the crossing shall be designed for a minimum of 100-year frequency storm.

3. For portions of the system considered minor drainage systems, the allowable spread of water on Collector Streets for the design storm is limited to maintaining two (2) clear 10-foot moving lanes of traffic. One (1) lane is to be maintained on Local Roads, while other access lanes (such as a subdivision cul-de-sac) can have a water spread equal to one-half (1/2) of their total width.

4. To ensure access to buildings and allow the use of the roadway by emergency vehicles during storms larger than the design storm, an overflow channel / swale between sag inlets and overflow paths or basin shall be provided at sag inlets so that the maximum depth of water that might be ponded in the street sag shall not exceed seven (7) inches measured from elevation of gutter. All water shall be contained in the right-of-way (ROW) for a 100-year storm.

5. Facilities functioning as a major drainage system as defined in **Appendix A,** in the floodway of a stream, must also meet IDNR design standards in addition to the City of Plymouth standards. In case of discrepancy, the most restrictive requirements shall apply.

6. New drain tiles and pipes, smaller than twenty-four (24) inches in diameter, shall be placed in a twenty (20)-foot easement (ten (10) feet from centerline on each side) and shall be designated on the record plat as twenty (20)-foot Drain Easement. Pipes that are twenty-four (24) inches or larger in diameter, shall be placed in a thirty (30)-foot easement (fifteen (15) feet from centerline on each side) and shall be designated on the record plat as thirty (30)-foot Drain Easement. Wider easements may be required by the City of Plymouth or designee when the depth of pipe is greater than six (6) to ten (10) feet, depending on the pipe size.

B. Determination of Hydraulic Capacity for Storm Sewers Using Manning's Equation

Determination of hydraulic capacity for storm sewers sized by the Rational Method analysis must be done using Manning's Equation:

$$V = (1.486/n) (R^{2/3}) (S^{1/2})$$

Then:

$$Q = (V) (A)$$

Where:

Q = Capacity in Cubic Feet Per Second (Ft^3/s or cfs)
V = Mean Velocity of Flow in Feet Per Second (Ft/s or fps)
A = Cross Sectional Area in Square Feet (Ft^2)
R = Hydraulic Radius in Feet
S = Slope of The Energy Grade Line in Feet Per Foot (Ft/Ft)
N = Manning's "n" Or Roughness Coefficient

The hydraulic radius, R, is defined as the cross-sectional area of flow divided by the wetted flow surface or wetted perimeter. Allowable "n" values and full-flow maximum permissible velocities for storm sewer materials are listed in **Table 4-1**.

Table 4-1

Typical Values of Manning's "n"		
Material	Manning's "n"	Maximum Velocities (Feet / Second)
Closed Conduits		
Concrete	0.013	10
Vitrified Clay*	0.013	10
HDPE & Polypropylene	0.012	10
PVC	0.012	10
Circular CMP, Annular Corrugations, 2 2/3 x ½ inch		
Unpaved	0.024	7
25% Paved	0.021	7
50% Paved	0.018	7
100% Paved	0.013	7
Concrete Culverts	0.013	10
HDPE or PVC	0.012	10

Open Channels		
Concrete, Trowel Finish	0.013	10
Concrete, Broom Finish	0.015	10
Gunite	0.018	10
Riprap Placed	0.030	10
Riprap Dumped	0.035	10
Gabion	0.028	10
New Earth[1]	0.025	4
Existing Earth[2]	0.030	4
Dense Growth of Weeds	0.040	4
Dense Weeds and Brush	0.040	4
Swale with Grass	0.035	4

Source of Manning "n" Values: HERPICC Stormwater Drainage Manual, July 1995.

[1]New Earth (Uniform, Sodded, Clay Soil)

[2]Existing Earth (Fairly Uniform, with Some Weeds)

*Note clay pipe is not recommended and requires City of Plymouth prior approval.

C. Backwater Method for Pipe System Analysis

For hydraulic analysis of existing or proposed storm drains which possess submerged outfalls, a more sophisticated design / analysis methodology than Manning's equation will be required. The backwater analysis method provides a more accurate estimate of pipe flow by calculating individual head losses in pipe systems that are surcharged and / or have submerged outlets. These head losses are added to a known downstream water surface elevation to give a design water surface elevation for a given flow at the desired upstream location.

Tailwater conditions assumed for this computation should be verified by appropriate calculations. Total head losses may be determined as follows:

Total Head Loss = Frictional Loss + Manhole Loss + Velocity Head Loss + Junction Loss

Various computer modeling programs such as HYDRA, ILLUDRAIN, and STORMCAD are available for analysis of storm drains under these conditions. Computer models to be utilized, other than those listed, must be accepted by the City of Plymouth or designee.

D. Minimum Size for Storm Sewers

The minimum diameter of all storm sewers is twelve (12) inches. When the minimum twelve (12)-inch diameter pipe will not limit the rate of release to the required amount, the rate of release for detention storage must be controlled by an orifice plate or other device, subject to acceptance of the City of Plymouth.

E. Pipe Cover, Grade, and Separation from Sanitary Sewers

Pipe grade shall be such that, in general, a minimum of two (2) feet of cover is maintained over the top of the pipe. If the pipe is to be placed under pavement, or within five (5) feet of the pavement, then the minimum pipe cover shall be two (2) feet from top of pavement to top of pipe. Uniform slopes shall be maintained between inlets, manholes and inlets to manholes. Final grade shall be set with full consideration of the capacity required, sedimentation problems, and other design parameters. Minimum and maximum allowable slopes shall be those capable of producing velocities of between three (3) and ten (10) feet per second (Ft/s or fps), respectively, when the sewer is flowing full. Maximum permissible velocities for various storm sewer materials are listed in **Table 4-1**. A minimum of one (1) foot of vertical separation between storm sewers and sanitary sewers shall be required. When this is not possible, the sanitary sewer must be encased in concrete or ductile steel within five (5) feet, each side, of the crossing centerline.

F. Alignment

Storm sewers shall be straight between manholes and / or inlets. Deflection of pipe sections shall not be permitted.

G. Manholes and Inlets

Manholes and / or inlets must be installed to provide human access to continuous underground storm sewers for the purpose of inspection and maintenance. The casting access minimum inside diameter must be no less than twenty-two (22) inches or a rectangular opening of no less than twenty-two (22) inches by twenty-two (22) inches. Steps shall be provided in structures deeper than four (4) feet, with the first step at the depth of two (2) feet and the following steps spaced every one (1) foot until the bottom is reached. When grade adjustments of manholes and inlets are required in the field to meet finish design or existing curb grade, adjustment rings with a maximum height of twelve (12) inches may be used. Manholes must be provided at the following locations:

1. Where two (2) or more storm sewers converge
2. Where pipe size or the pipe material changes
3. Where a change in horizontal alignment occurs
4. Where a change in pipe slope occurs
5. At intervals in straight sections of sewer, not to exceed the maximum allowed. The maximum distance between storm sewer manholes is shown in **Table 4-2**.

Table 4-2

Maximum Distance Between Manholes	
Pipe Diameter (Inches)	Maximum Distance (Feet)
12 through 42	400
48 and Larger	500

Manhole / inlet inside sizing shall be according to the City of Plymouth Standard Construction Details.

In addition to the above requirements, a minimum drop of 0.1 foot through manholes and inlet structures should be provided. When changing pipe size, match crowns of pipes, unless detailed modeling of hydraulic grade line shows that another arrangement would be as effective. Pipe slope should not be so steep that inlets surcharge (i.e. hydraulic grade line should remain below rim elevation).

Plans should note that all Inlets and castings must be pre-stamped with a "clean water" message.

Manhole / inlet inside sizing shall be as shown in **Table 4-3**.

Table 4-3

Manhole / Inlet Inside Sizing		
Depth of Structure	Minimum Diameter	Minimum Opening
Less Than 5 Feet	24 Inches	36 Inches x 24 Inches
5 Feet or More	48 Inches	48 Inches x 48 Inches

H. Inlet Sizing and Spacing

Inlets or drainage structures must be utilized to collect surface water through grated openings and convey it to storm sewers, channels, or culverts. The inlet grate opening provided must be adequate to pass the design 10-year flow with 50% of the sag inlet areas clogged. Inlets must be provided so that surface water is not carried across or around any intersection nor for a distance greater than 500 feet. An overflow channel from sag inlets to the overflow channel or basin must be provided at sag inlets. Inlet design and spacing may be done using the hydraulic equations by manufacturers or orifice / weir equations. Use of the U.S. Army Corps of Engineers HEC-12 computer program is also an acceptable method. Gutter spread on continuous grades may be determined using the Manning's equation, or by using **Figure 4-1**. Flow through the gutter should be compared to inlet grate capacity using orifice/weir flow equations, or by using the equations in **Figure 4-4**. Further guidance regarding gutter spread calculation may be found in the latest edition of LTAP Stormwater Drainage Manual, available from the Local Technical Assistance Program (LTAP). At the time of printing of this document, contact information for LTAP was:

Indiana LTAP
Purdue University
Toll-Free: (800) 428-7369 (Indiana only)
Phone: (765) 494-2164
Fax: (765) 496-1176
Email: inltap@ecn.purdue.edu
Website: www.purdue.edu/INLTAP/

I. Installation and Workmanship

The point of commencement for laying a storm sewer pipe shall be the lowest point in the proposed sewer line. All pipes must be laid, without break, upgrade from structure to structure. All storm sewer pipe outlets must have poured in place toewalls with anchor bolts. Bedding and backfill materials around storm sewer pipes, sub-drains, and the associated structures shall be per **Figure 4-2** and **Figure 4-3**. The specific location requirements for the use of these materials are dependent on pipe location in relation to pavement structures and on pipe material as detailed in **Figure 4-2** and **Figure 4-3**. The specifications for the construction of storm sewers and sub-drains, including backfill requirements, must not be less stringent than those set forth in the latest edition of the INDOT "Standard Specifications". Additionally, ductile iron pipe shall be laid in accordance with American Water Works Association (AWWA) C-600 and in accordance with either American Society of Testing Materials (ASTM) or the appropriate American Association of State Highway and Transportation Officials (AASHTO) specifications.

Dips / sags on newly installed storm systems are not allowed. Also, infiltration from cracks, missing pieces, and joints are not allowed. Variations from these standards must be justified and receive written acceptance from the City of Plymouth. To verify that all enclosed drains and sewers are functioning properly, all storm sewers including sub-drains shall be cleaned and televised with visual recordings (via closed circuit television). If video recording suggests deformation, a passing of 95% mandrel test may be required.

Based on the review of visual recordings, the City of Plymouth shall determine the need for additional inspection of the storm sewers or sub-drains to assess the condition of the system. Newly installed storm systems shall meet the minimum requirements established in the AASHTO Culvert and Storm Drain Inspection Guide constituting a Condition Rating of 1 or “Good” as defined in Section 4, Condition Rating System and as established in **Table 4-4**.

Table 4-4

Pipe Inspection Criteria	
Material	Good Condition Rating of “1”
HDPE, PVC or PP	Barrel maintains round shape with less than 5% vertical deformation of original ID. No indication of wear, abrasion, splits, or cracking.
Concrete	No measurable crack width greater than 0.01 inches. No spalling, slabbing, delamination, scaling, or abrasion.
CMP	Barrel maintains round shape with less than 5% vertical deformation of inside diameter. No dents, rust, abrasion, or localized damage.

J. Materials

Storm sewer manholes and inlets shall be constructed of cast-in-place (CIP) concrete or pre-cast reinforced concrete. Material and construction must conform to the latest edition of the Indiana Department of Transportation (INDOT) "Standard Specifications", Sections 702 and 720.

Pipe and fittings used in storm sewer construction must conform to the latest edition of the INDOT "Standard Specifications", Section 907 and 908. In addition, ductile iron pipe and fittings must conform to AWWA C-151. Polyethylene pipe used for private storm sewers shall conform to ASTM F2648 or INDOT specifications. Other pipe and fittings not specified herein or in Sections 907-908 of the latest edition of the INDOT "Standard Specifications" may be used only when specifically authorized by the City of Plymouth. Pipe joints shall be flexible and leak resistant as defined in AASHTO R82, with a maximum leakage rate of 200 gallons / inch diameter / mile / day. **If the storm sewer pipe is to be placed within a road ROW or in an area subject to loading, the pipe and fittings shall be reinforced concrete (RCP) or corrugated polypropylene (CPP).**

K. Special Hydraulic Structures

Special hydraulic structures required to control the flow of water in storm runoff drainage systems include junction chambers, drop manholes, stilling basins, and other special structures. The use of these structures shall be limited to those locations justified by prudent planning and by careful and thorough hydraulic engineering analysis. Certification of special structures by a certified Structural Engineer may also be required.

L. Connections to Storm Sewer System

To allow any connections to the storm sewer system, provisions for the connections shall be shown in the drainage calculations for the system. Specific language must be provided in the protective covenants, on the record plat, or with the parcel deed of record, noting the ability or inability of the system to accommodate any permitted connections, for example, sump pumps and footing drains.

1. Sump pumps installed to receive and discharge groundwater or other stormwater must be connected to a storm sewer. Sump pumps installed to receive and discharge floor drain flow or other sanitary sewage shall be connected to the sanitary sewers. A stormwater sump pump must only be used for the discharge of stormwater.
2. Footing drains and perimeter drains shall be connected to the sub-surface drain provided by the developer.
3. All down spouts or roof drains shall discharge onto the ground or be connected to the storm sewer. No down spouts or roof drains shall be connected to the sanitary sewers.

   Commercial, industrial, and institutional buildings that have their own private storm sewers may direct such drains directly to their systems.
4. Garage and basement floor drains shall be connected to the sanitary sewer(s).
5. Swimming pool drains must not be connected to the storm sewer.

In addition, none of the above-mentioned drains can be connected to any road sub-drain / street underdrains, unless specifically authorized by the City of Plymouth. Sub-drains may be required on new developments, as deemed necessary by the City.

M. Drainage System Overflow Design

Any drainage to Marshall County ditches as well as emergency overflow conveyances to the County Legal drain must be piped, per County request. A 10- to 30 foot (ft)-wide surface emergency conveyance path should be shown on the plans with sufficient erosion control measures (e.g., erosion control blanket or riprap). Pipes shall be sized to prevent overtopping of the detention basin banks. The Marshall County Storm Drainage and Sediment Control Ordinance (2011) is linked below:

Marshall County Storm Drainage and Sediment Control Ordinance (2011)
https://www.co.marshall.in.us/egov/documents/1533494328_91719.pdf

Figure 4-1

Street and Gutter Capacities (Continuous Grade)

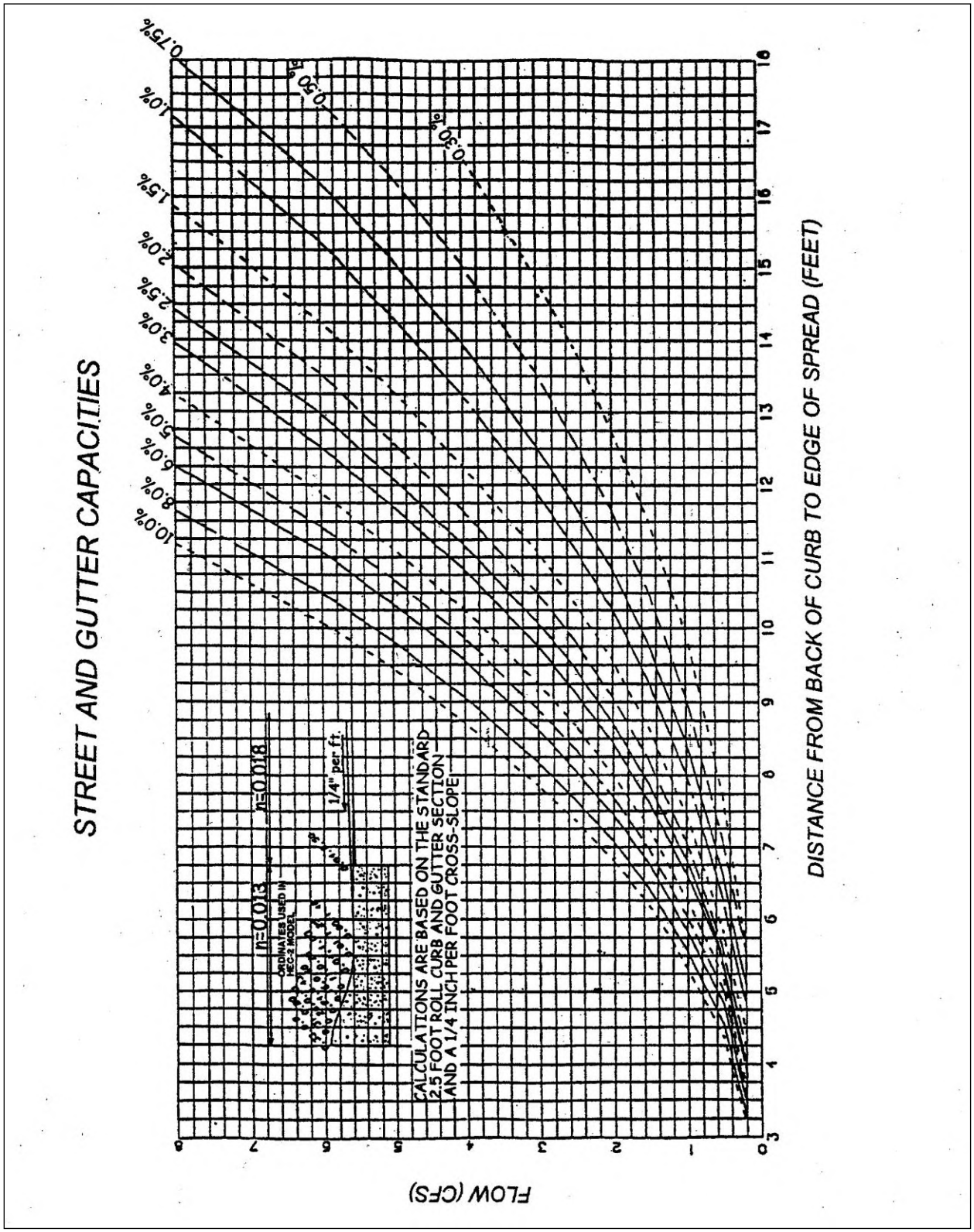
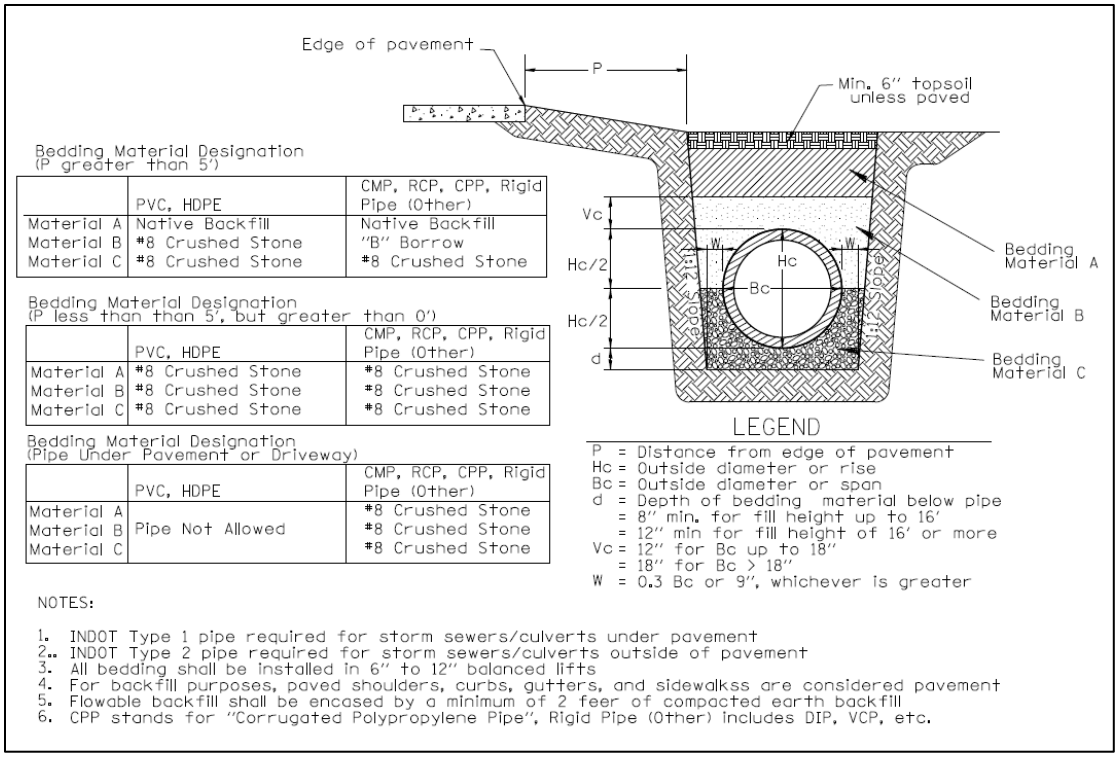


Figure 4-2
Bedding and Backfill Standards for Storm Sewers



Bedding Material Designation
(P greater than 5')

	PVC, HDPE	CMP, RCP, CPP, Rigid Pipe (Other)
Material A	Native Backfill	Native Backfill
Material B	#8 Crushed Stone	"B" Borrow
Material C	#8 Crushed Stone	#8 Crushed Stone

Bedding Material Designation
(P less than than 5', but greater than 0')

	PVC, HDPE	CMP, RCP, CPP, Rigid Pipe (Other)
Material A	#8 Crushed Stone	#8 Crushed Stone
Material B	#8 Crushed Stone	#8 Crushed Stone
Material C	#8 Crushed Stone	#8 Crushed Stone

Bedding Material Designation
(Pipe Under Pavement or Driveway)

	PVC, HDPE	CMP, RCP, CPP, Rigid Pipe (Other)
Material A	Pipe Not Allowed	#8 Crushed Stone
Material B		#8 Crushed Stone
Material C		#8 Crushed Stone

LEGEND

P = Distance from edge of pavement
Hc = Outside diameter or rise
Bc = Outside diameter or span
d = Depth of bedding material below pipe
= 8" min. for fill height up to 16'
= 12" min for fill height of 16' or more
Vc = 12" for Bc up to 18"
= 18" for Bc > 18"
W = 0.3 Bc or 9", whichever is greater

NOTES:

1. INDOT Type 1 pipe required for storm sewers/culverts under pavement
2. INDOT Type 2 pipe required for storm sewers/culverts outside of pavement
3. All bedding shall be installed in 6" to 12" balanced lifts
4. For backfill purposes, paved shoulders, curbs, gutters, and sidewalkss are considered pavement
5. Flowable backfill shall be encased by a minimum of 2 feer of compacted earth backfill
6. CPP stands for "Corrugated Polypropylene Pipe", Rigid Pipe (Other) includes DIP, VCP, etc.

Figure 4-3
Bedding and Backfill Standards for Sub-Drains Under Swales

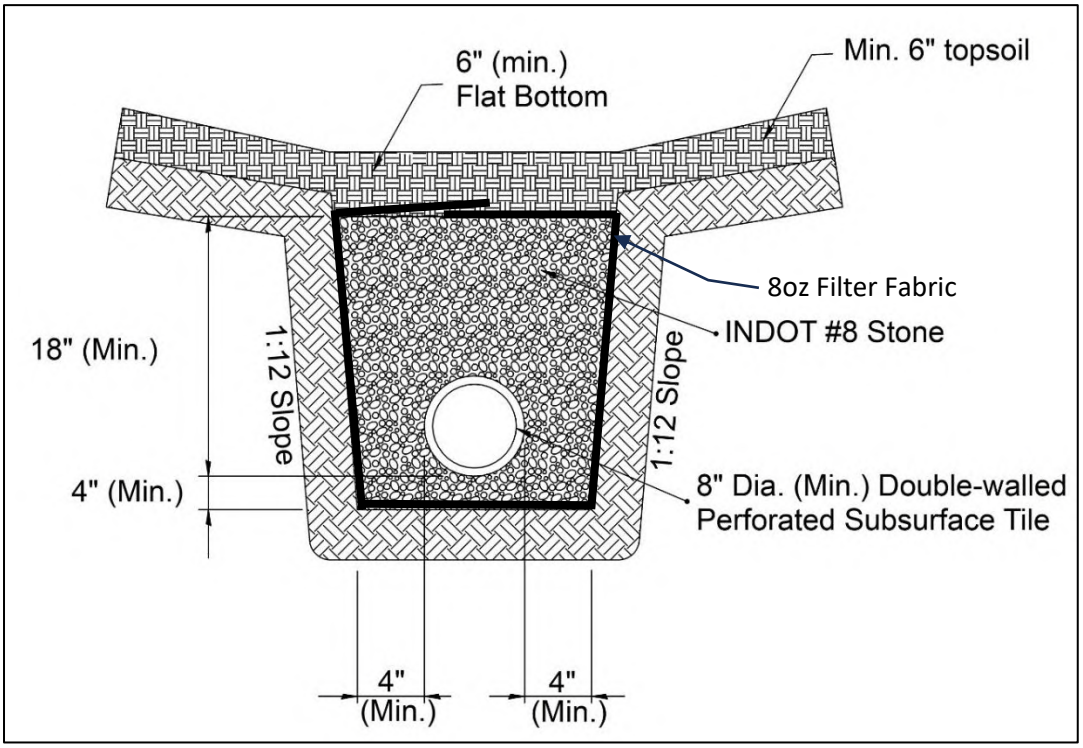
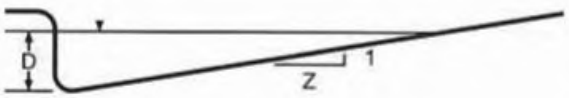


Figure 4-4
Inlet Grate Capacity & Flow Rate



$$Q = \frac{.56}{N} ZD^{8/3} S^{1/2} \text{ (MODIFIED MANNING EQUATION)}$$

Q = Channel flow in CFS
Z = Reciprocal of transverse slope
D = Depth in feet
S = Longitudinal slope
N = Roughness coefficient at constant .016 (value for concrete or asphalt)

$Q = KD^{5/3}$ (NEENAH EQUATION)
Q = Grate capacity in CFS
K = Grate coefficient from "Inlet Grate Capacities" manual
D = Depth of flow in feet

Chapter 5 | Open Channel Design Standards and Specifications

All channels, whether private or public, and whether constructed on private or public land, must conform to the design standards and other design requirements contained herein. Unless specifically referenced in a particular provision, the standards contained in this Chapter refer to open channels and not swales. Proposed open channels must be sized and designed to convey at least the 10-year frequency on-site stormwater runoff, as well as the anticipated 10-year frequency tributary off-site stormwater runoff based on the future developed condition (using **Table 3-1** of the Technical Standards). An analysis of the emergency routing of stormwater runoff through the subject development must be provided to confirm that the development will not obstruct the free flow of floodwaters from the tributary off-site property in its current condition and after development. In addition, the Drainage System Overflow Design must be completed in accordance with Section M of Chapter 4 to ensure the safe routing of flood waters through the subject development with the tributary off-site property in its current condition and after development.

A. Design Storm Frequencies

1. All channels and swales shall accommodate, at a minimum, peak runoff from a 10-year return frequency storm calculated based on methodology described in Chapter 2.
2. Channels with a carrying capacity of more than thirty (30) Ft^3/s (cfs) at bank-full stage shall be capable of accommodating peak runoff for a 50-year return frequency storm within the drainage easement.
3. Channel facilities functioning as a major drainage system, as defined in **Appendix A**, in the floodway of a stream, must also meet IDNR design standards in addition to the City of Plymouth standards. In case of discrepancy, the most restrictive requirements shall apply.
4. The 10-year storm design flow for residential rear and side lot swales shall not exceed four (4) Ft^3/s (cfs). Unless designed as a post-construction stormwater quality BMP, the maximum length of rear and side lot swales before reaching any inlet must not exceed 400 feet.
5. Regardless of minimum design frequencies stated above, the performance of all parts of drainage system must be checked for the 100-year flow conditions to ensure that all buildings are properly located outside the 100-year flood boundary and that flow paths are confined to designated areas with sufficient easement.
6. A minimum of twenty-five (25) feet from top of the bank on each side of a new channel must be designated on the recorded plat as a Drainage Easement. No landscaping is allowed within any Drainage Easement, except for a minimum twenty-five (25)-foot width of filter strip or suitable grass that must be installed along the top of bank.

B. Determination of Channel Waterway Area Using Manning's Equation:

$$\mathbf{A = Q / V}$$

Where:

A = Waterway Area of Channel in Square Feet (Ft^2)

Q = Discharge in Cubic Feet Per Second (Ft^3/s or cfs)

V = Steady-State Channel Velocity, as Defined by Manning's Equation (See Chapter 4)

C. Backwater Method for Drainage System Analysis

The determination of 100-year water surface elevation along channels and swales must be based on accepted methodology and computer programs designed for this purpose. Computer programs HEC-RAS, HEC-2, and ICPR are preferred programs for conducting such backwater analysis. The use of other computer models must be accepted in advance by the City of Plymouth.

D. Channel Cross-Section and Grade

1. The required channel cross-section and grade are determined by the design capacity, the material in which the channel is to be constructed, and the requirements for maintenance. A minimum depth may be required to provide adequate outlets for subsurface drains, tributary ditches, or streams. The channel grade shall be such that the velocity in the channel is high enough to prevent siltation but low enough to prevent erosion. Velocities less than one and one-half (1.5) feet per second (Ft/s or fps) for the design storm are not acceptable, as siltation will take place and ultimately reduce the channel cross-section area. The maximum permissible velocities in vegetated-lined channels are shown in **Table 5-1**. In addition to existing runoff, the channel design should incorporate increased runoff due to the proposed development.

2. Where depth of design flow is slightly below critical depth, channels shall have freeboard adequate to cope with the effect of hydraulic jumps.

3. Along the streets and roads, the bottom of the ditch should be low enough to install adequately sized driveway culverts without creating "speed bumps". The driveway culvert inverts must be designed to adequately consider upstream and downstream culvert elevations. Use of open channels and swales within the road right of way is only allowed in special circumstances when no other viable option exists. The applicant must consult with the City of Plymouth's Engineer prior to design.

4. Flow of a channel into a closed system is prohibited, unless runoff rate and head loss computations demonstrate the closed conduit to be capable of carrying the 100-year channel flow for developed conditions, either entirely or in combination with a defined overflow channel, with no reduction of velocity.

Table 5-1

Maximum Possible Velocities in Vegetal–Lined Chanels[1]			
Cover	Channel Slope Range (%)	Permissible Velocity[2]	
		Erosion Resistant Soils (Ft/s)	Easily Eroded Soils (Ft/s)
Bermuda Grass	0-5 5-10 Over 10	8 7 6	6 5 4

Bahia Buffalo Grass Kentucky Bluegrass Smooth Brome Blue Grama	0-5 5-10 Over 10	7 6 5	5 4 3
Grass Mixture Reed Canary Grass	0-5[3] 5-10	5 4	4 3
Lespedeza Sericea Weeping Lovegrass Yellow Bluestem Redtop Alfalfa Red Fescue	0-5 5-10[4]	3.4 -	2.5 -
Common Lespedeza[5] Sudangrass[5]	0-5[6]	3.5	2.5

[1] From Soil Conservation Service, SCS TP 61, "Handbook of Channel Design for Soil and Water Conservation".

[2] Use Velocities Exceeding 5 Feet Per Second (Ft/s or fps) Only Where Good Channel Ground Covers and Proper Maintenance Can Be Obtained.

[3] Do Not Use on Slopes Steeper Than 10% Except For Vegetated Side Slopes in Combination with a Stone, Concrete, or Highly Resistant Vegetative Center Section.

[4] Do Not Use on Slopes Steeper Than 5% Except For Vegetated Side Slopes in Combination with a Stone, Concrete, or Highly Resistant Vegetative Center Section.

[5] Annuals Use on Mild Slopes or as Temporary Protection Until Permanent Covers are Established.

[6] Use on Slopes Steeper Than 5% is Not Recommended.

E. Side Slopes

1. Earthen channel side slopes must be no steeper than 3 to 1 (3:1). Flatter slopes may be required to prevent erosion and for ease of maintenance.
2. Where channels will be lined with riprap, concrete, or other acceptable lining method, side slopes must be no steeper than 1.5 to 1 (1.5:1) with adequate provisions made for weep holes.
3. Side slopes steeper than 1.5 to 1 (1.5:1) may be used for lined channels provided that the side lining is designed and constructed as a structural retaining wall with provisions for live and dead load surcharge.
4. When the design discharge produces a depth of greater than three (3) feet in the channel, appropriate safety precautions must be added to the design based on reasonably anticipated safety needs at the site.

F. Channel Stability

1. Characteristics of a stable channel are:

    a. It neither promotes sedimentation nor degrades the channel bottom and sides.
    b. The channel banks do not erode to the extent that the channel cross-section is changed appreciably.
    c. Excessive sediment bars do not develop.
    d. Excessive erosion does not occur around culverts, bridges, outfalls, or elsewhere.
    e. Gullies do not form or enlarge due to the entry of uncontrolled flow to the channel.

2. Channel stability must be determined for an aged condition and the velocity must be based on the design flow or the bankfull flow, whichever is greater, using an "n" value for various channel linings as shown in **Tables 4-1** and **5-1**. In no case is it necessary to check channel stability for discharges greater than that from a 100-year frequency storm.

3. Channel stability must be checked for conditions representing the period immediately after construction. For this stability analysis, the velocity must be calculated for the expected flow from a 10-year frequency storm on the watershed, or the bankfull flow, whichever is smaller, and the "n" value for the newly constructed channels in fine-grained soils and sands may be determined in accordance with the "National Engineering Handbook 5, Supplement B, Soil Conservation Service" and must not exceed 0.025. The allowable velocity in the newly constructed channel may be increased by a maximum of 20% to reflect the effects of vegetation to be established under the following conditions:

    a. The soil and site in which the channel is to be constructed are suitable for rapid establishment and support of erosion controlling vegetation.
    b. Species of erosion controlling vegetation adapted to the area, and proven methods of establishment are shown.
    c. The channel design includes detailed plans for establishment of vegetation on the channel side slopes.

G. Drainage of Swales

All swales must meet the following requirements and the specifications of the City of Plymouth or designee:

1. Minimum swale flow line slopes are 1%. Swales with longitudinal slopes that are flatter than 1% shall consist of six (6)-inch-thick rebar-reinforced concrete. The side slopes of swales must not be steeper than a 3 to 1 (3:1) slope.

2. Maximum swale flow line slopes are 7%.

3. Minimum swale top width must be six (6) feet.

4. All flow shall be confined to the specific easements associated with each rear and side lot swale that are part of the minor drainage system.

5. Unless designed to act as a stormwater quality BMP or lined with concrete, vegetated swales must have a double-walled eight (8)-inch sub-surface drain with a minimum cover of eighteen (18) inches to dry the swales. Typical detail of a swale with sub-surface drain is shown on **Figure 4-3**. Tile lines may be outletted through a drop structure at the ends of the swale or through a standard tile outlet. Before reaching an open channel, the perforated drain shall be connected to ten (10) feet of SDR # 40. Also, cleanout risers are to be installed at the high point as well as end of run.
6. A minimum of twenty (20) feet along the swale (ten (10) feet from each side of the centerline) must be designated on the recorded plat as Drainage Easement.
7. Further guidance regarding this subject may be found in the latest edition of the Indiana Drainage Handbook.

H. Appurtenant Structures

The design of channels will include provisions for operation and maintenance and the proper functioning of all channels, laterals, travelways, and structures associated with the project. Recessed inlets and structures needed for entry of surface and subsurface flow into channels without significant erosion or degradation must be included in the design of channel improvements. The design will also provide for necessary floodgates, water level control devices, and any other appurtenance structure affecting the functioning of the channels and the attainment of the purpose for which they are built.

The effects of channel improvements on existing culverts, bridges, buried cables, pipelines, and inlet structures for surface and subsurface drainage on the channel being improved and laterals thereto must be evaluated to determine the need for modification or replacement. Culverts and bridges which are modified or added as part of channel improvement projects must meet reasonable standards for the type of structure and must have a minimum capacity equal to the design discharge or governmental agency design requirements, whichever is greater.

I. Disposal of Soil

Spoil material resulting from clearing, grubbing, and channel excavation must be disposed of in a manner that will:

1. Minimize overbank wash.
2. Provide for the free flow of water between the channel and floodplain boundary unless the valley routing and water surface profiles are based on continuous dikes being installed.
3. Not hinder the development of travelways for maintenance.
4. Leave the ROW in the best condition feasible, consistent with the project purposes, for productive use by the owner.
5. Improve the aesthetic appearance of the site to the extent feasible.

6. Be approved by the IDNR or US Army Corps of Engineers (whichever is applicable) if deposited in the floodway.

J. Materials

Materials acceptable for use as channel lining are:

1. Grass
2. Revetment Riprap
3. Concrete
4. Hand-Laid Riprap
5. Grouted Riprap
6. Precast Cement Concrete Riprap
7. Gabions
8. Straw, Coconut Mattings, or Other Accepted Material (Only Until Grass is Established)

Other lining materials must be accepted in writing by the City of Plymouth. Materials shall comply with the latest edition of the INDOT, "Standard Specifications".

K. Drainage System Overflow Design

See Chapter 4, Section M.

Chapter 6 | Construction Sites Stormwater Pollution Prevention Standards

The requirements contained in this Chapter are intended to prevent stormwater pollution resulting from soil erosion and sedimentation or from mishandling of solid and hazardous waste. Practices and measures included herein should assure that no foreign substance, (e.g., sediment, construction debris, chemicals) be transported from a site and allowed to enter any drainageway, whether intentionally or accidentally, by machinery, wind, rain, runoff, or other means.

A. Pollutants of Concern During Construction

The major pollutant of concern during construction is sediment. Natural erosion processes are accelerated at a project site by the construction process for a number of reasons, including the loss of surface vegetation and compaction damage to the soil structure itself, resulting in reduced infiltration and increased surface runoff. Clearing and grading operations also expose subsoils which are often poorly suited to re-establish vegetation, leading to longer-term erosion problems.

Problems associated with construction site erosion include transport of pollutants attached to transported sediment; increased turbidity (reduced light) in receiving waters; recreational use impairment. The deposited sediment may pose direct toxicity to wildlife or smother existing spawning areas and habitat. This siltation also reduces the capacity of waterways, resulting in increased flood hazards to the public.

Other pollutants of concern during the construction process are hazardous wastes or hydrocarbons associated with the construction equipment or processes. Examples include concrete washout, paints, solvents, and hydrocarbons from refueling operations. Poor control and handling of toxic construction materials pose an acute (short-term) or chronic (long-term) risk of death to aquatic life, wildlife, and the general public.

B. Erosion and Sediment Control Requirements

<u>General and Implementation Requirements</u>
The following general and implementation requirements apply to all land-disturbing activities and must be considered in the preparation of a Stormwater Pollution Prevention Plan (SWPPP) within the corporate boundaries of the City of Plymouth.

1. Trained Individuals must be utilized for activities associated with the development and design of the SWPPP, stormwater measure implementation, and stormwater project management.
2. Minimize the potential for soil erosion by designing a development that fits the topography and soils of the site. Unless needed to meet requirements and goals of the development, steep slopes should be avoided, and natural contours should be followed.
3. All activities on a site should be conducted in a logical sequence and in accordance with the site's construction phasing plan so that the smallest practical area of land will be exposed for the shortest practical period of time during development.

4. The length and steepness of designed slopes should be minimized to reduce erosion potential. Drainage channels and swales must be designed and adequately protected so that their final gradients and resultant velocities will not cause erosion in the receiving channel or at the outlet. Methods for determining acceptable velocities are included in this Stormwater Technical Standards Manual as well as in the IDEM Storm Water Quality Manual.

5. Sediment-laden water which otherwise would flow from the project site must be managed by appropriate erosion and sediment control measures to minimize sedimentation to receiving waters and adjacent properties as discussed in the IDEM Storm Water Quality Manual and other authoritative sources.

6. Public roadways and roadways not exclusive to construction traffic must be kept cleared of accumulated sediment that is a result of runoff or tracking. The following minimum conditions are applicable:

    a. Clearing of sediment must not include the utilization of mechanical methods that will result in mobilization of dust off the project site or flushing the area with water unless the flushed water is directed to an appropriate sediment control measure.

    b. Cleared sediment must be redistributed or disposed of in a manner that is in accordance with all applicable statutes and regulations.

    c. Sediment discharged or tracked onto roadways that are open to traffic must be removed as directed by a regulatory authority or at a minimum, removed by the end of the same day.

7. Phasing of construction activities must be used, when feasible, to minimize the footprint of disturbed unstable areas.

8. Collected runoff leaving a project site must be either discharged directly into a well-defined, stable receiving channel, or diffused and released to adjacent property without causing an erosion or pollutant problem to the adjacent property owner.

9. Natural features, including wetlands and sinkholes (karst features), must be protected from pollutants associated with stormwater runoff, through appropriate stormwater management and / or treatment measures.

10. Soil compaction is to be minimized, especially in areas where permanent vegetation will be re-established and / or areas that are designated to infiltrate stormwater for the post-construction phase.

11. Topsoil must be preserved, unless infeasible.

12. Existing natural buffers that are adjacent to waters of the state must be preserved to promote infiltration and provide protection of the water resource, unless infeasible. Activities performed by a county drainage board under IC 36-9-27 are excluded.

    a. Natural buffers must be preserved, including the entire buffer bordering and / or surrounding the water resource. Existing buffers:

        i. 50 feet or more in width must be preserved to a minimum of 50 feet.

ii. less than 50 feet in width must be preserved in their entirety. May be enhanced with vegetation that is native and promotes ecological improvement and sustainability.

b. Runoff directed to the natural buffer must be:

i. treated with appropriate erosion and sediment control measures prior to discharging to the buffer.

ii. managed with appropriate runoff control measures to prevent erosion from occurring within the buffer area.

c. Further information regarding buffer requirements is contained in IDEM's "Implementation of Buffers" guidance document.

13. Minimize the generation of dust through dust suppression techniques to prevent deposition into waters of the state and areas located beyond the permitted boundaries of the site as discussed in the IDEM Storm Water Quality Manual and other authoritative sources.

14. A stable construction site access measure must be provided at all points of construction traffic ingress and egress to the project site. Where the selected measure is not effective, an alternative measure or additional controls must be utilized to minimize tracking. Alternative measures may include, but are not limited to, wheel wash systems and rumble strips.

15. During the period of construction activities, all stormwater management measures necessary to meet the requirements of this permit must be maintained. Alternative measures must be selected and implemented, as necessary.

16. Discharge water from dewatering of groundwater from excavations, trenches, foundations, etc. must not be discharged when:

a. Sediment-laden water is not first directed to an appropriate sediment control measure or a series of control measures, as per IDEM Storm Water Quality Manual and other authoritative sources, that minimizes the discharge of the sediment.

b. A visible sheen and / or pollutants are present at a level that requires additional treatment and / or an alternate permit.

17. Appropriate measures must be implemented to eliminate wastes or unused building materials including, but not limited to garbage, debris, cleaning wastes, wastewater, concrete washout, mortar / masonry products, soil stabilizers, lime stabilization materials, and other substances from being carried from a project site by runoff or wind. Wastes and unused building materials must be managed and disposed of in accordance with all applicable statutes and regulations.

18. Construction and domestic waste must be managed to prevent the discharge of pollutants and windblown debris. Surplus plastic or hardened concrete / cementitious materials are not required to be placed in trash receptacles and are considered clean fill that may be

reused, disposed of on-site, or recycled in accordance with applicable state and federal regulations. Management of waste materials may include, but are not limited to:

a. Waste containers (trash receptacles), when selected to manage waste, must be managed to reduce the discharge of pollutants and blowing of debris. Receptacles that are not appropriately managed will require alternatives that include but are not limited to:

    i. A cover (e.g., lid, tarp, plastic sheeting, temporary roof) to minimize exposure of wastes to precipitation or

    ii. A similarly effective method designed to minimize the discharge of pollutants.

b. Waste that is not disposed of in trash receptacles must be removed at the end of the day from the site and disposed of properly.

19. Concrete and cementitious wash water areas, where cementitious fluids are permissible, must be identified for the site and the locations clearly posted. Wash water must be directed into leak-proof containers or leak-proof containment areas which are located and designed to divert runoff away from the measure and sized to prevent the discharge and / or overflow of the cementitious wash water. If not evaporated, wash water must be removed (pumped) for appropriate off-site disposal.

20. Fertilizer applications associated with the stabilization plan for the project must meet the following requirements:

    a. Apply fertilizer at a rate and amount as determined by a soil analysis or in accordance with the Indiana Stormwater Quality Manual or similar guidance documents.

    b. Apply fertilizer at an appropriate time of year for the project location, taking into consideration proximity to a waterbody, and preferably timed to coincide with the period of maximum vegetative uptake and growth.

    c. Avoid applying fertilizer immediately prior to precipitation events that are anticipated to result in stormwater runoff from the application area.

21. Proper storage and handling of materials, such as fuels or hazardous wastes, and spill prevention and clean-up measures must be implemented to minimize the potential for pollutants to contaminate surface or groundwater or degrade soil quality. To meet this requirement:

    a. A spill prevention and response plan, meeting the requirements in 327 IAC 2-6.1, must be completed.

    b. Proper project management and the utilization of appropriate measures including, but not limited to, eliminating a source or the exposure of materials must be completed.

    c. Manage the following activities:

        i. Fueling and maintenance of equipment.

        ii. Washing of equipment and vehicles.

        iii. Storage, handling, and disposal of construction materials, products, and wastes.

iv. Application of pesticides, herbicides, insecticides, and fertilizers.

v. Dispensing and utilization of diesel fuel, oil, hydraulic fluids, other petroleum products, and other chemicals.

vi. Handling and disposal of hazardous wastes, including, but not limited to paints, solvents, petroleum-based products, wood preservatives, additives, curing compounds, and acids.

vii. Washing of applicators and containers used for paint, grout, or other materials.

22. Personnel associated with the project must be informed of the terms and conditions of this permit and the requirements within the SWPPP. The permittee is required to document this process. Information must be provided through written notification, contracts, or other means (i.e., pre-construction meetings) that effectively communicates the provisions and requirements of the permit and SWPPP. Personnel may include, but are not limited to:

    a. General contractors, construction management firms, grading or excavating contractors, and trade industry representatives (i.e., concrete industry) associated with the overall project.

    b. Contractors or individual lot operators that have primary oversight on individual building lots.

    c. Those responsible for the implementation of the SWPPP and the installation, repair, and maintenance of stormwater measures.

    d. Those responsible for the application and storage of treatment chemicals.

    e. Those responsible for administering the self-monitoring program (SMP).

23. A notice must be posted near the main entrance of the project site or at a publicly accessible location. For linear project sites, such as a pipeline or highway, the notice must be placed in a publicly accessible location near the project field office.

    The notice must be maintained in a legible condition and include:

    a. A copy of the completed IDEM NOI or a document, such as the Permit Summary Report & Notice of Sufficiency letter produced by IDEM's online ePortal system, that at a minimum contains the information referenced in Section 6(c)1 of the City of Plymouth's Stormwater Management Ordinance.

    b. The NPDES permit number(s), upon receipt.

    c. The location of the construction plan / SWPPP if the project site does not have an on-site location to store the plan.

24. The use of anionic polymers (cationic polymers are not authorized for use) on the project site are authorized for sediment control provided their use is in conformance with current State of Indiana standards and specifications and the use is identified in the SWPPP. If use of a polymer is not in the SWPPP and is selected at a later date, notification to IDEM and the City of Plymouth is required. An email notification prior to the use of the polymer

to the IDEM Stormwater Program is acceptable. For projects regulated by a MS4, notification must follow the local process for the use of polymers.

25. Restoration and / or clean-up may be required for those areas impacted by sediment or other pollutant discharges. These activities will be performed as directed by the inspecting authority and may require:

    a. Development and submittal of a removal and restoration plan to ensure the methodology chosen will not result in further degradation of the resource.

    b. Permission by a property owner when the restoration activity requires access to a property owned by another entity or individual.

    c. Additional permits prior to initiation of the work.

Stabilization Requirements

The following stabilization requirements apply to all land-disturbing activities:

1. Un-vegetated areas that are left idle or scheduled to be left inactive must be temporarily or permanently stabilized with measures appropriate for the season to minimize erosion potential. To meet this requirement, the following apply:

    a. Stabilization must be initiated by the end of the seventh (7^{th}) day the area is left idle. The stabilization activity must be completed within fourteen (14) days after initiation. Initiation of stabilization includes, but is not limited to, the seeding and / or planting of the exposed area and applying mulch or other temporary surface stabilization methods where appropriate. Areas that are not accessible due to an unexpected and disruptive event that prevents construction activities are not considered idle.

    b. Areas that have been compacted may be excluded from the stabilization requirement when the areas are intended to be impervious surfaces associated with the final land use, provided runoff from the area is directed to appropriate sediment control measures.

2. Final stabilization of a project site is achieved when:

    a. All land-disturbing activities have been completed and a uniform (evenly distributed, without large bare areas) perennial vegetative cover with a density of 70% has been established on all unpaved disturbed areas, and areas not covered by permanent structures, or equivalent permanent stabilization measures have been employed. This requirement does not apply to:

        i. Landscaping that is part of the final project plan is considered stable when the plan has been fully implemented and areas not being vegetated are stable with a non-erosive material and / or product.

        ii. Projects or specific stormwater measures that utilize native vegetation and / or special vegetative plantings that are either required by a water quality permit / authorization or part of the design and functionality of a stormwater measure provided the activity does not pose a threat that will result in off-site sedimentation.

iii. Projects on land used for agricultural purposes when:

a) Stabilization is completed in accordance with the above Stabilization Requirements (in 1. a. and b.) as land-disturbance progresses. Land that is returned to agricultural production must be temporarily or permanently seeded upon completing land-disturbing activities. Stabilization requirements may be waived by the inspecting authority if the project site does not pose a threat of discharging sediment.

b) Disturbed areas, not previously used for agricultural production, such as filter strips, must be returned to their pre-land disturbance use.

b. Specific projects, due to function and / or operation may necessitate that an area remain disturbed. Only the minimum operational area is allowed to remain disturbed. This option primarily applies to off-road recreational commercial operations but may apply to other land use types upon determination by the City of Plymouth or applicable regulating entity.

3. Erosion Control Blanket or other slope protection shall be utilized on any slopes steeper than 4ft horizontal to 1 ft vertical and more than 6 inches in elevation change. No seeding and mulch allowed for these areas.

<u>Design Requirements</u>

The following design requirements apply to all land-disturbing activities and must be considered in the selection, design, and implementation of all stormwater quality and management measures contained in the SWPPP:

1. Sound engineering, agronomic, and scientific principles must be utilized for measures contained in the SWPPP.
2. Appropriate measures must be planned, designed, and installed as part of an erosion and sediment control system and in accordance with the site's construction phasing plan.
3. Stormwater runoff leaving the project site must be discharged in a manner that is consistent with the City of Plymouth's Stormwater Management Ordinance, State, or Federal law.
4. Collected runoff leaving the project site should be directed to an established vegetated area, when feasible and applicable, to increase pollutant removal and maximize stormwater infiltration and then either discharged directly into a well-defined, stable receiving conveyance or diffused and released without causing erosion at the point of discharge.
5. Conveyance systems must be designed taking into consideration both peak flow and total volume and must be adequately protected so that their final gradients and resultant velocities are unlikely to cause erosion at the outlet or in the receiving channel, based on known conditions of the discharge at the time of design to accommodate post-construction conditions.

6. Sediment basins, where feasible, must withdraw water from the surface of the water column (i.e., draw water from the top, where all settling out or separation of material has occurred by gravity, as opposed to drawing water from a pipe at the bottom, which has the "unsettled" water) unless equivalent sediment reduction can be achieved by use of alternative measures. Alternative measures include but are not limited to increasing the basin length to width ratio to 4:1 or greater, implementation of porous baffles, use of flocculants / polymers, and / or phasing of project land disturbance that also incorporates a rapid stabilization program. During freezing conditions, the implementation of alternative withdrawal methods may be utilized.

Monitoring and Management Requirements

A trained individual, acceptable to the City of Plymouth must monitor project construction and stormwater activities. These include:

1. A written evaluation of the entire project site, with the exception of those areas that are considered unsafe. The evaluation must be performed by a trained individual and completed:
    a. Twenty-four (24) hours prior to a qualifying precipitation event or by the end of the next business day following each measurable storm event (excludes accumulated snow events); which is defined as a precipitation accumulation equal to, or greater than, one-half (0.50) inch of rainfall within a twenty-four (24)-hour period. If no rain event occurs within the work week a minimum of one (1) inspection must occur for that week. In the event of multiple qualifying events during the work week, no more than three (3) inspections would be required to meet the self-monitoring commitment.
    b. At a minimum of one (1) time per month for specific areas within the project which are stabilized with permanent vegetative cover at 70% percent density and / or erosion resistant armoring is installed. A reduction to once (1x) per month is also applicable for the entire project site for stabilized common areas, basins, conveyances, outfalls, and inactive building sites. Prior to reducing the monitoring to monthly, records must identify the area and the date the area became eligible for monthly monitoring. Weekly monitoring as identified in (a) above must resume if one or more of the following occurs:
        i. The vegetative cover fails or there is evidence of erosion in the identified area.
        ii. IDEM or the City of Plymouth requires monitoring to resume.
2. A complete written evaluation report which must include:
    a. Name of the individual performing the evaluation, including printed name, title, and signature (electronic signatures are acceptable).
    b. Date of the evaluation.
    c. Amount of precipitation, when the evaluation is conducted after a measurable storm event. Recorded rainfall may be documented utilizing an on-site rain gauge or storm event information from a weather station that is representative of the project location.
    d. Observations of project performance in relation to:

i. Implementation of the SWPPP.

ii. Assessment of existing stormwater measures based on industry standards and maintenance standards to ensure each measure is operational and functioning properly.

iii. Additional measures necessary in the event an existing measure fails or is not present in the landscape

iv. Impacts including, but not limited to, sediment discharges, erosion, discharges that results in bank erosion, and operational activities that have the potential to generate pollutants and unauthorized discharges.

e. Documentation of an actual discharge that is visible during the assessment, the location of the discharge and a visual description of the discharge. The visual description includes, but is not limited to, color (turbidity reading is an option), odor, floatables, settled / suspended solids, foam, oil sheen, and any other visible sign that may be attributed to operations occurring on the project site.

f. Detail of corrective action recommended and / or completed. Corrective action includes, but is not limited to:

i. Repairing, modifying, or replacing any stormwater management measure.

ii. Clean-up and proper disposal of spills, releases, or other deposits.

iii. Remedying a permit violation.

iv. Taking reasonable steps to remediate, minimize or prevent the discharge of pollutants associated with the construction activity until a permanent corrective solution is initiated.

v. Restoring an impacted area and / or removing accumulated sediment, provided appropriate permission and permits are obtained to conduct the activity.

g. A timeline for which the corrective action will occur to remediate the discharge of pollutants. The established corrective action, at a minimum, must be initiated:

i. On the day the deficiency was discovered or when it is not practical to initiate on the discovery date, no later than 48 hours for the repair of a measure or installation of a temporary measure until a new and / or replacement measure is installed as specified in item ii) below.

ii. Within seven (7) days of discovery for the installation of a new (alternative) measure or replacement of an existing measure unless a shorter timeframe is required as part of a regulatory inspection. The City of Plymouth may also allow additional time to take corrective action.

iii. If corrective action cannot be achieved within the timelines outlined in i) or ii) above, a reason for incompletion must be provided and documented, including the anticipated completion date.

h. Documentation of corrective action taken from the previous self-monitoring report.

3. Maintaining the SMP reports at the site or at an easily accessible location (refer to Project Documentation Requirements below).

4. Providing all reports for the project site to the City of Plymouth within 48 hours of a request. Electronic copies are acceptable, provided they are in a format consistent with the paper record.

Project Documentation Requirements

The following project documentation must be developed and maintained:

1. Maintain a project management log that contains:

    a. Information related to all off-site borrow sites, disposal areas, and staging areas, including the location of each activity as it is identified and / or selected.

    b. Information related to all project activities including, but not limited to:
        i. SMP reports.
        ii. Regulatory inspections.
        iii. Responses to a compliance action or enforcement action.
        iv. Records showing the dates of all SWPPP modifications. The records must include the name of the person authorizing each change and a summary of all changes.

2. Ensure the SWPPP and supporting documentation associated with the SMP and project management log are accessible at the project site office or in the possession of on-site individuals with responsibility for the overall project management or associated with the management and operations of construction activities. This information must be provided to the City of Plymouth within 48 hours of a request.

C. Common Control Practices

All erosion control and stormwater pollution prevention measures required to comply with the City of Plymouth's Stormwater Management Ordinance and / or these Technical Standards must meet the design criteria, standards, and specifications similar to, or the same as, those outlined in the Indiana Storm Water Quality Manual, or other comparable and reputable references. Please note that pursuant to IC 13-18-27, MS4-designated entities may not require erosion and sediment control measures / BMPs that are more stringent than the measures / BMPs required by the IDEM general permit.

D. Individual Lot or Parcel Controls

From the time construction on an individual lot begins, until the individual lot is stabilized, the builder must take steps to:

- protect adjacent properties from sedimentation
- prevent mud / sediment from depositing on the street
- protect drainageways from erosion and sedimentation
- prevent sediment laden water from entering storm sewer inlets

This can be accomplished using numerous erosion and sediment control measures. Individual lot permit request form, plot plan review form and detailed requirements, including a standard erosion control plan for individual lots, are provided in **Appendix B**. The standard plan includes perimeter silt fence, stabilized construction entrance, drop inlet protection, stockpile containment, stabilized drainage swales, downspout extensions, temporary seeding and mulching, and permanent vegetation. Every relevant measure should be installed at each individual lot site. Note that construction site discharge must be contained and treated within each individual lot (or a group of individual lots being constructed by one builder) and is not allowed to be discharged offsite.

Construction sequence on individual lots and parcels should be as follows:

1. Clearly delineate areas of trees, shrubs, and vegetation that are to be undisturbed. To prevent root damage, the areas delineated for tree protection should be at least the same diameter as the crown.
2. Install perimeter silt fence at construction limits. Position the fence to intercept runoff prior to entering drainage swales.
3. Avoid disturbing drainage swales if vegetation is established. If drainage swales are bare, install erosion control blankets or sod to immediately stabilize.
4. Install drop inlet protection for all inlets on the property.
5. Install stable construction entrance that extends from the street to the building pad.
6. Perform primary grading operations.
7. Contain erosion from any soil stockpiles created on-site with an appropriate sediment control measure around the base.
8. Establish temporary seeding and straw mulch on disturbed areas.
9. Construct the home and install utilities.
10. Install downspout extenders once the roof and gutters have been constructed. Extenders should outlet to a stabilized area.
11. Re-seed any areas disturbed by construction and utilities installation with temporary seed mix within three (3) days of completion of disturbance.
12. Grade the site to final elevations.
13. Install permanent seeding or sod.

All erosion and sediment control measures must be properly maintained throughout construction. Temporary and permanent seeding should be watered as needed until established. For further information on individual lot and parcel erosion and sediment control, see the “Individual Lot Erosion and Sediment Control Plan and Certification” form and other resources in **Appendix B** as well as the latest information posted on IDEM’s website.

Chapter 7 | Post-Construction Stormwater Quality Management Standards

A. Introduction

It is recognized that developed areas, as compared to undeveloped areas, generally have increased imperviousness, decreased infiltration rates, increased runoff rates, and increased concentrations of pollutants such as fertilizers, herbicides, greases, oil, salts, and other pollutants. As new development and re-development continues within areas subject to City of Plymouth jurisdiction, measures must be taken to intercept and filter pollutants from stormwater runoff prior to reaching regional creeks, streams, and rivers. Through the use of appropriate BMPs, stormwater runoff will be filtered, and harmful amounts of sediment, nutrients, and contaminants will be removed.

The information provided in this Chapter establishes performance criteria for stormwater quality management and procedures to be followed when preparing a BMP plan for compliance. Post-construction BMPs must be sized to treat the water quality volume (WQV) and for flow-through BMPs the water quality discharge rate (QWQ), as appropriate. The methodology for calculating the WQV and QWQ values is provided in Chapter 8.

B. Post-Construction BMPs Performance Criteria

The City of Plymouth has established a minimum standard that the measurement of the effectiveness of the control of post-construction stormwater runoff quality will be based on removal of floatables in stormwater runoff and treatment, to the maximum extent practicable, of all major pollutants of concern expected for the proposed land use and / or those identified in the SWPPP for the site (including, if applicable, those pollutants found to be the cause of the receiving stream to be listed in IDEM 303(d) list) for the first (1st) inch of rainfall at the site. The above-noted "maximum extent practicable" criterion is subject to a minimum of 80% removal of Total Suspended Solids (TSS). These requirements are adopted as the basis of the City of Plymouth stormwater quality management program for all areas of its jurisdiction.

For the purpose of these Technical Standards, the control of post-construction stormwater runoff quality is assumed satisfactory when the appropriate number of pre-approved structural BMPs, tiered in accordance to the total site disturbed area as shown below, are designed, installed, and operated in accordance with the fact sheets provided within the appendices provided in the Indiana Office of Community and Rural Affairs (OCRA) Green Infrastructure Curriculum and Training web resources (Appendix C – BMP Fact Sheets.pdf (in.gov)).

Total Property/Development Area	Post-Construction BMP Requirement[1]
0 to less than 1 acre	Water Quality BMP Not Required
At least 1 acre	At least 1 Water Quality BMP

[1]These BMPs are in addition to any pre-treatment that may be required for hot spots.

C. Pollutants of Concern After Construction Stabilization

There are three (3) major sources of pollutants for a stabilized construction site:

- Deposition of atmospheric material (including wind-eroded material and dust)
- General urban pollution (thermal pollution, litter)
- Pollutants associated with specific land uses

It should be noted that some pollutants accumulate on impervious surfaces. This accumulated material is then subject to being washed into watercourses during storm events. It is for this reason that fish kills often occur during a rain event with a substantial previously rainless period.

Pollutants of concern include:

- Sediment is the major pollutant of concern during active construction. Natural erosion processes are accelerated at a project site by the construction process for a number of reasons, including the loss of surface vegetation and compaction damage to the soil structure itself, resulting in reduced infiltration and increased surface runoff. After the construction is completed, other chemicals that are released to surface waters from industrial and municipal discharges and polluted runoff from urban and agricultural areas continue to accumulate to harmful levels in sediments.

- Toxic chemicals from illegal dumping and poor storage and handling of materials. Industrial sites pose the most highly variable source of this pollution due to the dependency of the specific process to the resulting pollution amounts and constituents. As during construction, these chemicals can pose acute (short-term) or chronic (long-term) risk to aquatic life, wildlife, and the general public.
- Bacteria can originate from illicit sanitary connections to storm sewer systems, combined sewers, leaking septic systems, wildlife, domestic animal waste, and the like. Bacteria pathogens pose a direct health risk to humans and aquatic life.

- Nutrients can be released from leaking septic systems or applied in the form of fertilizers. Golf courses, manicured landscapes, and agricultural sources are the primary land uses associated with excess fertilization. Excessive nutrients in the local ecosystem are the source of algal blooms in ponds and lakes. These excessive nutrients also lead to acceleration of the eutrophication process, reducing the usable lifespan of these water bodies. Nitrogen and phosphorous are the primary nutrients of concern.

- Oxygen demand can be impacted by chemicals transported on sediment, by nutrients, and other pollutants (such as toxic chemicals). Reduced levels of oxygen impair or destroy aquatic life.

- Oils and hydrocarbons accumulate in streets from vehicles. They can also be associated with fueling stations and illicit dumping activities. Oils and hydrocarbons pose health risk to both aquatic and human health.

- Litter, including floatables, can result in a threat to aquatic life. The aesthetic impact can also reduce the quality of recreational use.

- Metals can be associated with vehicular activity (including certain brake dusts), buildings, construction material storage, and industrial activities. Metals are often toxic to aquatic life and threaten human health.

- Chlorides (salts) are historically associated with deicing activities. Chlorides are toxic to native aquatic life (verses saltwater aquatic life). Communities should consider a combination of cinders or sand to replace or supplement their deicing activities with chlorides. In addition, chloride stockpiles should remain covered.

- Thermal effects can be introduced by the removal of shade provided by riparian trees, as well as impervious channel linings, such as concrete, which release stored heat to water passing over them. Other sources of elevated temperature include effluent from power plant and industrial activities. Thermal pollution can threaten aquatic habitat, including fish species and beneficial water insects. Of particular concern are salmonid streams, due to the effect of thermal pollution on spawning for this particular species.

D. Water Quality Characteristics by Land Use

Direct water quality sampling is not generally required at this time by IDEM. However, water quality characteristics are strongly tied to land use. For the purpose of these Technical Standards, all proposed developments and re-developments must be assumed to involve increased levels of floatables, TSS, TP, TN, and metals. Additional pollutants may also be expected at certain types of developments and specific sites, as identified in the SWPPP for the site (including, if applicable, those pollutants found to be the cause of the receiving stream to be listed in IDEM 303(d) list).

E. Conventional Approach Procedure

Provide BMPs to Address Water Quality Management

The City of Plymouth has designated a number of pre-approved structural BMP methods (listed in **Table 7-1** for Conventional Approach) to be used alone or in combination to achieve the stormwater quality performance criteria noted in Section B of this Chapter for runoff generated from up to first (1^{st}) inch of rainfall on the entire site (excluding any protected undisturbed areas acceptable to the City of Plymouth). Details regarding the applicability and design of these pre-approved BMPs, including the effectiveness of these BMPs in treating pollutants of concern (including, if applicable, those pollutants found to be the cause of the receiving stream to be listed in the IDEM 303(d) list), are contained within fact sheets presented within the appendices provided in the Indiana OCRA Green Infrastructure Curriculum and Training web resources (https://www.in.gov/ocra/2367.htm). Additional information on recommended plant lists and recommended materials used for construction of stormwater BMPs are also provided in those appendices.

Innovative BMPs, including but not limited to, BMPs not previously accepted by the City of Plymouth must be certified by a Professional Engineer licensed in the State of Indiana and approved through the City of Plymouth. ASTM standard methods must be followed when verifying performance of new measures. New BMPs, individually or in combination, must meet the performance criteria noted in Section B of this Chapter, including the capture and removal of floatables. All innovative BMPs must have a low to medium maintenance requirement to be considered by the City of Plymouth. Testing to establish the pollutant removal rate must be conducted by an independent testing facility, not the

BMP manufacturer. The accepted design flow rate for a Water Quality Device is the flow value at which the claimed removal rate for TSS is equaled or exceeded based on the unit's efficiency curve (flow rate versus removal rate graph).

Note that a single BMP measure may not be adequate to achieve the water quality requirements (as noted above) for a project. It is for this reason that a "treatment train", a number of BMPs in series, is often required for a project. The pollutant removal efficiency of a number of BMPs in series may be determined from the following equation:

$$\mathbf{E_{series} = 1 - [(1-E_1)\ (1-E_2)\ (1-E_3)\ldots]}$$

Where,

E_{series} = Removal Efficiency of the BMP Series Combined (in Decimal Form)
E_1, E_2, and E_3 = Removal Efficiency of Units 1, 2, and 3, Respectively (in Decimal Form)

Table 7-1

Pre-Approved Post-Construction BMPs		
BMP[A]	Typical % Removal Efficiency[B]	Maintenance Easement Requirements
	TSS	
Bioretention	90[C]	25 Feet Wide Along the Perimeter
Constructed Wetland	67[C]	25 Feet Wide Along the Outer Perimeter of Forebay & 30 Feet Wide Along Centerline of Outlet
Underground Detention	70	20 Feet Wide Strip from Access Easement to Tank's Access Shaft & 30 Feet Wide Along Centerline of Inlet and Outlet
Extended Detention / Dry Pond	72	25 Feet Wide Along the Outer Perimeter of Forebay & 30 Feet Wide Along Centerline of Outlet
Infiltration Basin (Including Retention Ponds with Pretreatment)	90[C]	25 Feet Wide Along the Perimeter
Infiltration Trench	90[C]	25 Feet Wide Along the Perimeter
Constructed (Sand) Filter	70[C]	25 Feet Wide Along the Perimeter
Water Quality Device	Varies[D]	20 Feet Wide Strip from Access Easement to Chamber's Access Shaft
Vegetated Filter Strip	78[C]	25 Feet Wide Along the Length on The Pavement Side
Vegetated Swale	81[C]	25 Feet Wide Along the Top of Bank on One Side
Wet Ponds / Retention Basin	80	25 Feet Wide Along the Outer Perimeter of Forebay & 30 Feet Wide Along Centerline of Outlet

Notes:

A. Detailed specifications for these BMPs are provided in the fact sheets contained in the appendices provided in the Indiana OCRA Green Infrastructure Curriculum and Training web resources (Appendix C – BMP Fact Sheets.pdf (in.gov)) and are provided on the City of Plymouth's website, for convenience. Approval of use shall be at the discretion of the City of Plymouth's Engineering Department.

B. Removal rates shown are based on typical results. Unless otherwise shown, data extracted from various data sources. These rates are also dependent on proper design, installation, and maintenance. The ultimate responsibility for determining whether additional measures must be taken to meet the City of Plymouth Stormwater Management Ordinance requirements for site-specific conditions rests with the applicant.

C. IDEM Stormwater Quality Manual, 2007

D. The removal rate for this category varies widely between various models and manufacturers. The acceptable treatment rate for these devices shall be based on that currently certified by the New Jersey Department of Environmental Protection (NJDEP). Further details on acceptable Water Quality Devices and their treatment rates are provided in Chapter 8.

F. Special Provisions For “Hot Spot” Land Uses

For all those projects involving land uses considered to be high pollutant producers or “hot spots” (see **Table 8-2** e.g., vehicle service and maintenance facilities, vehicle salvage yards and recycling facilities, vehicle and equipment cleaning facilities, fleet storage areas for buses, trucks, etc., industrial / commercial or any hazardous waste storage areas or areas that generate such wastes, industrial sites, restaurants and convenience stores, any activity involving chemical mixing or loading / unloading, outdoor liquid container storage, public works storage areas, commercial container nurseries, and some high traffic retail uses characterized by frequent vehicle turnover), additional water quality requirements may be imposed by the City of Plymouth in addition to those included in water quality criteria in order to remove potential pollutant loadings from entering either groundwater or surface water systems. These pre-treatment options are included in **Table 7-2** and **Table 7-3**.

Table 7-2

Pre-Treatment Options for Stormwater Hot Spots	
Stormwater Hot Spots	Minimum Pre-Treatment Options
Vehicle Maintenance and Repair Facilities	A, E, F, G
Vehicle Fueling Stations	A, D, G
Drive-Through Restaurants, Pharmacies, Convenience Stores	B, C, D, I, K
Outdoor Chemical Mixing or Handling	G, H
Outdoor Storage of Liquids	G
Commercial Nursery Operations	I, J, L
Other Uses or Activities Designated by Appropriate Authority	As Required

Table 7-3

Minimum Pre-Treatment Options	
A	Oil / Water Separators / Hydrodynamic Separators
B	Sediment Traps / Catch Basin Sumps[1]
C	Trash / Debris Collectors in Catch Basins
D	Water Quality Inserts for Inlets (Inspected and Cleaned After Every Significant Rainfall Event)
E	Use of Drip Pans and / or Dry Sweep Material Under Vehicles / Equipment
F	Use of Absorbent Devices to Reduce Liquid Releases
G	Spill Prevention and Response Program
H	Diversion of Stormwater Away from Potential Contamination Areas
I	Vegetated Swales / Filter Strips
J	Constructed Wetlands
K	Stormwater Filters (Sand, Peat, Compost, etc.)
L	Stormwater Collection and Reuse (Especially for Irrigation)
M	BMPs that are a part of a SWPPP Under a NPDES Permit

1. Sumps (1 foot) are required in all storm catch basins and manholes; drywells are allowed only if the bottom is designed 1 foot higher than the proposed site groundwater elevation.

G. Construction Sequencing Considerations

BMPs noted in this Chapter refer to post-construction BMPs, which continue to treat stormwater after construction has been completed and the site has been stabilized. Installing certain BMPs, such as bioretention areas and sand filters, prior to stabilization can cause failure of the measure due to clogging from sediment. If such BMPs are installed prior to site stabilization, they should be protected by traditional erosion control measures.

In those instances, the construction sequence must require that the pond is cleaned out with pertinent elevations and storage and treatment capacities reestablished as noted in the accepted stormwater management plan.

H. Inspection and Maintenance Requirements

Inspection checklists for various types of BMPs are provided within the appendices provided in the Indiana OCRA Green Infrastructure Curriculum and Training web resources (https://www.in.gov/ocra/2367.htm). A sample of the required Stormwater Management Maintenance Agreement is provided in **Appendix B**. This agreement will need to be further customized, signed, notarized, and recorded so that it can be a part of the property's deed. The maintenance agreement and the O&M Manual shall be transferred to the new owner as the ownership and BMP maintenance responsibilities change hands.

Installation and maintenance of appropriate signage of all green infrastructure BMP sites is also recommended both to alert the maintenance crew and to educate the public.

Chapter 8 | Methodology for Determination of Required Sizing of BMPs

A. Introduction

This Chapter describes the acceptable methods for calculating WQV and Flow-Through BMP Flow Rate associated with Conventional Stormwater Management Approach. Structural Water Quality treatment is achieved by treating the first (1^{st}) inch of rainfall, either through retention / detention BMPs or by flow-through BMPs. Detention / Retention BMPs impound (pond) the runoff to be treated, while flow-through BMPs treat the runoff through some form of filtration process.

B. Retention / Detention BMP Sizing

1. WQV

Water Quality Detention BMPs must be designed to store the WQV for treatment. The WQV is the storage needed to capture and treat the runoff from the first (1^{st}) inch of rainfall. The WQV is equivalent to 1 inch of rainfall multiplied by the volumetric runoff coefficient (Rv) multiplied by the site area.

Methodology for calculating of WQV:

$$\mathbf{WQV = (P)\ (Rv)\ (A) / 12}$$

Where:

WQV= Water Quality Volume for Each Site's Outlet (Acre-Feet)
P = 1 Inch
Rv = Volumetric Runoff Coefficient
A = Site Area in Acres

The volumetric runoff coefficient is a measure of imperviousness for the contributing area, and is calculated as:

$$\mathbf{Rv = 0.05 + 0.009(I)}$$

Where:

I = Percent Impervious Cover

For example, a proposed commercial site will be designed to drain to three (3) different outlets, with the following drainage areas and impervious percentages:

Subarea ID	On-site Contributing Area (Acres)	Impervious Area %	Off-Site Contributing Area (Acres)
A	7.5	80	0.0
B	4.3	75	0.0
C	6.0	77	0.0

Calculating the volumetric runoff coefficient for subareas A, B and C yields:

Rv (Subarea A) = 0.05+0.009(80) = 0.77
Rv (Subarea B) = 0.05+0.009(75) = 0.73
Rv (Subarea C) = 0.05+0.009(77) = 0.74

The WQV for these three (3) areas are then calculated as:

WQV (Subarea A) = (1)(0.77)(7.5)/12 = 0.47 acre-feet
WQV (Subarea B) = (1)(0.73)(4.3)/12 = 0.26 acre-feet
WQV (Subarea C) = (1)(0.74)(6.0)/12 = 0.37 acre-feet

Note that this example assumed no offsite sources of discharge through the water quality BMPs. If there were significant sources of off-site runoff (sometimes called "run-on" for upstream areas draining to the site), the designer would have the option of bypassing off-site runoff around the on-site systems, or the detention BMP should be sized to treat the water quality volume for the off-site sources.

C. Flow-Through BMP Sizing

Flow-through BMPs are designed to treat runoff at a calculated water quality treatment flow rate through the system. Examples of flow through BMPs include catch basin inserts, sand filters, and grassed channels. Another flow through BMP is a manufactured treatment device such as a hydrodynamic separator, manufactured infiltration chambers, or other similar type of device discussed in the Water Quality Devices Fact Sheet (https://www.in.gov/ocra/2367.htm)

1. Requirements for Manufactured Treatment Devices

Stormwater Manufactured Treatment Devices (MTD), also known as Hydrodynamic separators are proprietary, and usually include a pollutant-water separation component. The MTD should be sized to treat flows up to, and including, the Water Quality Treatment Rate (QWQ) calculated for each project site outlet. To be acceptable, the MTD must meet the following criteria:

- The MTD must be offline and located in an area that is suitable to be maintained easily and regularly.
- The MTD must provide complete and unobstructed access to the entire bottom of the system from grade level, if applicable, for ease of maintenance.
- The MTD, or the treatment train (if applicable) that includes the MTD as one of its components, must have the ability to capture or skim pollutants including but not limited to: floating oils / immiscible materials.
- The MTD, or the treatment train (if applicable) that includes the MTD as one of its components, must have the ability to capture both floating and suspended solid material (trash, organic material, etc.) and other pollutants.
- The MTD must be a manufactured system currently certified by the New Jersey Department of Environmental Protection (NJDEP). A list of NJDEP- MTDs certified for 50% and 80% TSS removal are provided in a table located at: http://www.nj.gov/dep/stormwater/treatment.html

To obtain the maximum flow rate for various models of a MTD that is listed in the NJDEP-certified list, the latest verification report from NJCAT Verification Database must be used.

In summary, the following steps should be used to determine whether a proposed MTD unit is certified and to determine the accepted maximum flow rate for that unit.

1. Submittal for review and approval at the discretion of the City Engineering Department.
2. Design criteria: Flow rate (Q) determined using Rational Method and I = 1 in/hr, or SCS hydrograph method using 1 in design depth for type 2 storm.
3. Submittal of Manufacturer Recommended Cleaning/Maintenance schedule and practices/methods.
4. 50% TSS removal for operation with other treatment BMPs, 80% for stand-alone use.
5. MTD allowed for Offline use only.

2. Calculating the Required Treatment Flow Rate (QWQ)

The following procedure should be used to estimate peak discharges for flow through BMPs (adopted from Maryland, 2000). It relies on the volume of runoff computed using the Small Storm Hydrology Method (Pitt, 1994) and utilizes the NRCS, TR-55 Method.

Using the WQV methodology, a corresponding Curve Number (CNwq) is computed utilizing the following equation:

$$CNwq = \frac{1000}{\left[10 + 5P + 10Qa - 10\sqrt{Qa^2 + 1.25QaP}\right]}$$

Where:

CNwq = Curve Number for Water Quality Storm Event
P = 1Inch (Rainfall for Water Quality Storm Event)
Qa = Runoff Volume, in Inches = 1 Inch × Rv = Rv (Inches)
Rv = Volumetric Runoff Coefficient (See Previous Section)

Due to the complexity of the above equation, the CNwq is represented as a function of percent imperviousness in **Figure 8-1**.

The CNwq is then used in conjunction with the standard calculated time-of-concentration, Tc, and drainage area as the basis input for TR-55 calculations. Using the SCS Type 2 distribution for 1 inch of rainfall in 24-hours, the water quality treatment rate, QWQ, can then be calculated.

References

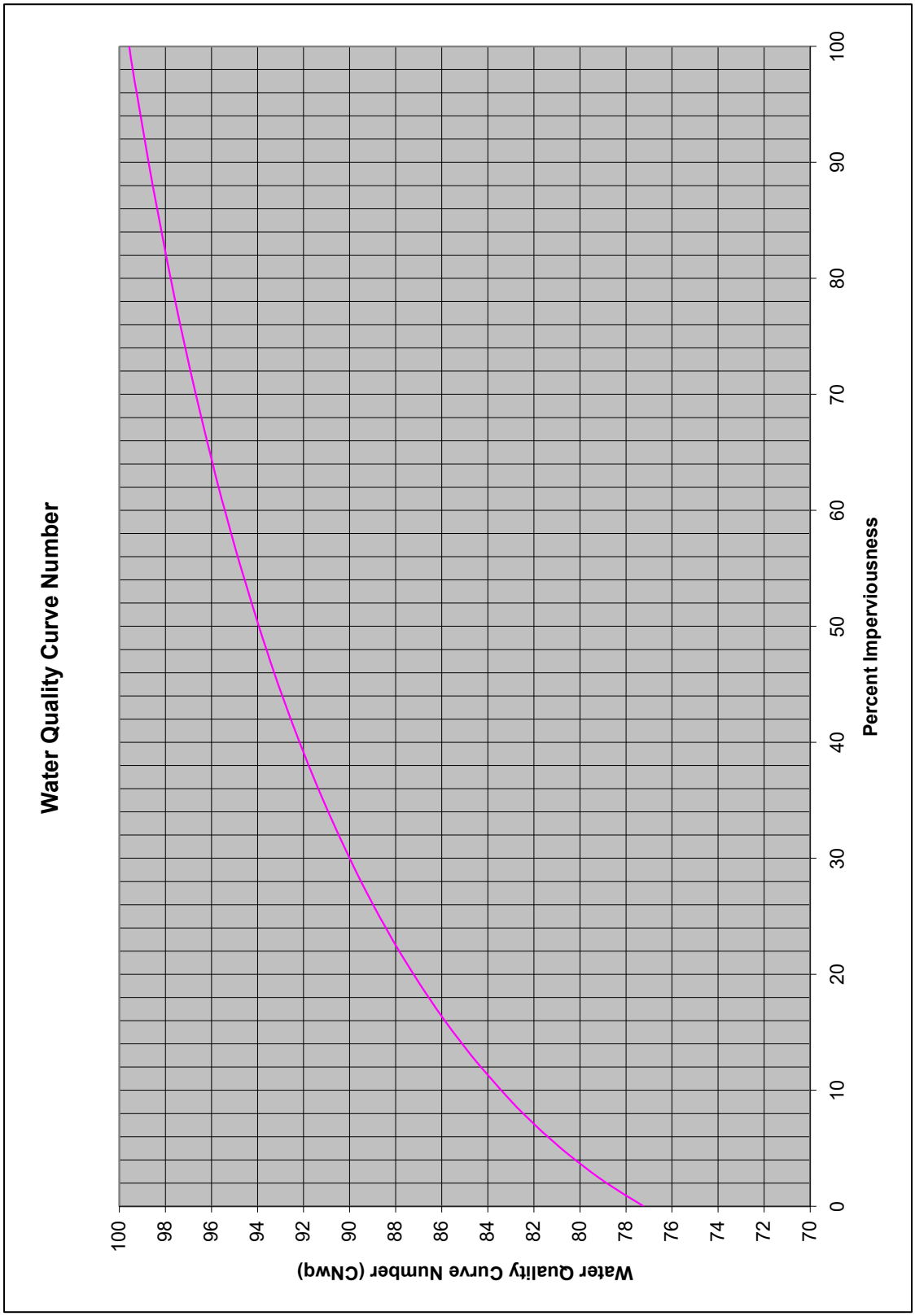
1. Maryland Stormwater Design Manual, Volume II, Appendix D.10, 2000

2. Pitt, R., 1994, Small Storm Hydrology. University of Alabama - Birmingham. Unpublished manuscript. Presented at design of stormwater quality management practices. Madison, WI, May 17-19, 1994.

3. Schueler, T.R. and R.A. Claytor, 1996, Design of Stormwater Filter Systems. Center for Watershed Protection, Silver Spring, MD.

4. United States Department of Agriculture (USDA), 1986. Urban Hydrology for Small Watersheds. Soil Conservation Service, Engineering Division. Technical Release 55 (TR-55).

Figure 8-1
Curve Number (CNwq) Calculation for Water Quality Storm Event



Chapter 9 | Lot / Building Grading and Drainage Standards

A. Grading and Building Pad Elevations

Minimum Federal and State Requirements

For all structures located in the Special Flood Hazards Area (SFHA) as shown on the FEMA maps and / or that mapped by the IDNR based on Best Available Data, the lowest floor elevation, including basement, must be at or above the flood protection grade (FPG) and therefore have a minimum of 2 feet of freeboard above the 100-year flood elevation. Additional requirements for buildings within SFHA may be contained in the City of Plymouth's Flood Hazard Areas Ordinance.

Additional Local Requirements

FPG for all buildings located within or adjacent to SFHA shall be shown on the secondary plat.

For all structures located outside SFHA or an IDNR designated floodplains that are subject to flooding from a detention / retention pond, the lowest adjacent grade (LAG) of all residential, commercial, or industrial buildings must have a minimum of 2 feet of freeboard above the 100-year flood elevation or the emergency overflow weir elevation, whichever is higher.

For all structures located outside SFHA or an IDNR designated floodplains that are subject to flooding from a stream or an open ditch or other waterway (an area along a stream with no floodplain designation or an area adjacent to a designated floodplain with ground elevation below 100-year flood elevation plus 2 feet), the LAG of all residential, commercial, or industrial buildings must have a minimum of 2 feet of freeboard above the 100-year flood elevation of that open ditch or waterway.

For all structures fronting a flooding source other than a swale or an emergency flood route, the floor of any basements or crawl spaces (if provided) must be a minimum of 1 foot above the normal pool level (if pond) or the 2-year flood level (if a stream or an open ditch). In addition, special considerations, based on detailed geotechnical analysis, should be made prior to considering placement of any basement below the 100-year flood elevation of an adjacent flooding source or pond.

For all structures adjacent to an emergency flood route (also referred to as overflow path / ponding areas), the minimum adjacent grade of the portion of the structure (the ground elevation next to the building after construction is completed that sits adjacent to the emergency flood route or may be subject to flooding by the emergency flood route) must be a minimum of 1 foot above the estimated 100-year elevation of the emergency flood route assuming that all stormwater inlets and pipes are fully clogged, with no discharge into the storm sewer system. The building adjacent grade requirements (including default elevations above the overflow route bottom) for buildings adjacent to overflow path / ponding areas are further discussed in Chapter 4 of this Manual.

For all structures adjacent to a road, the building's lowest entry elevation that is adjacent to and facing a road must be a minimum of 2 feet above the road elevation (elevation of the gutter at the center of the lot) so that the road drainage is not directed against the building.

There must be a positive slope drainage away from the building. Finished floor elevation or the lowest building entry elevation must be no less than 6 inches above finished grade around the building.

In addition to any other provisions in the City of Plymouth Stormwater Management Ordinance, Flood Hazard Areas Ordinance, or these Technical Standards, no buildings can be placed within 25 feet from the top of the bank of any existing or proposed stream, drain, or watercourse, regardless of the contributing drainage area or the bank height.

B. Lot Drainage

All lots must be laid out so as to provide drainage away from all buildings, and individual lot drainage must be coordinated with the general stormwater drainage pattern for the subdivision. Drainage must be designed so as to avoid the concentration of stormwater runoff from a lot onto adjacent lots. Each lot owner must maintain the lot grade, as it relates to stormwater drainage, in compliance with the approved construction plans.

No part of the lot area of any lot may contain land that is utilized as retention or detention facility or drainage pond, contains a watercourse, or is within a floodway. Where a watercourse separates the buildable area of the lot from the street by which it has access, provisions must be made for the installation of a culvert or other appropriate structure, as approved by the City of Plymouth or designee. If a subdivision contains an existing or to be developed waterbody, watercourse, or portion thereof, appropriate documentary assurances acceptable to the City of Plymouth must be provided for the maintenance of such waterbody or watercourse. It must be the property owners' responsibility to maintain the natural features on their lots and to take preventive measures against any and all erosion and / or deterioration of natural or manmade features on their lots.

C. Acceptable Outlet and Adjoining Property Impacts Policies

Design and construction of the stormwater facility must provide for the discharge of the stormwater runoff from off-site land areas as well as the stormwater from the area being developed (on-site land areas) to an acceptable outlet(s) (as determined by the City of Plymouth) having capacity to receive upstream (off-site) and on-site drainage. A roadside ditch is generally not considered an adequate outlet. The flow path from the development outfall(s) to a regulated drain, a City of Plymouth storm drain, or natural watercourse (as determined or approved by the City of Plymouth) must be provided on an exhibit that includes topographic information. Any existing field tile encountered during the construction must also be incorporated into the proposed stormwater drainage system or tied to an acceptable outlet.

If an adequate outlet is not located on-site, then off-site drainage improvements may be required. Those improvements may include, but are not limited to, extending storm sewers, clearing, dredging and / or removal of obstructions to open drains or natural water courses, and the removal or replacement of undersized culvert pipes as required by the City of Plymouth.

Chapter 10 | River Corridor, Bluffs, and Floodplain Protection Standards

A. Development Within Floodways and Fluvial Erosion Hazard Corridors

Regulatory floodway and Fluvial Erosion Hazard (FEH) Corridor along regulated drains or any natural stream watercourse are considered Impact Drainage Areas and must be preserved to the extent possible to minimize flooding and erosion impacts within the stream system.

Basis for Establishing Floodways

No policy at this time.

Basis for Establishing Fluvial Erosion Hazard (FEH) Corridors

No policy at this time.

Special Requirements within Floodways

No disturbance (fill or excavation) associated with a new development or redevelopment is permitted within regulatory floodways, unless such modifications are part of a stream-wide restoration plan or a watershed master plan. This restriction does not apply to stormwater or tile outfalls.

B. Additional Consideration for Development Within or Adjacent To Bluff Zones And Steep Slopes

No policy at this time.

C. Floodplain Avoidance / No Net Loss Floodplain Storage Policy

Floodplains exist adjacent to all natural and man-made streams, regardless of contributing drainage area or whether they have been previously identified or mapped. Due to potential impacts of floodplain loss on peak flows in streams and on the environment, floodplains are considered Impact Drainage Areas and disturbance to these areas should be avoided. When the avoidance of floodplain disturbance is not practical, the natural functions of the floodplain should be preserved to the extent possible.

In an attempt to strike a balance between the legitimate need for economic development within the City of Plymouth jurisdictional area and the ever-increasing importance of and the need to preserve the natural functions of floodplains to the extent possible (especially given the ongoing and projected climate change impacts), compensatory excavation equivalent to the floodplain storage lost shall be required for all activities within area inundated by the 100-year flood elevation plus 2 feet along floodplain of streams located in the City of Plymouth where drainage area of the stream is equal to or larger than one square mile (mi^2). The City of

Plymouth may alter the compensation ratio, based on extenuating circumstances, for a specific project.

General Requirements

Note that by definition, compensatory storage is the replacement of the existing floodplain and, in rare exceptions, the floodway storage lost due to fill. Compensatory storage is required when a portion of the floodplain is filled, occupied by a structure, or when as a result of a project a change in the channel hydraulics occurs that reduces the existing available floodplain storage.

Compensatory storage must:

- Be provided regardless of whether the flooding source is mapped or whether flood elevations are published or not. When flood elevations are not available for a flooding source that has a drainage area equal to or larger than 1 square mile (mi^2, 640 acres), the applicant is to determine the 10-year and 100-year flood elevations at the site and get them approved by the IDNR prior to use for floodplain compensation calculations.
- Equal at least 1 times (1x) the volume of flood storage lost below the 10-year elevation and storage lost between 10-year and 100-year plus 2 feet flood elevations;
- Be operational prior to placement of fill, structures, or other materials temporarily or permanently placed in the regulatory floodplain;
- Be provided in the immediate vicinity of the flood storage lost, where practical;
- Be provided in such a way to mimic as close as possible the function provided by the lost floodplain storage. If the floodplain storage is to be lost outside the active flow conveyance path, then it must be compensated for outside the flow conveyance path (e.g., a flood conveyance shelf / 2-stage ditch, while improving conveyance and erosion, is not an appropriate compensation for floodplain storage lost in the floodway fringe area).
- Be provided in addition to the site retention / detention volume; and
- Drain freely and openly to the waterway.

Compensatory storage is also required to be provided incrementally such that:

- All floodplain storage / conveyance capacity lost within the floodway must be compensated for within the floodway;
- All floodplain storage lost within the floodway fringe must be compensated for within the floodway fringe;
- All floodplain storage lost below the existing 10-year flood elevation must be compensated for below the proposed 10-year flood elevation; and
- All floodplain storage lost above the existing 10-year flood elevation must be compensated for above the proposed 10-year flood elevation.

Note that compensatory storage is required for activities in the regulatory floodplain. There is no threshold to compensatory storage; any volume of fill requires compensatory storage be provided. However, the compensatory storage requirement does not apply to specific activities in the regulatory floodplain, such as the floodproofing of an existing building, where the floodproofing measures such as berms or floodwalls are within 10 feet of the building, or crossing improvements, where artificially created storage is lost due to a reduction in head loss.

Computing Compensatory Storage

Computations must show 1 times (1x) compensation for floodplain storage volume lost for 10-year and 100-year storm plus 2 feet events. Storage lost between the existing ground and the existing 10-year flood elevation must be compensated by providing 1 times (1x) the amount lost and be placed between the existing ground elevation and the proposed 10-year floodplain elevation. Storage lost between the existing 10-year and the existing 100-year plus 2 feet elevation must be compensated by providing 1 times (1x) the amount lost and be placed between the proposed 10-year elevation and proposed 100-year plus 2 feet elevation.

When preparing a grading plan, thought should be given to how compensatory storage will be quantified. The most common methodology is the use of cross-sections and the "average end area method". The following requirements should be followed when preparing cross sections:

1. Prepare a detailed topographic survey tied to North American Vertical Datum of 1988 and the City of Plymouth Survey Control Network benchmarks.

2. Locate cross-sections parallel to each other and perpendicular to a reference line, often times a property line or fence line. Cross-sections used in a hydraulic model are always perpendicular to flood flows, and not always parallel to each other. Therefore, these are often not suitable for computing flood fringe compensatory storage volumes.

3. Plot cross sections at a standard engineering scale so as to allow the reviewer to verify areas. Horizontal scale should be a maximum of 1" = 50' and vertical scale should be a maximum of 1" =5', or as approved by the City of Plymouth.

4. Show existing grades, proposed grades, existing and proposed 10-year flood elevations, existing and proposed 100-year flood elevations, normal water level, a reference line, and floodway limits on the cross sections on the plans.

5. Locate cross-sections no more than 150 feet apart, with a minimum of three (3) cross-sections per cut / fill area, or as necessary to accurately quantify cuts and fills.

6. Locate cross sections to pick up critical features such as berms, ditches, and existing and proposed structures.

7. Each cross-section should be numbered or lettered and referenced on the plans.

This information is then utilized to compute the areas of cut and fill. A sample grading plan, a typical cross-section, and associated compensatory storage calculations for the 10-year flood are provided on **Figures 10-1**, **Figure 10-2**, and **Table 10-1**, respectively.

Volume of fill between cross-sections are calculated by finding the average fill cross-sectional area and multiplying it by the distance between the two cross-sections. For example, the fill volume between cross-sections A and B is calculated as follows:

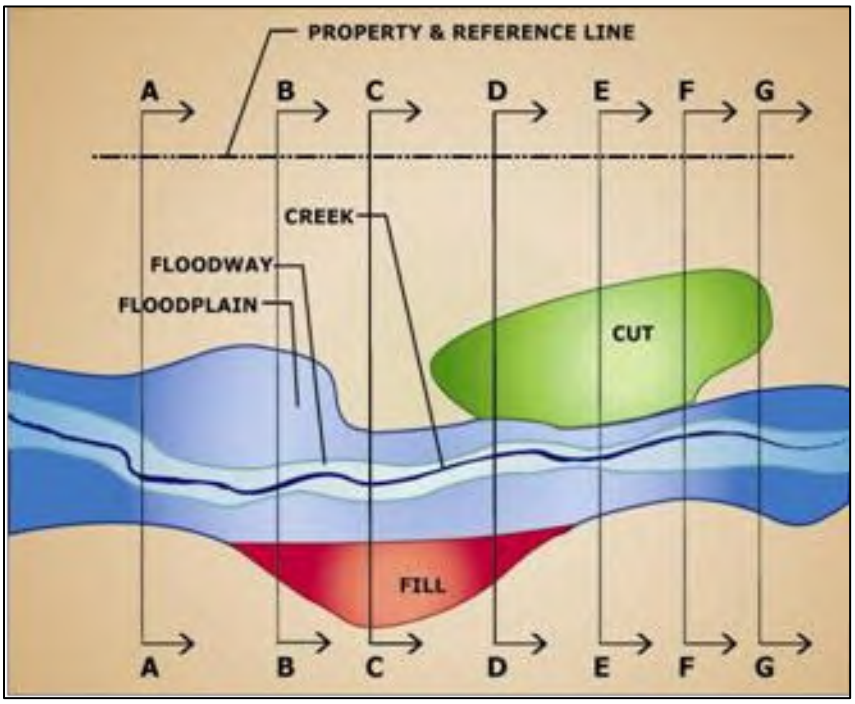
$$\text{Average Fill Area} = (\text{Fill Area "A"} + \text{Fill Area "B"}) / 2 = (0\ Ft^2 + 100\ Ft^2) / 2 = 50\ Ft^2$$

$$\text{Volume of Fill} = (\text{Average Fill Area}) \times (\text{Distance}) = (50\ Ft^2) \times (150\ Ft) = 7{,}500\ Ft^3$$

Once the total volume of fill placed, for this example, between the 0- and 10-yr flood elevations is determined, the total required compensatory storage can be calculated and compared against the total compensatory storage volume provided by the design as shown in the table. For this example:

Required Compensatory Storage = (1) × (Total Volume of Fill) = (1) × (36,250 Ft^3) = 36,250 Ft^3

Figure 10-1
Example Compensatory Storage Grading Plan



* Not to Scale & Topography not shown for clarity.

Figure 10-2
Example Cross-Section D-D

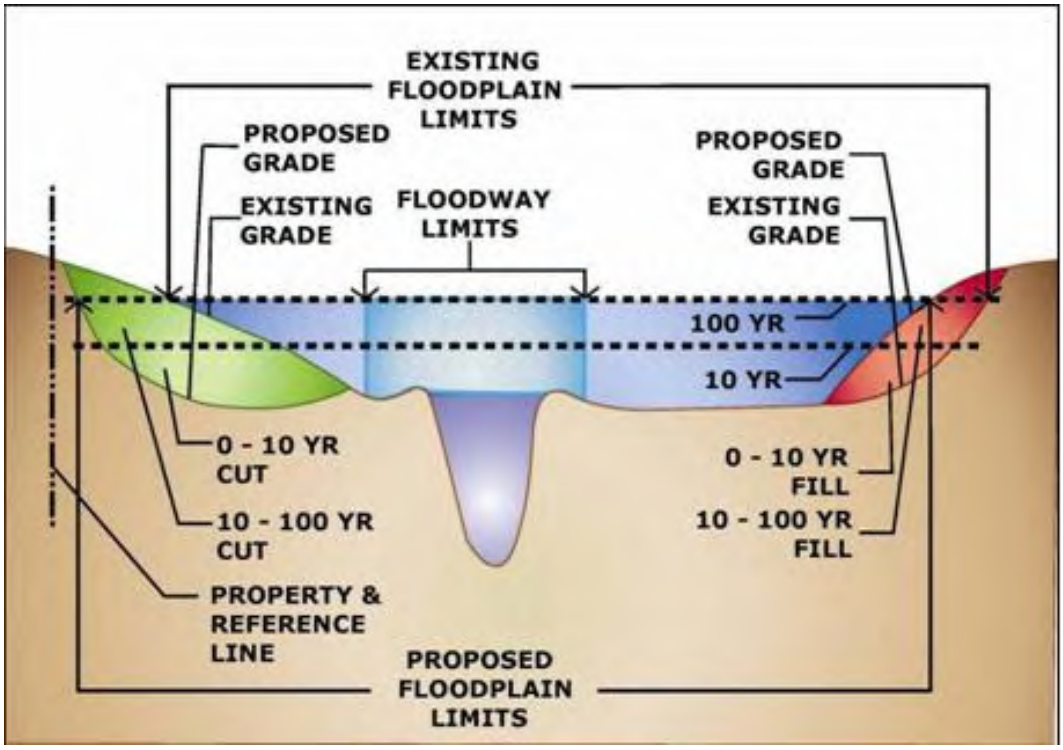


Table 10-1

Example Compensatory Storage Calculations for 0 to 10-Year Event							
Cross-Section	Distance Between Sections (Ft.)	Fill Area (Ft^2)	Average Fill Area (Ft^2)	Volume of Fill (Ft^3)	Cut Area (Ft^2)	Average Cut (Ft^2)	Volume of Cut (Ft^3)
A		0			0		
	150		50	7,500		0	
B		100			0		
	90		125	11,250		20	1,800
C		150			40		
	100		125	12,500		65	6,500
D		100			90		
	100		50	5,000		100	10,000
E		0			110		
	100		0	0		120	12,000
F		0			130		
	85		0	0		85	7,225
G		0			40		
Total Fill				36,250	Total Cut		37,525

Since the total amount of cut provided (37,525 Ft^3 as shown in the table) is larger than that required (36,250 Ft^3), the design meets the compensatory storage requirement for the 10-year flood. An additional table and calculation should be completed for the flood storage between the

10-year and 100-year plus two (2) feet flood elevation in a similar manner to determine whether the design meets the compensatory storage requirement for the area above the 10-year flood elevation.

Location of Compensatory Storage

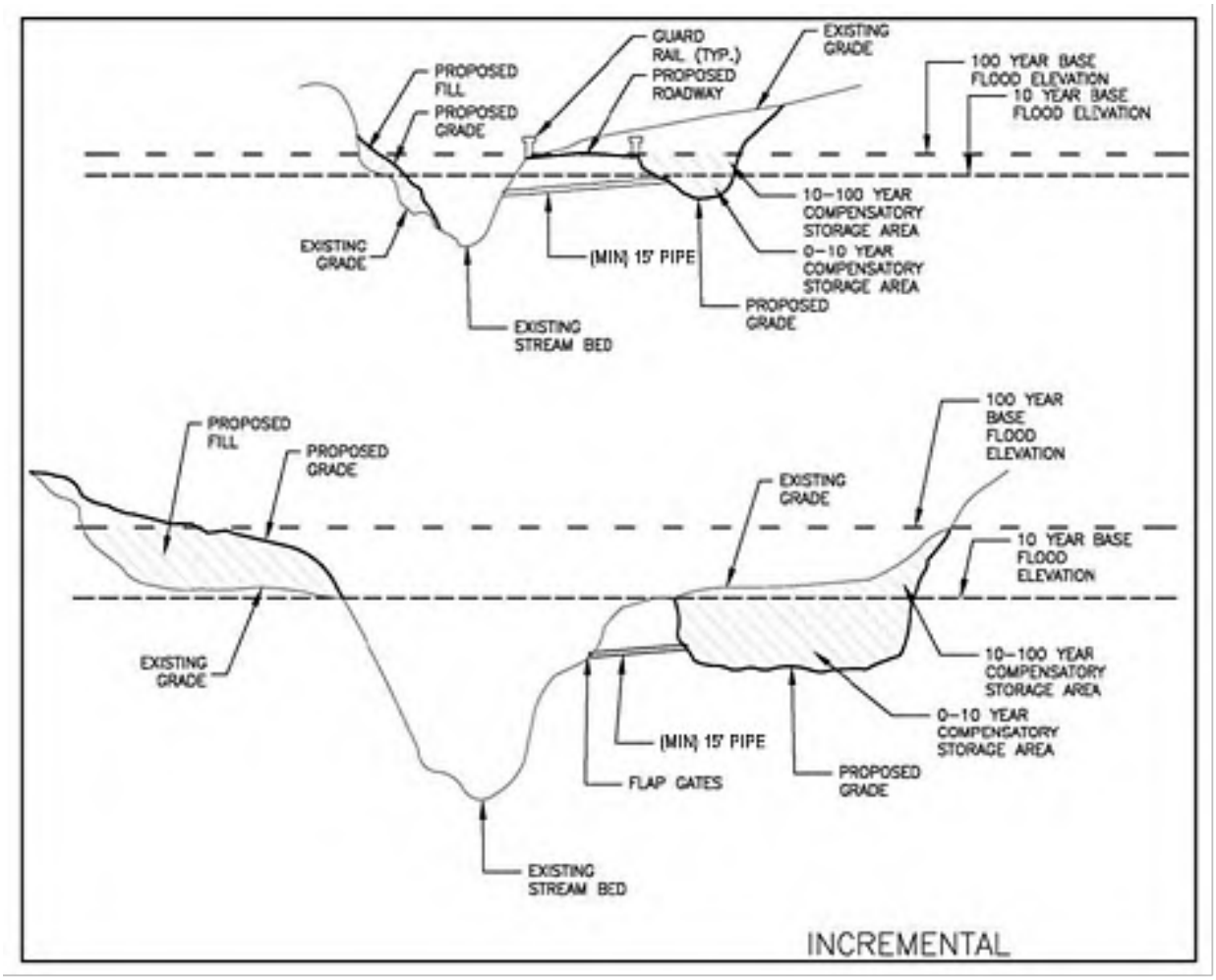
Compensatory storage must be located on-site and adjacent to or opposite the areas filled or occupied by a structure. In those rare instances when compensatory storage cannot be located adjacent to or opposite to the areas filled or occupied, engineering computations demonstrating that hydraulically equivalent compensatory storage has been provided in a nearby area in the watershed is required. These computations must show that no increase in flood flows or flood depths will result as a result of the location of the proposed compensatory storage.

Compensatory storage must be constructed to drain freely and openly to watercourses. In some rare cases it may be necessary to install pipes to construct and / or operate a compensatory storage basin. This may occur when site constraints, such as a roadway or sidewalk, separate the waterway from the compensatory storage area. This is illustrated in the top half of **Figure 11-3**.

Another scenario may occur when a site cannot meet the incremental storage requirements discussed in this document. If incremental storage requirements from the 10-year to 100-year plus 2 feet elevations cannot be met, pipes could be installed with a flap gate to prevent the water from entering from the stream bed at lower elevations. The berm could then be set at the elevation of the 10-year flood elevation, thus allowing the storage to only become effective above the 10-year flood elevation. This is illustrated in the bottom half of the illustration in **Figure 11-3**.

The use of pipes in compensatory storage will require approval by the City of Plymouth. Pipes must be a minimum of 15 inches in diameter so as to allow water to enter and exit freely with a minimum head differential. If the compensatory storage is proposed to be combined with detention, it must be demonstrated the compensatory storage and detention do not interfere with one another.

Figure 10-3
Example of Compensatory Storage Connection to Stream Through Pipe



Compensatory Storage in the Regulatory Floodway

Only fill associated with appropriate uses of the regulatory floodway may be allowed to be placed within the limits of the floodway. When in rare circumstances, fill is allowed, all provisions discussed above relating to compensatory storage must be met in addition to the items discussed below.

- Any fill placed within the existing floodway must be compensated for within the proposed floodway.
- All floodway storage lost below the existing 10-year base flood elevation must be replaced below the proposed 10-year base flood elevation.

- All floodway storage lost between the existing 10-year flood elevation and the existing 100-year plus 2 feet flood elevation must be replaced between the proposed 10-year and proposed 100-year plus 2 feet flood elevation.

Should excavation within a floodway be allowed in special circumstances, there must be no reduction in floodway surface area as a result of a floodway modification, unless such modification is part of a stream-wide plan or necessary to reduce flooding at an existing structure.

Chapter 11 | Standards Associated with Dams and Levees

A. Policy on Dams and Levees

No policy at this time.

Chapter 12 | Gasoline Outlets and Refueling Areas

Per the Stormwater Management Ordinance (approved June 2025), Gasoline outlets and refueling areas must install appropriate practices to reduce lead, copper, zinc, and polyaromatic hydrocarbons in stormwater runoff. These requirements will apply to all new facilities and existing facilities that replace their tanks, regardless of the size of the facility.

Since pollutants associated with fueling areas can cause significant problems with water quality, designers of post-construction plans shall thoroughly document how the selected BMPs will reduce the specified pollutants. The plan must also provide documentation that indicates how the maintenance of the selected BMPs will be done, and how to ensure that operators of the site will be aware of the maintenance requirements of the BMPs. While this is required for all post-construction pollution control plans, the BMP maintenance is particularly important for these sites due to the potential long term pollution problems that can result from fuels entering the storm drainage system. See **Table 7-2** and **7-3** for Fueling Station pre-treatment options. For suggestions regarding appropriate post-construction BMPs, refer to Chapter 8 of the "Indiana Storm Water Quality Manual."

IDEM Stormwater Manual (Chapter 8):
https://www.in.gov/idem/stormwater/files/stormwater_manual_chap_08.pdf

It should be noted, Hydrodynamic separators alone may not remove all the targeted pollutants from a site. For example, these systems alone may not remove 80% of the lead, copper, or zinc in addition to the polyaromatic hydrocarbons associated with fueling areas. A secondary in-line filter system (treatment train) may be required to achieve these objectives.

Appendix A

ABBREVIATIONS

BFE	Base Flood Elevation
BMP	Best Management Practice
BOWS	Plymouth Board of Works and Safety
CFS	Cubic Feet Per Second
CLOMR	Conditional Letter of Map Revision (from FEMA)
CLOMR-F	Conditional Letter of Map Revision Based on Fill (from FEMA)
CN	Curve Number
COE	United States Army Corps of Engineers
CSMP	Comprehensive Stormwater Management Program
CSO	Combined Sewer Overflow
CWA	Clean Water Act
ERM	Elevation Reference Mark
E&SC	Erosion and Sediment Control
EPA	Environmental Protection Agency
ETJ	Extraterritorial Jurisdiction
FBFM	Flood Boundary and Floodway Map
FEH	Fluvial Erosion Hazard
FEMA	Federal Emergency Management Agency
FHBM	Flood Hazard Boundary Map
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
FPG	Flood Protection Grade
FPS	Feet Per Second
GIS	Geographical Information System
GPS	Global Positioning System
HGL	Hydraulic Grade Line
HHW	Household Hazardous Waste
HUC	Hydrologic Unit Code

IDEM	Indiana Department of Environmental Management
IDNR	Indiana Department of Natural Resources
INDOT	Indiana Department of Transportation.
LAG	Lowest Adjacent Grade
LOMA	Letter of Map Amendment (from FEMA)
LOMR	Letter of Map Revision (from FEMA)
LOMR-F	Letter of Map Revision Based on Fill (from FEMA)
MCM	Minimum Control Measure
MS4	Municipal Separate Storm Sewer System
NAVD	North American Vertical Datum of 1988
NFIP	National Flood Insurance Program
NGVD 1929	National Geodetic Vertical Datum of 1929
NRCS	USDA-Natural Resources Conservation Service
NPDES	National Pollutant Discharge Elimination System
NPS	Non-point source
POTW	Publicly Owned Treatment Works
SFHA	Special Flood Hazard Area
SWCD	Soil and Water Conservation District
SWPPP	Stormwater Pollution Prevention Plan
SWQMP	Stormwater Quality Management Plan
Tc	Time of Concentration
TMDL	Total Maximum Daily Load
TN	Total Nitrogen
TP	Total Phosphorus
TSS	Total Suspended Solids
USCS	Unified Soil Classification System
USDA	United States Department of Agriculture
USFWS	United States Fish and Wildlife Service
WQV	Water Quality Volume

DEFINITIONS

Acre-Foot (AF). A measure of water volume equal to the inundation of a flat one-acre area to a depth of one foot (43,560 cubic feet).

Administering Authority. The designated unit of government given the authority to issue permits.

Agricultural Land Disturbing Activity. Tillage, planting, cultivation, or harvesting operations for the production of agricultural or nursery vegetative crops. The term also includes pasture renovation and establishment, the construction of agricultural conservation practices, and the installation and maintenance of agricultural drainage tile.

Agricultural Land Use Conservation Practices. Use of land for the production of animal or plant life, including forestry, pasturing or yarding of livestock, and planting, growing, cultivating, and harvesting crops for human or livestock consumption. Practices that are constructed on agricultural land for the purposes of controlling soil erosion and sedimentation. These practices include grass waterways, sediment basins, terraces, and grade stabilization structures.

Amortization Period. The length of time used to repay a debt or mortgage or to depreciate an initial cost.

Antecedent Runoff Condition. The index of runoff potential before a storm event. The index, developed by the Soil Conservation Service (SCS), is an attempt to account for the variation of the SCS runoff curve number (CN) from storm to storm.

Backflow Preventer. Device that allows liquids to flow in only one direction in a pipe. Backflow preventers are used on sewer pipes to prevent a reverse flow during flooding situations.

Backwater. The rise in water surface elevation caused by some obstruction such as a narrow bridge opening, buildings or fill material that limits the area through which the water shall flow.

Base Flood Elevation. The water surface elevation corresponding to a flood having a one percent probability of being equaled or exceeded in a given year.

Base Flood. See "Regulatory Flood".

Base Flow. Stream discharge derived from groundwater sources as differentiated from surface runoff. Sometimes considered to include flows from regulated lakes or reservoirs.

Basement. A building story that is all or partly underground but having at least one-half of its height below the average level of the adjoining ground. A basement shall not be counted as a story for the purpose of height regulations.

Benchmark. A marked point of known elevation from which other elevations may be established.

Best Management Practices. Design, construction, and maintenance practices and criteria for stormwater facilities that minimize the impact of stormwater runoff rates and volumes, prevent erosion, and capture pollutants.

Buffer Strip. An existing, variable width strip of vegetated land intended to protect water quality and habitat.

Building. See “structure”.

Capacity of a Storm Drainage Facility. The maximum flow that can be conveyed or stored by a storm drainage facility without causing damage to public or private property.

Catch Basin. A chamber usually built at the curb line of a street for the admission of surface water to a storm drain or subdrain, having at its base a sediment sump designed to retain grit and detritus below the point of overflow.

Centerline of Channel. The thalweg of a channel.

Channel Improvement. Alteration, maintenance, or reconstruction of the channel area for the purpose of improving the channel capacity or overall drainage efficiency. The noted "improvement" does not necessarily imply water quality or habitat improvement within the channel or its adjacent area.

Channel Modification. Alteration of a channel by changing the physical dimensions or materials of its bed or banks. Channel modification includes damming, rip-rapping or other armoring, widening, deepening, straightening, relocating, lining, and significant removal of bottom or woody vegetation. Channel modification does not include the clearing of dead or dying vegetation, debris, or trash from the channel. Channelization is a severe form of channel modification typically involving relocation of the existing channel (e.g., straightening).

Channel Stabilization. Protecting the sides and bed of a channel from erosion by controlling flow velocities and flow directions using jetties, drops, or other structures and/or by fining the channel with vegetation, riprap, concrete, or other suitable lining material.

Channel. A portion of a natural or artificial watercourse which periodically or continuously contains moving water, or which forms a connecting link between two bodies of water. It has a defined bed and banks which serve to confine the water.

Class V Injection Well. A type of well, which typically has a depth greater than its largest surface dimension, emplaces fluids into the subsurface, and does not meet the definitions of Class I through Class IV wells as defined under 40 CFR 146.5. While the term includes the specific examples described in 40 CFR 144.81, septic systems that serve more than one (1) single-family dwelling or provide service for non-domestic waste, dug wells, bored wells, improved sinkholes, french drains, infiltration sumps, and infiltration galleries, it does not include surface impoundments, trenches, or ditches that are wider than they are deep.

Closed Conduit. A pipe, tube, or tile used for transmitting water.

Combined Sewer Overflow. A system designed and used to receive and transport combined sewage so that during dry periods the wastewater is carried to a treatment facility. During storm events, the excess water is discharged directly into a river, stream, or lake without treatment.

Compensatory Storage. An artificial volume of storage within a floodplain used to balance the loss of natural flood storage capacity when artificial fill or substructures are placed within the floodplain.

Compost. Organic residue (or a mixture of organic residue and soil) that has undergone biological decomposition until it has become relatively stable humus.

Comprehensive Stormwater Management Program. A comprehensive stormwater program for effective management of stormwater quantity and quality throughout the community.

Constructed Wetland. A manmade shallow pool that creates growing conditions suitable for wetland vegetation and is designed to maximize pollutant removal.

Construction Activity. Land disturbing activities, and land disturbing activities associated with the construction of infrastructure and structures. This term does not include routine ditch or road maintenance or minor landscaping projects.

Construction Plan. A representation of a project site and all activities associated with the project. The plan includes the location of the project site, buildings and other infrastructure, grading activities, schedules for implementation and other pertinent information related to the project site. A storm water pollution prevention plan is a part of the construction plan.

Construction Site Access. A stabilized stone surface at all points of ingress or egress to a project site, for the purpose of capturing and detaining sediment carried by tires of vehicles or other equipment entering or exiting the project site.

Construction Support Activities. Include but are not limited to the following: concrete or asphalt batch plants, equipment staging yards, material storage areas, excavated material disposal areas, borrow areas. Such activities must not support multiple, unrelated projects, be a commercial/industrial operation, or continue to operate beyond the completion of the construction activity for the project it supports.

Contiguous. Adjoining or in actual contact with.

Contour Line. Line on a map which represents a contour or points of equal elevation.

Contour. An imaginary line on the surface of the earth connecting points of the same elevation.

Contractor or Subcontractor. An individual or company hired by the project site or individual lot owner, their agent, or the individual lot operator to perform services on the project site.

Control Structure. A structure designed to control the rate of flow that passes through the structure, given a specific upstream and downstream water surface elevation.

Conveyance. Any structural method for transferring stormwater between at least two points. The term includes piping, ditches, swales, curbs, gutters, catch basins, channels, storm drains, and roadways.

Convolution. The process of translating precipitation excess into a runoff hydrograph.

Crawl Space. Low space below first floor of a house where there has not been excavation deep enough for a basement, usually less than seven (7) feet in depth, but where there is access for pipes, ducts, utilities and similar equipment.

Critical Duration Analysis. The process of testing different rainfall durations to find that "critical duration", which produces the highest peak runoff or the highest storage volume.

Cross-Section. A graph or plot of ground elevation across a stream valley or a portion of it, usually along a line perpendicular to the stream or direction of flow.

Crown of Pipe. The elevation of top of pipe.

Cubic Feet Per Second (CFS). Used to describe the amount of flow passing a given point in a stream channel. One cubic foot per second is equivalent to approximately 7.5 gallons per second.

Culvert. A closed conduit used for the conveyance of surface drainage water under a roadway, railroad, canal or other impediment.

Curve Number (CN). The Soil Conservation Service index that represents the combined hydrologic effect of soil, land use, land cover, hydrologic condition and antecedent runoff condition.

Dam. A barrier to confine or impound water for storage or diversion, to prevent gully erosion, or to retain soil, sediment, or other debris.

Damage. Measurable rise in flood heights on buildings currently subject to flooding, flooding of buildings currently not subject to flooding and increases in volume or velocity to the point where the rate of land lost to erosion and scour is substantially increased.

Datum. Any level surface to which elevations are referred, usually Mean Sea Level.

Dechlorinated Swimming Pool Discharge. Chlorinated water that has either sat idle for seven (7) days following chlorination prior to discharge to the MS4 conveyance, or, by analysis, does not contain detectable concentrations (less than five-hundredths (0.05) milligram per liter) of chlorinated residual.

Depressional Storage Areas. Non-riverine depressions in the earth where stormwater collects. The volumes are often referred to in units of acre-feet.

Design Storm. A selected storm event, described in terms of the probability of occurring once within a given number of years, for which drainage or flood control improvements are designed and built.

Detention Basin. A facility constructed or modified to restrict the flow of storm water to a prescribed maximum rate, and to detain concurrently the excess waters that accumulate behind the outlet.

Detention Facility. A facility designed to detain a specified amount of stormwater runoff assuming a specified release rate. The volumes are often referred to in units of acre-feet.

Detention Storage. The temporary detaining of storage of stormwater in storage facilities, on rooftops, in streets, parking lots, school yards, parks, open spaces or other areas under predetermined and controlled conditions, with the rate of release regulated by appropriately installed devices.

Detention Time. The theoretical time required to displace the contents of a tank or unit at a given rate of discharge (volume divided by rate of discharge).

Detention. Managing stormwater runoff by temporary holding and controlled release.

Detritus. Dead or decaying organic matter; generally contributed to stormwater as fallen leaves and sticks or as dead aquatic organisms.

Developer. Any person financially responsible for construction activity, or an owner of property who sells or leases, or offers for sale or lease, any lots in a subdivision.

Development. Any man-made change to improved or unimproved real estate including but not limited to:

1. Construction, reconstruction, or placement of a building or any addition to a building;
2. Construction of flood control structures such as levees, dikes, dams or channel improvements;
3. Construction or reconstruction of bridges or culverts;
4. Installing a manufactured home on a site, preparing a site for a manufactured home, or installing a recreational vehicle on a site for more than hundred eight (180) days;
5. Installing utilities, erection of walls, construction of roads, or similar projects;
6. Mining, dredging, filling, grading, excavation, or drilling operations;
7. Storage of materials; or
8. Any other activity that might change the direction, height, or velocity of flood or surface waters.

“Development” does not include activities such as the maintenance of existing buildings and facilities such as painting, re-roofing, resurfacing roads, or gardening, plowing and similar agricultural practices that do not involve filling, grading, excavation, or the construction of permanent buildings.

Direct Release. A method of stormwater management where runoff from a part or the entire development is released directly to the receiving stream without providing detention.

Discharge. In the context of water quantity provisions, usually the rate of water flow, i.e., a volume of fluid passing a point per unit time commonly expressed as cubic feet per second, cubic meters per second, gallons per minute, or millions of gallons per day. In the context of water quality provisions, the discharge means any addition of liquids or solids to a water body or a flow conveyance facility.

Disposal. The discharge, deposit, injection, spilling, leaking, or placing of any solid waste or hazardous waste into or on any land/or water so that the solid waste or hazardous waste, or any constituent of the waste, may enter the environment, be emitted into the air, or be discharged into any waters, including

Ditch. A man-made, open drainageway in or into which excess surface water or groundwater drained from land, stormwater runoff, or floodwaters flow either continuously or intermittently.

Drain. A buried slotted or perforated pipe or other conduit (subsurface drain) or a ditch (open drain) for carrying off surplus groundwater or surface water.

Drainage Area. The area draining into a stream at a given point. It may be of different sizes for surface runoff, subsurface flow and base flow, but generally the surface runoff area is considered as the drainage area.

Drainage Classification (soil). As a natural condition of the soil, drainage refers to both the frequency and duration of periods when the soil is free of saturation. Soil drainage conditions are defined as:

- Well-drained: Excess water drains away rapidly, and no mottling occurs within 36 inches of the surface.
- Moderately Well Drained: Water is removed from the soil somewhat slowly resulting in small but significant periods of wetness, and mottling occurs between 18 and 36 inches.
- Poorly Drained: Water is removed so slowly that it is wet for a large part of the time, and mottling occurs between 0 and 8 inches.
- Somewhat Poorly Drained: Water is removed from the soil slowly enough to keep it wet for significant periods but not all of the time, and mottling occurs between 8 to 18 inches.
- Very Poorly Drained: Water is removed so slowly that the water table remains at or near the surface for the greater part of the time; there may also be periods of surface ponding; the soil has a black to gray surface layer with mottles up to the surface.

Drainage. The removal of excess surface water or groundwater from land by means of ditches or subsurface drains. Also see Natural drainage.

Drop Manhole. Manhole having a vertical drop pipe connecting the inlet pipe to the outlet pipe. The vertical drop pipe shall be located immediately outside the manhole.

Dry Well. A type of infiltration practice that allows stormwater runoff to flow directly into the ground via a bored or otherwise excavated opening in the ground surface.

Dry-Bottom Detention Basin. A basin designed to be completely dewatered after having provided its planned detention of runoff during a storm event.

Duration. The time period of a rainfall event.

Earth Embankment. A man-made deposit of soil, rock, or other material often used to form an impoundment.

Elevation Certificate. A form published by the Federal Emergency Management Agency that is used to certify the 100-year or base flood elevation and the lowest elevation of usable space to which a building has been constructed.

Elevation Reference Mark (ERM). Elevation benchmark tied to the National Geodetic Vertical Datum of 1929 and identified during the preparation of a Flood Insurance Study prepared for the Federal Emergency Management Agency.

Emergency Spillway. Usually, a vegetated earth channel used to safely convey flood discharges around an impoundment structure.

Energy Dissipater. A device to reduce the energy of flowing water.

Environment. The sum total of all the external conditions that may act upon a living organism or community to influence its development or existence.

Erosion And Sediment Control Measure. A practice, or a combination of practices, to control erosion and resulting sedimentation. and/or off-site damages.

Erosion And Sediment Control System. The use of appropriate erosion and sediment control measures to minimize sedimentation by first reducing or eliminating erosion at the source and then as necessary, trapping sediment to prevent it from being discharged from or within a project site.

Erosion Control Plan. A written description and site plan of pertinent information concerning erosion control measures designed to meet the requirements of the Ordinance or these Standards.

Erosion. The wearing away of the land surface by water, wind, ice, gravity, or other geological agents. The following terms are used to describe different types of water erosion:

- Accelerated Erosion: Erosion much more rapid than normal or geologic erosion, primarily as a result of the activities of man.
- Channel Erosion: An erosion process whereby the volume and velocity of flow wears away the bed and/or banks of a well-defined channel.

- Gully Erosion: An erosion process whereby runoff water accumulates in narrow channels and, over relatively short periods, removes the soil to considerable depths, ranging from 1-2 ft. to as much as 75-100 ft.
- Rill Erosion: An erosion process in which numerous small channels only several inches deep are formed; occurs mainly on recently disturbed and exposed soils (see Rill).
- Splash Erosion: The spattering of small soil particles caused by the impact of raindrops on wet soils; the loosened and spattered particles may or may not be subsequently removed by surface runoff.
- Sheet Erosion: The gradual removal of a fairly uniform layer of soil from the land surface by runoff water.

Extraterritorial Jurisdiction (ETJ). Areas located outside the corporate limits of a community over which the community has statutory development authority.

Farm or Field Tile. A pipe installed in an agricultural area to allow subsurface drainage of farmland for the purpose of agricultural production.

FEMA. The Federal Emergency Management Agency.

Filter Strip. Usually a long, relatively narrow area (usually, 20-75 feet wide) of undisturbed or planted vegetation used near disturbed or impervious surfaces to filter stormwater pollutants for the protection of watercourses, reservoirs, or adjacent properties.

Final Stabilization. The establishment of permanent vegetative cover or the application of a permanent non-erosive material to areas where all land disturbing activities have been completed and no additional land disturbing activities are planned under the current permit.

Floatable. Any solid waste that will float on the surface of the water.

Flood (or Flood Waters). A general and temporary condition of partial or complete inundation of normally dry land areas from the overflow, the unusual and rapid accumulation, or the runoff of surface waters from any source.

Flood Boundary and Floodway Map (FBFM). A map prepared by the Federal Emergency Management Agency the depicts the FEMA designated floodways within a community. This map also includes delineation of the 100-year and 500-year floodplain boundaries and the location of the Flood Insurance Study cross-sections.

Flood Crest. The maximum stage or elevation reached or expected to be reached by the waters of a specific flood at a given time.

Flood Duration. The length of time a stream is above flood stage or overflowing its banks.

Flood Easement. Easement granted to identify areas inundated by the 100-year flood and prohibit or severely restrict development activities.

Flood Elevation. The elevation at all locations delineating the maximum level of high waters for a flood of given return period.

Flood Fighting. Actions taken immediately before or during a flood to protect human life and to reduce flood damages such as evacuation, emergency sandbagging and diking.

Flood Forecasting. The process of predicting the occurrence, magnitude and duration of an imminent flood through meteorological and hydrological observations and analysis.

Flood Frequency. A statistical expression of the average time period between floods equaling or exceeding a given magnitude. For example, a 100-year flood has a magnitude expected to be equaled or exceeded on the average of once every hundred years; such a flood has a one-percent chance of being equaled or exceeded in any given year. Often used interchangeably with "recurrence interval".

Flood Hazard Area. Any floodplain, floodway, floodway fringe, or any combination thereof which is subject to inundation by the regulatory flood; or any flood plain as delineated by Zone X on a Flood Hazard Boundary Map.

Flood Hazard Boundary Map (FHBM). A map prepared by the Federal Emergency Management Agency that depicts Special Flood Hazard Areas as a Zone A within a community. There are no study text, base flood elevations, or floodways associated with this map.

Flood Insurance Rate Map (FIRM). A map prepared by the Federal Emergency Management Agency that depicts Special Flood Hazard Areas within a community. This map also includes the 100-year or Base Flood Elevation at various locations along the watercourses. More recent versions of the FIMR may also show the FEMA designated floodway boundaries and the location of the Flood Insurance Study cross-sections.

Flood Insurance Study (FIS). A study prepared by the Federal Emergency Management agency to assist a community participating in the National Flood Insurance Program in its application of the program regulations. The study consists of a text which contains community background information with respect to flooding, a floodway data table, summary of flood discharges, flood profiles, a Flood Insurance Rate Map, and a Flood Boundary and Floodway Map.

Flood Profile. A graph showing the relationship of water surface elevation to a specific location, the latter generally expressed as distance above the mouth of a stream of water flowing in a channel. It is generally drawn to show surface elevation for the crest or a specific magnitude of flooding, but may be prepared for conditions at any given time or stage.

Flood Protection Grade (FPG). The elevation of the regulatory or 100-year flood plus two (2) feet of freeboard if the flooding source is a lake, pond, stream, or an open channel/ditch (or 1 foot of freeboard if the flooding source is an overflow path/ponding area provided that the elevation of the overflow path/ponding area is calculated based on the assumption of fully plugged storm pipe system).

Flood Resistant Construction (Flood Proofing). Additions, changes or adjustments to structures or property that are designed to reduce or eliminate the potential for flood damage.

Flood Storage Areas. Depressions, basins, or other areas that normally stand empty or partially empty, but fill with rainfall runoff during storms to hold the runoff and reduce downstream flow rates. The volumes are often referred to in units or acre-feet.

Floodplain Management. The operation of a program of corrective and preventive measures for reducing flood damage, including but not limited to flood control projects, floodplain land use regulations, flood proofing of buildings, and emergency preparedness plans.

Floodplain Regulations. General term applied to the full range of codes, ordinances and other regulations relating to the use of land and construction within floodplain limits. The term encompasses zoning ordinances, subdivision regulations, building and housing codes, encroachment laws and open area (space) regulations.

Floodplain. The channel proper and the areas adjoining the channel which have been or hereafter may be covered by the regulatory or 100-year flood. Any normally dry land area that is susceptible to being inundated by water from any natural source. The floodplain includes both the floodway and the floodway fringe districts.

Floodway Fringe. That portion of the flood plain lying outside the floodway, which is inundated by the regulatory flood.

Floodway. The channel of a river or stream and those portions of the floodplains adjoining the channel which are reasonably required to efficiently carry and discharge the peak flow of the regulatory flood of any river or stream.

Footing Drain. A drain pipe installed around the exterior of a basement wall foundation to relieve water pressure caused by high groundwater elevation.

Forebay (or Sediment Forebay). A small pond placed in front of a larger retention/detention structure such as a wet pond, dry pond, or wetland to intercept and concentrate a majority of sediment that is coming into the system before it reaches the larger structure.

Freeboard. An increment of height added to the base flood elevation to provide a factor of safety for uncertainties in calculations, unknown local conditions, wave actions and unpredictable effects such as those caused by ice or debris jams. (See Flood Protection Grade).

French Drain. A drainage trench backfilled with a coarse, water-transmitting material; may contain a perforated pipe.

Gabion. An erosion control structure consisting of a wire cage or cages filled with rocks.

Garbage. All putrescible animal solid, vegetable solid, and semisolid wastes resulting from the processing, handling, preparation, cooking, serving, or consumption of food or food materials.

Geographical Information System. A computer system capable of assembling, storing, manipulation, and displaying geographically referenced information. This technology can be used for resource management and development planning.

Geotextile Fabric. A woven or non-woven, water-permeable synthetic material used to trap sediment particles, prevent the clogging of aggregates with fine grained soil particles, or as a separator under road aggregate.

Geotextile Liner. A synthetic, impermeable fabric used to seal impoundments against leaks.

Global Positioning System. A system that provides specially coded satellite signals that is processed by a receiver, which determines position, velocity, and time. The system is funded and controlled by the U.S. Department of Defense.

Grade. (1) The inclination or slope of a channel, canal, conduit, etc., or natural ground surface usually expressed in terms of the percentage the vertical rise (or fall) bears to the corresponding horizontal distance. (2) The finished surface of a canal bed, roadbed, top of embankment, or bottom of excavation; any surface prepared to a design elevation for the support of construction, such as paving or the laying of a conduit. (3) To finish the surface of a canal bed, roadbed, top of embankment, or bottom of excavation, or other land area to a smooth, even condition.

Grading. The cutting and filling of the land surface to a desired slope or elevation.

Grass. A member of the botanical family Graminae, characterized by blade-like leaves that originate as a sheath wrapped around the stem.

Grassed Swale. A type of vegetative practice used to filter stormwater runoff via a vegetated, shallow-channel conveyance.

Grassed Waterway. A natural or constructed waterway, usually broad and shallow, covered with erosion-resistant grasses and used to conduct surface water from an area.

Ground Cover (Horticulture). Low-growing, spreading plants useful for low-maintenance landscape areas.

Groundwater Recharge. The infiltration of water into the earth. It may increase the total amount of water stored underground or only replenish supplies depleted through pumping or natural discharge.

Groundwater. Accumulation of underground water, natural or artificial. The term does not include

Habitat. The environment in which the life needs of a plant or animal are supplied.

Hard Surface. See “Impervious Surface.”

High Water. Maximum designed permitted, or regulated water level for an impoundment.

Household Hazardous Waste. Solid waste generated by households that is ignitable, toxic, reactive, corrosive, or otherwise poses a threat to human health or the environment.

Hydraulic Grade Line (HGL). For Channel flow, the HGL is equal to the water surface whereas for pressure flow it is the piezometric surface.

Hydraulics. A branch of science that deals with the practical application of the mechanics of water movement. A typical hydraulic study is undertaken to calculate water surface elevations.

Hydrodynamic Loads. Forces imposed on structures by floodwaters due to the impact of moving water on the upstream side of the structure, drag along its sides, and eddies or negative pressures on its downstream side.

Hydrograph. For a given point on a stream, drainage basin, or a lake, a graph showing either the discharge, stage (depth), velocity, or volume of water with respect to time.

Hydrologic Unit Code. A numeric United States Geologic Survey code that corresponds to a watershed area. Each area also has a text description associated with the numeric code.

Hydrology. The science of the behavior of water in the atmosphere, on the surface of the earth, and underground. A typical hydrologic study is undertaken to compute flow rates associated with specified flood events.

Hydrometeorologic. Water-related meteorological data such as rainfall or runoff.

Hydrostatic Loads. Those loads or pressures resulting from the static mass of water at any point of floodwater contact with a structure. They are equal in all direction and always act perpendicular to the surface on which they are applied. Hydrostatic loads can act vertically on structural members such as floors, decks and roofs, and can act laterally on upright structural members such as walls, piers, and foundations.

IDNR. Indiana Department of Natural Resources.

Illicit Discharge. Any discharge to a conveyance that is not composed entirely of stormwater except naturally occurring floatables, such as leaves or tree limbs.

Impact Areas. Areas defined or mapped that are unlikely to be easily drained because of one or more factors including but not limited to any of the following: soil type, topography, land where there is not adequate outlet, a floodway or floodplain, land within 75 feet of each bank of any regulated drain or within 75 feet from the centerline of any regulated tile ditch.

Impaired Waters. Waters that do not or are not expected to meet applicable water quality standards, as included on IDEM's CWA Section 303(d) List of Impaired Waters.

Impervious Surface. Surfaces, such as pavement and rooftops, which prevent the infiltration of stormwater into the soil.

Individual Building Lot. A single parcel of land within a multi-parcel development.

Individual Lot Operator. A contractor or subcontractor working on an individual lot.

Individual Lot Owner. A person who has financial control of construction activities for an individual lot.

INDOT. Indiana Department of Transportation. Generally used here to refer to specifications contained in the publication "INDOT Standard Specifications."

Infiltration Practices. Any structural BMP designed to facilitate the percolation of run-off through the soil to ground water. Examples include infiltration basins or trenches, dry wells, and porous pavement.

Infiltration. Passage or movement of water into the soil.

Infiltration Swales. A depressed earthen area that is designed to promote infiltration.

Inlet. An opening into a storm drain system for the entrance of surface storm water runoff, more completely described as a storm drain inlet.

Intermittent Stream. A stream which carries water a considerable portion of the time, but which ceases to flow occasionally or seasonally because bed seepage and evapotranspiration exceed the available water supply.

Invert. The inside bottom of a culvert or other conduit.

Junction Chamber. A converging section of conduit, usually large enough for a person to enter, used to facilitate the flow from one or more conduits into a main conduit.

Land Surveyor. A person licensed under the laws of the State of Indiana to practice land surveying.

Land-Disturbing Activity. Any man-made change of the land surface, including removing vegetative cover that exposes the underlying soil, excavating, filling, transporting and grading.

Larger Common Plan Of Development Or Sale. A plan, undertaken by a single project site owner or a group of project site owners acting in concert, to offer lots for sale or lease; where such land is contiguous, or is known, designated, purchased or advertised as a common unit or by a common name, such land shall be presumed as being offered for sale or lease as part of a larger common plan. The term also includes phased or other construction activity by a single entity for its own use.

Lateral Storm Sewer. A drain that has inlets connected to it but has no other storm drain connected.

Life Cycle Cost. Cost based on the total cost incurred over the system life including research, development, testing, production, construction, operation, and maintenance. Costs are normally determined on present worth or equivalent annual cost basis.

Low Entry Elevation. The elevation in a structure where overbank flooding can enter the structure.

Lowest Adjacent Grade. The elevation of the lowest grade adjacent to a structure, where the soil meets the foundation around the outside of the structure (including structural members such as basement walkout, patios, decks, porches, support posts or piers, and rim of the window well.

Lowest Floor. Refers to the lowest of the following:

1. The top of the basement floor;
2. The top of the garage floor, if the garage is the lowest level of the building;
3. The top of the first floor of buildings constructed on a slab or of buildings elevated on pilings or constructed on a crawl space with permanent openings; or
4. The top of the floor level of any enclosure below an elevated building where the walls of the enclosure provide any resistance to the flow of flood waters unless:

    a) The walls are designed to automatically equalize the hydrostatic flood forces on the walls by allowing for the entry and exit of flood waters, by providing a minimum of two opening (in addition to doorways and windows) having a total area of one (1) square foot for every two (2) square feet of enclosed area subject to flooding. The bottom of all such openings shall be no higher than one (1) foot above grade.

    b) Such enclosed space shall be usable only for the parking of vehicles or building access.

Low Impact Development. LID is a land planning and engineering design approach with a goal of replicating the pre-development hydrologic regime of urban and developing watersheds. The primary goal of LID is to mimic a site's predevelopment hydrology by reducing the impervious surface, infiltrating, filtering, storing, evaporating, and detaining runoff close to its source.

Major Drainage System. Drainage system carrying runoff from an area of one or more square miles.

Manhole. Storm drain structure through which a person may enter to gain access to an underground storm drain or enclosed structure.

Manning Roughness Coefficient or Manning's "n" Value. A dimensionless coefficient ("n") used in the Manning's equation to account for channel wall frictional losses in steady uniform flow.

Measurable Storm Event. A precipitation event that results in a total measured precipitation accumulation equal to, or greater than, one-half (0.5) inch of rainfall.

Minimum Control Measure. Minimum measures required by the NPDES Phase II program. The six (6) MCMs are: Public education and outreach, Public participation and involvement, Illicit discharge detection and elimination, Construction site runoff control, Post-construction runoff control, and Pollution prevention and good housekeeping.

Minor Drainage Systems. Drainage system carrying runoff from an area of less than one square mile.

Minor Subdivision. See Subdivision, Minor.

Mulch. A natural or artificial layer of plant residue or other materials covering the land surface which conserves moisture, holds soil in place, aids in establishing plant cover, and minimizes temperature fluctuations.

Multi-Family. Any structure which contains three or more dwelling units. A dwelling unit is any structure, or part of a structure, which is constructed to a house a family.

Municipal Separate Storm Sewers (MS4). An MS4 meets all the following criteria: (1) is a conveyance or system of conveyances owned by the state, county, City, or other public entity; (2) discharges to waters of the U.S.; (3) is designed or used for collecting or conveying stormwater; (4) is not a combined sewer; and, (5) is not part of a Publicly Owned Treatment Works (POTW).

Municipal, State, Federal, Or Institutional Refueling Area. An operating gasoline or diesel fueling area whose primary function is to provide fuel to either municipal, state, federal, or institutional equipment or vehicles.

Mutual Drain. A drain that: (1) Is located on two or more tracts of land that are under different ownership; (2) was established by the mutual consent of all the owners; and (3) was not established under or made subject to any drainage statute.

National Flood Insurance Program (NFIP). The NFIP is a federal program enabling property owners to purchase flood insurance. The Federal Emergency Management Agency administers the NFIP in communities throughout the Unites States. The NFIP is based on an agreement between local communities and the Federal government which states that if a community will implement floodplain management measures to reduce future flood risks to new construction and substantially improved structures in flood hazard areas, the Federal government will make flood insurance available within the community as a financial protection against flood losses that do occur.

National Geodetic Vertical Datum of 1929. The nationwide, Federal Elevation datum used to reference topographic elevations to a known value.

National Pollution Discharge Elimination System (NPDES). A permit developed by the U.S. EPA through the Clean Water Act. In Indiana, the permitting process has been delegated to IDEM. This permit covers aspects of municipal stormwater quality.

Natural Drainage. The flow patterns of stormwater run-off over the land in its pre-development state.

Nonagricultural Land Use. Commercial use of land for the manufacturing and wholesale or retail sale of goods or services, residential or institutional use of land intended primarily to shelter people, highway use of land including lanes, alleys, and streets, and other land uses not included in agricultural land use.

Nonpoint Source Pollution. Pollution that enters a water body from diffuse origins on the watershed and does not result from discernable, confined, or discrete conveyances.

Normal Depth. Depth of flow in an open conduit during uniform flow for the given conditions.

North American Vertical Datum of 1988 (NAVD 1988). The nationwide, Federal Elevation datum used to reference topographic elevations to a known value.

Nutrient(s). (1) A substance necessary for the growth and reproduction of organisms. (2) In water, those substances (chiefly nitrates and phosphates) that promote growth of algae and bacteria.

Off-Site. Everything not located at or within a particular site.

Off-Site Land Areas. Those areas that by virtue of existing topography naturally shed surface water onto or through the developing property.

100-Year Frequency Flood. See “regulatory flood”.

On-Site. Located within the controlled or urbanized area where runoff originates.

Open Drain. A natural watercourse or constructed open channel that conveys drainage water.

Open Space. Any land area devoid of any disturbed or impervious surfaces created by industrial, commercial, residential, agricultural, or other manmade activities.

Orifice. A device which controls the rate of flow from a detention basin.

Outfall Scouring. The deterioration of a streambed or lakebed from an outfall discharge to an extent that the excessive settling of solid material results and suitable aquatic habitat is diminished.

Outfall. The point, location, or structure where a pipe or open drain discharges to a receiving body of water.

Outlet. The point of water disposal from a stream, river, lake, tidewater, or artificial drain.

Overland Flow. Consists of sheet flow, shallow concentrated flow and channel flow.

Peak Discharge (or Peak Flow). The maximum instantaneous flow from a given storm condition at a specific location.

Percolation. The movement of water through soil.

Perennial Stream. A stream that maintains water in its channel throughout the year.

Permanent Stabilization. The establishment, at a uniform density of seventy percent (70%) across the disturbed area, of vegetative cover or permanent non-erosive material that will ensure the resistance of the soil to erosion, sliding, or other movement.

Permeability (Soil). The quality of a soil that enables water or air to move through it. Usually expressed in inches per hour or inches per day.

Pervious. Allowing movement of water.

Pesticides. Chemical compounds used for the control of undesirable plants, animals, or insects. The term includes insecticides, herbicides, algicides, rodenticides, nematicides, fungicides, and growth regulators.

pH. A numerical measure of hydrogen ion activity, the neutral point being 7.0. All pH values below 7.0 are acid, and all above 7.0 are alkaline.

Phasing Of Construction. Sequential development of smaller portions of a large project site, stabilizing each portion before beginning land disturbance on subsequent portions, to minimize exposure of disturbed land to erosion.

Phosphorus (Available). Inorganic phosphorus that is readily available for plant growth.

Piping. The formation of "pipes" by underground erosion. Water in the soil carries the fine soil particles away, and a series of eroded tubes or tunnels develop. These openings will grow progressively larger and can cause a dam failure.

Planimetric Data. Horizontal measurements involving distances or dimensions on a diagram, map, Plat of Survey or topographic map. Normally in units of feet.

Plat of Survey. A scaled diagram showing boundaries of a tract of land/or subdivision. This may constitute a legal description of the land and be used in lieu of a written description.

Point Source. Any discernible, confined, and discrete conveyance including but not limited to any pipe, ditch, channel, tunnel, conduit, well, discrete fissure, or container from which pollutants are or maybe discharged (P.L. 92-500, Section 502[14]).

Pollutant Of Concern. Any pollutant that has been documented via analytical data as a cause of impairment in any waterbody.

Porosity. The volume of pore space in soil or rock.

Porous Pavement. A type of infiltration practice to improve the quality and reduce the quantity of storm water run-off via the use of manmade, pervious pavement which allows run-off to percolate through the pavement and into underlying soils

Private Drain. A drain that: (1) Is located on land owned by one person or by two or more persons jointly; and (2) was not established under or made subject to any drainage statute.

Professional Engineer. A person licensed under the laws of the State of Indiana to practice professional engineering.

Programmatic Indicator. Any data collected by an MS4 entity that is used to indicate implementation of one (1) or more minimum control measures.

Project Site Owner. The person required to submit a stormwater permit application and required to comply with the terms of the ordinance or these Technical Standards, including a developer or a person who has financial and operational control of construction activities, and project plans and specifications, including the ability to make modifications to those plans and specifications.

Project Site. The entire area on which construction activity is to be performed.

Probable Maximum Flood. The most severe flood that may be expected from a combination of the most critical meteorological and hydrological conditions that are reasonably possible in the drainage basin. It is used in designing high-risk flood protection works and citing of structures and facilities that shall be subject to almost no risk of flooding. The probable maximum flood is usually much larger than the 100-year flood.

Publicly Owned Treatment Works (POTW). A municipal operation that breaks down and removes contaminants in the wastewater prior to discharging to a stream through primary and/or secondary treatment systems.

Qualified Professional. An individual who is trained and experienced in storm water treatment techniques and related fields as may be demonstrated by state registration, professional certification, experience, or completion of coursework that enable the individual to make sound, professional judgments regarding storm water control or treatment and monitoring, pollutant fate and transport, and drainage planning.

Radius of Curvature. Length of radius of a circle used to define a curve.

Rain Garden. A vegetative practice used to alter impervious surfaces, such as roofs, into pervious surfaces for absorption and treatment of rainfall.

Rainfall Intensity. The rate at which rain is falling at any given instant, usually expressed in inches per hour.

Reach. Any length of river, channel or storm drain.

Receiving Stream or Receiving Water. The body of water into which runoff or effluent is discharged. The term does not include private drains, unnamed conveyances, retention and detention basins, or constructed wetlands used as treatment.

Recharge. Replenishment of groundwater reservoirs by infiltration and transmission from the outcrop of an aquifer or from permeable soils.

Recurrence Interval. A statistical expression of the average time between floods equaling or exceeding a given magnitude.

Redevelopment. Alterations of a property that change a site or building in such a way that there is disturbances of one (1) acre or more of land. The term does not include such activities as exterior remodeling.

Regional Pond. A detention/retention basin sized to detain/retain the runoff from the entire watershed, on-site and off-site, tributary to the pond's outlet.

Regulated Area. Area under the stormwater regulatory jurisdiction of the City of Plymouth.

Regulated Drain. A drain subject to the provisions of the Indiana Drainage Code, I.C.-36-9-27.

Regulatory or 100-Year Flood. The discharge or elevation associated with the 100-year flood as calculated by a method and procedure which is acceptable to and approved by the Indiana Department of Natural Resources and the Federal Emergency Management Agency. The "regulatory flood" is also known as the "base flood".

Regulatory Floodway. See Floodway.

Release Rate. The amount of storm water release from a storm water control facility per unit of time.

Reservoir. A natural or artificially created pond, lake or other space used for storage, regulation or control of water. May be either permanent or temporary. The term is also used in the hydrologic modeling of storage facilities.

Retail Gasoline Outlet. An operating gasoline or diesel fueling facility whose primary function is the resale of fuels. The term applies to facilities that create five thousand (5,000) or more square feet of impervious surfaces, or generate an average daily traffic count of one hundred (100) vehicles per one thousand (1,000) square feet of land area.

Retention Basin. A type of storage practice, that has no positive outlet, used to retain storm water run-off for an indefinite amount of time. Runoff from this type of basin is removed only by infiltration through a porous bottom or by evaporation.

Retention. The storage of stormwater to prevent it from leaving the development site. May be temporary or permanent.

Retention Facility. A facility designed to completely retain a specified amount of stormwater runoff without release except by means of evaporation, infiltration or pumping. The volumes are often referred to in units of acre-feet.

Return Period - The average interval of time within which a given rainfall event will be equaled or exceeded once. A flood having a return period of 100 years has a one percent probability of being equaled or exceeded in any one year.

Revetment. Facing of stone or other material, either permanent or temporary, placed along the edge of a stream to stabilize the bank and protect it from the erosive action of the stream. Also see Revetment riprap.

Right-of-Way (ROW) for a County Drain. The statutory right of way as defined by Indiana Code for a regulated drain.

Riparian Habitat. A land area adjacent to a waterbody that supports animal and plant life associated with that waterbody.

Riparian Zone. Of, on, or pertaining to the banks of a stream, river, or pond.

Riprap. Broken rock, cobble, or boulders placed on earth surfaces, such as the face of a dam or the bank of a stream, for protection against the action of water (waves). Revetment riprap is material graded such that: (1) no individual piece weighs more than 120 lbs. and (2) 90-100% will pass through a 12-inch sieve, 20-60% through a 6-inch sieve, and not more than 10% through a 12-inch sieve.

River Restoration. Restoring the channel of a stream or ditch to its perceived original, non-obstructed capacity by means of clearing & snagging, obstruction removal, and inexpensive streambank protection measures. The term "restoration", as noted, does not necessarily imply restoration or improvement of water quality or habitat within the channel or its adjacent area.

Riverine. Relating to, formed by, or resembling a stream (including creeks and rivers).

Runoff Coefficient - A decimal fraction relating the amount of rain which appears as runoff and reaches the storm drain system to the total amount of rain falling. A coefficient of 0.5 implies that 50 percent of the rain falling on a given surface appears as storm water runoff.

Runoff. That portion of precipitation that flows from a drainage area on the land surface, in open channels, or in stormwater conveyance systems.

Sand. (1) Soil particles between 0.05 and 2.0 mm in diameter. (2) A soil textural class inclusive of all soils that are at least 70% sand and 15% or less clay.

Sanitary Backup. The condition where a sanitary sewer reaches capacity and surcharges into the lowest area.

Scour. The clearing and digging action of flowing water.

Sediment. Solid material (both mineral and organic) that is in suspension, is being transported, or has been moved from its site of origin by air, water, gravity, or ice and has come to rest on the earth's surface.

Sediment Forebay. See "Forebay".

Sedimentation. The process that deposits soils, debris and other unconsolidated materials either on the ground surfaces or in bodies of water or watercourses.

Seepage. The passage of water or other fluid through a porous medium, such as the passage of water through an earth embankment or masonry wall.

Sensitive Water. A water body in need of priority protection or remediation base on its:

- providing habitat for threatened or endangered species,
- usage as a public water supply intake,
- relevant community value,
- usage for full body contact recreation,
- exceptional use classification as found in 327 IAC 2-1-11(b), outstanding state resource water classification as found in 327 IAC 2-1-2(3) and 327 IAC 2-1.5-19(b).

Settling Basin. An enlargement in the channel of a stream to permit the settling of debris carried in suspension.

Silt Fence. A fence constructed of wood or steel supports and either natural (e.g. burlap) or synthetic fabric stretched across area of non-concentrated flow during site development to trap and retain on-site sediment due to rainfall runoff.

Silt. (1) Soil fraction consisting of particles between 0.002 and 0.05 mm in diameter. (2) A soil textural class indicating more than 80% silt.

Siphon - A closed conduit or portion of which lies above the hydraulic grade line, resulting in a pressure less than atmospheric and requiring a vacuum within the conduit to start flow. A siphon utilizes atmospheric pressure to effect or increase the flow of water through a conduit. An inverted siphon is used to carry storm water flow under an obstruction such as a sanitary sewer.

Site. The entire area included in the legal description of the land on which land disturbing activity is to be performed.

Silvicultural. the practice of controlling the establishment, growth, composition, health, and quality of forests to meet diverse needs and values.

1. Nonpoint activities include source silvicultural activities such as nursery operations, site preparation, reforestation and subsequent cultural treatment, thinning, prescribed burning, pest and fire control, harvesting operations, surface drainage, or road construction and maintenance from which there is natural runoff. Some of these activities (such as stream crossing for roads) may involve the placement of dredged or fill material which may require a CWA section 404 permit and a 401 Water Quality Certification.
2. Point source activities include any discernible, confined and discrete conveyance related to rock crushing, gravel washing, log sorting, or log storage facilities which are operated in connection with silvicultural activities and from which pollutants are discharged into waters of the United States or the State.

Slope. Degree of deviation of a surface from the horizontal, measured as a numerical ratio or percent. Expressed as a ratio, the first number is commonly the horizontal distance (run) and the second is the vertical distance (rise)--e.g., 2:1. However, the preferred method for designation of slopes is to clearly identify the horizontal (H) and vertical (V) components (length (L) and Width (W) components for horizontal angles). Also note that according to international standards (Metric), the slopes are presented as the vertical or width component shown on the numerator--e.g., 1V:2H. Slope expressions in the Ordinance or these Technical Standards follow the common presentation of slopes--e.g., 2:1 with the metric presentation shown in parenthesis--e.g., (1V:2H). Slopes can also be expressed in "percent". Slopes given in percent are always expressed as (100*V/H) --e.g., a 2:1 (1V:2H) slope is a 50% slope.

Soil and Water Conservation District. A public organization created under state law as a special-purpose district to develop and carry out a program of soil, water, and related resource conservation, use, and development within its boundaries. A subdivision of state government with a local governing body, established under IC 14-32.

Soil. The unconsolidated mineral and organic material on the immediate surface of the earth that serves as a natural medium for the growth of land plants.

Solid Waste. Any garbage, refuse, debris, or other discarded material.

Special Flood Hazard Area. An area that is inundated during the 100-Year flood.

Spill. The unexpected, unintended, abnormal, or unapproved dumping, leakage, drainage, seepage, discharge, or other loss of petroleum, hazardous substances, extremely hazardous substances, or objectionable substances. The term does not include releases to impervious surfaces when the substance does not migrate off the surface or penetrate the surface and enter the soil.

Spillway. A waterway in or about a hydraulic structure, for the escape of excess water.

Standard Project Flood. A term used by the U.S. Army Corps of Engineers to designate a flood that may be expected from the most severe combination of meteorological and hydrological conditions that are considered reasonable characteristics of the geographical area in which the drainage basin is located, excluding extremely rare combinations. The peak flow for a standard project flood is generally 40 – 60 percent of the probable maximum flood for the same location.

Stilling Basin - A basin used to slow water down or dissipate its energy.

Storage Practices. Any structural BMP intended to store or detain stormwater and slowly release it to receiving waters or drainage systems. The term includes detention and retention basins.

Storm Drain Signing. Any marking procedure that identifies a storm sewer inlet as draining directly to a receiving waterbody so as to avoid dumping pollutants. The procedures can include painted or cast messages and adhesive decals.

Storm Duration. The length of time that water may be stored in any stormwater control facility, computed from the time water first begins to be stored.

Storm Event. An estimate of the expected amount of precipitation within a given period of time. For example, a 10-yr. frequency, 24-hr. duration storm event is a storm that has a 10% probability of occurring in any one year. Precipitation is measured over a 24-hr. period.

Storm Frequency. The time interval between major storms of predetermined intensity and volumes of runoff--e.g., a 5-yr., 10-yr. or 20-yr. storm.

Storm Sewer. A closed conduit for conveying collected storm water, while excluding sewage and industrial wastes. Also called a storm drain.

Stormwater Drainage System - All means, natural or man-made, used for conducting storm water to, through or from a drainage area to any of the following: conduits and appurtenant features, canals, channels, ditches, storage facilities, swales, streams, culverts, streets and pumping stations.

Stormwater Facility. All ditches, channels, conduits, levees, ponds, natural and manmade impoundments, wetlands, tiles, swales, sewers and other natural or artificial means of draining surface and subsurface water from land.

Stormwater Pollution Prevention Plan. A plan developed to minimize the impact of storm water pollutants resulting from construction activities.

Stormwater Quality Management Plan. A comprehensive written document that addresses stormwater runoff quality.

Stormwater Quality Measure. A practice, or a combination of practices, to control or minimize pollutants associated with storm water runoff.

Stormwater Runoff. The water derived from rains falling within a tributary basin, flowing over the surface of the ground or collected in channels or conduits.

Stormwater. Water resulting from rain, melting or melted snow, hail, or sleet.

Stream Gauging. The quantitative determination of streamflow using gauges, current meters, weirs, or other measuring instruments at selected locations (see Gauging station').

Stream Length. The length of a stream or ditch, expressed in miles, from the confluence of the stream or ditch with the receiving stream to the upstream extremity of the stream or ditch, as indicated by the solid or dashed, blue or purple line depicting the stream or ditch on the most current edition of the seven and one-half (72) minute topographic quadrangle map published by the United States Geological Survey, measured along the meanders of the stream or ditch as depicted on the map.

Stream. See intermittent stream, Perennial stream, Receiving stream.

Streambanks. The usual boundaries (not the flood boundaries) of a stream channel. Right and left banks are named facing downstream.

Strip development. A multi-lot project where building lots front on an existing road.

Structure. Refers to a structure that is principally above ground and is enclosed by walls and a roof. The term includes but is not limited to, a gas or liquid storage tank, a manufactured home or a prefabricated building, and recreational vehicles to be installed on a site for more than 180 days.

Structural Engineer. A person licensed under the laws of the State of Indiana to engage in the designing or supervising of construction, enlargement or alteration of structures or any part thereof.

Structural Floodplain. Management Measures. Those physical or engineering measures employed to modify the way foods behave, (e.g., dams, dikes, levees, channel enlargements and diversions).

Subarea/Sub-Basin. Portion of a watershed divided into homogenous drainage units which can be modeled for purposes of determining runoff rates. The subareas/sub-basins have distinct boundaries, as defined by the topography of the area.

Subdivision. Any land that is divided or proposed to be divided into lots, whether contiguous or subject to zoning requirements, for the purpose of sale or lease as part of a larger common plan of development or sale.

Subdivision, Minor. The subdivision of a parent parcel into any combination of not more than three (3) contiguous or non-contiguous new residential, commercial, or industrial building sites. The parcel shall front upon an existing street which is an improved right-of-way maintained byt eh County or other governmental entity and not involve any new street.

Subsoil. The B horizons of soils with distinct profiles. In soils with weak profile development, the subsoil can be defined as the soil below which roots do not normally grow.

Subsurface Drain. A pervious backfield trench, usually containing stone and perforated pipe, for intercepting groundwater or seepage.

Subwatershed. A watershed subdivision of unspecified size that forms a convenient natural unit. See also Subarea.

Sump Failure. A failure of the sump pump that results in inundation of crawl space or basement.

Sump Pump. A pump that discharges seepage from foundation footing drains.

Surcharge. Backup of water in a sanitary or storm sewer system in excess of the design capacity of the system.

Surface Runoff. Precipitation that flows onto the surfaces of roofs, streets, the ground, etc., and is not absorbed or retained by that surface but collects and runs off.

Suspended Solids. Solids either floating or suspended in water.

Swale. An elongated depression in the land surface that is at least seasonally wet, is usually heavily vegetated, and is normally without flowing water. Swales conduct stormwater into primary drainage channels and may provide some groundwater recharge.

Tailwater. The water surface elevation at the downstream side of a hydraulic structure (i.e. culvert, bridge, weir, dam, etc.).

Temporary Stabilization. The covering of soil to ensure its resistance to erosion, sliding, or other movement. The term includes vegetative cover, anchored mulch, or other non-erosive material applied at a uniform density of seventy percent (70%) across the disturbed area.

Thalweg. The deepest point (or centerline) of a channel.

Tile Drain. Pipe made of perforated plastic, burned clay, concrete, or similar material, laid to a designed grade and depth, to collect and carry excess water from the soil.

Tile Drainage. Land drainage by means of a series of tile lines laid at a specified depth, grade, and spacing.

Time of Concentration (tc). The travel time of a particle of water from the most hydraulically remote point in the contributing area to the point under study. This can be considered the sum of an overland flow time and times of travel in street gutters, storm sewers, drainage channels, and all other drainage ways.

Topographic Map. Graphical portrayal of the topographic features of a land area, showing both the horizontal distances between the features and their elevations above a given datum.

Topography. The representation of a portion of the earth's surface showing natural and man-made features of a give locality such as rivers, streams, ditches, lakes, roads, buildings and most importantly, variations in ground elevations for the terrain of the area.

Topsoil. (1) The dark-colored surface layer, or a horizon, of a soil; when present it ranges in depth from a fraction of an inch to 2-3 ft. (2) Equivalent to the plow layer of cultivated soils. (3) Commonly used to refer to the surface layer(s), enriched in organic matter and having textural and structural characteristics favorable for plant growth.

Total Maximum Daily Load. Method used to establish allowable loadings for specified pollutants in a surface water resource to meet established water quality standards.

Toxicity. The characteristic of being poisonous or harmful to plant or animal life. The relative degree or severity of this characteristic.

TP-40 Rainfall. Design storm rainfall depth data for various durations published by the National Weather Service in their Technical Paper 40 dated 1961.

Trained Individual. An individual who is trained and experienced in the principles of storm water quality, including erosion and sediment control as may be demonstrated by state registration, professional certification (such as CESSWI and/or CPESC certification), or other documented and applicable experience or coursework as deemed sufficient by the City of Plymouth that enable the individual to make judgments regarding storm water control or treatment and monitoring.

Transition Section. Reaches of the stream of floodway where water flows from a narrow cross-section to a wide cross-section or vice-versa.

Tributary. Based on the size of the contributing drainage area, a smaller watercourse which flows into a larger watercourse.

Turbidity. (1) Cloudiness of a liquid, caused by suspended solids. (2) A measure of the suspended solids in a liquid.

Underdrain. A small diameter perforated pipe that allows the bottom of a detention basin, channel or swale to drain.

Unified Soil Classification System. A system of classifying soils that is based on their identification according to particle size, gradation, plasticity index, and liquid limit.

Uniform Flow. A state of steady flow when the mean velocity and cross-sectional area remain constant in all sections of a reach.

Unit Hydrograph. A unit hydrograph is the hydrograph that results from one inch of precipitation excess generated uniformly over the watershed at a uniform rate during a specified period of time.

Urban Drain. A drain defined as “Urban Drain” in Indiana Drainage Code.

Urbanization. The development, change or improvement of any parcel of land consisting of one or more lots for residential, commercial, industrial, institutional, recreational or public utility purposes.

Vegetative Practices. Any nonstructural or structural BMP that, with optimal design and good soil conditions, utilizes various forms of vegetation to enhance pollutant removal, maintain and improve natural site hydrology, promote healthier habitats, and increase aesthetic appeal. Examples include grass swales, filter strips, buffer strips, constructed wetlands, and rain gardens.

Vegetative Stabilization. Protection of erodible or sediment producing areas with: permanent seeding (producing long-term vegetative cover), short-term seeding (producing temporary vegetative cover), or sodding (producing areas covered with a turf of perennial sod-forming grass).

Water Course. Any river, stream, creek, brook, branch, natural or man-made drainage way in or into which stormwater runoff or floodwaters flow either regularly or intermittently.

Water Quality. A term used to describe the chemical, physical, and biological characteristics of water, usually in respect to its suitability for a particular purpose.

Water Resources. The supply of groundwater and surface water in a given area.

Water Table. (1) The free surface of the groundwater. (2) That surface subject to atmospheric pressure under the ground, generally rising and failing with the season or from other conditions such as water withdrawal.

Waterbody. Any accumulation of water, surface, or underground, natural or artificial.

Watercourse. Any river, stream, creek, brook, branch, natural or man-made drainageway in or into which stormwater runoff or floodwaters flow either continuously or intermittently.

Watershed Area. All land and water within the confines of a drainage divide. See also Watershed.

Watershed. The region drained by or contributing water to a specific point that could be along a stream, lake or other stormwater facilities. Watersheds are often broken down into subareas for the purpose of hydrologic modeling.

Waterway. A naturally existing or manmade open conduit or channel utilized for the conveyance of water.

Weir. A channel-spanning structure for measuring or regulating the flow of water.

Wellhead Protection Area. Has the meaning set forth at 327 IAC 8-4.1-1(27).

Wet-Bottom Detention Basin (Retention Basin) - A basin designed to retain a permanent pool of water after having provided its planned detention of runoff during a storm event.

Wetlands. Areas that are inundated or saturated by surface water or groundwater at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions.

Appendix B

B1 –Stormwater Permit Forms

Drainage and SWPPP Performance Bond Form

Application / Checklist / Invoice for Stormwater Management Permit (Drainage Plan and SWPPP)

Construction / SWPPP Technical Review Form

Petition to County Drainage Board and Engineer's Statement

Statement of Financial Responsibility

BMP Maintenance Agreement

Project Check-In Checklist

Enforcement Matrix

B2 – Construction Inspection/Completion Forms

Construction Self-Monitoring Report - Plymouth

Construction Self-Monitoring Report - IDEM

Completion Affidavit for SWPPP Notice of Termination

B3 – Individual Lot/Parcel Permit Forms

Instructions for Individual Lot Permit Request

Individual Lot Permit Request Application

Individual Lot Typical Erosion & Sediment Control Plan and Certification

Plot Plan Review Form

Individual Lot Stormwater Pollution Prevention Requirements

B1 – Stormwater Permit Forms

SITE IMPROVEMENT AND DEVELOPMENT BOND Bond No. _____

We, _____, as Developer, and _____, a corporation organized under the laws of the State of _____, as Surety, are held and firmly bound unto **CITY OF PLYMOUTH, IN**, a municipal corporation, as the City, in the sum of:

_____ Dollars ($_____),

to be paid to the City, for which payment well and truly to be made, we bind ourselves, our heirs, executors and administrators, successors and assigns, pursuant to the terms of this Site Improvement and Development Bond.

WHEREAS, the Developer has submitted to the City for its approval the following development project:

_____ (the "Development Project"); and

WHEREAS, the City has approved or is about to approve the Development Project plans in its records, a copy of which is by reference made a part hereof; and

WHEREAS, the City is requiring the Developer to post a bond equal to the total cost to complete the following Development Project requirements: **DRAINAGE AND SWPPP IMPRVOVEMENTS,** The manner in which the aforementioned requirements shall be completed is set forth, as applicable, in the Development's approved plans, any Development Agreement, any permits or approval letters issued for the Development, amendments made during construction to the satisfaction of the City, and any other applicable law or regulation. The Bond as determined from the sum of:

- Drainage Improvements - $_____
- SWPPP Improvements - $_____

The Surety agrees that the bond shall remain in full force and effect until such time as the City releases such bond, which release shall occur upon both of the following occurring: (1) City receives all completion documents including asbuilt plans, verified compliance with proposed plans, certificate of completion, City issuance to the Developer of a notice of initial acceptance or approval stating that all of the public improvement obligations have been completed; and (2) delivery to the City of assurance that the Developer shall maintain and repair all post-construction improvements that the City has initially accepted unless public infrastructure is dedicated to the city. Public infrastructure dedicated to the City must meet all written City requirements for dedication.

Notification that the bond has been released or reduced, requests for funds pursuant to the bond, and other requests or inquiries regarding the bond will be directed to the following:

Issuing Company Name: _____
Mailing Address: _____
Phone Number: _____ Email: _____

NOW, THEREFORE, the condition of this obligation is such that if the Developer shall truly and faithfully perform to the satisfaction of the City its obligations set forth above, then this obligation shall be void; otherwise to remain in full force and effect.

DATED this ____ day of _____, 20___.

DEVELOPER:

_____

_____

SURETY:

_____

_____

By:

_____

, Attorney-in-Fact

Construction Stormwater Manager
Department of Engineering
900 Oakhill Avenue, P.O. BOX 492 Plymouth, Indiana 46563
Phone: (574) 936-3614 | Fax: (574) 936-3017

Requirements for Drainage and Storm Water Pollution Prevention Plan (SWPPP) Development and Completion

To Whom it May Concern,

Please find below a list of the attached information and forms pertinent to the City of Plymouth's drainage and SWPPP submittals and procedures. For additional information, please see the City of Plymouth's Stormwater Management Ordinance (approved June 2025).

Drainage Plan:

Drainage Plan Application	*To be submitted to the Engineering Office along with Drainage plans*
Drainage Plan Checklist	*Basis of the design plan submission review*
Drainage Plan Invoice	

(Email a digital copy of all the above submissions to the City Engineer)

SWPPP:

SWPPP Application	*To be submitted to the Engineering Office along with SWPPP plans*
SWPPP Checklist	*Basis of the design plan submission review*
SWPPP Invoice	
IDEM Notice of Intent (NOI)	*Use latest form downloaded from IDEM's website*
IDEM Notice of Sufficiency	*Will receive from IDEM after NOI approval, email copy to the Engineering Office*

(Email a digital copy of all the above submissions to the City Engineer)

Closeout:

Drainage Completion Affidavit	*To be submitted to the Engineering Office*
Drainage As-Builts Drawings	*To be submitted to the Engineering Office*
SWPPP Completion Affidavit	*To be submitted to the Engineering Office*
SWPPP As-Built Drawings	*To be submitted to the Engineering Office*
SWPPP Notice of Termination (NOT)	*Use latest form downloaded from IDEM's website*

(Email a digital copy of all the above submissions to the Engineer Department)

Construction Stormwater Manager
Department of Engineering
900 Oakhill Avenue, P.O. BOX 492 Plymouth, Indiana 46563
Phone: (574) 936-3614 | Fax: (574) 936-3017

(Continued on Next Page)

General Information:

- Submitted hard copies can be mailed and/or emailed to:

Clerk-Treasurer 124 N Michigan St Plymouth, IN 46563 clerktreas@plymouthin.com	Construction Stormwater Manager 900 Oakhill Ave. Plymouth, IN 46563 stormwater@plymouthin.com

- Digital copies of all material shall be emailed to the Engineering Department at publicworks@plymouthin.com and gis@plymouthin.com

- Financial Guarantees
  - Financial Guarantees must be in place before approvals of drainage and/or SWPPP are finalized.
  - No Building Permits or Improvement Location Permits will be issued by the City of Plymouth without a drainage and/or SWPPP application signed by the Engineering Department.

- For more information refer to:
  - Stormwater Management Ordinance (approved June 2025): https://www.plymouthin.com/department/index.php?structureid=39
  - The City of Plymouth Enforcement Matrix (attached)

- Plan Review
  - Currently, MS4's are not subject to IDEM's 28-day review period requirement.
    - Per Indiana House Enrolled Act No. 1266 - Erosion and Sediment Control in Construction Projects (effective July 2019), a review authority has 10 working days for plan review of small projects* and 14 working days for large projects*.
    - If a submission is found to be incomplete, the City of Plymouth will notify the project site owner immediately; the plan review timeline resets from the day corrections / additions are received by the City of Plymouth.
  - From time to time the City of Plymouth may use outside consultants to review submissions. To avoid delays, follow the checklist and ensure all the pertinent information is easily identifiable on the plans and in the calculations.

MCM Implication		GP Requirement	Annual Report Requirement	Ordinance Implication
☐ Public Education / Involvement	☐ Post-Construction Site Control	☒ Yes, Section(s): 4.5(c)	☐ Yes, Section(s):	☒ Yes [x] Current [ ] New
☐ Illicit Discharge	☐ Good Housekeeping	☐ No	☐ No	☐ No
☒ Construction Site Control				

Project Number

(Completed by City of Plymouth)

Drainage Plan - Application

Part 1 of 3

To: City of Plymouth
Clerk-Treasurer
124 North Michigan Street, P.O. Box 492
Plymouth, Indiana 46563
clerktreas@plymouthin.com

Copy: City of Plymouth
Department of Engineering
c/o: Construction Stormwater Manager
900 Oakhill Avenue, P.O. BOX 492
Plymouth, Indiana 46563
stormwater@plymouthin.com

Applicant: _____
Street: _____
City: _____
State: _____
Zip Code: _____
Phone: _____
Email: _____

Property Owner: _____
Street: _____
City: _____
State: _____
Zip Code: _____
Phone: _____
Email: _____

REQUIREMENT: This form must be completed and approved prior to the issuance of a building permit and/or construction or installation of any structure, building, site improvement, or combination thereof, which has a ground area of 4,000 square feet or greater located within the city limits of the City of Plymouth. This includes drives, parking areas, or other impervious or semi-impervious improvements.

Name of Project: _____ **Address:** _____

Type of Construction: ☐ Building ☐ Residential ☐ Commercial ☐ Industrial ☐ Subdivision
☐ Street, Driveway, Parking Lot ☐ Other Improvement; Specify _____

Site/Lot Size: _____ ☐ Square Feet / ☐ Acres

Existing Paved Area: _____ Square Feet of; ☐ Asphalt/Concrete ☐ Stone/Gravel

Existing Hard Surface area: _____ Square Feet of; ☐ Buildings ☐ Other _____

Final Size of Buildings: _____ Square Feet of Ground Floor Area

Final Area of Paving: _____ Square Feet of; ☐ Asphalt/Concrete ☐ Stone/Gravel

Total Coverage: _____ Square Feet of Impervious or Semi-Impervious Surface Area

Estimated Start Date of Construction: _____ **Estimated Completion date:** _____

Drainage Outlet into County Regulated Drain?: ☐ Yes ☐ No (If Yes attach County Drainage Board approval)

Construction in Flood Hazard Area?: ☐ Yes ☐ No (If Yes attach Indiana Department of Natural Resources (IDNR) approval)

Approval requested: ☐ Preliminary ☐ Final ☐ Amendment to Approved Plan

Financial Guarantee: Type: _____ Amount: _____

Signature: _____ **Date:** _____

This application must be accompanied by plans and specifications prepared by an Indiana licensed professional engineer, architect, or land surveyor in accordance with the requirement of the City of Plymouth, Indiana, Stormwater Management Ordinance.

THIS SECTION TO BE COMPLETED BY THE CITY OF PLYMOUTH

Scheduled Date for Consideration by Board of Public Works and Safety: _____ Time: _____
Disposition of Application: ☐ Approval ☐ Disapproval ☐ Conditional Approval (Attached) ☐ Waivered Financial
Guarantee Received Date: _____ Amount: _____ Type: _____
Date approved by Board of Public Works and Safety: _____
Authorized Signature: _____ Date: _____

Project Number

(Completed by City of Plymouth)

Stormwater Pollution Prevention Plan (SWPPP) - Application
Part 1 of 3

Project Information

Name: _____ Plan Submittal Date: _____

Hydrologic Unit Code: _____ Latitude: _____ Longitude: _____

Address: _____ City: _____ Zip: _____ County: _____

Civil Township: _____ Quarter: _____ Section: _____ Township: _____ Range: _____

Location Description: _____

_____

Contact Information

Applicant

Name: _____

Company: _____

Address: _____

City: _____

State: _____ Zip: _____

Phone: _____

Email: _____

Property Owner

Name: _____

Company: _____

Address: _____

City: _____

State: _____ Zip: _____

Phone: _____

Email: _____

SWPPP Preparer

Name: _____

Company: _____

Address: _____

City: _____

State: _____ Zip: _____

Phone: _____

Email: _____

Construction BMP Contact

Name: _____

Company: _____

Address: _____

City: _____

State: _____ Zip: _____

Phone: _____

Email: _____

Project Description

☐ Residential Single Family ☐ Commercial ☐ Other: _____

☐ Residential Multi-Family ☐ Industrial

Area of Site Improvements

Lot Size: _____ ☐ Ft^2 ☐ Acre(s) Area of Disturbed Land: _____ ☐ Ft^2 ☐ Acre(s)

Area of Buildings: _____ ☐ Ft^2 ☐ Acre(s) Area of Parking: _____ ☐ Ft^2 ☐ Acre(s)

Total Area of Impervious / Semi-Impervious Surface(s): _____ ☐ Ft^2 ☐ Acre(s)

Drainage Outlet and Floodway Implications

☐ County Legal Drain ☐ City MS4 ☐ No Outlet ☐ Other: _____

Is This Site in a Flood Hazard / Plain Area? ☐ Yes ☐ No If Yes, Attach IDNR Approval

Project Administration

Estimated Start Date: _____ Estimated Completion Date: _____

Financial Guarantee Type: _____ Amount: _____

Applicant Signature: _____ Date: _____

Project Number

(Completed by City of Plymouth)

Drainage Plan - Checklist
Part 2 of 3

Name of Project: _____

Address or Location Description: _____

_____

This form is the second part of a three-part application. Part 1 is the Application and Part 3 is the Invoice. This sheet is part of the Checklist utilized in the Substantial Submittal Review and the Detailed Review.

1. Topographic and Soils Map		Yes	N/A	Page Number	Section, Paragraph
a.	Soils are to be identified. If SCS is utilized, identify hydrologic classification.				
b.	Contour intervals shall be 1Ft. when slopes are less than 4%. If slopes are greater than 4%, utilize 2-Ft. contours.				
c.	Location of streams and other flood water runoff channels. Identify 100-year flood / runoff elevations.				
d.	Normal water level (elevation) and delineation of lakes, ponds, swamps, detention basins, and flood plains. Include lines of inflow and outflow.				
e.	Location of any recorded or visible regulated drains, farm drains, inlets, and outfalls.				
f.	Location of any recorded or visible storm, sanitary, and combined sewers and outfalls.				
g.	Location of any recorded or visible septic tank systems and outlets.				
h.	Location of any recorded or visible seeps, springs, flowing and other wells.				

2. Preliminary Drainage Plan		Yes	N/A	Page Number	Section, Paragraph
a.	Extent and area of each watershed affecting the design of the retention / detention facilities.				
b.	Preliminary layout and design of proposed storm sewers and outlet locations. Show receiving stream(s) / channel(s).				
c.	Design of proposed street system, including depressed pavements used to convey or temporarily store overflow from heavier storms.				
d.	Locations, cross sections, and waterway openings and the basis for design (calculations) of culverts, bridges, and storm sewers.				
e.	Materials, elevations, waterway openings and basis for design (calculations) of culverts, bridges, and storm sewers.				
f.	Existing ponds and basins to be maintained, enlarged, or altered as well as new ponds or basins to be constructed and the basis for their design (calculations).				
g.	The estimated depth and the amount of storage required in the new ponds or basins (water elevations and supporting calculations).				
h.	The estimated location and amount (area) of imperious surface to be constructed.				
i.	Any interim plan which is to be incorporated into the development, pending completion of the final drainage plan.				

Plymouth INDIANA

Project Number

(Completed by City of Plymouth)

3. Valley Cross-Section		Yes	N/A	Page Number	Section, Paragraph
a.	Typical cross-sections of existing and proposed channels. Identify 100 -year runoff / flood levels.				

4. Site Plan		Yes	N/A	Page Number	Section, Paragraph
a.	Site plan is drawn to scale and identifies site improvements and drainage facilities.				

5. Final Drainage Plans		Yes	N/A	Page Number	Section, Paragraph
a.	Extent and area of each watershed affecting the design of the retention / detention facilities.				
b.	Design of storm sewers and outlet locations. Show receiving stream(s) / channel(s).				
c.	Design of proposed street system including depressed pavements used to convey or temporarily store overflow from heavier storms.				
d.	Existing streams and floodplains to be maintained and new channels to be constructed. Locations, cross-sections, and profiles.				
e.	Proposed culverts and bridges to be built. Their materials, elevations, waterway openings, and basis for design.				
f.	Existing storage basins and ponds to be maintained, enlarged, or otherwise altered.				
g.	The estimated location and amount (area) of imperious surface to be constructed.				
h.	Slope, type, and size of all sewers and other waterways.				
i.	Plot or tabulation of storage volume(s) corresponding with water surface elevation(s). A tabulation of outflow rates.				
j.	Adequate erosion control during and post-construction				
k.	Adequate emergency overflow spillway and detention basin top of bank Freeboard (6” higher than peak water surface across spillway) shown.				

6. Report		Yes	N/A	Page Number	Section, Paragraph
a.	Description of the proposed development.				
b.	Current land use conditions.				
c.	Method of hydraulic and hydrologic analysis used and any special assumptions of special conditions.				
d.	Results of the analysis (basis for design calculations).				
e.	Recommended drainage control facilities.				

7. Permission to Connect to Legal Drain		Yes	N/A	Page Number	Section, Paragraph
a.	Written permission from the County Surveyor must be obtained to outlet into a legal drain (this includes any work in the legal drain easement).				

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Project Number (Completed by City of Plymouth)

8. Stormwater Detention Calculations		Yes	N/A	Page Number	Section, Paragraph
a.	Discharge from Site at a 10-year release rate or less (confirm with City Engineering).				
b.	Proper discharge piping modeling or Orifice calculations included.				
c.	Detention storage requirements (Peak 100-yearr event) calculated. Required storage calculations and proposed pond volumes.				
d.	Retention storage requirements calculated. Required storage calculations and proposed pond volumes (to hold the 100-year, 24-hour storm event).				
e.	Proper Stage-Discharge storage provided (with a 6% “oversize” safety factor).				
f.	Adequate Emergency Overflow Spillway calculations provided.				

9. Stormwater Conveyance		Yes	N/A	Page Number	Section, Paragraph
a.	Flow and velocity of runoff calculated in pipe systems and open channels.				
b.	Grade elevations set to drain into inlets; manhole rim elevations set.				
c.	Manhole invert elevations calculated and set.				
d.	Watershed areas shown for all inlets and culverts.				
e.	Inlet grate flow capacity checked				
f.	Proper outlet and channel protection.				

10. Water Quality (WQ) (For Construction Sites over 1 acre)		Yes	N/A	Page Number	Section, Paragraph
a.	Water quality measures were appropriately selected, designed according to OCRA Green Infrastructure Curriculum (Appendix C) and calculated to meet 80% TSS removal efficiency.				
b.	Extended Detention sized correctly. (WQV=(P)(Rv)(A)/12				
c.	Detention basin designed and shaped to maximize detention time for solids removal.				
d.	Low flow orifice sized to detain Calculated WQV for at least 24hrs?				
e.	Flow-Through BMP selected, sized accordingly, and maintenance accounted for?				

11. Itemized Drainage Financial Guarantee		Yes	N/A	Page Number	Section, Paragraph
a.	Provide an itemized list of financial guarantee.				
The Itemized Financial Guarantees shall be a list of drainage elements such as structures, conveyance system(s), detention / retention basins, etc. indicating the: amount to be installed, unit price, total cost for each item, and total cost for the project. Cost for closeout as-built drawings shall be included.					

Project Number

(Completed by City of Plymouth)

SWPPP - Checklist
Part 2 of 3

Name of Project: _____

Address or Location Description: _____

_____

This form is the second part of a three-part application. Part 1 is the Application and Part 3 is the Invoice. This sheet is part of the Checklist utilized in the Review..
See IDEM's CSGP Guidance, for more information: https://www.in.gov/idem/stormwater/files/csgp_development_guidance.pdf

Project Information		Yes	N/A	Page Number	Section, Paragraph
1.	Provide a written description of the project's construction components and a summary of the proposed land use.				

Construction Plan – General Plan Components (Section A)		Yes	N/A	Page Number	Section, Paragraph
1.	Index of the location of required plan elements in the construction plan (this form may be used as the index).				
2.	A vicinity map depicting the project site location in relationship to recognizable local landmarks, towns, and major roads.				
3.	Narrative of the nature and purpose of the project.				
4.	Latitude and longitude to the nearest fifteen (15) seconds.				
5.	Legal description of the project site.				
6.	11 by 17-inch plat showing building lot numbers / boundaries and road layout / names.				
7.	Boundaries of the one hundred (100) year floodplains, floodway fringes, and floodways.				
8.	Land use of all adjacent properties (especially upstream watershed(s)).				
9.	Identification of a U.S. EPA approved or established Total Maximum Daily Load (TMDL).				
10.	Name(s) of the receiving water(s).				
11.	Identification of discharges to a water on the current 303(d) list of impaired waters and the pollutant(s) for which it is impaired.				
12.	Soils map of the predominate soil types.				
13.	Identification and location of all known wetlands, lakes, and water courses on or adjacent to the project site (construction plan, existing site layout).				
14.	Identification of any other state or federal water quality permits or authorizations that are required for construction activities.				
15.	Identification and delineation of existing cover, including natural buffers.				
16.	Existing site topography at an interval appropriate to indicate drainage patterns.				
17.	Location(s) where run-off enters the project site.				
18.	Location(s) where run-off discharges from the project site prior to land disturbance.				
19.	Location of all existing structures on the project site.				
20.	Existing permanent retention or detention facilities, including manmade wetlands, designed for the purpose of stormwater management.				

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21.	Locations where stormwater may be directly discharged into groundwater, such as abandoned wells, sinkholes, or karst features.				
22.	Size of the project area expressed in acres.				
23.	Total expected land disturbance expressed in acres.				
24.	Proposed final topography.				
25.	Locations and approximate boundaries of all disturbed areas.				
26.	Locations, size, and dimensions of all stormwater drainage system such as culverts, stormwater sewer, and conveyance channels.				
27.	Locations of specific points where stormwater and non-stormwater discharges will leave the project site.				
28.	Location of all proposed site improvements, including roads, utilities, lot delineation and identification, proposed structures, and common areas.				
29.	Location of all on-site and off-site soil stockpiles and borrow areas.				
30.	Construction support activities that are expected to be part of the project.				
31.	Location of any in-stream activities that are planned for the project including, but not limited to, stream crossings and pump arounds.				

SWPPP – Construction Component (Section B)		Yes	N/A	Page Number	Section, Paragraph
1.	Description of the potential pollutant generating sources and pollutants, including all potential non-stormwater discharges.				
2.	Stable construction entrance / exit locations and specifications.				
3.	Specifications for temporary and permanent stabilization.				
4.	Sediment control measures for concentrated flow areas.				
5.	Sediment control measures for sheet flow areas.				
6.	Run-off control measures.				
7.	Stormwater outlet protection location and specifications.				
8.	Grade stabilization structure locations and specifications.				
9.	Dewatering applications and management methods.				
10.	Measures utilized for work within waterbodies.				
11.	Maintenance guidelines for each proposed stormwater quality measure.				
12.	Planned construction sequence that describes the implementation of stormwater quality measures in relation to land disturbance.				
13.	Provisions for erosion and sediment control on individual residential building lots regulated under the proposed project.				
14.	Material handling and spill prevention and spill response plan meeting the requirements in 327 IAC 2-6.1.				
15.	Material handling and storage procedures associated with construction activity.				

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Project Number

(Completed by City of Plymouth)

SWPPP – Post-Construction Component (Section C)		Yes	N/A	Page Number	Section, Paragraph
1.	Description of pollutants and their sources associated with the proposed land use.				
2.	Description of proposed post-construction stormwater measures (including calculating for water quality volume (WQV) for projects with added impervious surfaces.				
3.	Plan details for each stormwater measure.				
4.	Sequence describing stormwater measure implementation.				
5.	Maintenance guidelines for proposed post-construction stormwater measures.				
6.	Entity that will be responsible for operation and maintenance of the post-construction stormwater measures.				

Post-construction is <u>not</u> required for:

- Land-disturbing activities where there will be no additional impervious surfaces added (as calculated in Section A 23, above).
- Single-family residential developments of four (4) lots or less.
- Individual building lots within a permitted project site.
- Single family residences and private ponds that are not part of a larger common plan of development or sale.

Itemized SWPPP Financial Guarantee		Yes	N/A	Page Number	Section, Paragraph
1.	Provide an itemized list of financial guarantee.				

The Itemized Financial Guarantees shall be a list of: Best Management Practices (BMPs), amount to be installed, unit price, total cost for each BMP, and total cost for the project. BMP inspections, maintenance, and removal shall be included in the price.

Invoice Number (Completed by City of Plymouth)	Project Number (Completed by City of Plymouth)

Drainage Plan - Invoice

Part 3 of 3

Name of Project: _____

Address or Location Description: _____

Date Submitted: _____

Drainage Plan Submittal Review
☐ Drainage Plan Application is completed / signed and is being submitted as part of this packet.
☐ Drainage Plan was prepared and certified by a Licensed Architect, Engineer, or Surveyor
☐ An itemized cost evaluation of the Drainage Plan is being submitted for Financial Guarantees as part of this packet.
☐ A copy of the Drainage Plan calculations is being submitted as part of this packet.
☐ A copy of the Drainage Plan drawings is being submitted as part of this packet.
☐ Construction Site Area: _____ Acre(s)
The Plan Review fee is based upon the Construction Limits Area, as defined in the Stormwater Management Ordinance (Approved June 2025).

NOTE: All Project submittal documents and review fees (as identified in the applicable Ordinance(s)) shall be submitted to the City of Plymouth Clerk-Treasurer's Office as well as the Construction Stormwater Manager. Both submissions shall include both one (1) hard copy and one (1) electronic copy.

The ten (10) or fourteen (14)-working day timeframe for preliminary determination of substantial completeness shall not start until the next business day after the submittal. For example, if the submittal is received by the City of Plymouth Clerk-Treasurer's Office and the Construction Stormwater Manager on a Friday, the review timeframe would not start until the following Monday. If deficiencies are found during the preliminary review, the ten (10) or fourteen (14)-working day timeframe is null and void and a thirty (30)-day review period is in effect from the time of the most recent submittal.

If the preliminary determination of substantial completeness review is found to be satisfactory, the Notice of Intent (NOI) letter may be submitted to IDEM (with a digital copy of proof of submittal sent to both the Clerk-Treasurer's Office and the Construction Stormwater Manager) and construction activities may begin no sooner than 48 hours after the NOI submittal. The Financial Guarantee is to be submitted within ten (10) working days of the NOI, otherwise a Stop Work Order may be issued until the Financial Guarantee is received.

If any deficiencies are found during the preliminary review, the Applicant and Owner will be notified via email and will have ten (10) working days to address the deficiencies and resubmit. If the resubmission is not made within ten (10) working days, a Stop Work Order may be issued. **Note:** the deficiency resubmission may be submitted via email only.

Drainage Project Plan Review Fee

See *Chapter 151.51 – Project Review Fees* of the City of Plymouth Code of Ordinances, for more information.

Invoice Number (Completed by City of Plymouth)	Project Number (Completed by City of Plymouth)

SWPPP - Invoice

Part 3 of 3

Name of Project: _____

Address or Location Description: _____

Date Submitted: _____

SWPPP Submittal Review

- ☐ SWPPP Application is completed / signed and is being submitted as part of this packet.
- ☐ SWPPP was prepared and certified by a competent, trained individual.
- ☐ Indiana Stormwater Quality Manual is referenced for BMP installation & maintenance.
- ☐ An itemized cost evaluation of BMPs is being submitted for Financial Guarantees as part of this packet.
- ☐ A copy of the IDEM CSGP Notice of Intent NOI is being submitted.
- ☐ A copy of the SWPPP is being submitted as part of this packet.
- ☐ Construction Limits, Number of Acres: _____ Acre(s)

The SWPPP Submittal Review fee is based upon the Construction Limits Area, as defined in the Stormwater Management Ordinance (Approved June 2025).

NOTE: All Project submittal documents and review fees (as identified in the applicable Ordinance(s)) shall be submitted to the City of Plymouth Clerk-Treasurer's Office as well as the Construction Stormwater Manager. Both submissions shall include both one (1) hard copy and one (1) electronic copy.

The ten (10) or fourteen (14)-working day timeframe for preliminary determination of substantial completeness shall not start until the next business day after the submittal. For example, if the submittal is received by the City of Plymouth Clerk-Treasurer's Office and the Construction Stormwater Manager on a Friday, the review timeframe would not start until the following Monday. If deficiencies are found during the preliminary review, the ten (10) or fourteen (14)-working day timeframe is null and void and a thirty (30)-day review period is in effect from the time of the most recent submittal.

If the preliminary determination of substantial completeness review is found to be satisfactory, the Notice of Intent (NOI) letter may be submitted to IDEM (with a digital copy of proof of submittal sent to both the Clerk-Treasurer's Office and the Construction Stormwater Manager) and construction activities may begin no sooner than 48 hours after the NOI submittal. The Financial Guarantee is to be submitted within ten (10) working days of the NOI, otherwise a Stop Work Order may be issued until the Financial Guarantee is received.

If any deficiencies are found during the preliminary review, the Applicant and Owner will be notified via email and will have ten (10) working days to address the deficiencies and resubmit. If the resubmission is not made within ten (10) working days, a Stop Work Order may be issued. **Note:** the deficiency resubmission may be submitted via email only.

SWPPP Submittal Review Fee

See *Chapter 151.51 – Project Review Fees* of the City of Plymouth Code of Ordinances, for more information.

Plymouth INDIANA	**Construction Stormwater Pollution Prevention Plan Technical Review** **Plymouth Stormwater Management Ordinance *2025-2243*** www.plymouthin.com **IDEM Construction Stormwater General Permit:** https://www.in.gov/idem/stormwater/construction-land-disturbance-permitting/ (INR040000 effective 12/18/2021)

Construction/Stormwater Pollution Prevention Plan Technical Review and Comment

Project Name: **Scope of Project:** **County(ies):** **Latitude:** **Longitude:**	**Plan Submittal Date:** **Click here to enter a date.** **Plan Review Date:** **Click here to enter a date.**

Plan Preparer: **Affiliation:**
Address:
City: **State:** **Zip:**
Phone: **Cell Phone:** **Email:**

Project Site Owner: **Company Name (if applicable):**
Address:
City: **State:** **Zip:**
Phone: **Cell Phone:** **Email:**

Applicable MS4: City of Plymouth (Permit Number: INR040064)

Plan Reviewer: **Affiliation:** **On behalf of:** IDEM
Address:
City: **State:** **Zip:**
Phone: **Cell Phone:** **Email:**
Assisting Plan Reviewer: **Affiliation:** **On behalf of:**

Plan Review Status:		
☐	**Plan is Adequate**	A comprehensive plan review has been completed, and it has been determined that the plan satisfies the minimum requirements of the City of Plymouth Stormwater Management Ordinance 2025-2243 and the Construction Stormwater General Permit INR040000 (Effective 12-18-2021).
☐	**Preliminary Review**	A comprehensive review will not be completed at this time. The plan review authority reserves the right to perform a comprehensive review at a later date, and revisions may be required at that time.
☐	**Conditional Acceptance**	Acceptance of the plan is conditional. The conditional acceptance is contingent upon addressing the issues identified in the comment sections.
☐	**Plan is Deficient**	Significant deficiencies were identified and must be addressed. Refer to the comment sections.

Action(s):	
☐	**Submit a Notice of Intent:** Submit the Notice of Intent (NOI) online through the IDEM Regulatory ePortal: (https://stormwater.idem.in.gov/ncore/external/home) *It is required to upload a copy of this review form when submitting the NOI through the IDEM Regulatory ePortal.*
☐	**Do Not File a Notice of Intent or Commence Land-Disturbing Activities:** Deficiencies must be adequately addressed, and an acceptable plan review completed.
☐	**Comments:** Refer to Plan Review Comments Sections of this document.
☐	**Revisions:** Update and submit the revised Construction / Stormwater Pollution Prevention Plan as indicated below. *Revisions must be submitted through the Regulatory ePortal: (https://stormwater.idem.in.gov/ncore/external/home)*
	☐ Update and submit a complete plan set that addresses plan deficiencies.
	☐ Update and submit a document (narrative and / or plan sheets) that address plan deficiencies.
	☐ Update and submit a complete plan set that addresses plan deficiencies. A comprehensive plan review will not be completed.

Plan Review Information

- *The technical review and comment process is intended to evaluate the completeness of the Construction / Stormwater Pollution Prevention Plan (Plan) for the project. The Plan submitted was not reviewed for the adequacy of engineering design. All measures included in the Plan, as well as those recommended in the comments should be evaluated as to their feasibility by a qualified individual with structural measures designed by a qualified engineer. The Plan has not been reviewed for other local, state, or federal permits that may be required to proceed with this Project.*
- *Additional information, including design calculations may be requested to further evaluate the plan.*
- *All proposed stormwater pollution prevention measures and those referenced in this review must meet the design criteria and standards set forth in the "Indiana Stormwater Quality Manual" from the Indiana Department of Environmental Management or similar Guidance Documents.*
- *Construction activities and unforeseen weather conditions may affect the performance of the erosion and sediment control system, individual measures, or the effectiveness of the Plan. The Plan must be a flexible document, with provisions to modify or substitute measures as necessary to ensure compliance.*

Section A: Construction Plan – General Plan Components

Adequate	Deficient	N/A	A	*The construction plan elements include general information associated with the project site that are critical for the evaluation of the stormwater pollution prevention plan component. This information includes, but is not limited to an index, resource information, reference maps, grading information, project layout and design, and drainage plan*
☐	☐	☐	**1**	Index of the location of required plan elements in the construction plan
☐	☐	☐	**2**	A vicinity map depicting the project site location in relationship to recognizable local landmarks, towns, and major roads
☐	☐	☐	**3**	Narrative of the nature and purpose of the project
☐	☐	☐	**4**	Latitude and longitude to the nearest fifteen (15) seconds
☐	☐	☐	**5**	Legal description of the project site
☐	☐	☐	**6**	11 X 17-inch plat showing building lot numbers/boundaries and road layout / names
☐	☐	☐	**7**	Boundaries of the one hundred (100) year floodplains, floodway fringes, and floodways
☐	☐	☐	**8**	Land use of all adjacent properties
☐	☐	☐	**9**	Identification of a U.S. EPA approved or established TMDL
☐	☐	☐	**10**	Name(s) of the receiving water(s)
☐	☐	☐	**11**	Identification of discharges to a water on the current 303(d) list of impaired waters and the pollutant(s) for which it is impaired
☐	☐	☐	**12**	Soils map of the predominate soil types
☐	☐	☐	**13**	Identification and location of all known wetlands, lakes, and water courses on or adjacent to the project site (construction plan, existing site layout)
Adequate	Deficient	N/A	A	*The construction plan elements include general information associated with the project site that are critical for the evaluation of the stormwater pollution prevention plan component. This information includes, but is not limited to an index, resource information, reference maps, grading information, project layout and design, and drainage plan.*
☐	☐	☐	**14**	Identification of any other state or federal water quality permits or authorizations that are required for construction activities
☐	☐	☐	**15**	Identification and delineation of existing cover, including natural buffers
☐	☐	☐	**16**	Existing site topography at an interval appropriate to indicate drainage patterns
☐	☐	☐	**17**	Location(s) where run-off enters the project site
☐	☐	☐	**18**	Location(s) where run-off discharges from the project site prior to land disturbance
☐	☐	☐	**19**	Location of all existing structures on the project site
☐	☐	☐	**20**	Existing permanent retention or detention facilities, including manmade wetlands, designed for the purpose of stormwater management

☐	☐	☐	**21**	Locations where stormwater may be directly discharged into ground water, such as abandoned wells, sinkholes, or karst features
☐	☐	☐	**22**	Size of the project area expressed in acres
☐	☐	☐	**23**	Total expected land disturbance expressed in acres
☐	☐	☐	**24**	Proposed final topography
☐	☐	☐	**25**	Locations and approximate boundaries of all disturbed areas
☐	☐	☐	**26**	Locations, size, and dimensions of all stormwater drainage system such as culverts, stormwater sewer, and conveyance channels
☐	☐	☐	**27**	Locations of specific points where stormwater and non-stormwater discharges will leave the project site
☐	☐	☐	**28**	Location of all proposed site improvements, including roads, utilities, lot delineation and identification, proposed structures, and common areas
☐	☐	☐	**29**	Location of all on-site and off-site soil stockpiles and borrow areas
☐	☐	☐	**30**	Construction support activities that are expected to be part of the project
☐	☐	☐	**31**	Location of any in-stream activities that are planned for the project including, but not limited to, stream crossings and pump arounds

Adequate	Deficient	N/A	A	
				The construction plan elements include general information associated with the project site that are critical for the evaluation of the stormwater pollution prevention plan component. This information includes, but is not limited to an index, resource information, reference maps, grading information, project layout and design, and drainage plan.
☐	☐	☐	**32**	Pre-construction and post-construction estimate of peak discharge (10- year storm event), utilizing the Rational Method, for project areas less than 200 acres

Section A – Comments:

- Evaluate areas with potential waters of the state and, where required, verify if permits / authorizations are required prior to any impacts to waters of the state. These potential resources include areas with hydric soil, hydrophytic vegetation, pooling water, or evidence of flowing water such as swales, ditches, drains, or natural conveyances. Evaluation of hydric soil, hydrophytic vegetation, or pooling water should conform to the US Army Corps of Engineers Wetlands Delineation Manual," Technical Report Y-87-1, and the applicable regional supplement https://www.usace.army.mil/Missions/Civil-Works/Regulatory-Program-and-Permits/reg_supp/. Avoidance and minimization of impacts to waters of the state should be prioritized.
-

Section B: Stormwater Pollution Prevention Plan – Erosion and Sediment Control/Project Site Management				
Adequate	Deficient	N/A	B	*The construction component of the Stormwater Pollution Prevention Plan includes stormwater quality measures to address erosion, sedimentation, and other pollutants associated with land disturbance and construction activities. Proper implementation of the plan, maintenance of measures, and administering a self-monitoring program is required to manage the project site to minimize the discharge of sediment and other pollutants. Construction activities and unforeseen weather conditions may affect the performance of the erosion and sediment control system, individual measures, or the effectiveness of the plan. The plan must be a flexible document, with provisions to modify or substitute measures as necessary to ensure compliance.* *Where applicable, Items in 2 through 10 below will be evaluated for location, dimensions, detailed specifications, and construction details*
☐	☐	☐	**1**	Description of the potential pollutant generating sources and pollutants, including all potential non-stormwater discharges
☐	☐	☐	**2**	Stable construction entrance locations and specifications
☐	☐	☐	**3**	Specifications for temporary and permanent stabilization
☐	☐	☐	**4**	Sediment control measures for concentrated flow areas
☐	☐	☐	**5**	Sediment control measures for sheet flow areas
☐	☐	☐	**6**	Run-off control measures
☐	☐	☐	**7**	Stormwater outlet protection location and specifications
☐	☐	☐	**8**	Grade stabilization structure locations and specifications
☐	☐	☐	**9**	Dewatering applications and management methods
☐	☐	☐	**10**	Measures utilized for work within waterbodies
☐	☐	☐	**11**	Maintenance guidelines for each proposed stormwater quality measure
☐	☐	☐	**12**	Planned construction sequence that describes the implementation of stormwater quality measures in relation to land disturbance
☐	☐	☐	**13**	Provisions for erosion and sediment control on individual residential building lots regulated under the proposed project
☐	☐	☐	**14**	Material handling and spill prevention and spill response plan meeting the requirements in 327 IAC 2-6.1
☐	☐	☐	**15**	Material handling and storage procedures associated with construction activity

Section C: Stormwater Pollution Prevention Plan – Post-Construction				
Adequate	Deficient	N/A	C	*The post-construction component of the Stormwater Pollution Prevention Plan includes the implementation of stormwater quality measures to address pollutants that will be associated with the final project land use. Post-construction stormwater measures should be functional upon completion of the project. Long term functionality of the measures is critical to their performance and should be monitored and maintained.*
☐	☐	☐	**1**	Description of pollutants and their sources associated with the proposed land use
☐	☐	☐	**2**	Description of proposed post-construction stormwater measures
☐	☐	☐	**3**	Plan details for each stormwater measures
☐	☐	☐	**4**	Sequence describing stormwater measure implementation
☐	☐	☐	**5**	Maintenance guidelines for proposed post-construction stormwater measures
☐	☐	☐	**6**	Entity that will be responsible for operation and maintenance of the postconstruction stormwater measures

Section C – Comments:

- Post-construction stormwater quality and quantity measures have not been evaluated for adequacy of design. The proposed measures included in this SWP3 are being accepted based on the design engineer's submittal.
- The rate of stormwater run-off and/or volume from the project site must meet local requirements to address stormwater quantity as established by ordinance or other regulatory mechanism. When a local requirement does not exist, the post-development run-off discharge from the project site must not exceed the pre-development discharge based on the two-year, ten-year, and one-hundred-year peak storm events.
- Post-construction is not required for:
  - Land-disturbing activities where there will be no additional impervious surfaces added.
  - Single-family residential developments of four lots or less.
  - Single-family residential strip developments.
  - Individual building lots within a permitted project site.
  - Single family residences and private ponds that are not part of a larger common plan of development or sale.

Review completed by:

Name / Signature File Here
Construction Stormwater Manager
900 Oakhill Ave. | Plymouth, IN 46563
(574) 936-3614 (Office) | (574) 936-3017 (Fax)
City of Plymouth | www.plymouthin.com

Plymouth INDIANA

PETITION TO COUNTY DRAINAGE BOARD

THE UNDERSIGNED, as owner(s) of the proposed subdivision to be known as:

_____

final number of lots: _____, situated in Section _____ _____,

Township _____, Range _____, _____ Township,

Marshall County, Indiana, and more particularly described as follows:

(ATTACH LEGAL DESCRIPTION OF PROPERTY)

do(es) herby petition the *Marshall County* Drainage Board to accept the drainage system and the easements therefore in and for said proposed Subdivision into the County Regulated Drainage System, and to establish a maintenance fund therefore.

STATE OF INDIANA)

)

COUNTY OF _____)

The undersigned, having been duly sworn upon oath, say(s) that the above information is true and correct and is (their) (his) (her) voluntary act and deed.

_____ _____

Owner Signature Owner Signature

_____ _____

Owner Printed Owner Printed

_____ _____

Address Address

Subscribed and sworn to before me, a Notary Public, within and for said County and State, this _____ day of _____, 20_____.

_____

Notary Public

(Notary Stamp or Seal)

_____ My Commission Expires_____

ENGINEER'S STATEMENT

I hereby certify to the best of my knowledge and belief:

1. That the plans and specifications which are attached to and made a part of this petition have been prepared by me or under my direct supervision.
2. That these plans and specifications have been prepared in accordance with all applicable Codes and Ordinances.
3. That I will perform reasonable and regular inspections of this project during construction as required to enable me to certify, upon completion, that the work was done in accordance with approved construction plans and specifications.

DATE:_____ SIGNATURE:_____

Telephone:_____ Typed Name:_____

Indiana Registration Number:_____

Address:_____

This petition given primary approval this ___day of _____, 20_____.

* * * * * * * * * * * *

This petition given approval and the above-mentioned drains accepted/not accepted into the *Marshall County* Regulated Drainage System this_____ day of _____, 20_____.

_____

_____

MARSHALL COUNTY DRAINAGE BOARD

Statement of Financial Responsibility

The undersigned of the proposed project to be known as

_____

do hereby agree to take full responsibility of financial payment of review fees incurred on the above project. I am aware that the review fees will begin as soon as primary and/or secondary application is filed and continues until the project is approved and/or withdrawn. I understand that if the project is withdrawn the review fees are still due and payable from the application date to the date on the letter of withdrawal. No project will receive an approval letter nor will the plat be approved for recording until all fees are paid. All review fees are made payable to the *City of Plymouth.*

The undersigned, having duly sworn upon oath, that the above information has been read and fully understood to be true and correct and is (undersigned) voluntary act and deed. The undersigned assumes responsibility for the aforementioned fees.

_____

Signature Mailing Address

_____

Signature Printed City, State, Zip Code

STATE OF INDIANA)

)

COUNTY OF_____)

Subscribed and sworn before me, a Notary Public, within and for said County and State, this _____ day of _____, 20_____.

My Commission Expires_____

Notary Public_____

(Notary Stamp or Seal)

Stormwater Management BMPs Maintenance Agreement

THIS AGREEMENT is made this _____day of _____ , 20____ , by _____ [Owner Name] of _____ [Company Name] with principal offices located _____ [Owner/Company Address], hereinafter "Owner".

In accordance with *The City of Plymouth's* Stormwater Ordinance and Technical Standards, the Owner agrees to install and maintain stormwater management practice(s) (also known as BMPs) on the subject property, known as _____ [Property's Common Name] located at _____ [Property's Address], hereinafter "Property" in accordance with Exhibit A. The Owner further agrees to the terms stated in this document to ensure that the stormwater management practice(s) continues serving the intended function in perpetuity. This Agreement includes the following exhibit:

Exhibit A: BMP Operation and Maintenance Manual ("Manual").

Note: This agreement and all Exhibits shall be recorded with the deed of property by the Owner at the *Marshall County* Recorder's Office and two (2) copies of the recorded document provided to the *City of Plymouth* Office, hereinafter "Community".

Through this Agreement, the Owner hereby subjects the Property to the following covenants, conditions, and restrictions:

1. The Owner shall be solely responsible for the installation, maintenance, and repair of the stormwater management practices, drainage easements, and associated landscaping identified in the Manual.

2. No alterations or changes to the stormwater management practice(s) identified in the Manual shall be permitted unless they are deemed to comply with this Agreement and are approved in writing by the Community.

3. The Owner shall retain the services of a qualified individual or company to operate and ensure the maintenance of the stormwater management practice(s) identified in the Manual.

4. The Owner shall annually, by December 30th, provide to the Community records of inspections, maintenance, and repair of the stormwater management practices in accordance with the Manual.

5. The Community or its designee is authorized to access the property as necessary to conduct inspections of the stormwater management practices or drainage easements to ascertain compliance with the intent of this Agreement and the activities prescribed in the Manual. Upon written notification by the Community or its designee of required maintenance or repairs, the Owner shall complete the specified maintenance or repairs within a reasonable time frame determined by the Community. The Owner(s) shall be liable for the failure to undertake any maintenance or repairs so that the public health, safety and welfare shall not be endangered nor the road improvement damaged.

6. If the Owner fails to properly maintain the stormwater management practice(s) in accordance with the Manual and this Agreement, the Community is authorized, but not required, to perform the specified inspections, maintenance, or repairs in order to preserve the intended functions of the practice(s) and prevent the practice(s) from becoming a threat to public health, safety, general welfare or the environment. In the case of an emergency, as determined by the Community, no notice shall be required prior to the Community performing emergency maintenance or repairs. The Community may levy the costs and expenses of such inspections, maintenance, or repairs plus a ten percent (10%) administrative fee against the Owner. The Community at the time of entering upon said stormwater management practice for the purpose of maintenance or repair may file a notice of lien in the office of the Register of Deeds of *Marshall County* upon the property affected by the lien. If said costs and expenses are not paid by the Owner, the Community may pursue the collection of same through appropriate court actions and in such a case, the Owner shall pay in addition to said costs and expenses all costs of litigation, including attorney fees.

7. The Owner hereby conveys to the Community an easement over, on, and in the Property or otherwise grants perpetual access rights for the purpose of access to the stormwater management practice for the inspection, maintenance, and repair thereof, should the Owner fail to properly inspect, maintain, and repair the practice(s).

8. The Owner agrees that this Agreement shall be recorded and that the Property shall be subject to the covenants and obligations contained herein, and this Agreement shall bind all current and future owners of the property.

9. The Owner agrees in the event that the Property is sold, transferred, or leased to provide information to the new owner, operator, or lessee regarding proper inspection, maintenance, and repair of the stormwater management practice(s). The information shall accompany the first deed transfer and include this Agreement and all Exhibits. The transfer of this information shall also be required with any subsequent sale, transfer, or lease of the Property.

10. The Owner agrees that the rights, obligations, and responsibilities hereunder shall commence upon execution of the Agreement.

11. The Owner whose signatures appear below hereby represent and warrant that they have the authority and capacity to sign this agreement and bind the respective parties hereto.

12. The Owner, its agents, representatives, successors, and assigns shall defend, indemnify and hold the Community harmless from and against any claims, demands, actions, damages, injuries, costs or expenses of any nature whatsoever, hereinafter “Claims”, fixed or contingent, known or unknown, arising out of or in any way connected with the design, construction, use, maintenance, repair or operation (or omissions in such regard) of the stormwater management practice(s) referred to in Exhibit A which are the subject of this Agreement. This indemnity and hold harmless shall include any costs, expenses, and attorney fees incurred by the Community in connection with such Claims or the enforcement of this Agreement.

IN WITNESS WHEREOF, the Owner has executed this Agreement on the day and year first above written.

_____ _____

Owner Signature Date

_____ _____

Printed Name Company

_____

Title

STATE OF INDIANA)
) SS:
COUNTY OF *Marshall*)

BEFORE ME, the undersigned, a Notary Public in and for said County and State, personally appeared _____ Owner subscribed and sworn before this_____ day of _____, 20 ___.

_____ _____

Commission Expiration Date County of Residence

_____

Signature

_____

Printed Name

WHEN RECORDED, RETURN FILE STAMPED COPY TO:

The City of Plymouth, Engineering Department

……

Accepted by *The City of Plymouth*:

Signature: _____ Date: _____

Printed Name: _____ Title: _____

Project Check-In Checklist

Project Name: _____

Engineer: _____ Contact Person: _____

Address: _____

Phone Number: _____ Fax Number: _____

Developer/Owner: _____ Contact Person: _____

Address: _____

Phone Number: _____ Fax Number: _____

Project Information:

Section: _____ Township: _____ Range: _____

Township: _____ City: _____

Number of Lots: _____ Acres: _____

Parcel Number: _____ Reg. Drain: _____

Accepted Drainage Footage:

Open: _____ Tile: _____

Checklist:

1 Copy of Petition Signed & Notarized by Owner: _____

1 Copy of Legal Description: _____ 2 Sets of Plans: _____

1 Copy of Engineer Statement: _____ 1 CD (with complete project submittal): _____

1 Copy of application checklist: 1 Check for Application Fee: _____

1 Copy of Regulated Drain Exhibit (if applicable): _____

1 Copy of Stormwater Drainage Technical Report: _____

1 Copy of Stormwater Pollution Prevention Plan for Construction Sites: _____

1 Copy of Post-Construction Stormwater Pollution Prevention Plan: _____

Original Letter and 1 copy of Financial Responsibility: _____

1 Copy of Project Check-In Checklist (this sheet): _____

OFFICE USE ONLY

Assessment: _____ **Revisions:** _____

Approval: _____ **Denial:** _____

Construction Stormwater Manager
Department of Engineering
900 Oakhill Avenue, P.O. BOX 492 Plymouth, Indiana 46563
Phone: (574) 936-3614 | Fax: (574) 936-3017

Per IDEM's MS4GP Section 4.5(e), instances of non-compliance with the City of Plymouth's Stormwater Management Ordinance, an approved stormwater management design plan or permit, or any other regulatory mechanism, are subject to the following schedule of escalating enforcement.

This schedule is for quick reference only, the City of Plymouth's Stormwater Management Ordinance should be reviewed in full, for complete understanding.

Enforcement Matrix, Non-Illicit Discharges		**Fee Schedule**
Site Notice	When any person / construction site operator has failed to comply, they may be served a written **Site Notice**, specifying the particular item(s) to be addressed. The person / construction site operator shall immediately investigate the matter and seek a resolution whereby the identified item(s) may be reinstated to proper working conditions within the specified timeframe.	1st Offense: $250
Warning Notice	When any person / construction site operator continues to fail, they may be served a written **Warning Notice**, specifying the particular item(s) to be addressed. The person / construction site operator shall immediately investigate the matter and seek a resolution whereby the identified item(s) may be reinstated to proper working conditions within the specified timeframe.	2nd Offense: $500 3rd Offense: $1,000 4th Offense: $2,000
Notice of Violation	When any person / construction site operator continues to fail, the City of Plymouth may order compliance by written **Notice of Violation** to the responsible person. Civil penalties are incurred as follows: 1. $2,500.00 (depending on the severity of the violation) for the first day (24 hours) the violation remains unremedied after receipt of the **Warning Notice** or **Notice of Violation** the specified timeline. 2. $7,500.00 for each day the violation remains unremedied thereafter.	$2,500 Initial $7,500 Daily
Compensatory Action	In lieu of enforcement proceedings, penalties, etc., the City of Plymouth may impose upon a violator alternative **Compensatory Action**, such as storm drain stenciling, attendance at compliance workshops, creek cleanup, etc.	-
Stop Work Order	The City of Plymouth may, following 72 hours* from written notice and non-resolution of identified items, suspend work on a project by issuing a **Stop Work Order**. If the violator fails to comply with a **Stop Work Order**, the City of Plymouth may take such steps as deemed necessary to prevent or minimize damage to the MS4 or Waters of the U.S. (WOTUS), or to minimize danger to persons, etc. Only work being performed on the erosion control / illicit discharge items shall be permitted during the timeframe of the **Stop Work Order**. *Per Indiana House Enrolled Act 1266, the 72 hours of written notice does not apply to situations where public hazard(s) or safety hazard(s) are present.	See Ordinance

Nothing in this document, besides the requirements of a **Stop Work Order**, shall limit the authority of the City of Plymouth to take any action, including emergency action or any other enforcement action, without first issuing a **Site Notice**.

In the case of an illicit discharge from the site, a **Stop Work Order** may be issued immediately, without 72 hours of prior written notice.

Updated March 2024

B2 – Construction Inspection / Completion Forms

Self-Monitoring Program

Background

The City of Plymouth, pursuant to Section 3.6 of the IDEM Construction Stormwater General Permit (CSGP) issued on December 18, 2021, has developed a Self-Monitoring Program. The Self-Monitoring Program (and relevant Reports) is a mechanism to manage construction projects so as to ensure appropriate steps are taken to implement and maintain measures that will reduce the discharge of sediment and other pollutants associated with construction activities into sewers, waterways, and the like.

IDEM CSGP Link: https://www.in.gov/idem/stormwater/construction-land-disturbance-permitting/

In the event of a complex site evaluation, or at the direction of IDEM or the City of Plymouth, utilize IDEM's Self-Monitoring Report, attached.

Important Notes

1. Self-Monitoring Reports, per IDEM CSGP Section 3.6 (a), must be completed (unless an area is considered unsafe):
    a. 24 hours prior to a qualifying precipitation event or by the end of the next business day following each measurable storm event (excludes accumulated snow events); which is defined as a precipitation accumulation equal to, or greater than, one-half (½) inch of rainfall within a 24-hour period.
        i. In the event of multiple qualifying events during the work week, no more than three (3) inspections would be required to meet the self-monitoring requirement.
        ii. If no rain event occurs within the work week, a minimum of one (1) inspection must occur for that week.
    b. At a minimum of one (1) time per month for specific areas within the project which are stabilized with permanent vegetative cover at 70% density and / or erosion resistant armoring is installed.
        i. A reduction to once (1x) per month is also applicable for the entire project site for stabilized common areas, basins, conveyances, outfalls, and inactive building sites. Prior to reducing the monitoring and reporting to monthly, records must identify the area and the date the area became eligible for monthly monitoring. Weekly monitoring, as identified in (b.) above, must resume if one (1) or more of the following occurs:
            1. The vegetative cover fails or there is evidence of erosion in the identified area.
            2. IDEM or the inspecting authority (City of Plymouth) requires monitoring to resume.
2. Self-Monitoring Reports, per IDEM CSGP Section 3.6 (a)(3) and (4), must be maintained at the site or at an easily accessible location (refer to IDEM CSGP Section 3.7) and must be provided within 48 hours of a request by the inspecting authority (City of Plymouth). Electronic Copies are acceptable, provided they are in a format consistent with the paper record.

3. Location information can be a narrative or used in conjunction with a map that identifies areas within the project. If a map is used, the map should be included as part of the Self-Monitoring Report.
4. Per IDEM CSGP Section 3.6 (a)(2)(G), corrective action (to remediate the discharge of pollutants) must be initiated on the day discovery of deficiencies occurs, unless it is not practical to initiate on the discovery date, in which case:
    a. No later than 48 hours after the discovery date, for the repair of a measure or installation of a temporary measure until a new and / or replacement measure is installed as in item (b.) below.
    b. Within seven (7) days of discovery for the installation of a new (alternative) measure or replacement of an existing measure, unless a shorter timeframe is required as part of a regulatory inspection. The inspecting authority (City of Plymouth) may also allow additional time to take corrective action.
    c. If corrective action cannot be achieved within the timelines outlined in (a.) or (b.) above, a reason for incompletion must be provided and documented, including the anticipated completion date.
5. Self-Monitoring Inspection Reports must be prepared by a trained individual, as stipulated by IDEM.
6. A complete set of all Self-Monitoring Reports must be included with the SWPPP Completion Affidavit packet, after completion of the project, before final project acceptance, release of Financial Guarantee(s), and closeout can occur.

For any questions regarding this Self-Monitoring Program or the Self-Monitoring Report (included below), please contact the Construction Stormwater Manager as provided below. Paper and electronic submissions of the Self-Monitoring Reports should also be sent as provided below.

City of Plymouth
Department of Engineering
c/o: Construction Stormwater Manag
900 Oakhill Avenue, P.O. BOX 492
Plymouth, Indiana 46563
stormwater@plymouthin.com

Plymouth INDIANA

Self-Monitoring Report (CSGP Section 3.6)

Part A: Inspection Information

Project Name: _____

Name of Evaluator: _____ Date of Inspection: _____

Evaluator's Title: _____ Time of Inspection: _____

Evaluator Email/Phone Number: _____

Inspection Type: ☐ 24 hr. (after ≥ 1/2" rain) Amount of Precipitation: _____ Inches

☐ 24 hr. (before ≥ 1/2" rain) ☐ Monthly (only applies to areas permanently stabilized)

☐ Weekly Status: _____

Part B: Project Performance – SWPPP Implementation

SWPPP Measure: _____ Observations/Notes:

Location(s): _____

☐ No Action Required ☐ Replace Measure

☐ Maintenance Required ☐ Alternative Measure

☐ Repair Measure ☐ Additional Measure

☐ Temporary Measure

Action Initiated Date: _____ Action Completed Date: _____

SWPPP Measure: _____ Observations/Notes:

Location(s): _____

☐ No Action Required ☐ Replace Measure

☐ Maintenance Required ☐ Alternative Measure

☐ Repair Measure ☐ Additional Measure

☐ Temporary Measure

Action Initiated Date: _____ Action Completed Date: _____

Part C: Project Performance – Discharges and Corrective Actions

Location(s): _____ Observations/Corrective Actions:

☐ Sediment/Bank Erosion ☐ Foam/Oil Sheen

☐ Settled/Suspended Solids ☐ Color/Turbid Discharges

☐ Odor ☐ Operational Activities

☐ Floatables/Trash ☐ Other: _____

Action Initiated Date: _____ Action Completed Date: _____

Location(s): _____ Observations/Corrective Actions:

☐ Sediment/Bank Erosion ☐ Foam/Oil Sheen

☐ Settled/Suspended Solids ☐ Color/Turbid Discharges

☐ Odor ☐ Operational Activities

☐ Floatables/Trash ☐ Other: _____

Action Initiated Date: _____ Action Completed Date: _____

Signature of Evaluator: _____ **Date:** _____

SELF-MONITORING PROGRAM

The Indiana Construction General Permit (INRA00000) was issued on December 18th, 2021, by the Indiana Department of Environmental Management. The construction stormwater general permit (CSGP) is a performance-based regulation that contains specific requirements. One of the requirements is to develop and implement a self-monitoring program. The self-monitoring program is a mechanism to mange the construction project to ensure appropriate steps are taken to implement and maintain measures that will reduce the discharge of sediment and other pollutants associated with construction activities.

IDEM has developed a sample form that can be used by permittees and/or their representatives to implement the self-monitoring program. This form is not required by IDEM but is being provided to permittees that are required to comply with the CSGP and meet the requirements to conduct a self-monitoring program.

The requirements of the self-monitoring program can be found in the CSGP Section 3.6. The CSGP is available at: https://www.in.gov/idem/stormwater/construction-land-disturbance-permitting/

The form contains specific information that is required as part of a self-monitoring program. The form contains specific requirements that can be targeted to specific locations on the project site. Location information can be a narrative or used in conjunction with a map that identifies areas within the project. If a map is used, the map should be included as part of the self-monitoring report. The self-monitoring program documentation is required to be part of the project management log. Information on the project management log can be found in Section 3.7 of the CSGP.

Corrective action must be initiated within 48 hours of discovery of deficiencies when maintenance is required and within seven (7) days of discovery of deficiencies when a new (alternative) or replacement measure is required.

If the project is within and regulated by a Municipal separate Storm Sewer System (MS4), the MS4 may allow the use of this form to meet the local MS4 stormwater ordinance. It is the responsibility of the permittee and/or their representative to verify if this form is allowed for projects regulated by the MS4 or if the MS4 requires a form specific to meet the needs of the MS4 stormwater ordinance.

Self-Monitoring Report

PART A: INSPECTION INFORMATION

	Inspection Type	Inspection Date and Time
Project Name: **CSGP Project #** Number**:** **Permit # Assigned by MS4:** **Name of MS4:** **County:** **Name of Evaluator:** **Title of Evaluator:** **Affiliation:** **Email/Phone Number:**	☒ 24 hr (after >.5" rain) ☐ 24 hr (before >.5" rain) ☐ Weekly ☐ Monthly *(only applies to areas permanently stabilized)*	Click here to enter a date. **Date of Last or Forecasted (circle one) Precipitation:** Click here to enter a date. **Amount of Last or Forecasted (circle one) Precipitation:** Click here to enter a date.

PART B: SITE CONDITIONS

Current Site Information (check all that apply): ☐ Clearing ☐ Grubbing ☐ Grading ☐ Building Construction ☐ Installation of Infrastructure ☐ Utility Work ☐ Vegetative Establishment ☐ Other:

Soil Conditions: ☐ Dry ☐ Moist ☐ Wet ☐ Muddy ☐ Partial Snow Cover ☐ Snow Cover ☐ Frozen ☐ Freeze/Thaw ☐ Other:

PART C: PROJECT MANAGEMENT

(1) Was the SWP3 accessible at the time of the inspection?	☒ Yes ☐ No
(2) Is the SWP3 current and/or updated to reflect the current stage of development?	☐ Yes ☐ No
(3) Have all action items identified on the preceding reports been resolved? (a) If not identity which items require repair and provide a resolution timeline:	☐ Yes ☐ No
(4) Is the project posting information posted in accordance with the CSGP Section 3.7?	☐ Yes ☐ No

PART D: GUIDANCE FOR COMPLETION OF FORM

- Identify all areas of the site that currently have an erosion/sediment control measure in place. Using Part E, evaluate the current condition of the measure and using the middle column, determine what action needs to be taken.
- Identify all areas of the site where stormwater run-off leaves site or where any discharge occurs. In each location where run-off leave the site, evaluate:
  - If an erosion/sediment control measure is in place: evaluate each action step required in Part E
  - If no erosion/sediment control measure is in place or a measure is required, use Part E to include specifics about the location and type of measure to be implemented.
- For any area of the site where there is run-off, or a discharge provide a description and location using Part F.
  - If any sedimentation is occurring note where it is discharging: Off site, to a waterbody (on or off-site) or other sensitive area
- For Part H in the Evaluation of sheet flow and concentrated run-off:
  - Identify location of discharge/run-off and check any visual descriptions that apply to the discharge. If the discharge/run-off will resolve with the repair of a sediment or erosion control measure or good housekeeping practices, you do not need to list an action in the observations/notes section. If the discharge/run-off will not be solved with repair/replacement of a sediment control measure or good housekeeping practices, you will need to list the action taken to resolve the run-off and any additional pollutants visible in the discharge.
  - Print out additional pages for Parts E and F.

PART E: SEDIMENT CONTROL AND RUN-OFF MANAGEMENT		
Measure: Location(s):	☐ No Action Required ☐ Maintenance Required ☐ Repair Measure ☐ Temporary Measure ☐ Replace Measure ☐ Alternative Measure ☐ Additional Measure	Observations/Notes:
Action Initiated Date: Click here to enter a date. Initials:		Action Completed Date: Click here to enter a date. Initials
Measure: **Location(s):**	☐ No Action Required ☐ Maintenance Required ☐ Repair Measure ☐ Temporary Measure ☐ Replace Measure ☐ Alternative Measure ☐ Additional Measure	Observations/Notes:
Action Initiated Date: Click here to enter a date. Initials:		Action Completed Date: Click here to enter a date. Initials
Measure: Location(s):	☐ No Action Required ☐ Maintenance Required ☐ Repair Measure ☐ Temporary Measure ☐ Replace Measure ☐ Alternative Measure ☐ Additional Measure	Observations/Notes:
Action Initiated Date: Click here to enter a date. Initials:		Action Completed Date: Click here to enter a date. Initials
Measure: Location(s):	☐ No Action Required ☐ Maintenance Required ☐ Repair Measure ☐ Temporary Measure ☐ Replace Measure ☐ Alternative Measure ☐ Additional Measure	Observations/Notes:
Action Initiated Date: Click here to enter a date. Initials:		Action Completed Date: Click here to enter a date. Initials
Measure: Location(s):	☐ No Action Required ☐ Maintenance Required ☐ Repair Measure ☐ Temporary Measure ☐ Replace Measure ☐ Alternative Measure ☐ Additional Measure	Observations/Notes:
Action Initiated Date: Click here to enter a date. Initials:		Action Completed Date: Click here to enter a date. Initials

PART F: SURFACE STABILIZATION		
Location(s):	☐ Permanent Vegetative Cover: ☐ Continue to Monitor ☐ 70 Percent Density Achieved ☐ Perform Seeding/Reseed ☐ Temporary ☐ Permanent ☐ Apply straw mulch and anchor ☐ Install Erosion Control Blanket ☐ Repair Erosion ☐ Utilize Alternative Stabilization Method	Observations/Notes:
Action Initiated Date: Click here to enter a date. Initials:		Action Completed Date: Click here to enter a date. Initials
Location(s):	☐ Permanent Vegetative Cover: ☐ Continue to Monitor ☐ 70 Percent Density Achieved ☐ Perform Seeding/Reseed ☐ Temporary ☐ Permanent ☐ Apply straw mulch and anchor ☐ Install Erosion Control Blanket ☐ Repair Erosion ☐ Utilize Alternative Stabilization Method	Observations/Notes:
Action Initiated Date: Click here to enter a date. Initials:		Action Completed Date: Click here to enter a date. Initials
Location(s):	☐ Permanent Vegetative Cover: ☐ Continue to Monitor ☐ 70 Percent Density Achieved ☐ Perform Seeding/Reseed ☐ Temporary ☐ Permanent ☐ Apply straw mulch and anchor ☐ Install Erosion Control Blanket ☐ Repair Erosion ☐ Utilize Alternative Stabilization Method	Observations/Notes:
Action Initiated Date: Click here to enter a date. Initials:		Action Completed Date: Click here to enter a date. Initials
Location(s):	☐ Permanent Vegetative Cover: ☐ Continue to Monitor ☐ 70 Percent Density Achieved ☐ Perform Seeding/Reseed ☐ Temporary ☐ Permanent ☐ Apply straw mulch and anchor ☐ Install Erosion Control Blanket ☐ Repair Erosion ☐ Utilize Alternative Stabilization Method	Observations/Notes:
Action Initiated Date: Click here to enter a date. Initials:		Action Completed Date: Click here to enter a date. Initials

PART G: GOOD HOUSEKEEPING		
Site Ingress/Egress Location(s):	☐ Maintain Construction Entrance ☐ Remove Tracked Sediment (do not flush sediment) ☐ Install Additional Measures	Observations/Notes:
Action Initiated Date: Click here to enter a date. Initials:		Action Completed Date: Click here to enter a date. Initials
Trash Location(s):	☐ Continue to Monitor and Manage ☐ Cover Trash Receptacles ☐ Clean Up Wind-blown Trash ☐ Other	Observations/Notes:
Action Initiated Date: Click here to enter a date. Initials:		Action Completed Date: Click here to enter a date. Initials
Spills or Leaks Location(s):	Type of Leak/Spill ☐ Concrete and/or Cementitious Washout ☐ Fuel ☐ Other	Observations/Notes/Action Taken:
Action Initiated Date: Click here to enter a date. Initials:		Action Completed Date: Click here to enter a date. Initials

PART H: EVALUATION OF SHEET FLOW AND CONCENTRATED RUN-OFF (DISCHARGES)		
Location(s):	☐ Sediment ☐ Oil Sheen ☐ Odor ☐ Floatables/Trash ☐ Foam ☐ Color/Turbid Discharge ☐ Other	Observations/Notes:
Action Initiated Date: Click here to enter a date. Initials:		Action Completed Date: Click here to enter a date. Initials
Location(s):	☐ Sediment ☐ Oil Sheen ☐ Odor ☐ Floatables/Trash ☐ Foam ☐ Color/Turbid Discharge ☐ Other	Observations/Notes:
Action Initiated Date: Click here to enter a date. Initials:		Action Completed Date: Click here to enter a date. Initials

PART I: CHANGES TO SWP3	
Does the corrective action based on this inspection require modification to the SWP3? ☒ Yes ☐ No Date of SWP3 update: Brief description of the changes:	
Action Initiated Date: Click here to enter a date. Initials:	Action Completed Date: Click here to enter a date. Initials
I certify that Part A-H of this evaluation were evaluated by me as a trained individual. To the best of my knowledge and belief, the information documented in the report is true, accurate, and complete.	
Evaluator Name and Title: **Signature and Date:** _____	

SELF-MONITORING PROGRAM

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IDEM has developed a sample form that can be used by permittees and/or their representatives to implement the self-monitoring program. This form is not required by IDEM but is being provided to permittees that are required to comply with the CSGP and meet the requirements to conduct a self-monitoring program.

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The form contains specific information that is required as part of a self-monitoring program. The form contains specific requirements that can be targeted to specific locations on the project site. Location information can be a narrative or used in conjunction with a map that identifies areas within the project. If a map is used, the map should be included as part of the self-monitoring report. The self-monitoring program documentation is required to be part of the project management log. Information on the project management log can be found in Section 3.7 of the CSGP.

Corrective action must be initiated within 48 hours of discovery of deficiencies when maintenance is required and within seven (7) days of discovery of deficiencies when a new (alternative) or replacement measure is required.

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Plymouth INDIANA

Project Number
(Completed by City of Plymouth)

Completion Affidavit for SWPPP Notice of Termination (NOT)

Project Information

Name: _____

Address: _____

Location Description: _____

Contact Information

Applicant

Name: _____
Company: _____
Address: _____
City: _____
State: _____ Zip: _____
Phone: _____
Email: _____

Property Owner

Name: _____
Company: _____
Address: _____
City: _____
State: _____ Zip: _____
Phone: _____
Email: _____

SWPPP Preparer

Name: _____
Company: _____
Address: _____
City: _____
State: _____ Zip: _____
Phone: _____
Email: _____

Construction BMP Contact

Name: _____
Company: _____
Address: _____
City: _____
State: _____ Zip: _____
Phone: _____
Email: _____

I hereby certify that the SWPPP Construction BMPs and the Post – Construction BMPs for the subject Project have been installed and utilized according to the plans and specifications prepared by the above-mentioned SWPPP Preparer and approved by the City of Plymouth on _____.

Attached is a set (1) of 'As-Built' plans, prepared and certified by an Indiana licensed Professional Engineer, Architect or Land Surveyor. The plans include all components relevant to the completed SWPPP, including:

1. Identification and dates of installation and removal of all Construction BMPs
2. Identification and installation date of Post-Construction BMPs
3. Certified statement on the plans verifying the SWPPP utilized substantially complies with the SWPPP approved by the City of Plymouth.

Also included is one (1) complete set of the Self-Monitoring Inspection Reports, as prepared by a qualified individual as stipulated by IDEM.

Applicant Signature: _____
Date: _____

This Section to Be Completed by the City of Plymouth

Approval for the IDEM NOT submittal and City release of Financial Guarantees remarks: _____

_____

Signature: _____ **Date:** _____
Title: _____

Project Number

(Completed by City of Plymouth)

Requirements for SWPPP Completion Affidavit

After completion of the Project, and before final acceptance can be made, the following documentation is required to be submitted to the City of Plymouth Clerk-Treasurer and Construction Stormwater Manager for review:

1. Completion Affidavit (signed)
2. Self-Monitoring Inspection Reports (set)
3. Professionally prepared and certified "As Built" drawings (one (1) set)
4. IDEM NOT Form 51514 (completed, for concurrence)

The "As Built" drawings shall include all pertinent data relevant to the completed storm drainage system and stormwater management facilities, as noted in the City of Plymouth Stormwater Management Ordinance (approved July 2026).

All such required documentation shall be reviewed for compliance within thirty (30) days after submission to the City of Plymouth Construction Stormwater Manager and will be maintained in the City of Plymouth's files.

This review is required per the City of Plymouth's Stormwater Management Ordinance. The City of Plymouth, as the reviewing agency, needs to approve the NOT prior to its submission to IDEM. A copy of the final NOT submission to IDEM is required to also be sent to the City of Plymouth; this final submittal may be sent via email to the City of Plmouth Construction Stormwater Manager,as indicated below.

Please submit the applicable NOT documentation, as required, to the following contacts:
City of Plymouth
Department of Engineering
Construction Stormwater Manager
900 Oakhill Avenue, P.O. BOX 492

Plymouth, Indiana 46563
stormwater@plymouthin.com

B3 - Individual Lot / Parcel Permit Forms

INSTRUCTIONS FOR INDIVIDUAL LOT PERMIT REQUEST

1. Request shall be made to the *City of Plymouth.*
2. Request shall be made on standard form only, supplied by the *City of Plymouth.*
3. The form shall be completely filled out, including the following information:

a. Name of subdivision/minor plat and lot number.
b. Section/Township/Range
c. Parcel number of property involved.
d. Project name (if none then put individual's name).
e. Contact person(s).
f. Type of residential lot and/or improvement. This office may require more details depending on the type of improvements being proposed.

4. A certified Plot Plan, including site layout is required to be submitted with each permit application.
5. An erosion and sediment control plan is required to be submitted with each permit application.
6. Applicant will need to provide the name, address telephone number, and list of qualifications of the trained individual in charge of the mandatory stormwater pollution prevention self-monitoring program for the project site.
7. Applicant will need to abide by any additional requirements set forth in the *City of Plymouth* Stormwater Management Ordinance and/or Technical Standards.
8. The applicant or an agent of the applicant must sign the form.
9. Check or money order is to be made payable to the *City of Plymouth*. The correct amount of fee, based on the Fee Ordinance, must be included with the application package.

INDIVIDUAL LOT PERMIT REQUEST

Name of Subdivision/Minor Plat & Lot #: _____
Parcel Number: _____

Project Name: _____
Section/Township/Range _____
Township Name: _____

Applicant's Name: _____
Address: _____

Property Owner: _____
Property **Address**:
_____
_____

Phone: (___)_____
Fax: (___)_____

Phone: (___)_____
Fax: (___)_____

Contractor/Builder: _____
Address: _____
Phone: (___)_____
Contact Person: _____

Fax: (___)_____
Cell Phone: _____

Type of Residential Lot or **Improvement**: _____

Trained Individual in Charge of the Mandatory Stormwater Pollution Prevention
ProgramName:_____ **Address**:_____ Phone #:_____
List of Qualifications): _____

_____
_____
_____

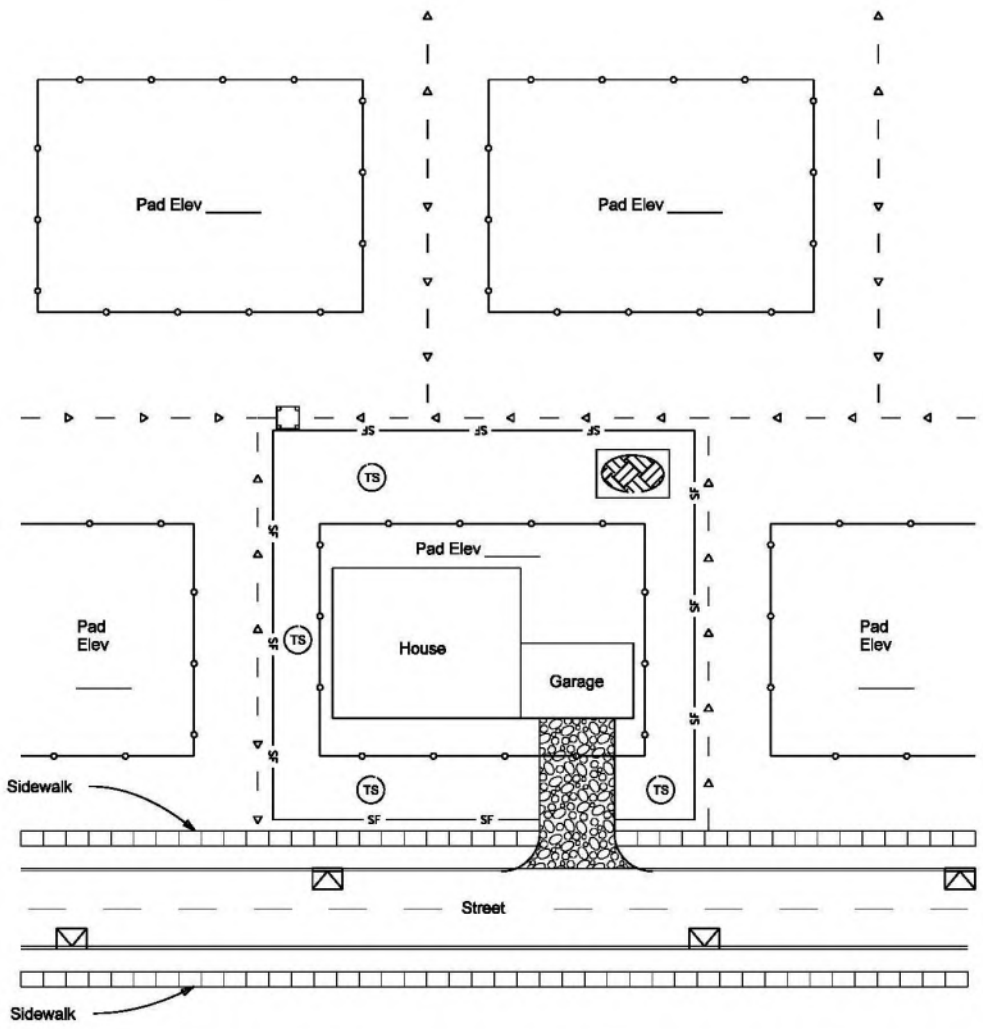
The individual lot operator is responsible for installation and maintenance of all erosion and sediment control measures until the site is stabilized.

_____ _____
Signature **Date**

Check Title: Owner _____ Contractor _____ Engineer _____ Agent _____ Other _____

*** **For Office Use Only** *** Engineering Firm: _____
Permit # _____ Plan Project # _____ Check # _____

5/8/2018
K:\INDY\CADD\Details\Erosion Control\erosion and sed plan.dgn



Legend and Check List

- SF — Silt Fence ☐
- Gravel Construction Entrance ☐
- Drop Inlet Protection ☐
- Curb Inlet Protection ☐
- TS — Temporary Seeding ☐
- Property Lines / Drainage Swale ☐
- Constructed Building Pad ☐
- Soil Stockpile Protection ☐

Notes:

- Draw in any "Do Not Disturb" areas.
- Provide pad elevations for subject property and adjacent properties.
- Erosion Control Measures must be functional and maintained throughout construction.

I hereby certify that the drainage ways, pad elevations, and erosion and sediment control measures are consistant with the overall development plans.

Signature _____ Date _____

Individual Lot Erosion and Sediment Control Plan and Certification

Plot Plan Review

Landowner_____ Parcel #_____
Legal Description_____ Erosion Fee _____

- Legal Description
- Stamped by Engineer/Surveyor_____
- "N" arrow and scale
- Site/location map
- Flood Zone _____ Map # _____
- Check for Wetland
- Dimension of Lot
    - House finish floor 2' above road elevation
    - House finish floor in relationship to pond normal pool
    - Driveway location not over curb inlets or storm sewers
    - Show utilities
        - Water
        - Sanitary
        - Septic with perimeter tile location & outlet with elevations
        - 8" tile, storm piping, or regulated drain
        - Swales
    - Elevations-centerline of road
        - Inlet/outlet elevations
        - Benchmark
    - Easements
    - Field verified elevations
- Erosion Control
    * Temporary Driveway or construction entrance
    * Lot Stabilization – silt fence or equivalent
    * Curb or Yard inlet stabilization
    * Pond protection
    * Street protection
    * Seeding-permanent, temporary (timeline)
    * Undisturbed area marked
    * Stockpile area marked and protected

*Endangered species area

* Responsible Individual _____

Additional Comments: _____

_____

Reviewed By:_____ Date Received: _____
Date Reviewed: _____ Revisions: _____ Approval Number:_____
Date Approved:_____ Date Taken to Planning:_____
Called: _____ Faxed Comments: _____

Individual Lot/Parcel
Stormwater Pollution Prevention Requirements

FOR COMPLIANCE WITH *City of Plymouth* STORMWATER ORDINANCE

THE INDIVIDUAL LOT OWNER OR OPERATOR IS RESPONSIBLE FOR THE INSTALLATION AND MAINTENANCE OF STORMWATER POLLUTION PREVENTION CONTROLS UNTIL THE ENTIRE LOT IS COMPLETE AND 100% STABILIZED.

- Initial Stormwater controls such as construction entrances, curbside and rear of lot storm inlet protection (geo-textile wrapped under grate is prohibited), and perimeter controls (typically silt fence) must be in place BEFORE ANY LOT DISTURBANCE begins.

- All construction materials must be staged off of the street and on the lot behind perimeter controls.

- Portable toilets must be kept off of the street and should be placed on even ground on the lot behind perimeter controls.

- All lots must provide and utilize appropriate trash containment for site waste.

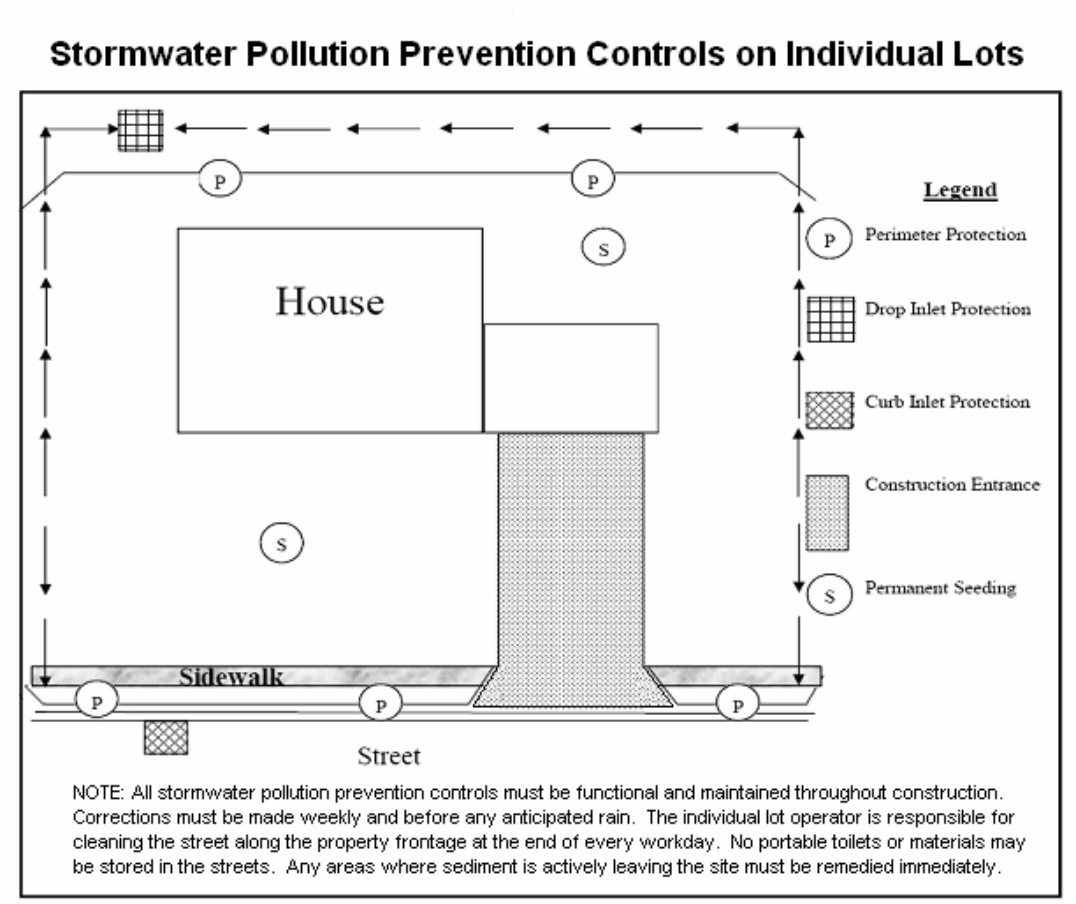
- Contractors must use appropriate practices for concrete, mortar, and paint washout. These materials must be properly contained and NOT enter the storm drains or other conveyances.

- Any off site tracking of sediment into the street, or off site sedimentation into swales or drains MUST be cleaned as soon as possible and by no later than the end of the day.

- Lot frontage should be cleaned and acceptable in appearance at the end of every business day.

- Areas where operations have impacted adjacent lots or rear yard swales MUST be repaired to design condition and 100% stabilized.

PLEASE NOTE: ANY INSPECTION WILL FAIL IF THE CORRECT STORMWATER POLLUTION PREVENTION CONTROLS ARE NOT IN PLACE AND PROPERLY MAINTAINED.



Construction Sequence

1. Install construction entrance.
   a. Use #2 stone. Flare entrance at street so it can handle vehicle turn radius. See Detail.
2. Install perimeter protection.
   a. Protection along the sides of the lot is only necessary if the adjacent lot is built out or if stormwater runoff will drain in that direction.
   b. Make sure perimeter protections are turned into the lot where they terminate to create a ponding area. See above diagram.
   c. Rear of lot perimeter protection should be installed to protect the rear yard swale.
3. Install protections on storm inlets at curbside and at rear of lot.
   a. Geo-textile or “fabric” wrapped underneath the grate is <u>PROHIBITED.</u>
   b. Make sure curbside inlet protection leaves the top 3-4 inches of the storm grate exposed to allow for overflow to enter the storm inlet, preventing ponding.
4. Lot disturbance may begin once the controls listed above are in place.
5. Maintain lot controls at all times and repair as soon as possible when a correction is needed.
   a. If sediment is actively leaving the site due to a failing control such as tracking or an operation such as dewatering, it must be corrected <u>IMMEDIATELY.</u>
6. Stabilize all exposed soils with vegetation, mulch, or stone when construction is complete.

a. Lot is considered stabilized once vegetation has reached 100% coverage and 70 % density.

7. Remove temporary stormwater pollution prevention controls.

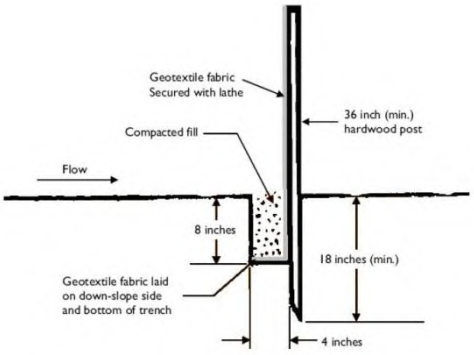
a. These may be removed when exposed soils have been stabilized with vegetation, sod, or mulch.

b. Seed alone is not a stabilization measure until it germinates and achieves proper coverage.

BMP DETAILS

Silt Fence

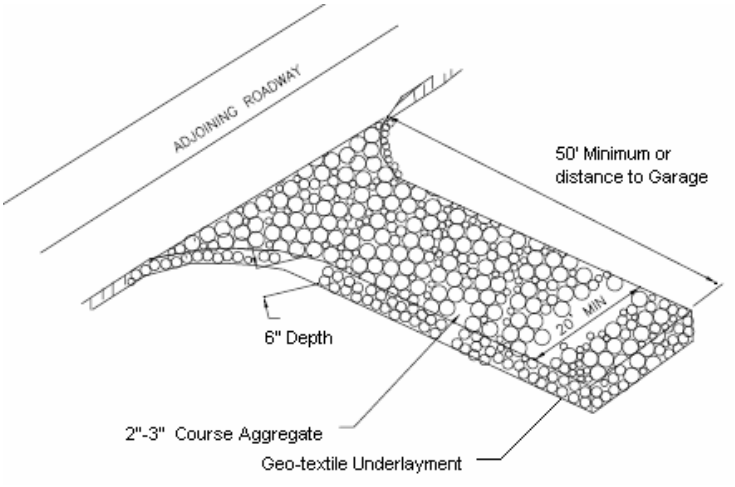
1. Install silt fence parallel to the contour of the land.
2. Extend ends of silt fence upslope 3-4 feet to allow for ponding areas behind the fence.
3. Excavate trench 8 inches deep and 4 inches wide.
4. Install with stakes on the down slope side of fence.
5. Bury 12 inches of fabric in the trench, extending the bottom 4 inches toward the upslope side.
6. Backfill trench on both sides of fence and compact.
7. Join silt fence sections by wrapping two posts and driving them in together. Do not use any other method of joining.



8. Inspect weekly and within 24 hrs of a ½" of rain. Silt fence should be cleaned out when the sediment has reached 1/3 the height of the exposed fencing. Repair silt fence where torn or damaged. Complete repairs before the next anticipated rain and by no later than one week from the date they are noticed.

Construction Entrance

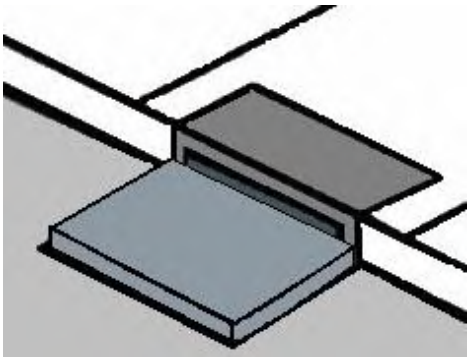
1. Install construction entrance from street to face of proposed building or at a 50’ minimum length. Use #2 stone at a 6” minimum depth.
2. A geo-textile is required underneath the entrance to extend its functionality.
3. Flare out entrance where it meets the street so that vehicle turn radiuses do not travel over disturbed ground.



4. Perimeter Controls (silt fence) should be turned into the lot for a few feet where they meet the construction entrance.
5. Inspect weekly and within 24 hrs of a ½” of rain. Freshen or replace stone as needed to prevent off site tracking. If offsite tracking is occurring, clean up immediately, and correct the reason why the drive is failing as soon as possible. Complete repairs before the next anticipated rain and by no later than one week from the date they are noted.

1. Install inlet protection on all curbside and rear of lot storm

2. Curbside inlet protection should be installed so that 3 – 4 inches of the top of the casting is exposed to allow for overflow, preventing excessive ponding.
3. Wrapping geo-textile underneath the grate for protection or straw bale barriers are PROHIBITED practices.
4. Make sure inlet protection is securely fastened to the storm grate and installed per the manufacturer’s recommendations.



5. Inspect weekly and within 24 hours of a ½" of rain. Sediment accumulation or standing water around the inlet can indicate the need for maintenance. Clean protection when clogged with sediment or when it reaches ½ of the storage capacity or height of the control. Replace protection if torn or worn. Clean sediment from street around the storm inlet and place back onto lot behind perimeter controls. Complete repairs before the next anticipated rain and by no later than one week from the day they are noted.

Appendix C

PERFORMANCE SUMMARY OF GREEN INFRASTRUCTURE PRACTICES

This table provides a quick summary of the expected performance for each of the green infrastructure best management practices (BMPs). The following fact sheets provide additional details.

		POTENTIAL APPLICATIONS							STORMWATER QUANTITY FUNCTIONS			STORMWATER QUALITY FUNCTIONS				COST	MAINTENANCE	COLD WEATHER
		Residential	Commercial	Ultra Urban	Industrial	Retrofit	Road	Rec	Volume	Ground-water Recharge	Peak Rate	TSS	TP	TN[1]	Temp			
SITE ASSESSMENT & SITE DESIGN BMPS	Cluster-Type Development	YES	YES[2]	LIMITED	LIMITED	NO	NO	LIMITED	HIGH	HIGH	HIGH	HIGH	HIGH	HIGH	HIGH	LOW	LOW/MED	HIGH
	Minimize Soil Compaction	YES	YES	LIMITED	YES	LIMITED	LIMITED	YES	MED/HIGH	MED/HIGH	LOW/MED	MED/HIGH	MED/HIGH	LOW	MED/HIGH	LOW/MED	LOW	LOW/MED
	Minimize Total Disturbed Area	YES	YES	LIMITED	YES	LIMITED	LIMITED	YES	HIGH	HIGH	HIGH	HIGH	HIGH	HIGH	HIGH	LOW	LOW	HIGH
	Protect Natural Flow Waterways	YES	YES	NO	YES	YES	YES	YES	LOW/MED	LOW	MED/HIGH	LOW/MED	LOW/MED	LOW	LO	LOW	LOW/MED	LOW/MED
	Protect Riparian Buffer Areas	YES	YES	LIMITED	YES	YES	LIMITED	YES	LOW/MED	LOW/MED	LOW/MED	HIGH	HIGH	MED	HIGH	LOW/MED	LOW	HIGH
	Protect Sensitive Areas	YES	YES	LIMITED	YES	NO	LIMITED	YES	HIGH	HIGH	HIGH	HIGH	HIGH	LOW	HIGH	LOW/MED	LOW/MED	HIGH
	Reduce Impervious Surfaces	YES	YES	LIMITED	YES	LIMITED	YES	YES	HIGH	HIGH	HIGH	MED	LOW	LOW	MED	LOW	LOW	HIGH
	Stormwater Disconnection	YES	YES	LIMITED	LIMITED	LIMITED	LIMITED	YES	HIGH	HIGH	HIGH	HIGH	HIGH	LOW/MED	HIGH	LOW	LOW	LOW
SOURCE-CONTROL & WATER QUALITY TREATMENT BMPS	Bioretention (Rain Garden)	YES	YES	LIMITED	LIMITED	YES	YES	YES	MED/HIGH	MED/HIGH	MEDIUM	HIGH	MEDIUM	MEDIUM	HIGH	MEDIUM	MEDIUM	MEDIUM
	Detention Basin-Constructed Wetland	YES	YES	YES	YES	YES	YES	YES	LOW	LOW	HIGH	HIGH	MEDIUM	MEDIUM	LOW/MED	HIGH	LOW/MED	MED/HIGH
	Detention Basin -Dry Pond	YES	YES	YES	YES	YES	YES	YES	LOW	LOW	HIGH	MEDIUM	MEDIUM	LOW	LOW	HIGH	Sediment-LOW, Vegetation-High	MED/HIGH
	Detention Basin -Wet Pond	YES	YES	YES	YES	YES	YES	YES	LOW	LOW	HIGH	HIGH	MEDIUM	MEDIUM	LOW/MED	HIGH	LOW/MED	MED/HIGH
	Infiltration Practice- Basin	YES	YES	LIMITED[2]	YES	LIMITED	LIMITED	NO	HIGH	HIGH	HIGH	HIGH[4]	MED/HIGH	MED (NO_3)	HIGH	LOW/MED	LOW/MED	MED/HIGH
	Infiltration Practice-Subsurface Bed	YES	YES	YES	YES	YES	LIMITED	NO	HIGH	HIGH	HIGH	HIGH[4]	MED/HIGH	LOW	HIGH	HIGH	MEDIUM	HIGH
	Infiltration Practice-Trench	YES	YES	YES	YES	YES	YES	NO	MEDIUM	HIGH	LOW/MED	HIGH[4]	MED/HIGH	LOW/MED	HIGH	MEDIUM	LOW/MED	HIGH
	Infiltration Practice-Dry Well	YES	YES	YES	LIMITED	YES	NO	NO	MEDIUM	HIGH	MEDIUM	HIGH[4]	MED/HIGH	LOW/MED	HIGH	MEDIUM	LOW/MED	HIGH
	Pervious Pavement (Pavers, Blocks, Concrete, Asphalt)	YES[3]	YES	YES	YES[3]	YES[3]	LIMITED[3]	YES	HIGH	HIGH	MED/ HIGH	HIGH[4]	MED/HIGH	LOW	HIGH	MEDIUM	HIGH	MEDIUM
	Planter Box/Tree Box	YES	YES	YES	LIMITED	YES	NO	LIMITED	LOW/MED	MEDIUM[5]	MEDIUM	MEDIUM	LOW/MED	LOW/MED	HIGH	MEDIUM	MEDIUM	MEDIUM
	Rainwater Harvesting	YES	YES	YES	YES	YES	NO	YES	HIGH	LOW	LOW[3]	MED[4]	MEDIUM	MED (NO_3)	MEDIUM	Rain Barrel-LOW-Cistern-MED	MEDIUM	MEDIUM
	Vegetated Filter Strip	YES	YES	LIMITED[2]	LIMITED	YES	YES	YES	LOW	LOW	LOW	MED/HIGH	MED/HIGH	MED/HIGH (NO_3)	MED/HIGH	LOW	LOW/MED	HIGH
	Vegetated Roof (Green Roof)	LIMITED	YES	YES	YES	YES	N/A	YES	MED/ HIGH	LOW[6]	MEDIUM	MEDIUM	MEDIUM	MEDIUM	HIGH	HIGH	MEDIUM	MEDIUM
	Vegetated Swale (Bioswale)	YES	YES	LIMITED[2]	YES	LIMITED	YES	YES	LOW/MED	LOW/MED	LOW/MED	MED/HIGH	LOW/HIGH	MEDIUM	MEDIUM	LOW/MED	LOW/MED	MEDIUM
SITE RESTORATION	Native Revegetation	YES	YES	LIMITED	YES	YES	LIMITED	YES	LOW/MED/ HIGH	LOW/MED/ HIGH	LOW/MED	HIGH	HIGH	MED/HIGH	MEDIUM	LOW/MED	LOW	MEDIUM
	Riparian Buffer Restoration	YES	YES	YES	YES	YES	LIMITED	YES	LOW/MED	LOW/MED	LOW/MED	MED/HIGH	MED/HIGH	MED/HIGH (NO_3)	MED/HIGH	LOW/MED	LOW	HIGH
	Soil Restoration	YES	YES	YES	YES	LIMITED	YES	YES	MED	LOW	MEDIUM	HIGH	HIGH	MED/(NO_3)	MEDIUM	MEDIUM	LOW	HIGH

(*Source: compiled by Christopher B. Burke Engineering, LLC from multiple sources)*

NOTES:

1 – Reported as TN except as noted as (NO_3)

2 – Difficult to apply due to space limitations typically associated with these land uses.

3 – Applicable with special design considerations

4 – This assumes TSS loads and their debris have been managed properly before entering the BMP to prevent clogging

5 – Requires infiltration planter box.

6 – Although vegetated roofs can be used very successfully in combination with infiltration systems.

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BMP Fact Sheet

CLUSTER-TYPE DEVELOPMENT

Cluster-type development (also known as open space development) concentrates development on smaller lots on a portion of a larger site. Clustering allows the site planner to avoid resource sensitive and constrained areas at a site, such as steep slopes and water-sensitive areas including riparian buffers, wetlands, and floodplains without sacrificing the level of development. Clustering reduces the amount of required infrastructure and various development-related costs. Clustering lends itself to residential development, with greatest potential in municipalities where large-lot residential development is typical. Clustering can reduce total impervious area and total disturbed areas at development sites, thereby reducing stormwater peak rates of runoff, reducing total volume of runoff, and reducing nonpoint source pollutant loads.



Figure 1 Pinehills Development, Plymouth, MA (MA Office of Energy and Environmental Affairs)

Potential Applications		Stormwater Quantity Functions	
Residential	Yes	Volume	High
Commercial	Yes*	Groundwater Recharge	High
Ultra Urban	Limited	Peak Rate	High
Industrial	Limited	Stormwater Quality Functions	
Retrofit	No	TSS	High
Highway/Road	No	TP	High
Recreational	Limited	NO_3	High
		Temperature	High

**Depending upon site size, constraints, and other factors.*

Additional Considerations	
Cost	Low
Maintenance	Low/Med
Winter Performance	High

Variations

- Clustering as an option
- Clustering mandated by the municipality
- Clustering with incentives such as density bonuses

Key Design Features

- Develop inventory
- Map sensitive areas
- Reduce total site disturbance and develop cluster plan
- Increase undisturbed open space

Benefits

- Reduces required infrastructure
- Increases open space
- Protects environmentally sensitive natural resources

Limitations

- Site specific based on land topography and individual conditions

Description and Function

Cluster-type development is driven by reducing minimum lot size, though not necessarily changing the total number of lots or amount of development occurring. As lot sizes decrease, the portion of the site which remains as undisturbed open space increases. If clustering is done carefully, this remaining open space can and should include those areas which are most sensitive environmentally and/or which offer special value functions not otherwise protected from development (e.g., high-quality woodlands areas).

Variations

One variation to a typical cluster-type development allows for a density bonus to incentivize use of this technique. A density bonus allows for additional lots to be added to the site beyond what the yield plan would show with a conventional subdivision. Proponents of this method state that allowing an additional lot or two may be the incentive needed to increase implementation of this technique. Opponents of this variation state that a density bonus is not needed since the development already costs less due to less stormwater and transportation infrastructure.

A second clustering variation for municipalities to consider, subject to legal review, is establishing clustering as the baseline requirement, at least in some zoning categories. Conventional non-clustered development would still be an option (variance, conditional use, etc.), but only if a variety of performance standards are satisfied.

A third variation for consideration relates to the nature and extent of development types subject to clustering provisions. As discussed above, single-family residential development at lower densities/on larger lots is ready-made for clustering. However, clustering concepts can provide LID/GI benefits in larger corporate office parks, in retail centers, and other uses. Often this clustering concept takes on its own nomenclature e.g., New Urbanist, Smart Growth, Planned Integrated Development, and others. In these cases, not only are individual lots reduced in size, but the physical form of the development typically undergoes change (i.e., 50,000 square feet of retail can move from a one-story box to stacked development with a much different New Urbanist configuration). Depending upon the nature and extent of the uses involved, "clustering" of nonresidential uses (e.g., daytime offices with evening/ weekend retail), if carefully planned can offer potential for reduced parking requirements.

Applications

Residential Clustering

The most common clustering option is residential clustering on new development. **Figure 2** illustrates a more traditional development scenario where lots are placed across the entire site. In this example, the lot and house placement does avoid major natural features such as floodplain and wetlands, but still substantially encroaches into woodlands and riparian buffer features. Such a development layout ("yield plan") provides an estimate of a site's capacity to accommodate lots and houses at the base density hypothetically allowed under a local zoning ordinance.



Figure 2 Conventional Development
Source: Growing Greener: Putting Conservation into Local Codes Natural Lands Trust, Inc., 1997

Figure 3 illustrates a "density-neutral" approach to clustering, where the number of lots and houses is held constant at 18 lots; however, the lot size has been reduced significantly allowing for 50 percent of open space area.

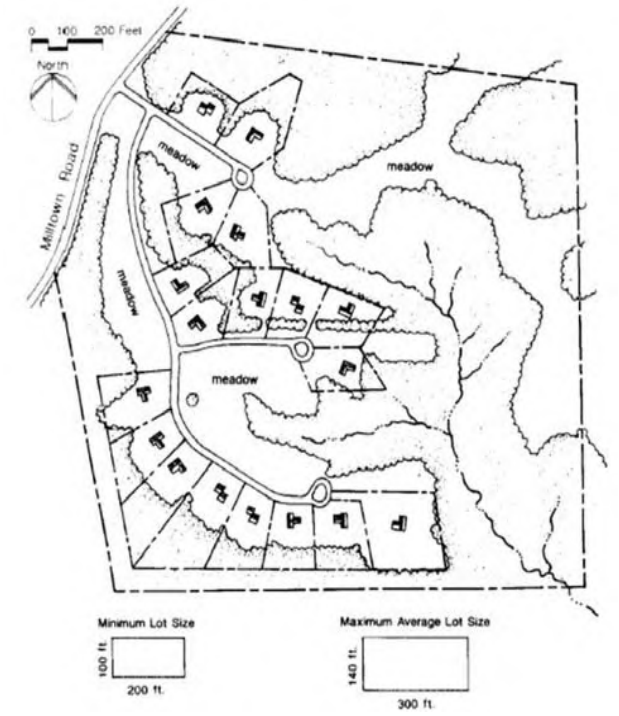


Figure 3 Clustered Development
Source: Growing Greener: Putting Conservation into Local Codes. Natural Lands Trust, Inc., 1997

Nonresidential Clustering

Conventional nonresidential development (e.g., retail commercial development) can also be configured in the form of low-rise (one story), relatively low-density strip or "big box" centers.

Design Considerations

The design process for implementing clustering at a proposed development site can occur in a variety of ways. Randall Arendt's *Growing Greener: Putting Conservation into Local Codes* (1997) provides clustering guidance in several straightforward steps. The process typically begins with the applicant applying existing conventional code to the site with any necessary net outs to develop a "yield plan." The purpose is to determine how many units can be developed conventionally:

- Step 1: Identify land to be protected: Primary conservation areas,
  - Identify land to be protected: Secondary conservation areas, and
  - Delineate potential development area.
- Step 2: Locate house sites on potential development area
- Step 3: Connect with streets and trails
- Step 4: Draw in lot lines

A major issue to address is the extent to which a clustering process is consistent with local ordinance requirements. How many house sites, and with what lot size, are going to be located in the potential development area?

If the existing local code is fully flexible, applicants can comprehensively "zone out" primary and secondary conservation areas and be confident that the baseline "yield plan" unit count can be loaded into the potential development area at whatever lot size is necessary (some applicants/developers believe that smaller lots translate into less valuable and marketable units and are reluctant to make considerable reductions in lot sizes). Often, however, such reduced lot sizes are less than the local ordinance allows. In such cases, the applicant is motivated to reduce primary and secondary conservation areas, so that the potential development area can be enlarged.

Stormwater Functions and Calculations

Volume and peak rate

Cluster-type development is a technique that results in increased open space, which reduces stormwater peak rate and volume. These open spaces are often associated with other BMPs from this manual, including preserving sensitive areas and protecting riparian corridors. These BMPs are not to be included in the disturbed stormwater management area when calculating runoff volume

Any portion of the open space that is mitigated or revegetated/reforested should be included in the disturbed stormwater management area, but may be granted runoff reduction recognition in accordance with the applicable BMP for native revegetation, soil restoration, minimize soil compaction, riparian buffer restoration, or minimize total disturbed area.

Water quality improvement

Clustering minimizes impervious areas and their associated pollutant loads, resulting in improved water quality. In addition, clustering preserves open space and other natural features, such as riparian corridors, which allow for increased infiltration of stormwater and removal of pollutant loads.

Maintenance

Preserving open space creates concerns regarding responsibility for maintenance activities. Legally, the designated open space may be conveyed to the municipality. More likely, ownership of these natural areas will be assumed by homeowners' associations or the specific individual property owners where these resources are located. Specific maintenance activities will depend on the type of vegetation present in the preserved natural area. For example, woodlands require little to no maintenance and open lawns require higher maintenance. An objective of cluster-type development is to conserve the existing natural systems with minimal, if any, intervention and disturbance.

Cost

Clustering is beneficial from a cost perspective. Costs to build 100 clustered single-family residential homes is less due to less land clearing and grading, less road and sidewalk construction (including curbing), less lighting and street landscaping, potentially less sewer and water line construction, potentially less stormwater collection system construction, and other economies of scale.

Post-construction, clustering also reduces costs. A variety of studies from Rutgers University's landmark *Costs of Sprawl* studies and later updates show that delivery of a variety of municipal services such as street maintenance, sewer and water services, and trash collection are more economical on a per person or per house basis when development is clustered. Furthermore, services such as police protection are made more efficient when residential development is clustered.

Additionally, clustering has been shown to positively affect land values. Analyses of market prices of conventional development over time in contrast with comparable clustered residential developments (where size, type, and quality of the house itself is held constant) indicate that clustered development increases in value at a more rapid rate than conventionally designed developments. This is partly due to the proximity to permanently protected open space.

Designer/Reviewer Checklist for Cluster-Type Development

ITEM	Page No.	YES	NO	N/A	NOTES
Has nonstructural BMP Protect Sensitive Resources been applied? If not, complete this BMP.	2, 3				
Has a baseline "yield plan" been developed by applicant?	2, 3				
What local ordinance provisions - obstacles and opportunities - exist for clustering?	3				
Has a Potential Development Area, or comparable, which avoids Sensitive Resources, been delineated?	3				
Has "yield plan" house/unit count been loaded into Potential Development Area?	3				
What clustered lot size assumptions are being used? Compatible with local ordinance?	3				
Compare disturbed area/developed area of "yield plan" with clustered plan?	3				

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BMP Fact Sheet

MINIMIZE SOIL COMPACTION

Minimizing soil compaction is the practice of protecting and minimizing damage to existing soil quality caused by the land development process. Enhancing soil composition with soil amendments and mechanical restoration after it has been damaged is addressed in a separate structural BMP.



Figure 1 Areas where heavy equipment is not allowed are fenced off to avoid compaction of soils and damages to vegetation (MD Custom Homes, WI)

Applications		Stormwater Quantity Functions	
Residential	Yes	Volume	Med/High
Commercial	Yes	Groundwater Recharge	Med/High
Ultra Urban	Limited	Peak Rate	Low/Med
Industrial	Yes	Stormwater Quality Functions	
Retrofit	Limited	TSS	Med/High
Highway/Road	Limited	TP	Med/High
Recreational	Yes	NO_3	Low
		Temperature	Med/High

Additional Considerations	
Cost	Low/Med
Maintenance	Low
Winter Performance	Low/Med

Key Design Features

- Reduce disturbance through design and construction practices
- Limit areas of heavy equipment
- Avoid extensive and unnecessary clearing and stockpiling of topsoil
- Use top quality topsoil; maintain topsoil quality during construction

Benefits

- Increases infiltration capacity
- Provides a healthy environment for vegetation
- Preserves low areas, which offer added benefit when runoff is directed there from impervious areas

Limitations

- Difficult to implement on small development sites

Description and Function

Minimizing soil compaction relates directly to reducing total site disturbance, site clearing, site earthwork, the need for soil restoration, and the size and extent of costly, engineered stormwater management systems. Ensuring soil quality can significantly reduce the cost of landscaping vegetation (higher survival rate, less replanting) and landscaping maintenance. Fencing off an area can help minimize unnecessary soil compaction.

Soil is a physical matrix of weathered rock particles and organic matter that supports a complex biological community. This matrix has developed over a long time period and varies greatly within the state. Healthy soils, which have not been compacted, perform numerous valuable stormwater functions, including:

- Effectively cycling nutrients,
- Minimizing runoff and erosion,
- Maximizing water-holding capacity,
- Reducing storm runoff surges,
- Absorbing and filtering excess nutrients, sediments, and pollutants to protect surface and groundwater,
- Providing a healthy root environment,
- Creating habitat for microbes, plants, and animals, and
- Reducing the resources needed to care for turf and landscape plantings.

Undisturbed soil consists of pores that have water-carrying and holding capacity. When soils are overly compacted, the soil pores are destroyed and permeability is drastically reduced. In fact, the runoff response of vegetated areas with highly compacted soils closely resembles that of impervious areas, especially during large storm events (Schueler, 2000). Recent research studies indicate that compacted soils from development practices end up as dense as concrete.

Applications

Minimizing soil compaction can be performed at any land development site during the design phase. It is especially suited for developments where significant “pervious” areas (i.e., post-development lawns and other maintained landscapes) are being proposed. If existing soils have already been excessively compacted, soil restoration is applicable (see Soil Restoration BMP Fact Sheet).

Design Considerations

Early in a project’s design phase, the designer should develop a soil management plan based on soil types and existing level of disturbance (if any), how runoff will flow off existing and proposed impervious areas, trees and natural vegetation that can be preserved, and tests indicating soil depth and quality. The plan should clearly show the following:

1. **No disturbance areas**. Soil and vegetation disturbance is not allowed in designated no disturbance areas. Protecting healthy, natural soils is the most effective strategy for preserving soil functions. Not only can the functions be maintained, but protected soil organisms are also available to colonize neighboring disturbed areas after construction.

2. **Minimal disturbance areas**. Limited construction disturbance occurs, but soil restoration may be necessary for such areas to be considered fully pervious after development. In addition, areas to be vegetated after development should be designated minimal disturbance areas. These areas may allow some clearing, but no grading due to unavoidable cutting and/or filing. They should be immediately stabilized, revegetated, and avoided in terms of construction traffic and related activity. Minimal disturbance areas do not include construction traffic areas.

3. **Construction traffic areas**. Construction traffic is allowed in these areas. If these areas are to be considered fully pervious following development, a soil restoration program will be required.

4. **Topsoil stockpiling and storage areas**. If these areas are needed, they should be protected and maintained. They are subject to soil restoration (including compost and other amendments) following development.

5. **Topsoil quality and placement**. Soil tests are necessary to determine if it meets minimum parameters. Critical parameters include: adequate depth (four inches minimum for turf, more for other vegetation), organic content (five percent minimum), and reduced compaction (1,400 kPa maximum) (Hanks and Lewandowski, 2003). To allow water to pass from one layer to the other, topsoil must be "bonded" (See Construction Guidelines #4 below) to the subsoil when it is reapplied to disturbed areas.

Construction Guidelines

1. At the start of construction, no disturbance and minimal disturbance areas must be identified with signage and fenced as shown on the construction drawings.

2. No disturbance and minimal disturbance areas should be strictly enforced.

3. No disturbance and minimal disturbance areas should be protected from excessive sediment and stormwater loads while adjacent areas remain in a disturbed state.

4. Topsoil stockpiling and storage areas should be maintained and protected at all times. When topsoil is reapplied to disturbed areas it should be "bonded" with the subsoil. This can be done by spreading a thin layer of topsoil (2-3 inches), tilling it into the subsoil, and then applying the remaining topsoil. Topsoil should meet locally available specifications/ requirements.

Stormwater Functions and Calculations

Volume and peak rate reduction

Minimizing soil compaction can reduce the volume of runoff by maintaining soil functions related to stormwater infiltration and evapotranspiration. Designers that use this BMP can select a lower runoff curve number for calculating runoff volume and peak rate from the area of minimized soil compaction. See the discussion under LID/GI Approach in Post-Construction Stormwater Quality Management Chapter of the Technical Standards on how to calculate the runoff reduction recognition for this BMP.

Where no disturbance areas are specified, which are also sensitive areas maintained in their presettlement state, there will be no net increase in stormwater runoff from that area. Calculation methodology to account for the protection of sensitive areas is provided under LID/GI Approach in Post-Construction Stormwater Quality Management Chapter of the Technical Standards.

Water quality improvement

Minimizing soil compaction improves water quality through infiltration, filtration, chemical and biological processes in the soil, and a reduced need for fertilizers and pesticides after development. See the discussion under LID/GI Approach in Post-Construction Stormwater Quality Management Chapter of the Technical Standards for information on how to calculate the volume of runoff that needs water quality treatment.

Maintenance

Sites that have minimized soil compaction properly during the development process should require considerably less maintenance than sites that have not. Landscape vegetation, either retained or re-planted, will likely be healthier, have a higher survival rate, require less irrigation and fertilizer, and have better aesthetics.

Some maintenance activities, such as frequent lawn mowing, can cause considerable soil compaction after construction and should be avoided whenever possible. Planting low-maintenance native vegetation is the best way to avoid damage due to maintenance. No disturbance areas on private property should have an easement, deed restriction, or other legal measure imposed to prevent future disturbance or neglect.

Cost

Minimizing soil compaction generally results in significant construction cost savings. Design costs may increase slightly due to a more time intensive design.

Criteria to Receive Runoff Reduction Recognition for Minimize Soil Compaction BMP

To receive runoff reduction recognition under the local regulation, areas of no disturbance and minimal disturbance must meet the following criteria:

- ☐ No disturbance and minimal disturbance areas are protected by having the limits of disturbance and access clearly shown on the Stormwater Plan, all construction drawings, and delineated/flagged/fenced in the field.
- ☐ No disturbance and minimal disturbance areas are not be stripped of existing topsoil.
- ☐ No disturbance and minimal disturbance areas are not be stripped of existing vegetation.
- ☐ No disturbance and minimal disturbance areas are not be subject to excessive equipment movement. Vehicle movement, storage, or equipment/material lay-down is not be permitted in these areas.
- ☐ Use of soil amendments and additional topsoil is permitted in other areas being disturbed, as described above. Light grading may be done with tracked vehicles that prevent compaction.
- ☐ Lawn and turf grass are acceptable uses. Planted meadow is an encouraged use.
- ☐ Areas receiving runoff reduction recognition are located on the development project.

Designer/Reviewer Checklist for Minimize Soil Compaction References

ITEM	Page No.	YES	NO	N/A	NOTES
Have no disturbance areas been defined on plans (see minimize total disturbed area BMP)?	2				
Have no disturbance areas been fenced/flagged in field?	3, 4				
Have minimal disturbance areas been defined on plans?	2				
Have construction traffic areas been defined on plans?	2				
Is soil restoration BMP committed to construction traffic areas, post-construction phase?	2				
Are soil stockpiling and storage areas defined on plan?	3				
Have proper topsoil quality and placement specifications been committed in the plans?	3				

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BMP Fact Sheet

MINIMIZE TOTAL DISTURBED AREA

A key component of LID/GI is to reduce the impacts during development activities such as site grading, removal of existing vegetation, and soil mantle disturbance. This can be achieved through developing a plan to contain disturbed areas.



Figure 1 Undisturbed areas are protected during construction (Lake County, OH Stormwater Management Department

Key Design Features

- Identify and avoid special value and environmentally sensitive areas (See Protect Sensitive Areas BMP)
- Maximize undisturbed open space
- Minimize disturbance lot-by-lot
- Maximize soil restoration and restore soil permeability
- Minimize and control construction traffic areas
- Minimize and control construction stockpiling and storage areas

Benefits

- Reduced runoff volume
- Reduced peak rates
- High water quality benefits
- Increased infiltration capacity
- Provides healthy environment for vegetation

Limitations

- Difficult to achieve on small development sites

Applications		Stormwater Quantity Functions	
Residential	Yes	Volume	High
Commercial	Yes	Groundwater Recharge	High
Ultra Urban	Limited	Peak Rate	High
Industrial	Yes	Stormwater Quality Functions	
Retrofit	Limited	TSS	High
Highway/Road	Limited	TP	High
Recreational	Yes	NO_3	High
		Temperature	High

Additional Considerations	
Cost	Low
Maintenance	Low
Winter Performance	High

Description and Function

Disturbance at a development site can occur through normal construction practices, such as grading, cutting, or filling. Minimizing the total disturbed area of the site requires the consideration of multiple BMPs, such as cluster development and identifying and protecting sensitive areas. These BMPs serve to protect area resources by reducing site grading and maintenance required for long-term operation of the site.

Minimizing the total disturbed area of a site specifically focuses on how to minimize the grading and overall site disturbance, maximizing conservation of existing native plant communities and the existing soil mantle of a site. If invasive plant species are present in the existing vegetation, proper management of these areas may be required in order for the vegetation to achieve its greatest hydrological potential.

Minimize grading

Reduction in grading can be accomplished in several ways, including the conformation of the site design with existing topography and land surface, where road alignments strive to follow existing contours as much as possible, varying the grade and alignment criteria as necessary to comply with safety limits.

Minimize overall site disturbance

Site design criteria have evolved in local jurisdictions to ensure that developments meet safety standards (i.e. sight distance and winter icing) as well as certain quality or appearance standards. Roadway design criteria should be flexible in order to optimize the fit for a given parcel and achieve optimal roadway alignment. The avoidance of environmentally sensitive resources, such as important woodlands, may be facilitated through flexible roadway layout.

From the single lot perspective, the conventional lot layout can impose added earthwork and grading. Although the intent of these local requirements is to provide privacy and spacing between units, the end result is often a cleared and graded lot, which reduces stormwater benefits. And although configuring lots in a rectilinear shape may optimize the number of units, local jurisdictions should consider requiring that the total site be made to fit the natural landscape as much as possible.

Local criteria that impose road geometry are usually contained within the subdivision and land development ordinance. Densities, lot and yard setbacks, and minimum frontages are usually contained in the zoning ordinance. Flexibility in the following land development standards will help to minimize site disturbance on an individual lot basis, thereby achieving area-wide stormwater quality and quantity results:

- Road vertical alignment criteria (maximum grade or slope)
- Road horizontal alignment criteria (maximum curvature)
- Road frontage criteria (lot dimensions)
- Building setback criteria (yards dimensions)



Figure 2 Minimally Disturbed Development
Source: Metropolitan Washington Council of Governments

Applications

Minimizing the total disturbed area of a site is best applied in lower density single-family developments, but can also be applied in residential developments of all types including commercial, office park, retail center, and institutional developments. Larger industrial park developments

can also benefit from this BMP. However, as site size decreases and density and intensity of development increases, this BMP is uniformly more difficult to apply successfully. At some larger sites where Ultra Urban, Retrofit, or Highway/Road development is occurring, limited application may be feasible.

Design Considerations

During the initial conceptual design phase of a land development project, the applicant's design engineer should provide the following information, ideally through development of a Minimum Disturbance/Minimum Maintenance Plan:

1. **Identify and Avoid Special Value/Sensitive Areas**
   Delineate and avoid environmentally sensitive resources using existing data from appropriate agencies, as applicable.

2. **Minimize Disturbance at Site** Modify road alignments (grades, curvatures, etc.), lots, and building locations to minimize grading, and earthwork as necessary to maintain safety standards and local code requirements. Minimal disturbance design should allow the layout to best fit the land form without significant earthwork, such as locating development in areas of the site that has been previously cleared, if possible. If cut/fill is required, the use of retaining walls is preferable to earthwork. Limits of grading and disturbance should be designated on plan documentation submitted to the local jurisdiction for review/approval and should be physically designated at the site during construction via flagging, fencing, etc.

   In addition, utilizing natural drainage features generally results in less disturbance and requires less revegetation.

3. **Minimize Disturbance at Lot**
   To decrease disturbance, grading should be limited to roadways and building footprints. Local jurisdictions should establish maximum setbacks from structures, drives, and walks. These setbacks should be designed to be rigorous but reasonable in terms of current feasible site construction practices. These standards may need to vary with the type of development being proposed and the context of that development (the required disturbance zone around a low density single-family home can be expected to be less than the disturbance necessary for a large commercial structure), given necessity for use of different types of construction equipment and the realities of different site conditions. For example, the U.S. Green Building Council's Leadership in Energy & Environmental Design Reference Guide (Version 2.0 June 2001) specifies:

   "…limit site disturbance including earthwork and clearing of vegetation to 40 feet beyond the building perimeter, 5 feet beyond the primary roadway curbs, walkways, and main utility branch trenches, and 25 feet beyond pervious paving areas that require additional staging areas in order to limit compaction in the paved area… "

Stormwater Functions and Calculations

Volume

Any portion of a site that can be maintained in its predevelopment state by using this BMP will not contribute increased stormwater runoff and will reduce the amount of treatment necessary for Channel Protection Volume and Water Quality Volume requirements. In addition, isolated trees within disturbed areas that are protected in accordance to this BMP or the "Protect Sensitive Areas" BMP requirements can get "runoff reduction recognition" by receiving a curve number reflecting a woodlot in "good" condition for the pre-developed underlying soil conditions.

Peak rate

Runoff from the minimized disturbed area may be excluded from peak rate calculations for rate control, provided that the runoff from the area is not conveyed to and/or through stormwater management control structures. If necessary, runoff from the minimized disturbed area should be directed around BMPs and stormwater pipes and

inlets by means of vegetated swales or low berms that direct flow to natural drainageways.

Water quality improvement

Water quality is benefited substantially by minimizing the disturbed area.

Maintenance

Minimizing site disturbance will result in a reduction of required maintenance of a site in both the short and long terms. Areas of the site left as intact native plant communities do not typically require replacement with hard surfaces or additional vegetation to retain function. On the other hand, artificial surfaces such as pavement or turf grass require varying levels of maintenance throughout the life of a development. Higher levels of disturbance will also typically require significant maintenance of erosion control measures during the active development of a parcel, thus adding to short-term development costs.

While intact natural areas may require small amounts of occasional maintenance to maintain function (typically through invasive species control), levels of maintenance required for hard surfaces or turf grass will remain static or, in most cases, increase over time. Avoiding disturbance to natural areas benefits the short-term developer and the long-term owner by minimizing time and money needed to maintain artificial surfaces.

Cost

The reduced costs of minimized grading and earthwork should benefit the developer. Cost issues include reduced grading and related earthwork as well as costs involved with site preparation, fine grading, and seeding.

Calculation of reduced costs is difficult due to the extreme variation in site factors, (amount of grading, cutting/filling, and haul distances for required trucking,). Some relevant costs factors are as follows (as based on R.S. Means, Site Work & Landscape Cost Data, 2007):

Site Clearing

- Cut & chip light trees to diameter $3,475/acre
- Grub stumps and remove $1,600/acre
- Cut & chip light trees to 24-inch diameter $11,600/acre
- Grub stumps and remove $6,425/acre

Strip Topsoil and Stockpile

- Ranges from $0.52- $1.78/yd^3 because of Dozer horse power, and ranges from ideal to adverse conditions
- Assuming six inches of topsoil, 500-ft haul: $2.75-9.86/yd^3
- Assuming six inches of topsoil, 500-ft haul: $9,922 -16,746 / acre

Site Preparation, Fine Grading, Seeding

- Fine grading w/ seeding: $2.91 /yd^2
- Fine grading w/ seeding: $14,084 /acre

In sum, total costs usually range from $29,000 - $49,000 per acre and could substantially exceed that figure at more challenging sites.

Criteria to Receive Runoff Reduction Recognition for Minimizing Total Disturbed Area

To receive runoff reduction recognition for protection of existing trees under a local regulation, the following criteria must be met:

- ☐ Area has not been subject to grading or movement of existing soils.
- ☐ Existing native vegetation are in a healthy condition as determined through a plant inventory and may not be removed.
- ☐ Invasive vegetation may be removed.
- ☐ Pruning or other required maintenance of vegetation is permitted. Additional planting with native plants is permitted.
- ☐ Area is protected by having the limits of disturbance clearly shown on all construction drawings and delineated in the field.
- ☐ Area is located on the development project.

Designer/Reviewer Checklist for Minimizing Total Disturbed Area

ITEM	Page No.	YES	NO	N/A	NOTES
Do local jurisdiction requirements for open space and related resource protection exist? Applied here?	2				
Have related BMPs (Protect Sensitive Areas, Natural Flow Pathways, Riparian Buffers, Clustering) been applied?	2				
Has Potential Development Area been defined?	3				
Have infrastructure connections/constraints been analyzed?	2				
On site, have roads been aligned to fit topography, to parallel contours and minimize cut/fill? On areas previously cleared? With terracing? Compatible with natural flow pathways?	3				
On lots, have buildings been located to reduce disturbance?	3				
On lots, have maximum disturbance radii been established and applied?	2				
No disturbance areas shall be clearly delineated on construction plans and flagged/fenced in field	3				
Have no disturbance zones been assessed qual-itatively for invasive management needs?	2				

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BMP Fact Sheet

PROTECT NATURAL FLOW PATHWAYS

A main component of LID/GI is to identify, protect, and use natural drainage features, such as swales, depressions, and watercourses to help protect water quality. Designers can use natural drainage features to reduce or eliminate the need for structural drainage systems.



Figure 1 Long swales utilized in areas of natural drainage pathways, Lenexa, KS (USEPA, picasaweb)

Applications	
Residential	Yes
Commercial	Yes
Ultra Urban	No
Industrial	Yes
Retrofit	Yes
Highway/Road	Yes
Recreational	Yes

Stormwater Quantity Functions	
Volume	Low/Med
Groundwater Recharge	Low
Peak Rate	Med/High
Stormwater Quality Functions	
TSS	Low/Med
TP	Low/Med
NO_3	Low
Temperature	Low

Additional Considerations	
Cost	Low
Maintenance	Low/Med
Winter Performance	Low/Med

Variations

- Check dams to slow velocity
- Earthen berms for additional storage
- Additional native vegetation for increased infiltration

Key Design Features

- Identifies and maps natural drainage features (e.g., swales, channels, ephemeral streams, depressions, etc.)
- Uses natural drainage features to guide site design
- Distributes non-erosive surface flow to natural drainage features
- Keeps non-erosive channel flow within drainage pathways
- Uses native vegetative buffers

Benefits

- Maximizes natural hydrological functions
- Reduces structural management practices
- Reduces management costs

Limitations

- Minimal water quality benefits

Description and Function

Many natural undeveloped sites have identifiable drainage features such as swales, depressions, and watercourses which effectively manage the stormwater that is generated on the site. By identifying, protecting, and using these features, a development can minimize its stormwater impacts. Instead of ignoring or replacing natural drainage features with engineered systems that rapidly convey runoff downstream, designers can use these features to reduce or eliminate the need for structural drainage systems.

Naturally vegetated drainage features tend to slow runoff and thereby reduce peak discharges, improve water quality through filtration, and allow some infiltration and evapotranspiration to occur. Protecting natural drainage features can provide for significant open space and wildlife habitat, improve site aesthetics and property values, and reduce the generation of stormwater runoff itself. If protected and used properly, natural drainage features generally require very little maintenance and can function effectively for many years.

Site designs should use and/or improve natural drainage pathways whenever possible to reduce or eliminate the need for stormwater pipe networks. This can reduce costs, maintenance burdens, and site disturbance related to pipe installation. Natural drainage pathways should be protected from significantly increased runoff volumes and rates due to development. The design should prevent the erosion and degradation of natural drainage pathways through the use of upstream volume and rate control BMPs, if necessary. Level spreaders, erosion control matting, revegetation, outlet stabilization, and check dams can also be used to protect natural drainage features.

Variations

Natural drainage features can also be made more effective through the design process. Examples include constructing slight earthen berms around natural depressions or other features to create additional storage, installing check dams within drainage pathways to slow runoff and promote infiltration, and planting additional native vegetation within swales and depressions.

Applications

As density and overall land disturbance decreases, this BMP can be used with a greater variety of land uses and development types. It is best used in residential development, particularly lower density single-family residential development. Where local jurisdiction ordinances already require a certain percentage of the undeveloped site to remain as undeveloped open space, this open space requirement can be overlain onto natural flow pathways/drainage features, as well as floodplains, wetlands, and related riparian areas. After minimizing runoff as much as possible, reduced runoff quantities can then be distributed into this natural flow pathway system, on a broadly distributed basis, lot by lot.

Other land uses such as commercial and industrial developments tend to be associated with higher density development. This results in higher impervious coverage and maximum site disturbance allowances, making protecting and conserving natural flow pathways/drainage areas more difficult.



Figure 2 Schematic of a site design protecting natural drainage features.
Source: Georgia Stormwater Management Manual, Volume 2: Technical Handbook, First Edition. August, 2001

Applications for both retrofit and highway/road are limited. In terms of retrofitting, some developed

sites may have elements of natural flow pathways/drainage features intact, although many presettlement site features may have been altered and/or eliminated. Developed sites of lower densities may offer limited retrofit potential. Similarly, highway/road projects are likely to be characterized by both limited site area, given the difficulties of right-of-way acquisition, as well as substantial disturbance of this limited site area.



Figure 3 Natural drainage features can guide the design
Source: Delaware Department of Natural Resources and Environmental Control - Conservation Design for Stormwater Management

Design Considerations

1. **Identify natural drainage features**. Identifying and mapping natural drainage features is generally done as part of a comprehensive site analysis. This process is an integral first step of site design (**Figure 3**. Subtle site features such as swales, drainage pathways, and natural depressions should be delineated in addition to commonly mapped hydrologic elements such as wetlands, perennial and intermittent streams, and waterbodies.

2. **Use natural drainage features to guide site design.** Instead of imposing a two-dimensional paper design on a particular site, designers can use natural drainage features to steer the site layout. Drainage features define contiguous open space and other undisturbed areas as well as road alignment and building placement. The design should minimize disturbance to natural drainage features. Drainage features that are to be protected should be clearly shown on all construction plans. Methods for protection, such as signage and fencing, should also be noted on applicable plans.

3. **Use native vegetation**. Natural drainage pathways should be planted with native vegetative buffers and the features themselves should include native vegetation where applicable. If drainage features have been previously disturbed, they can be restored with native vegetation and buffers.

4. **County Regulated Drain Considerations**: Special attention needs to be paid regarding the type and density of vegetation used if the natural drainage pathway is or will be a part of the County Regulated Drain system. The County may require that at least one side of the regulated drain is clear of woody vegetation, with continuous access provided for reconstruction and maintenance. Pre-coordination with the County Surveyor's Office is highly recommended.

Stormwater Function and Calculations

Volume reduction

Protecting natural flow pathways can reduce the volume of runoff in several ways. Reducing disturbance and maintaining a natural cover reduces the volume of runoff through infiltration and evapotranspiration. Using natural flow pathways further reduces runoff volumes through allowing increased infiltration to occur, especially during smaller storm events. Encouraging infiltration in natural depressions also reduces stormwater volumes. Employing strategies that direct non-erosive sheet flow onto naturally vegetated areas also promotes infiltration – even in areas with relatively impermeable soils.

Peak rate mitigation

Protecting natural flow pathways can reduce the peak rate of runoff in several ways. Reducing

disturbance and maintaining a natural cover reduces the runoff rate. Using natural flow pathways can lower discharge rates by slowing runoff and increasing onsite storage.

Water quality improvement

Protecting natural flow pathways improves water quality through filtration, infiltration, sedimentation, and thermal mitigation.

Maintenance

Natural drainage features that are properly protected and used as part of site development should require very little maintenance. However, periodic inspections are important. Inspections should assess erosion, bank stability, sediment/debris accumulation, and vegetative conditions, including the presence of invasive species. Problems should be corrected in a timely manner.

Protected drainage features on private property should have an easement, deed restriction, or other legal measure to prevent future disturbance or neglect.

Cost

Protecting natural flow pathways generally results in significant construction cost savings. Protecting these features results in less disturbance, clearing, and earthwork and requires less revegetation. Using natural flow pathways reduces the need and size of costly, engineered stormwater conveyance systems. Together, protecting and using natural flow pathways reduces and even eliminates the need for stormwater management facilities (structural BMPs), lowering costs even more.

Designer/Reviewer Checklist for Protect Natural Flow Pathways

ITEM	Page No.	YES	NO	N/A	NOTES
Identify in plan all natural flow pathways before proposed development?	2				
Identify in plan natural flow pathways protected post-development?	2				
Highlight in plan natural flow pathways which are integrated into stormwater management?	2, 3				
Have measures been taken to guarantee that natural pathways won't be negatively impacted by stormwater flows?	2				
Has runoff reduction recognition been calculated for natural flow pathways being protected?	3				

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BMP Fact Sheet

PROTECT RIPARIAN BUFFER AREAS

Riparian buffer areas are important elements of local communities' green infrastructure and/or LID/GI tool box. These areas are critical to the biological, chemical, and physical integrity of our waterways. Riparian buffer areas protect water quality by cooling water, stabilizing banks, mitigating flow rates, and providing for pollution and sediment removal by filtering overland sheet runoff before it enters the water. The Environmental Protection Agency defines buffer areas as, "areas of planted or preserved vegetation between developed land and surface water, [which] are effective at reducing sediment and nutrient loads."

Physical restoration of riparian buffer areas is discussed in a separate structural BMP. A detailed description of the characteristics of riparian buffer areas is combined with a discussion of their stormwater functions in the Riparian Buffer Restoration BMP.



Figure 1 Urban riparian buffer (Hamilton County, IN Urban Conservation Association)

Key Design Features

- Physical protection
- Protection through planning tools

Benefits

- Improves water quality
- Reduces runoff velocities
- Reduces flow
- Enhances site aesthetics, habitat
- Reduces shoreline and bank erosion
- Improves flood control
- Reduces water temperature

Limitations

- Limited in reducing total runoff volumes
- Size of lot and/or development site may reduce ability to protect riparian buffers

Applications		Stormwater Quantity Functions	
Residential	Yes	Volume	Low/Med
Commercial	Yes	Groundwater Recharge	Low/Med
Ultra Urban	Limited	Peak Rate	Low/Med
Industrial	Yes	Stormwater Quality Functions	
Retrofit	Yes	TSS	High
Highway/Road	Limited	TP	High
Recreational	Yes	NO_3	Medium
		Temperature	High

Additional Considerations	
Cost	Low/Med
Maintenance	Low
Winter Performance	High

Applications

As with the Protect Sensitive Areas nonstructural BMP, protecting riparian buffer areas has great value and utility for virtually all types of development proposals and land uses. This BMP works best on larger sites. Therefore, although riparian buffer programs should be advocated in the densest of settings, their application is likely to be limited in high density contexts. Creative design can maximize the potential of riparian buffers. Clustering and density bonuses are design methods available to increase the amount and connectedness of open space areas such as riparian buffers.

Design Considerations

Physical Design

Consider the following when protecting the proper riparian buffer area width and related specifications:

- Existing or potential value of the resource to be protected,
- Site, watershed, and buffer characteristics,
- Intensity of adjacent land use, and
- Specific water quality and/or habitat functions desired.

Riparian buffers can be divided into different zones (**Figure 2**) that include vegetation to enhance the quality of the body of water.

Zone 1: Also termed the "streamside zone," Zone 1 begins at the edge of the stream bank of the active channel and extends a minimum distance of 25 feet, measured horizontally on a line perpendicular to the water body. Undisturbed vegetated area aims to protect the physical and ecological integrity of the stream ecosystem. The vegetative target for the streamside zone is undisturbed native woody species with native plants forming canopy, understory, and duff layer. Where such forest does not grow naturally, the native vegetative cover appropriate for the area (such as grasses, forbs, or shrubs) is the vegetative target.

Zone 2: Also termed the "middle zone," Zone 2 extends immediately from the outer edge of Zone 1 for a minimum distance of 55 feet. This managed area of native vegetation protects key components of the stream ecosystem and provides distance between upland development and the streamside zone. The vegetative target for the middle zone is either undisturbed or managed native woody species or, in its absence, native vegetative cover of shrubs, grasses, or forbs. Undisturbed forest, as in Zone 1, is encouraged strongly to protect future water quality and the stream ecosystem.

Zone 3: Also termed the "outer zone," Zone 3 extends a minimum of 20 feet immediately from the outer edge of Zone 2. This zone prevents encroachment into the riparian buffer area, filters

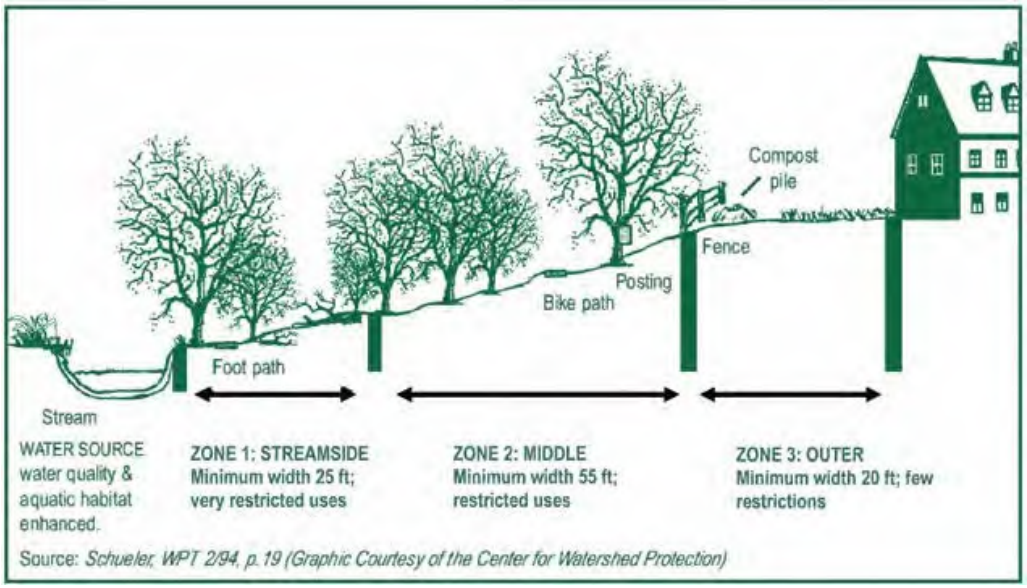


Figure 2 Buffer Width Recommendations
Source: Schueler, Watershed Protection Techniques, 1994

runoff from adjacent land, and encourages sheet flow of runoff into the buffer. The vegetative target for the outer zone is native woody and herbaceous vegetation to increase the total width of the buffer; native grasses and forbs are acceptable.

County Regulated Drain Considerations: Special attention needs to be paid regarding the type and density of vegetation used if the natural drainage pathway or stream is or will be a part of the County Regulated Drain system. The County may require that at least one side of the regulated drain is clear of woody vegetation, with continuous access provided for reconstruction and maintenance. Pre-coordination with the County Surveyor's Office is highly recommended.

Local Planning and Riparian Buffers

Numerous tools exist at the community level to protect riparian buffers, including ordinances, integrating buffers into plans, and public education.

Local buffer regulations

To effectively manage riparian buffer areas, a community must properly plan for these resources. Some Indiana communities have riparian buffer ordinances that explicitly regulate these areas. Typical components of a riparian ordinance include:

- Exemptions,
- Width requirements,
- Permitted and prohibited uses within the riparian buffer,
- Maintenance requirements,
- Waivers and variances, and
- Maintenance and construction of utilities and public roads along the stream corridor.

Natural features setback standards establish a minimum setback (buffer width) from natural features to prevent physical harm or destruction of the feature. These standards recognize the relationship between terrestrial and aquatic ecosystems and should be applied to both lakes and rivers. Each community establishes buffer width standards at their discretion.

In general, the wider the buffer, the greater the number of ecological functions the riparian zone will provide. Communities may choose to establish fixed or variable width buffers or a combination of the two (Oakland County Planning & Economic Development Services).

Key planning elements of a local riparian protection program*

- Provide ample setbacks for sanitary facilities on buffer areas
- The wider the riparian buffer, the greater the water quality protection and habitat value of the area
- Establish setbacks from rivers and streams
- Regulate road placement adjacent to the riparian buffer area
- Restrict clearing, construction, and development within the 100-year floodplain
- Zone areas adjacent to riparian buffer areas for low intensity development
- Establish minimum lot size, frontage, and width requirements
- Include reference to floodplain, soil, and sedimentation controls administered by other agencies in riparian regulations
- Screen new structures with native vegetation
- Limit industrial use along riparian corridors and regulate through special use permits subject to pre-designated standards
- Limit the amount of impervious surfaces allowed adjacent to the buffer area
- Clearly outline appropriate and inappropriate uses of riparian buffer areas
- Promote intergovernmental coordination of regulations among communities along the river corridor

* - Adapted from *Michigan Wetlands – Yours to Protect*

Integrating buffer protection into plans

In addition to implementing a riparian buffer ordinance, communities can include riparian buffer area protection in the following planning tools:

- Local master plans,
- Park and recreation plans, and
- Subdivision and land development ordinances.

Park and recreation plans can adopt the goals, policies, and objectives for riparian protection that are listed in the community master plan, or include its own park and recreation-specific recommendations for riparian buffer management. Content may focus on defining appropriate and inappropriate recreational uses for riparian areas located within parks. Park and recreation plans may also provide guidelines for proper construction and maintenance of river access points, and rules and regulations for public access as these topics relate to potential impacts on riparian buffers (Oakland County Planning & Economic Development Services).

Riparian buffer education

Educational opportunities for the general public are an important component in community planning. Informing riparian owners of the importance of buffer areas will help to ensure these areas are understood and maintained over time. Public education activities include hosting public meetings, direct mailings to riparian homeowners, and educational workshops. These activities can be developed to meet the specific needs of your community through partnerships with local watershed groups.

Design Measures

The following elements represent a menu of design measures for riparian and natural resource protection that communities may choose to encourage or require developers to incorporate during the site plan review process.

Conservation subdivision or open space regulations:

- Prepare natural features inventory on proposed project sites.
- Require certain percentage of total parcel acreage to be retained as open space.
- Reference minimum buffer widths for riparian buffer areas and identify upland areas adjacent to riparian buffer areas as preferred green space designated for low-impact residential recreation activities.
- Advocate cluster development that concentrates construction on land with less conservation value, and allows owners of house lots in the development to share undivided ownership of the portion of property remaining in a scenic and natural condition.
- Advocate lot averaging standards for retaining riparian resources and natural features on smaller sites.

Lot size and density regulations:

- Provide flexible lot size and density standards to guide development away from a stream buffer or other sensitive land.
- Provide developers with density bonuses for land-conserving design and density disincentives to actively discourage land-consuming layouts.

Minimum frontage and road setback regulations:

- Provide flexibility in frontage and road setback standards to minimize development intrusion on riparian buffer areas.

Stormwater management guidelines:

- Regulate erosion control before, during, and after construction.
- Encourage developers to retain natural vegetation already protecting waterways.
- Create a variable-width, naturally vegetated buffer system along lakes and streams that also encompasses critical environmental

features such as the 100-year floodplain, steep slopes, and wetlands.
- Limit clearing and grading of forests and native vegetation at a site to the minimum amount needed to build lots, allow access, and provide fire protection.
- Promote riparian buffer areas as part of stormwater management planning.

Source: Planning for Green River Corridors, Oakland County Planning & Economic Development Services.

Stormwater Functions and Calculations

Any portion of a site that can be maintained in its presettlement state by using this BMP will not contribute increased stormwater runoff and will reduce the amount of treatment necessary.

Volume

Protected riparian buffers are not to be included in the disturbed stormwater management area when calculating runoff volume

Any portion of a riparian buffer area that is mitigated or revegetated/reforested should be included in the disturbed stormwater management area, but may be granted runoff reduction recognition in accordance with the applicable BMP for native revegetation, soil restoration, minimize soil compaction, riparian buffer restoration, or minimize total disturbed area.

Peak rate

Runoff from the riparian buffers may be excluded from peak rate calculations for rate control, provided that runoff from the riparian buffers is not conveyed to and/or through stormwater management control structures. If necessary, runoff from riparian buffers should be directed around BMPs and stormwater pipes and inlets by means of vegetated swales or low berms that direct flow to natural drainageways.

Water quality improvement

Water quality is benefited substantially by avoiding negative impacts which otherwise would have resulted from impacts to riparian buffers (e.g., loss of water quality functions from riparian buffers, from wetland reduction, etc.).

Cost

The costs of protecting riparian areas relate to a reduction in land available for development. However, most riparian areas are located in wetlands or floodplains, restricting the amount of buildable area.

Designer/Reviewer Checklist for Protect Riparian Buffer Areas

ITEM	Page No.	YES	NO	N/A	NOTES
Define municipal programs requirements or resources for riparian buffer protection, if any.	3-4				
Based on above and relevant sources, establish riparian buffer protection standards for development site.	3-4				
Map riparian resources at the site which need buffer protection.	3				
Apply Zone1/Zone2/Zone3 determinations; adjust for steep slopes and/or other natural/made factors.	2				
Overlay development program onto site, avoiding/minimizing Riparian Buffer Zone impacts.	3				
Contacted the County Surveyor's Office if the drainage pathway or stream is a County Regulated Drain?	3				
Provide for Riparian Buffer Zone maintenance?	4				
Provide for Riparian Buffer Zone protection in perpetuity (deed restrictions? covenants? easements?)?	--				

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BMP Fact Sheet

PROTECT SENSITIVE AREAS

Protecting sensitive and special value features is the process of identifying and avoiding certain natural features during development. This allows these features to be used for various benefits, including reducing stormwater runoff.

Protecting sensitive areas can be implemented both at the site level and throughout the community. For prioritization purposes, natural resources and their functions may be weighted according to their functional value. Sensitive areas should be preserved in their natural state to the greatest extent possible and are not the appropriate place to locate stormwater infrastructure.



Figure 1 Tree Preservation during construction, Seattle, WA (USEPA, picasaweb)

Key Design Features

- Identify and map the following: floodplains, riparian areas, wetlands, woodlands, prairies, natural flow pathways, steep slopes, and other sensitive areas
- Identify and map potential development areas

Benefits

- Improved water quality
- Mitigation of runoff volume and peak rates

Limitations

- Difficult to implement on smaller sites

Applications		Stormwater Quantity Functions	
Residential	Yes	Volume	High
Commercial	Yes	Groundwater Recharge	High
Ultra Urban	Limited	Peak Rate	High
Industrial	Yes	Stormwater Quality Functions	
Retrofit	No	TSS	High
Highway/Road	Limited	TP	High
Recreational	Yes	NO_3	Low
		Temperature	High

Additional Considerations	
Cost	Low/Med
Maintenance	Low/Med
Winter Performance	High

Description and Function

Protecting sensitive areas challenges the site planner to inventory and then, to the greatest extent possible, avoid resource sensitive areas at a site, including riparian buffers, wetlands, hydric soils, floodplains, steep slopes, woodlands, valuable habitat zones, and other sensitive resource areas. Development, directed away from sensitive areas, can be held constant, if BMPs such as cluster development are also applied.

A major objective of LID/GI is to accommodate development with fewer impacts to the site. If development avoids encroachment upon, disturbance of, and impact to those natural resources which are especially sensitive to land development impacts and/or have special functional value, then low impact development can be achieved.

The first step in protecting sensitive areas is for the site planner to define, inventory, and map which resources are especially sensitive and/or have special value at a site proposed for development. Although many sensitive areas are common to all areas within Indiana, they can vary by region. The most detailed inventories are often compiled at the municipal or county level. For those areas without municipal or county-level data, state-level data can be used.

Preserving open space in multiple development areas throughout a community can ultimately evolve to form a unified open space system, integrating important conservation areas throughout the community and beyond. Many communities within Indiana are undertaking "green infrastructure" planning initiatives to proactively map these systems in order to restore or protect them as development occurs. The objective of these plans is to avoid impacting sensitive areas by: 1) carefully identifying and mapping these resources (resource areas, primary and secondary) from the start of the site planning process, and 2) striving to protect resource areas by defining other portions of the site free of these resources (potential development areas).

At the community level, local governments can implement community-wide regulations that protect sensitive areas such as wetlands, woodlands, riparian areas, and floodplains.

Potential Applications

Regardless of land use type, protecting sensitive areas is applicable across all types of land development projects, whether residential of varying densities or office park, retail center or industrial and institutional uses. As density and intensity of uses increases, ease of application of this BMP decreases. In such limited cases, it is especially important that sensitive areas be prioritized.

Design Considerations

1. **Identify, map, and inventory sensitive areas**. Mapping a site's sensitive areas is an important step in preserving them. These features often include wetlands, steep slopes, woodlands, floodplains, and riparian areas. These data may give the community a general idea of the sensitive resources that could be on the site. In addition, the mapping will help the site designer define a potential development area which avoids encroachment upon and disturbance of defined and mapped sensitive areas.

   The inventory of sensitive areas should also include an assessment of the *quality* of the existing natural communities. Because plant communities will exist in a variety of states based on historic disturbance and degradation, the quality of the given community needs to be considered in comparison to other similar communities.

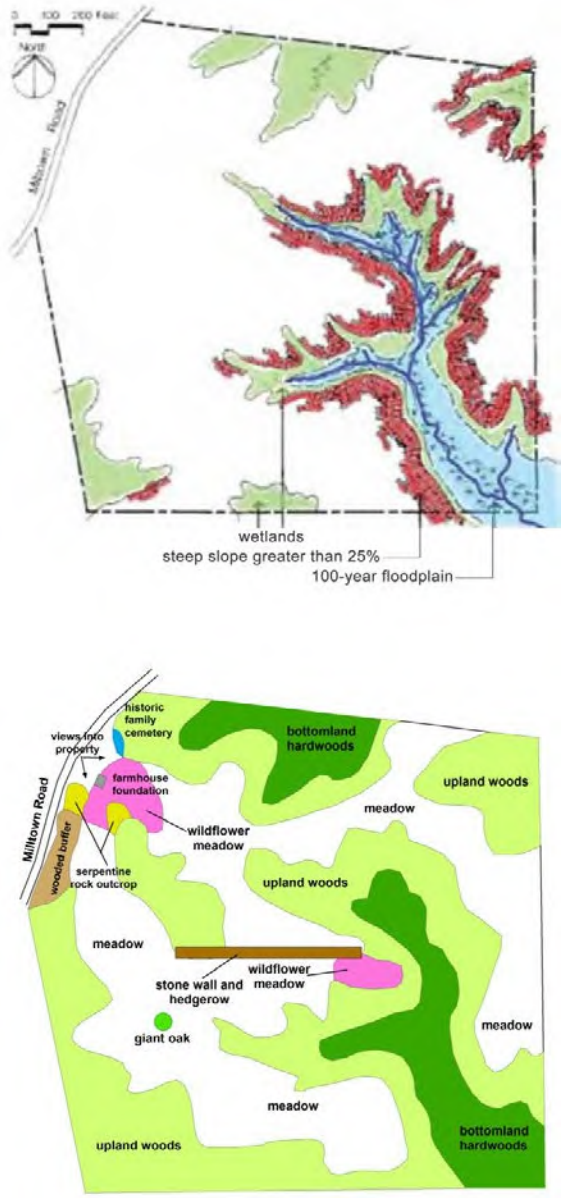


Figure 2 Map of Sensitive Areas (top) and Secondary Resources (bottom)
Source: Arendt, Randall. 1997

2. **Combine mapped natural features into a sensitive resource areas map, prioritizing areas to avoid development**. All sensitive resource mapping should be overlain to produce a sensitive areas map. In *Growing Greener*, Randall Arendt acknowledges prioritizing or weighting of sensitive areas by defining them as either Primary Conservation Areas (the most critical – avoid at all costs) or Secondary Conservation Areas (important resources which should be avoided when possible). Mapping the secondary resources of the site is an important step; the community can provide input to determine which features are important for preservation. Additionally, Primary and Secondary Conservation Areas can be defined in different ways, possibly varying with watershed context, (e.g., woodlands in some contexts may be classified as Primary Conservation Areas, rather than secondary).

3. **Map potential development areas; prioritize/ weight as necessary**. The potential development area should be delineated on the basis of protecting the primary and secondary resources on a site. Like the sensitive areas map, priorities and weightings may be reflected in the potential development area map. If sensitive areas have been prioritized, then weightings of potential development also may be established, varying with lack of degree of sensitivity measured by the resources themselves or overlapping of resources.

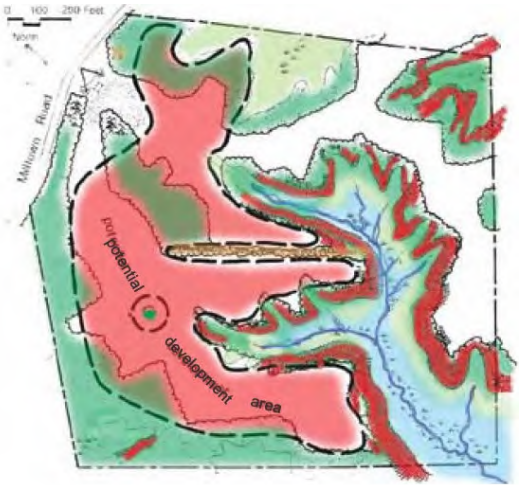


Figure 3 Map of Potential Development Areas
Source: Arendt, Randall. 1997

4. **Local regulation.** The level of regulation imposed on resource areas (primary and secondary) will likely vary by local jurisdiction. A local ordinance may prohibit and/or otherwise restrict development in primary and secondary resource areas, provided certain legal tests (such as a takings determination) are passed. Additional

activities include:

a. Conservation easement – Given to land conservancy or maintained by homeowners association.
b. Requirements in the master deed and bylaws for protection and preservation.
c. Boundary markers at edges of lots to minimize encroachment.
d. Cooperative agreements for stewardship of sensitive areas between homeowners' associations and local conservation organizations.

Stormwater Functions and Calculations

Any portion of a site that can be maintained in its predevelopment state by using this BMP will not contribute increased stormwater runoff and will reduce the amount of treatment necessary.

Volume

Protected sensitive areas are not to be included in the disturbed stormwater management area when calculating runoff volume.

Any portion of a sensitive area that is mitigated or revegetated/reforested should be included in the disturbed stormwater management area, but may be granted runoff reduction recognition in accordance with the applicable BMP for native revegetation, soil restoration, minimize soil compaction, riparian buffer restoration, or minimize total disturbed area.

Peak Rate

Runoff from the protected sensitive area may be excluded from peak rate calculations for rate control, provided that the runoff is not conveyed to and/or through stormwater management control structures. If necessary, runoff from protected sensitive areas should be directed around BMPs and stormwater pipes and inlets by means of vegetated swales or low berms that direct flow to natural drainageways.

Water Quality Improvement

Water quality is benefited substantially by avoiding negative impacts which otherwise would have resulted from impacts to sensitive areas (e.g., loss of water quality functions from riparian buffers, from wetland reduction, etc.)

Construction Guidelines

Although protecting sensitive areas happens early in the site plan process, it is equally important that the developer and builder protect these areas during construction.

The following guidelines describe good planning practices that will help ensure protection of a few common environmentally sensitive resources during construction.

Water Resources

- If vegetation needs to be reestablished, plant native species, or use hydroseed and mulch blankets immediately after site disturbance.
- Use bioengineering techniques, where possible, to stabilize stream banks.
- Block or protect storm drains in areas where construction debris, sediment, or runoff could pollute waterways.
- During and after construction activities, sweep the streets to reduce sediment from entering the storm drain system.
- Avoid hosing down construction equipment at the site unless the water is contained and does not get into the stormwater conveyance system.
- Implement spill control and clean-up practices for leaks and spills from fueling, oil, or use of hazardous materials. Use dry clean-up methods (e.g., absorbents) if possible. Never allow a spill to enter the stormwater conveyance system.
- Avoid mobile fueling of equipment. If mobile fueling is necessary, keep a spill kit on the fueling truck.
- Properly dispose of solid waste and trash to prevent it from ending up in our lakes and streams.
- When protecting riparian buffer areas, consider the three buffer zones in protection criteria:

Zone 1: Also termed the "streamside zone," Zone 1 begins at the edge of the stream bank of the active channel and extends a minimum distance of 25 feet, measured horizontally on a line perpendicular to the water body. Undisturbed vegetated area aims to protect the physical and ecological integrity of the stream ecosystem. The vegetative target for the streamside zone is undisturbed native woody species with native plants forming canopy, understory, and duff layer; where such forest does not grow naturally, then native vegetative cover appropriate for the area (such as grasses, forbs, or shrubs) is the vegetative target.

Zone 2: Also termed the "middle zone," Zone 2 extends immediately from the outer edge of Zone 1 for a minimum distance of 55 feet. This managed area of native vegetation protects key components of the stream ecosystem and provides distance between upland development and the streamside zone. The vegetative target for the middle zone is either undisturbed or managed native woody species or, in its absence, native vegetative cover of shrubs, grasses, or forbs. Undisturbed forest, as in Zone 1, is strongly encouraged to protect further water quality and the stream ecosystem.

Zone 3: Also termed the "outer zone," Zone 3 extends a minimum of 20 feet immediately from the outer edge of Zone 2. This zone prevents encroachment into the riparian buffer area, filters runoff from adjacent land, and encourages sheet flow of runoff into the buffer. The vegetative target for the outer zone is native woody and herbaceous vegetation to increase the total width of the buffer; native grasses and forbs are acceptable.

County Regulated Drain Considerations: Special attention needs to be paid regarding the type and density of vegetation used if the natural drainage pathway or stream is or will be a part of the County Regulated Drain system. The County may require that at least one side of the regulated drain is clear of woody vegetation, with continuous access provided for reconstruction and maintenance. Pre-coordination with the County Surveyor's Office is highly recommended.

Wetlands

- Avoid impacts to wetlands whenever possible. If impractical, determine if a wetland permit is needed from the state or local government. (If any permit requirements or wetland regulations conflict with these guidelines, comply with the permit or regulation).
- Excavate only what is absolutely necessary to meet engineering requirements. Do not put excavated material in the wetland. (Excavated material could be used in other areas of the site to improve seeding success).
- If construction activities need to occur within a wetland, activities should be timed, whenever possible, when the ground is firm and dry. Avoid early spring and fish-spawning periods.
- Install flagging or fencing around wetlands to prevent encroachment.
- Travel in wetlands should be avoided. Access roads should avoid wetlands whenever possible. Crossing a wetland should be at a single location and at the edge of the wetland, if possible.
- Never allow a spill to enter area wetlands.

Floodplains

- Design the project to maintain natural drainage patterns and runoff rates if possible.
- Maintain as much riparian vegetation as possible. If riparian vegetation is damaged or removed during construction, replace with native species.
- Use bioengineering techniques to stabilize stream banks.
- Keep construction activity away from wildlife crossings and corridors.
- Stockpile materials outside of the floodplain and use erosion control techniques

Woodlands and Isolated Trees in Disturbed Areas

- Protect trees on sites with severe design limitations, such as steep slopes and highly erodible soils.
- Preserve trees along watercourses to prevent bank erosion, decreased stream temperatures, and to protect aquatic life.
- Protect the critical root zone of trees during construction. This is the area directly beneath a tree's entire canopy. For every inch of diameter of the trunk, protect 1.5 feet of area away from the trunk.

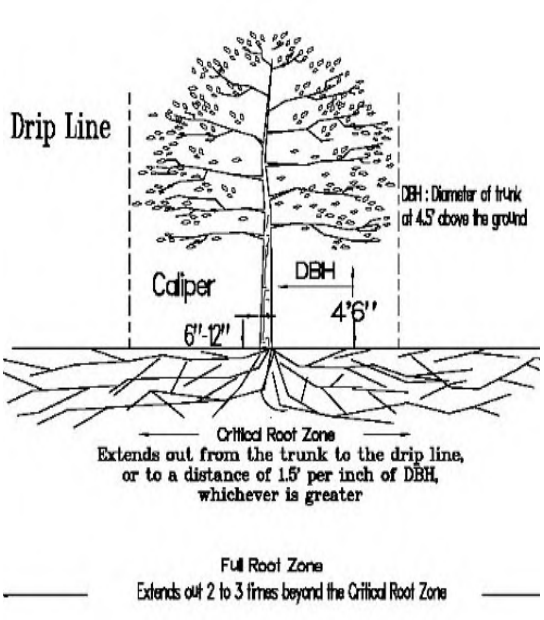


Figure 4 Critical Root Zone of a Tree
Source: www.ncufc.org/

- Avoid trenching utilities through the tree's critical root zone.
- Avoid piling excavated soil around any tree.
- Replace trees removed during construction with native trees.
- Conduct post-construction monitoring to ensure trees impacted by construction receive appropriate care.

General Construction Considerations

- Conduct a pre-construction meeting with local jurisdiction officials, contractors, and subcontractors to discuss natural resource protection. Communicate agreed-upon goals to everyone working on the project.
- Insert special requirements addressing sensitive natural areas into plans, specifications, and estimates provided to construction contractors. Note the kinds of activities that are not allowed in sensitive areas.
- Confine construction and staging areas to the smallest necessary and clearly mark area boundaries. Confine all construction activity and storage of materials to designated areas.
- Install construction flagging or fencing around sensitive areas to prevent encroachment.
- Excavate only what is absolutely necessary to meet engineering requirements. Do not put excavated material in sensitive areas. (Excavated material could be used in other areas of the site to improve seeding success.)
- Conduct onsite monitoring during construction to ensure sensitive areas are protected as planned. Conduct post-construction monitoring to ensure sensitive areas that were impacted by construction receive appropriate care.

Maintenance

The preservation of open space creates maintenance concerns related to who is required to perform the maintenance activities. Legally, the designated open space may be conveyed to the local government. More likely, ownership of these natural areas will be assumed by homeowners' associations or simply the specific individual property homeowners where these resources are located. Specific maintenance activities will depend upon the type of vegetation present in the preserved natural area where woodlands require little to no maintenance and open lawn require higher maintenance.



Figure 5 Protecting Woodlands

Cost

When development encroaches into sensitive areas, dealing with their special challenges invariably adds to development and construction costs. Sometimes these added costs are substantial, as in the case of working with wetlands or steep slopes.

Sometimes costs emerge only in longer-term operation, like encroachment in floodplains. This can translate into added risk of building damage for future owners, as well as health and safety impacts, insurance costs, and downstream flooding. If all short- and long-term costs of impacting sensitive areas were quantified and tallied, total real costs of sensitive area encroachment would increase substantially. Conversely, protecting sensitive areas results not only in cost savings, but also in water quality benefits.

At the same time, reduction in potential development areas resulting from protecting and conserving sensitive areas can have the effect of altering — even reducing — a proposed development program, thereby reducing development yield and profit. To address this, this BMP can be applied in tandem with the cluster development BMP.

Designer/Reviewer Checklist for Protect Sensitive Areas

ITEM	Page No.	YES	NO	N/A	NOTES
Define sensitive resources at proposed development site (see Key Design Features for list of sensitive resources)	1, 2				
Map sensitive resources at proposed development site	1, 2				
Prioritize/weight sensitive areas, as necessary and appropriate	3				
Develop potential development area map, or comparable, defined as converse/negative of sensitive areas, with priorities/weightings as necessary and appropriate.	3				
Determine baseline development plan, compatible with municipal ordinance.	2				
Iteratively fit baseline development plan to potential development area, minimizing sensitive area encroachment?	2				
Is this BMP process required by municipality? Yes or no, has applicant followed these steps, or comparable?	3, 4				

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BMP Fact Sheet

REDUCE IMPERVIOUS SURFACES

Reducing impervious surfaces includes minimizing areas such as streets, parking lots, and driveways. By reducing the amount of paved surfaces, stormwater runoff is decreased while infiltration and evapotranspiration opportunities are increased.



Figure 1 Grassed overflow parking areas (NEMO, University of Connecticut)

Applications		Stormwater Quantity Functions	
Residential	Yes	**Volume**	High
Commercial	Yes	**Groundwater Recharge**	High
Ultra Urban	Limited	**Peak Rate**	High
Industrial	Yes	**Stormwater Quality Functions**	
Retrofit	Limited	**TSS**	Medium
Highway/Road	Yes	**TP**	Low
Recreational	Yes	**NO_3**	Low
		Temperature	Medium

Additional Considerations	
Cost	Low
Maintenance	Low
Winter Performance	High

Key Design Features

Streets

- Evaluate traffic volumes and street parking requirements
- Consult with local fire department and road agencies
- If available, consider a private road ordinance, as necessary, to minimize width
- Minimize pavement widths and lengths by using alternative roadway layouts, restricting on-street parking, minimizing cul-de-sac radii, and using permeable pavers

Parking Lots

- Evaluate parking requirements considering average demand as well as peak demand
- Consider smaller parking stalls and/or compact parking spaces
- Analyze parking lot layout to evaluate the applicability of narrowed traffic lanes and slanted parking stalls
- If appropriate, minimize impervious parking area by using overflow parking areas constructed of pervious paving materials

Lot Level

- Use maximum lot coverage requirements to manage the amount of impervious surfaces
- Reduce front yard setbacks to allow for shorter driveways
- Use alternative materials for patios, sidewalks, driveways, as appropriate

Benefits

- Directly reduces runoff volumes and peak rates
- Reduces development and maintenance costs
- Enhances aesthetics and habitat

Limitations

- Must comply with local private road ordinances
- Must comply with vehicular safety standards

Description and Function

Reducing street imperviousness performs valuable stormwater functions in contrast to conventional development in the following ways:

- Increases infiltration,
- Decreases runoff volumes,
- Increases stormwater time of concentration,
- Improves water quality by decreasing nonpoint source pollutant loading, and
- Decreases the concentration and energy of stormwater.

Imperviousness greatly influences stormwater runoff volume and quality by increasing the rapid transport of stormwater and collecting pollutants from atmospheric deposition, automobile leaks, and additional sources.

Stream degradation has been observed at impervious levels as low as 10-20 percent watershed-wide when these areas are managed conventionally. Recent findings indicate that degradation is observed even at much lower levels of imperviousness. Reducing imperviousness improves an area's hydrology, habitat structure, and water quality.

Design Considerations

Street Width

Streets usually are the largest single component of imperviousness in residential development. Universal application of high-volume, high-speed traffic design criteria results in excessively wide streets. Coupled with the perceived need to provide both on-street parking and emergency vehicle access, the end result is residential streets that may be 36 feet or greater in width.

The American Society of Civil Engineers (ASCE) and the American Association of State Highway and Transportation Officials (AASHTO) recommend that low-traffic-volume roads (less than 50 homes or 500 daily trips) be as narrow as 22 feet. Some municipalities have reduced their lowest trafficable residential roads to 18 feet or less. Higher-volume roads are recommended to be wider. **Table 1** provides sample road widths from different jurisdictions.

Jurisdiction	Residential Street Pavement Width	Maximum Daily Traffic (trips/day)
State of New Jersey	20 ft. (no parking)	0-3,500
	28 ft. (parking on one side)	0-3,500
State of Delaware	12 ft. (alley)	--
	21 ft. (parking on one side)	--
Howard County, Maryland	24 ft. (parking not regulated)	1,000
Charles County, Maryland	24 ft. (parking not regulated)	---
Morgantown, West Virginia	22 ft. (parking on one side)	--
Boulder, Colorado	20 ft.	150
	20 ft. (no parking)	350-1,000
	22 ft. (parking on one side)	350
	26 ft. (parking on both sides)	350
	26 ft. (parking on one side)	500-1,000
Bucks County, Pennsylvania	12 ft (alley)	--
	16-18 ft. (no parking)	200
	20-22 ft. (no parking)	200-1,000
	26 ft. (parking on one side)	200
	28 ft. (parking on one side)	200-1,000

Table 1 Narrow Residential Street Widths
Source: Cohen, 1992; Bucks County Planning Commission, 1980; Center for Watershed Protection, 1998

Need for adequate emergency vehicle access, notably fire trucks, also leads to wider streets. While it is perceived that very wide streets are required for fire trucks, some local fire codes permit roadway widths as narrow as 18 feet (**Table 2**). Concerns also exist relating to other vehicles and maintenance activities on narrow streets. School buses are typically nine feet wide, mirror to mirror.

Source	Residential Street Width
U.S. Fire Administration	18-20 ft.
Baltimore County, Maryland Fire Department	16 ft. (no on-street parking)
	24 ft. (on-street parking)
Virginia State Fire Marshall	18 ft. minimum
	24 ft. (no parking)
Prince George's County, Maryland Department of	30 ft. (parking on one side)
Environmental Resources	36 ft. (parking on both sides)
	20 ft. (fire truck access)
Portland, Oregon Office of Transportation	18 ft. (parking on one side)

Table 2 Fire Vehicle Street Requirements
Source: Adapted from Center for Watershed Protection, 1998

Prince George's and Montgomery Counties in Maryland require only a 12-foot driving lane for buses (Center for Watershed Protection, 1998).

Similarly, trash trucks require only a 10.5-foot driving lane. Trash trucks have a standard width of nine feet (Waste Management, 1997; BFI, 1997). In some cases, road width for emergency vehicles may be added through use of permeable pavers for roadway shoulders.

Snow removal on narrower streets is readily accomplished with narrow, eight-foot snowplows. Restricting parking to one side of the street allows accumulated snow to be piled on the other side of the street. Safety concerns are also cited as a justification for wider streets, but increased vehicle-pedestrian accidents on narrower streets are not supported by research. In fact, wider streets have been shown to promote increased speeds and accidents. The Federal Highway Administration states that narrower streets reduce vehicle travel speeds, lessening the incidence and severity of accidents.

Higher density developments require wider streets, but alternative layouts can minimize street widths. For example, in instances where on-street parking is desired, impervious pavement is used for the travel lanes, with permeable pavers placed on the road apron for the parking lanes. The width of permeable pavers is often the width of a standard parking lane (six to eight feet). This design approach minimizes impervious area while also providing an infiltration and recharge area for the impervious roadway stormwater.

Street Length

Numerous factors influence street length, including clustering techniques. As with street width, street length greatly impacts the overall imperviousness of a developed site. While no one prescriptive technique exists for reducing street length, alternative street layouts should be investigated for options to minimize impervious cover. Successful clustering design consistently has shown to reduce required street lengths, holding development programs constant (i.e., 100 homes successfully clustered on a 100-acre property results in a significant reduction in street length and total imperviousness than 100 homes conventionally gridded in large-lot development format).

Cul-de-sacs

The use of cul-de-sacs introduces large areas of imperviousness into residential developments. Some communities require the cul-de-sac radius to be as large as 50 to 60 feet. Simply reducing the radius from 40 feet to 30 feet can reduce the imperviousness by 50 percent (Schueler, 1995).

When cul-de-sacs are necessary, three primary alternatives can reduce their imperviousness; reduce the required radius, incorporate a landscaped island into the center of the cul-de-sac, or create a T-shaped (or hammerhead) turnaround (**Figure 2**). To reduce the radius, many jurisdictions have identified required turnaround radii (**Table 3**).

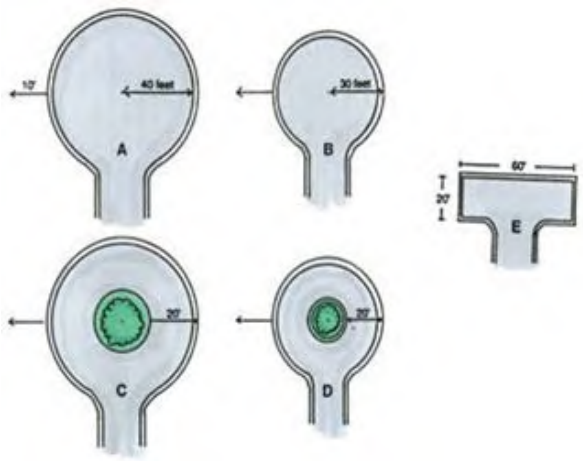


Figure 2 Five Cul-de-Sac Options
Source: Center for Watershed Protection, 1998

A landscaped island in the center of a cul-de-sac can provide the necessary turning radius, minimizing impervious cover. This island can be designed as a depression to accept stormwater runoff from the surrounding pavement, thus furthering infiltration. A flat apron curb will stabilize roadway pavement and allow for runoff to flow into the cul-de-sac's open center.

A T-shaped turnaround reduces impervious surface even further – yielding a paved area less than half that of a 30-foot radius turnaround. Since vehicles need to make a three-point turn to drive out, T-shaped turnarounds are most appropriate on streets with 10 or fewer homes.

Source	Radius
Portland, Oregon Office of Transportation	35 ft (with fire dept. approval)
Buck County, Pennsylvania Planning Commission	38 ft (outside turning radius)
Fairfax County, Virginia Fire and Rescue	45 ft
Baltimore County, Maryland Fire Department	35 ft (with fire dept. approval)
Montgomery County, Maryland Fire Department	45 ft
Prince George's County, Maryland Fire Department	43 ft

Table 3 Cul-de-sac Turning Radii
Source: Adapted from Center for Watershed Protection, 1998

Parking

Parking lots often comprise the largest percentage of impervious area. Parking lot size is dictated by lot layout, stall geometry, and parking ratios. Modifying any or all of these three aspects can serve to minimize the total impervious areas associated with parking lots. Parking ratio requirements and accommodating peak parking demand often provide parking capacity substantially in excess of average parking needs. This results in vast quantities of unused impervious surface. A design alternative to this scenario is to provide designated overflow parking areas.

The primary parking area, sized to meet average demand, might still be constructed on impervious pavement to meet local construction codes and American with Disabilities Act requirements. However, the overflow parking area, designed to accommodate increased parking requirements associated with peak demand, could be constructed on pervious materials (e.g., permeable pavers, grass pavers, gravel. See Porous Pavement BMP Fact Sheet). This design approach, focused on average parking demand, will still meet peak parking demand requirements while reducing impervious pavement.

Parking Ratios

Parking ratios express the specified parking requirements provided for a given land use. These specified ratios are often set as minimum requirements. Many developers seeking to ensure adequate parking provide parking in excess of the minimum parking ratios. Additionally, commercial parking is often provided to meet the highest hourly demand of a given site, which may only occur a few times per year. However, average parking demand is generally less than the typical required parking ratios (**Table 4**).

Parking spaces are comprised of five impervious components (Center for Watershed Protection, 1998):

1. The parking stall,
2. The overhang at the stall's edge,
3. A narrow curb or wheel stop,
4. The parking aisle that provides stall access, and

5. A share of the common impervious areas (e.g., fire lanes, traffic lanes).

Land Use	Parking Ratio	Average Parking Demand
Single Family Home	2 spaces per dwelling unit	1.1 spaces per dwelling unit
Shopping Center	5 spaces per 1,000 ft_2 of GFA	3.97 spaces per 1,000 ft_2 of GFA
Convenience Store	3.3 spaces per 1,000 ft_2 of GFA	Not available
Industrial	1 space per 1,000 ft_2 of GFA	1.48 spaces per 1,000 ft_2 of GFA
Medical/Dental Office	5.7 spaces per 1,000 ft_2 of GFA	4.11 spaces per 1,000 ft_2 of GFA

(GFA – Gross Floor Area, excluding storage and utility space)
Table 4 Example Minimum Parking Ratios
Source: Institute of Transportation Engineers, 1987; Smith, 1984: Wells, 1994

Of these, the parking space itself accounts for approximately 50 percent of the impervious area, with stall sizes ranging from 160 to 190 square feet.

Several measures can be taken to limit parking space size. First, jurisdictions can review standard parking stall sizes to determine their appropriateness. A typical stall dimension may be 10 feet by 18 feet, much larger than needed for many vehicles. The great majority of SUVs and vehicles are less than seven feet in width, providing opportunity for making stalls slightly narrower and shorter. In addition, a typical parking lot layout includes parking aisles that accommodate two-way traffic and perpendicularly oriented stalls. The use of one-way aisles and angled parking stalls can reduce impervious area.

Municipalities can also stipulate that parking lots designate a percentage of stalls as compact parking spaces. Smaller cars comprise a significant percentage of vehicles and compact parking stalls create 30 percent less impervious cover than average-sized stalls (Center for Watershed Protection, 1998).

Stormwater Functions and Calculations

Quantifying impervious areas at a proposed development site, pre- to post-development continues to dominate stormwater calculations. Stormwater calculations, are sensitive to pervious areas and their contribution to total volume of runoff, increased peak rate of runoff, and increased generation of nonpoint source pollutants. A reduction in imperviousness achieved through reduced street widths and lengths and reduced paved parking areas automatically reduces the volume and peak rate of runoff. To the extent that water quality is linked to runoff volume, reduction in imperviousness translates into a reduction in water quality management requirements as compared with standard design.

Maintenance

A reduction in impervious area results in decreased maintenance. For example, whether publicly or privately maintained, reducing roadway or parking lot imperviousness typically translates into reduction in all forms of maintenance required, from basic roadway repair to winter maintenance and snow removal.

Cost

Street Width

Costs for paving are estimated to be approximately $15 per square yard (Center for Watershed Protection, 1998), which would be considerably higher in current dollars. At this cost, for each one-foot reduction in street width, estimated savings are $1.67 per linear foot of paved street. For example, reducing the width of a 500-foot road by five feet would result in a savings of over $4,100, which would be considerably higher in current dollars. This cost is exclusive of other construction costs including grading and infrastructure.

Street Length

Factoring in pavement costs at $15 per square yard (as above), a 100-foot length reduction in a 25-foot-wide road would produce a savings in excess of $4,000 (much higher in current dollars).

In addition to pavement costs, costs for street lengths, including traditional curb and gutter and stormwater management controls, are

approximately $150 per linear foot of road (Center for Watershed Protection, 1998), which would be considerably higher in current dollars. Decreasing road length by 100 feet would save an additional $15,000, for a combined total of $19,100.

Parking

Estimates for parking construction range from $1,200 to $1,500 per space (Center for Watershed Protection, 1998), which would be significantly higher in current dollars. For example, assuming a cost of $1,200 per parking space, reducing the required parking ratio for a modest 20,000 square foot shopping strip from five spaces per 1,000 square feet to four spaces per 1,000 square feet would represent a savings of $24,000.

Designer/Reviewer Checklist for Reducing Impervious Surfaces

ITEM	Page No.	YES	NO	N/A	NOTES
Check municipal ordinances for requirements/ specifications for roads, drives, parking, walkways, other (problems vs. opportunities?), including safety requirements.	1				
Have both macro (e.g., clustering) and micro site planning (e.g., reduced setbacks) activities been applied fully?	1, 3				
Have LID/GI impervious reduction standards for roads, drives, parking, and other impervious areas been consulted and applied? ➢ *Street Width: See Tables 1 and 2 on Pages 2-3* ➢ *Cul-de-sac: See Table 3 on Page 4*	2-4				
Have roads and drives been reduced or narrowed as much as possible?	1-3				
Have macro parking ratios, lot layout, sharing strategies, and micro strategies (sizes/dimensions) been applied fully? ➢ *Parking Ratios: See Table 4 on Page 5*	4, 5				
Have pervious surfaces been applied for roads, drives, walks, parking, patios, and other hard surfaces, with maintenance been provided?	5				

➢ *Denotes Minimum Design Considerations*

References

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BMP Fact Sheet

STORMWATER DISCONNECTION

Minimize stormwater volume by disconnecting roof leaders, impervious roads, and driveways and direct runoff to other BMPs including vegetated areas that infiltrate at the site.



Figure 1 Curb cuts and vegetated areas along a Portland, OR street (USEPA, picasaweb)

Applications		Stormwater Quantity Functions	
Residential	Yes	Volume	High
Commercial	Yes	Groundwater Recharge	High
Ultra Urban	Limited	Peak Rate	High
Industrial	Limited	Stormwater Quality Functions	
Retrofit	Limited	TSS	High
Highway/Road	Limited	TP	High
Recreational	Yes	NO_3	Low/Med
		Temperature	High

Additional Considerations	
Cost	Low
Maintenance	Low
Winter Performance	Low

Variations

- Rooftop disconnection
- Driveway/walkway/small parking areas/patio disconnection
- Minor roads
- Distribute to existing vegetated services
- Distribute to existing depressions, re-graded areas
- Distribute via curb cuts/curb removal

Key Design Features

- Encourages shallow sheet flow through vegetated areas
- Directs flows into stabilized vegetated areas, including on-lot swales and bioretention areas
- Limits the contributing rooftop area to a maximum of 500 ft^2 per downspout
- Maximizes overland flows
- Minimizes use of curb and gutter systems and piped drainage systems

Site Factors

- Water table to bedrock depth: two-foot minimum
- Soils: A, B
- Slope: maximum 5 percent
- Potential hotspots: No
- Max. drainage area: rooftop area of 1,000 ft^2

Benefits

- Reduces runoff volume and peak rate
- Increases water quality benefits

Limitations

- Requires area for infiltration

Description and Function

Roofs, roads, and driveways account for a large percentage of post-development imperviousness. These surfaces influence stormwater quality and runoff volume by facilitating the rapid transport of stormwater and collecting pollutants from rainfall, automobile leaks, and additional sources.

Disconnecting roof leaders and routing road and driveway runoff from conventional stormwater conveyance systems allows runoff to be collected and managed onsite. Runoff can be directed to designed vegetated areas for onsite storage, treatment, and volume control. This is a distributed, low-cost method for reducing runoff volume and improving stormwater quality through:

- Increasing infiltration and evapotranspiration,
- Decreasing stormwater runoff volume, and
- Increasing stormwater time of concentration.

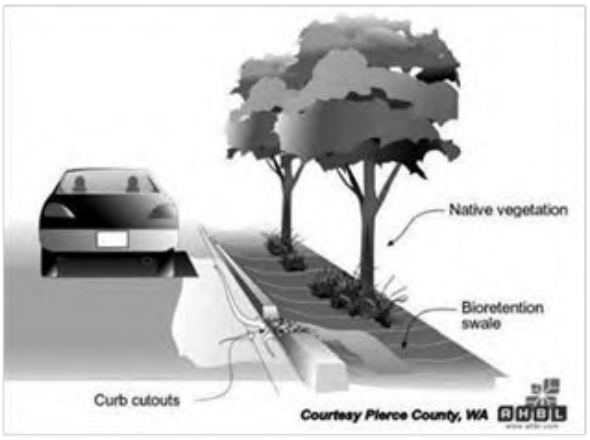


Figure 2 Curb cut-outs allow stormwater runoff from a parking lot to flow into a bioretention swale

The suitability of vegetated swales to receive runoff depends on land use, soil type, imperviousness of the contributing watershed, and dimensions and slope of the vegetated swale system. Use of natural low-lying areas is encouraged; natural drainage courses should be used and preserved.

Some ponding of water in areas receiving runoff may occur. It is important to take into account site usage when applying this BMP so that ponding does not unnecessarily interfere with expected site use (including backyard play areas). These areas should be shown on plan documents and protected with easements and deed restrictions.

Although this BMP can be applied in a variety of development settings, it will likely be more successful as lot size increases and density decreases. In situations where clustering has not been fully exercised and lots remain relatively large, these lots and the large areas of perviousness make perfect candidates for stormwater disconnection.

Variations

Disconnecting stormwater can be achieved through identifying the source of runoff and how it will be managed once disconnection occurs.

Source

Stormwater can flow from rooftop areas or from impervious areas such as driveways, walkways, small parking areas, minor roadways, and ancillary outdoor areas such as patios. (Note: Roads and highways, because of their greater runoff generation require Structural BMPs.)

Management practices

A common and successful management practice is to direct stormwater runoff to areas of existing vegetation. Vegetation can be of varying types, from established meadow to immature to mature woodland. A particular variation to consider is grading (crowning) of drives and minor roadways and

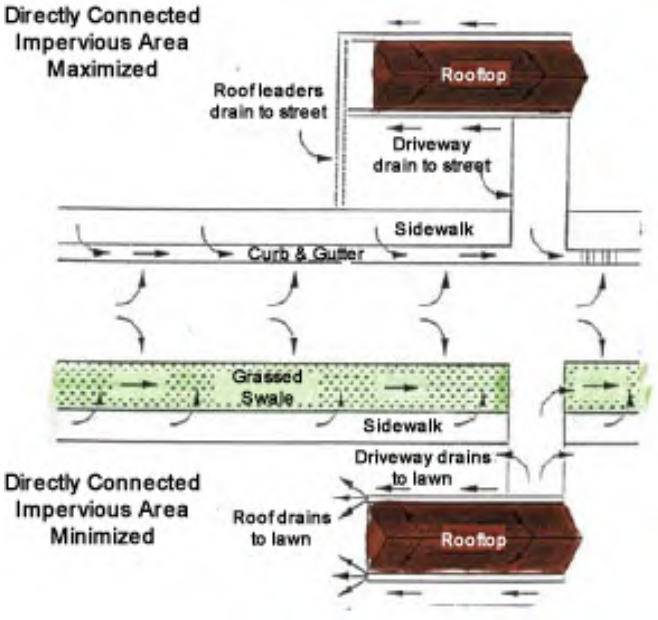


Figure 3 Difference between maximizing and minimizing runoff

eliminating curbing (or provision of curb cuts) so that runoff is allowed to flow in an even and non-concentrated manner onto adjacent vegetated areas.

In addition to directing runoff to vegetated areas, runoff may also be discharged to non-vegetated BMPs, such as dry wells, rain barrels, and cisterns for stormwater retention and volume reduction.

Another management practice includes routing runoff to existing grades and depressions that can be used to capture, store, and treat runoff. An important caveat is that applying this BMP should not prompt grading and disturbing areas which otherwise would not have been disturbed. However, assuming that grading and disturbance cannot be avoided, then subtle adjustments to grading may create additional management/storage opportunities for disconnected runoff.

An ideal coupling of BMPs is to minimize the total disturbed area of a site in coordination with stormwater disconnection. This not only reduces runoff volumes, peak rates, and pollutant loadings, but also provides multiple decentralized opportunities to receive disconnected flows.

Applications

Disconnection is ideal for most single-family developments, but can also be applied to many development sites, including larger office parks and retails centers. Industrial developments, with their larger impervious covers and greater runoff volumes, make stormwater disconnection a challenge. Even so, there are isolated applications which are beneficial and promote LID/GI objectives. Similarly, Ultra Urban and Highway/Road developments with large flows would be more limited in application.

If downspout disconnection is applied as a retrofit, downspouts should be extended away from the basement as many footing drains are attached to the sanitary sewer system.

Design Considerations

Careful consideration should be given to the design of vegetated collection areas. Concerns pertaining to basement seepage and water-soaked yards are warranted, with the potential arising for saturated depressed areas and eroded water channels. Proper design and use of bioretention areas, infiltration

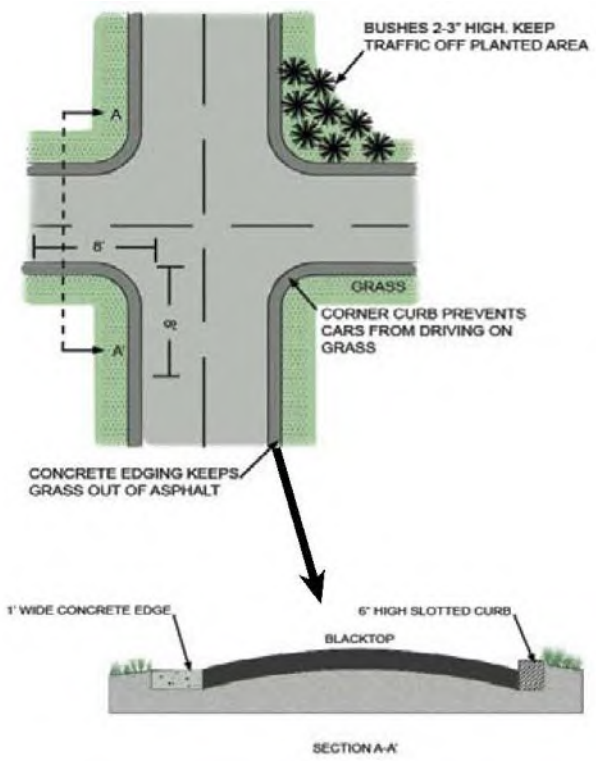


Figure 4 Curb cuts as a method of stormwater disconnection

trenches, and/or dry wells reduces or eliminates the potential for surface ponding and facilitates functioning during cold weather months. Where basements exist, consider the direction of groundwater flow and proximity.

Disconnection of small runoff flows can be accomplished in a variety of ways (Prince George's County Department of Environmental Protection, 1997; Maryland Department of the Environment, 1997; Cahill, 2008).

1. Encourage shallow sheet flow through vegetated areas.

2. Direct roof leader flow into BMPs designed specifically to receive and convey rooftop runoff.

3. Direct flows into stabilized vegetated areas, including on-lot swales and bioretention areas.

4. Rooftop runoff may also be directed to onsite depression storage areas.

5. The entire vegetated "disconnection" area should have a maximum slope of five percent.

6. Runoff should not be directed to vegetated areas if there is reason to believe that pollutant loadings will be elevated.

7. Roof downspouts or curb cuts should be at least 10 feet away from the nearest connected impervious surface to discourage "re-connections."

   a. Limit the contributing impervious area to a maximum of 1,000 sq. ft. per discharge point.

   b. Limit the contributing rooftop area to a maximum of 1,000 sq. ft. per downspout, where pervious area receiving runoff must be at least twice this size.

   c. For contributing areas greater than 1,000 sq. ft., leveling devices are recommended.

8. The maximum contributing impervious flow path length should be 75 feet.

9. For impervious areas, the length of the disconnection area must be at least the length of the contributing area (a minimum 75 feet for discharges which are concentrated; 25 feet for discharges which are not concentrated).

10. In all cases, flows from roof leaders should not contribute to basement seepage.

Stormwater runoff from disconnection needs to be monitored to ensure that flows do not become channel-ized that can result in erosion. Attention must be given to safe overflowing of larger storms, though clearly the more frequent smaller storms are of greatest interest and concern for successful design (use two-year storm for erosion analysis). Make sure flow of water and temporary ponding of water in management areas will not become a problem.

Stormwater Functions and Calculations

Peak rate and volume

This BMP reduces total volume and peak rates of runoff, as runoff is minimized from centralized stormwater management systems at the development site. Disconnection directly reduces volume and peak rates, which reduces the need for structural BMPs.

Water quality improvement

In terms of rooftop disconnection, this BMP has limited water quality benefit because rooftops typically have minimal pollution. In terms of other impervious area runoff sources being disconnected (driveways, walkways, ancillary areas, minor roads), water quality benefits can be significant given their greater pollutant loadings.

Maintenance

When disconnecting stormwater from rooftops or other impervious surfaces, maintaining the vegetated areas is required, but is limited.

If using structural BMPs, such as bioretention or vegetated swales, follow their specific maintenance activities. Typical maintenance of vegetation includes a biannual health evaluation of the vegetation and subsequent removal of any dead or diseased vegetation plus mulch replenishment, if included in the design. This can be incorporated into regular maintenance of the site landscaping. In some cases, if leaders are directing stormwater to lawn depressions, maintenance may be as simple as mowing.

Cost

Stormwater disconnection reduces both construction and maintenance costs due to less reliance on traditional stormwater management infrastructure. In addition, using existing or planned bioretention areas within a site creates a double usage of these BMPs.

Designer/Reviewer Checklist for Disconnection

ITEM	Page No.	YES	NO	N/A	NOTES
Are site factors conducive to disconnection (infiltration- related factors? slope? other?)	2, 3				
Is proposed development type (e.g., residential, commercial) conducive to disconnection? Free of hot spots?	3				
Have potential disconnection runoff sources been adequately reviewed/utilized in terms of proposed plan? ➢ *Max. slope for entire vegetated "disconnection" area:* ***5%*** ➢ *Minimum distance of roof downspouts or curb cuts from nearest connected impervious surface to discourage "re-connections":* ***10 feet*** ➢ *Max. contributing impervious area:* ***1000 ft^2 per discharge point*** ➢ *Max. contributing rooftop area (receiving pervious area must be twice this size):* ***1000 ft^2 per downspout*** ➢ *If contributing area > 1000 ft^2, leveling device is recommended* ➢ *Max. length of contributing impervious flow path:* ***75 feet*** ➢ *Minimum length of disconnection area to contributing impervious area:* ***75 feet for concentrated discharges; 25 feet for non-concentrated discharges***	3, 4				
Have potential disconnection management measures been used/exploited for all potential sources?	2, 3				
Have Criteria and Runoff Reduction Recognitions specifications for both rooftop and non-rooftop sources of disconnection been satisfied?	3				
Have disconnection calculation Runoff Reduction Recognitions been properly entered, as specified in Criteria and Runoff Reduction Recognitions?	3				

➢ *Denotes Minimum Design Consideration*

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BMP Fact Sheet

BIORETENTION (RAIN GARDENS)

Bioretention areas (often called rain gardens) are shallow surface depressions planted with specially selected native vegetation to capture and treat stormwater runoff from rooftops, streets, and parking lots.



Figure 1 Residential rain garden, Lenexa KS (USEPA, picasaweb)

Stormwater Quantity Functions			
Residential	Yes	Volume	Med/High
Commercial	Yes	Groundwater Recharge	Med/High
Ultra Urban	Limited	Peak Rate	Medium
Industrial	Yes	Stormwater Quality Functions	
Retrofit	Yes	TSS	High
Highway/Road	Yes	TP	Medium
Recreational	Yes	TN	Medium
		Temperature	High

Additional Considerations	
Cost	Medium
Maintenance	Medium
Winter Performance	Medium

Variations

- Subsurface storage/infiltration bed
- Use of underdrain
- Use of impervious liner

Key Design Features

- Flexible in size and infiltration
- Ponding depths 6-18 inches for drawdown within 48 hours
- Native plants
- Amend soil as needed
- Provide positive overflow for extreme storm events

Site Factors

- Water table/bedrock separation: two-foot minimum, four foot recommended
- Soils: HSG A and B preferred; C & D may require an underdrain (see Infiltration BMP)
- Feasibility on steeper slopes
- Potential hotspots: Yes with pretreatment and/or impervious liner
- Maximum drainage area: 5:1, not more than 1 acre to one area

Benefits

- Volume control and groundwater recharge, moderate peak rate control, filtration
- Versatile with broad applicability
- Enhance site aesthetics, habitat
- Potential air quality and climate benefits

Limitations

- Higher maintenance until vegetation is established
- Limited impervious drainage area
- Requires careful selection and establishment of plants

Description and Function

Bioretention is a method of managing stormwater by pooling water within a planting area and allowing the water to infiltrate the garden. In addition to managing runoff volume and reducing peak discharge rates, this process filters suspended solids and related pollutants from stormwater runoff. Bioretention can be implemented in small, residential applications or as part of a management strategy in larger applications.

Bioretention is designed into a landscape as a typical garden feature, to improve water quality while reducing runoff quantity. Rain gardens can be integrated into a site with a high degree of flexibility and can integrate nicely with other structural management systems including porous pavement parking lots, infiltration trenches, and other non-structural stormwater BMPs.

Bioretention vegetation serves to filter (water quality) and transpire (water quantity) runoff, and enhance infiltration. Plants absorb pollutants while microbes associated with the plant roots and soil break them down. The soil medium filters out pollutants and allows storage and infiltration of stormwater runoff, providing volume control. In addition, engineered soil media may serve as a bonding surface for nutrients to enhance pollutant removal.

Properly designed bioretention techniques provide a layer of compost that acts like a sponge to absorb and hold runoff. Vegetation in the rain garden can be diverse, through the use of many plant species and types, resulting in a system tolerant to insects, diseases, pollution, and climatic stresses.

The term "rain garden" is used to refer to smaller-scale bioretention facilities typically found on residential properties

Bioretention can accomplish the following:

- Reduce runoff volume
- Filter pollutants, through both soil particles (which trap pollutants) and plant material (which take up pollutants
- Provide habitat
- Recharge groundwater (if no underdrain is placed underneath)
- Reduce stormwater temperature impacts
- Enhance site aesthetics
- Higher maintenance until vegetation is established
- Limited impervious drainage area
- Requires careful selection and establishment of plants

Figure 2 illustrates a schematic of a relatively simple bioretention area (or rain garden). **Figure 3** illustrates a schematic of a bioretention area that is a more technically engineered structure, designed to complete specific stormwater management goals. Pond depth, soil mixture, infiltration bed, perforated underdrains, domed risers, and positive overflow structures may be designed according to the specific, required stormwater management functions.

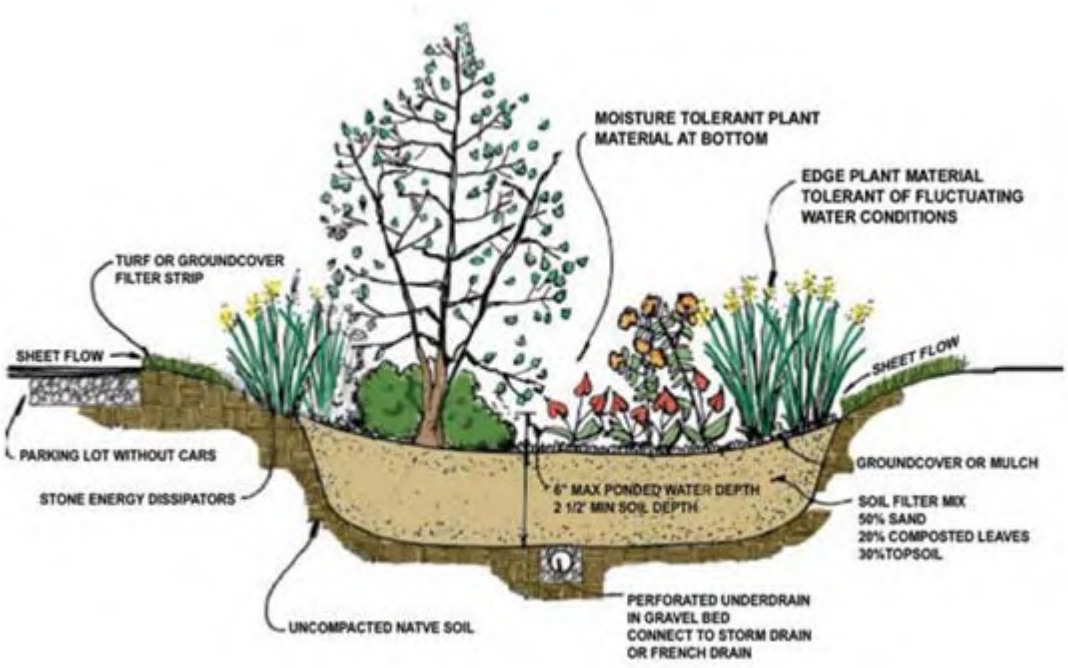


Figure 2 Schematic of a small residential rain garden

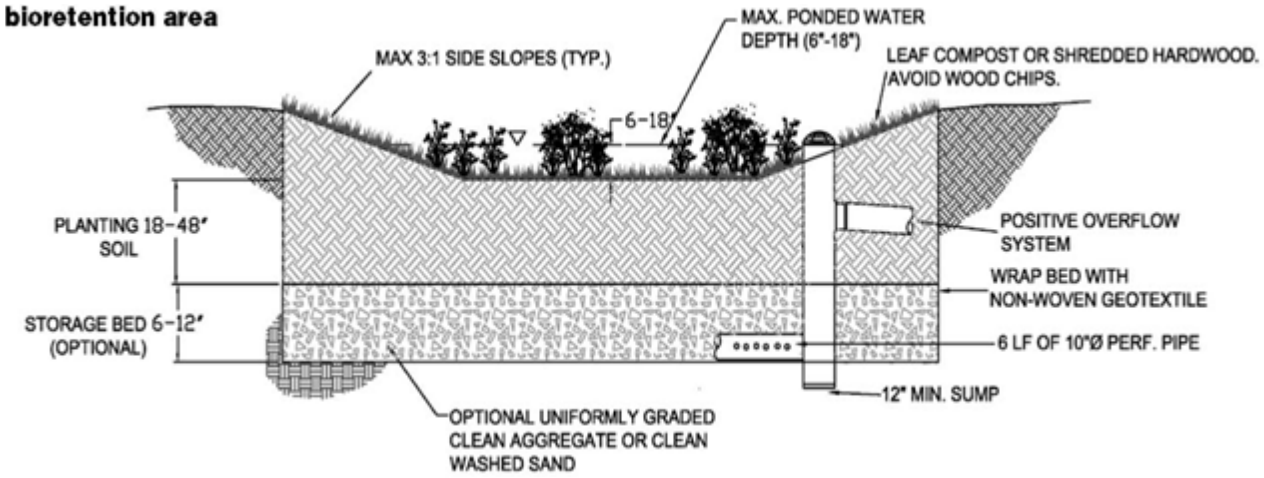


Figure 3 Schematic of a technically engineered bioretention area

Variations

A bioretention system is a depression in the ground planted like a garden that provides for the storage and infiltration of relatively small volumes of stormwater runoff, often managing stormwater on a lot-by-lot basis. This use of many small stormwater controls versus one large detention area promotes the low impact development goal of decentralized treatment of stormwater. But, if greater volumes of runoff must be managed or stored, a bioretention system can be designed with an expanded subsurface infiltration bed, or can be increased in size. Typically, the ratio of impervious area draining to the bioretention area should not exceed five-to-one, and the total impervious area draining to a single system should not be more than one acre. Variations noted relate to performance types, flow entrance, and positive overflow.

Performance types

Depending on varying site conditions, bioretention can be designed to allow for 1) complete infiltration, 2) infiltration/filtration, or 3) filtration. These variations will often determine the need for such design features as the gravel bed, double-walled underdrains, and impervious liners.

Bioretention using complete infiltration occurs in areas where groundwater recharge is beneficial and the soils have the permeability necessary to accommodate the inflow. This type of BMP is often less expensive to construct because there is no underdrain and the soils on site are often used.

The most common variation to this type of bioretention includes a gravel or sand bed underneath the planting bed and often accompanied by the use of a double-walled underdrain. This allows for additional storage or for areas with low permeability to use bioretention as infiltration, as well as, filtration (Figure 3). Some volume reduction will occur through infiltration, as well as evaporation and transpiration.

Another variation is to use bioretention primarily for filtration. This is often used in contaminated soils or hot spot locations using an impervious liner to prevent infiltration and groundwater contamination. The primary stormwater function then becomes filtration with some volume reduction through evaporation and transpiration.

For areas with low permeability, bioretention may achieve some infiltration while acting as detention with peak rate control for all storms up to the design storm.

Flow inlet

Pretreatment of runoff should be provided where sediment or pollutants entering the rain garden may cause concern or decreased BMP functionality. Soil erosion control mats, blankets, or rock must be used where runoff flows from impervious areas enter the rain garden.

Flow inlet: Trench drain

Trench drains can accept runoff from impervious surfaces and convey it to a rain garden. The trench drain may discharge to the surface of the rain garden or may connect directly to an aggregate infiltration bed beneath.

Educational Signage

Once a bioretention area is established, installing signage will help the general public and maintenance crews recognize LID/GI practices, which can help promote sustainable stormwater management. Educational signs can incorporate LID/GI goals and maintenance objectives, in addition to the type of LID/GI project being employed.

Flow inlet: Curbs and curb cuts

Curbs can be used to direct runoff from an impervious surface along a gutter to a low point where it flows into the rain garden through a curb cut. Curb cuts may be depressed curbs, or may be full height curbs with openings cast or cut into them.

Positive overflow

A positive overflow, via the surface or subsurface, is recommended to safely convey excessive runoff from extreme storm events.

Positive overflow: Domed riser

A domed riser may be installed to ensure positive, controlled overflow from the system. Once water ponds to a specified depth, it will begin to flow into the riser through a grate, which is typically domed to prevent clogging by debris.

Positive overflow: Inlet structure

An inlet structure may also be installed to ensure positive, controlled overflow from the system. Once water ponds to a specified depth, it will begin to flow into the inlet.

Applications

Bioretention areas can be used in a variety of applications, from small areas in residential lawns to extensive systems in commercial parking lots (incorporated into parking islands or perimeter areas). Industrial, retrofit, highway/road, and recreational areas can also readily incorporate bioretention. One key constraint in using bioretention in ultra-urban settings is space.

Residential

The residential property owner that wants to design and build a rain garden at home does not need to go through the engineering calculations listed under stormwater calculations and functions. Assistance with simple rain gardens is available from several sources listed under the Plant Selection portion of this BMP. **Figure 4** shows a typical rain garden configuration on a residential property.

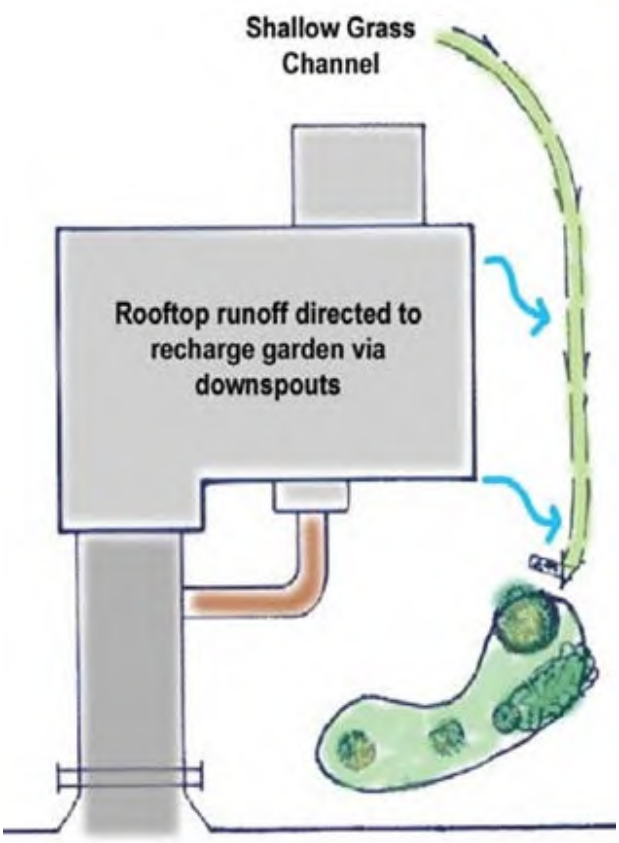


Figure 4 Single-family residential lot drainage schematic
Source: Claytor and Schueler, 1995 with modifications by Cahill

Another source of water for a small rain garden is connecting the roof leader from adjacent buildings. The stormwater may discharge to the surface of the bioretention area or may connect directly to an aggregate infiltration bed beneath.

Tree and shrub pits

Tree and shrub pits intercept runoff and provide shallow ponding in mulched areas around the tree or shrub (**Figure 5**). Mulched areas should typically extend to the tree's drip line. Plant material should be selected based on tolerance to standing water as identified in Recommended Plant List for BMPs Appendix.

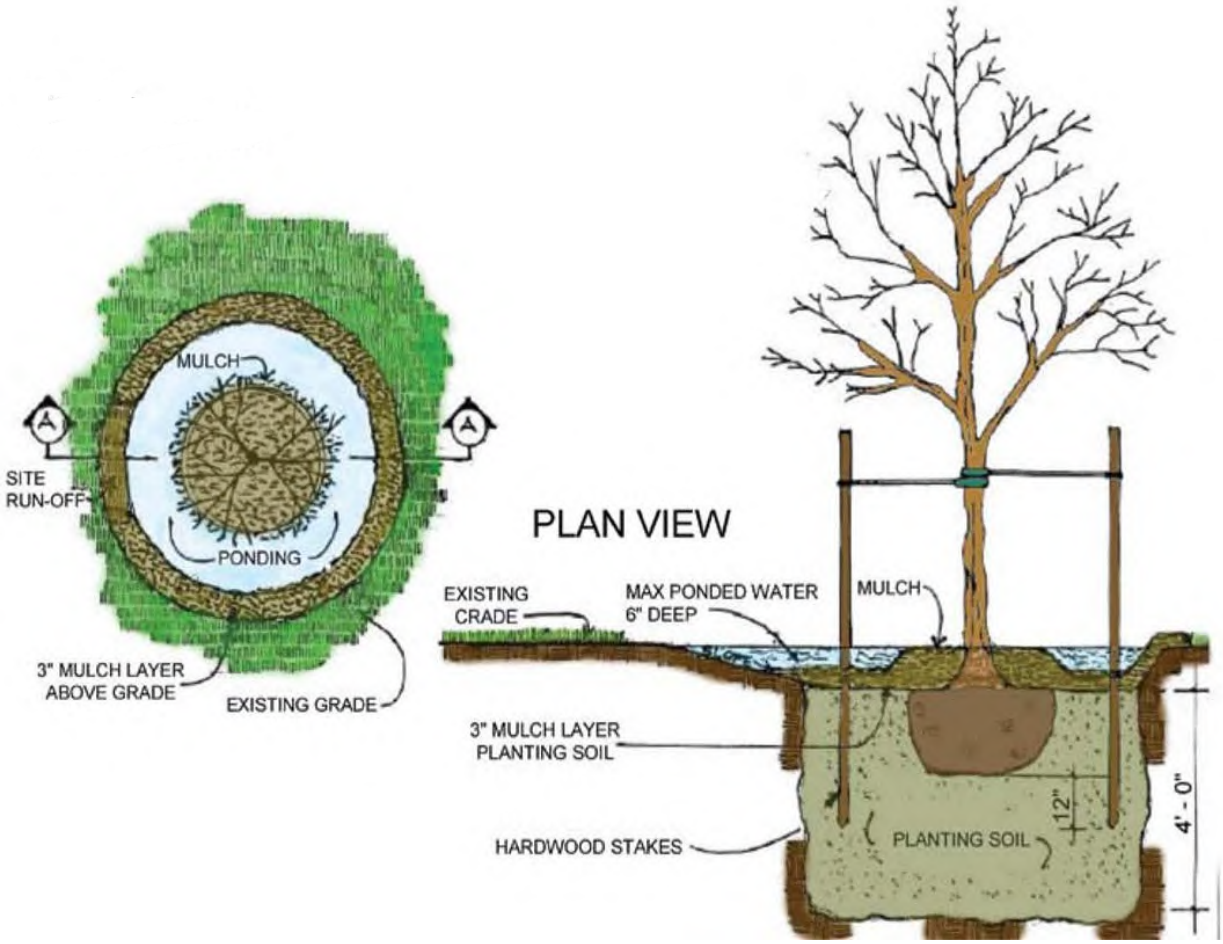


Figure 5 Tree planting detail

Roads and highways

Linear bioretention area feature may be used along a highway. Runoff is conveyed along the concrete curb until it reaches the end of the gutter, where it spills into the vegetated area.

Parking lot island bioretention

In parking lots for commercial, industrial, institutional, and other uses, stormwater management and green space areas are limited. In these situations, bioretention areas for stormwater management and landscaping may provide multiple benefits.

A bioretention area in a parking lot can occur in parking lots with no curbs and with curbs. The no-curb alternative allows stormwater to sheet flow over the parking lot directly into the bioretention area.

In a curbed parking lot, runoff enters the bioretention area through a curb cut. If the runoff volume exceeds the ponding depth available, water overflows the bioretention area and enters a standard inlet.

A variation on this design is a direct underground connection to the standard inlet from the underground aggregate infiltration bed via an overflow pipe.

Primary Components of a Bioretention System

1. Pretreatment (may be necessary to help prevent clogging)

- Sediment removal through a vegetated buffer strip, cleanout, stabilized inlet, water quality inlet, or sediment trap prior to runoff entry into the bioretention area

2. Flow inlet

- Varies with site use (e.g., parking island versus residential lot applications)
- Entering velocities must be non-erosive – use erosion control mats, blankets, or rock where concentrated runoff enters the bioretention area

3. Ponding area

- Provides temporary surface storage of runoff and allows sediment to settle
- Provides evaporation for a portion of runoff
- Depth no more than 6-18 inches for aesthetics, functionality, and safety

4. Plant material (see the Recommended Plant Lists Appendix)

- Absorbs stormwater through transpiration
- Root development creates pathways for infiltration
- Bacteria community resides in the root system creating healthy soil structure with water quality benefits
- Can improve aesthetics for site
- Provides habitat for animals and insects
- Reinforces long-term performance of subsurface infiltration
- Ensures plants are salt tolerant if in a location that would receive snowmelt chemicals
- Should be native plant species and placed according to drought and water tolerance
- Should be selected based on tolerance to standing water as identified in Recommended Plant List for BMPs Appendix

5. Organic layer or mulch

- Acts as a filter for pollutants in runoff
- Protects underlying soil from drying and eroding. Simulates leaf litter by providing environment for microorganisms to degrade organic material
- Provides a medium for biological growth, decomposition of organic material, adsorption and bonding of heavy metals
- Wood mulch should be shredded – compost or leaf mulch is preferred

6. Planting soil/volume storage bed

- Provides water/nutrients to plants
- Enhances biological activity and encourages root growth
- Provides storage of stormwater by the voids within the soil particles
- Provides surface for adsorption of nutrients

7. Positive overflow

- Provides for the direct discharge of runoff during large storm events when the subsurface/surface storage capacity is exceeded
- Examples of outlet controls include domed risers, inlet structures, and weirs

Design Considerations

Bioretention is flexible in design and can vary in complexity according to site conditions and runoff volume requirements. Design and installation procedures may vary from very simple for "backyard" rain gardens to highly engineered bioretention areas in ultra-urban areas.

Infiltration BMPs should be sited so that they minimize risk to groundwater quality and present no threat to subsurface structures. **Table 1** provides recommended setback distances of bioretention areas to various lot elements.

The distance from the bottom of the infiltration BMP to the seasonal high groundwater level or bedrock is recommended to be four feet. Two feet is allowable, but may reduce the performance of the BMP.

Setback from	Minimum distance (feet)
Property line	10
Building foundation*	10
Private well	50
Public water supply well**	50
Septic system drainfield***	100

* minimum with slopes directed away from building
** At least 200 feet from Type I or IIa wells, 75 feet from Type IIb and III wells
*** 50 feet for septic systems with a design flow of less than 1,000 gallons per day

Table 1 Recommended Setback Distances

Bioretention is best suited for areas with at least moderate infiltration rates (more than 0.25 inches per hour) – see Infiltration BMP. In extreme situations where permeability is less than 0.25 inches per hour, special variations may apply, such as using amended subsoils or double-walled underdrains (or using constructed wetlands instead). The following procedures should be considered when designing bioretention areas:

1. The **flow entrance** must be designed to prevent erosion in the bioretention area. Some alternatives include flared end sections, erosion control mats, sheet flow into the facility over grassed areas, rock at entrance to bioretention area, curb cuts with grading for sheet flow, and roof leaders with direct surface connection.

2. **A positive overflow system** should be designed to safely convey away excess runoff. The overflow can be routed to the surface in a non-erosive manner or to another stormwater system. Some alternatives include domed risers, inlet structures, weirs, and berms.

3. Sizing criteria
   a. **Surface area** is dependent upon storage volume requirements, but should generally not exceed a maximum loading ratio of 5:1 impervious drainage area to bioretention area and no more than one acre drainage area to one bioretention cell. However, for design purposes, the total volume of water generated from the contributing drainage area must be used, not just the impervious portion. See Infiltration BMP for additional guidance on loading ratios.

      The required bioretention surface area is determined by taking the volume of runoff to be controlled according to LID/GI criteria, and maintaining the maximum ponding depth, loading rate, and emptying time. Infiltration and evapotranspiration are increased by increasing the surface area of the bioretention area. The total surface area needed may be divided into multiple cells. This configuration may be useful to collect runoff from both the front and back of a building.

   b. Surface side slopes should be gradual. For most areas, a maximum of 3:1 side slopes are recommended.

   c. The recommended surface ponding depth is six inches. Up to 18 inches may be used if plant selection is adjusted to tolerate water depth. Drain within 24-48 hours.

   d. **Ponding area** should provide sufficient surface area to meet required storage volume without exceeding the design ponding depth. The subsurface infiltration

bed is used to supplement surface storage where appropriate.

4. **Planting soil depth** should generally be between 18 and 48 inches where only herbaceous plant species will be used. If trees and woody shrubs will be used, soil media depth may be increased, depending on plant species. Native soils can be used as planting soil or modified to be suitable on many sites. Small, backyard rain gardens can generally use existing soils without a specialized depth. Planting soil should be approximately four inches deeper than the bottom of the largest root ball.

5. **Planting soil** should be capable of supporting a healthy vegetative cover. Soils should be amended with a composted organic material. A recommended range of a soil mixture is 20-40 percent organic material (compost), 30-50 percent sand, and 20-30 percent topsoil, although any soil with sufficient drainage may be used for bioretention.

   Soils should also have a pH of between 5.5 and 6.5 (better pollutant adsorption and microbial activity), a clay content less than 10 percent (a small amount of clay is beneficial to adsorb pollutants and retain water although no clay is necessary if pollutant loadings are not an issue), be free of toxic substances and unwanted plant material, and have a 5-10 percent organic matter content. Additional organic matter can be added to the soil to increase water holding capacity.

   If brought from off site, **sand** should be clean, coarse, and conform to ASTM C-33 or AASHTO M-6 (Standard Specification for Concrete Aggregates).

   If the void space within an amended soil mix will be used in calculating runoff volume capacity in the system, tests should be conducted on the soil's porosity to determine the available storage capacity.

6. Proper **plant selection** is essential for bioretention areas to be effective. Typically, native floodplain or wet meadow plant species are best suited to the variable environmental conditions encountered in a bioretention area. Suggested species may include Cardinal Flower (*Lobelia cardinalis*), Blue Lobelia (*Lobelia siphilitica*), New England Aster (*Aster novae-angliae*), and Brown Fox Sedge (*Carex vulpinoidea*) (See Recommended Plant Lists Appendix for a detailed list).

   In most cases, seed is not the preferred method for establishing plants in a bioretention area. The fluctuating water levels make it difficult for the seed to readily establish, while the random nature of seeding produces a look which previous experience indicates is unacceptably "wild." Therefore, it is strongly recommended that live plant material in plug or gallon-potted form be used, and installed on 1-2 foot centers for a more formal appearance. Shrubs and trees are also recommended to be included in a bioretention area. Plant material should be selected based on tolerance to standing water as identified in Recommended Plant List for BMPs Appendix.

7. **Planting periods** will vary but, in general, trees and shrubs should be planted from mid-April through early June, or mid-September through mid-November. Native seed should be installed between October 1 and June 1. Live plant material (plugs or gallon pots) should be installed between May 1 and June 15. Planting dates may be lengthened if a regular water source can be provided. Likewise, planting should be ceased at an earlier date in the event of a drought year.

8. A maximum of 2-3 inches of shredded hardwood **mulch,** aged at least six months to one year, or leaf compost (or other comparable product) should be uniformly applied immediately after shrubs and trees are planted to prevent erosion, enhance metal removals, and simulate leaf litter in a natural forest system. Wood chips should be avoided as they tend to float during inundation periods. In order to maintain oxygen flow, mulch or compost should not exceed three

inches in depth or be placed directly against the stems or trunks of plants.

9. When working in areas with **steeper slopes**, bioretention areas should be terraced laterally along slope contours to minimize earthwork and provide level areas for infiltration.

10. A subsurface **storage/infiltration bed**, if used, should be at least six inches deep and constructed of clean gravel with a significant void space for runoff storage (typically 40 percent) and wrapped in geotextile fabric.

11. **Underdrains** are often not needed unless in-situ soils are expected to cause ponding lasting longer than 48 hours. If used, underdrains are typically small diameter (6-12 inches) perforated pipes in a clean gravel trench wrapped in geotextile fabric (or in the storage/infiltration bed). Underdrains should have a flow capacity greater than the total planting soil infiltration rate and should have at least 18 inches of soil/gravel cover. They can daylight to the surface or connect to another stormwater system. A method to inspect and clean underdrains should be provided (via cleanouts, inlet, overflow structure, etc.)

Stormwater Functions and Calculations

When designing a bioretention area, it is recommended to follow a two-step process:

1. Initial sizing of the bioretention area based on the principles of Darcy's Law.
2. Verify that the loading ratio and the necessary volume reductions are being met.

Initial sizing of the bioretention area

Bioretention areas can be sized based on the principles of Darcy's Law, as follows:

With an underdrain:

$$A_f = V * d_f / [k * (h_f + d_f) * t_f]$$

Without an underdrain:

$$A_f = V * d_f / [i * (h_f + d_f) * t_f]$$

Where:

A_f = surface area of filter bed (ft^2)
V = required storage volume (ft^3)
d_f = filter bed depth (ft)
k = coefficient of permeability of filter media (ft/day)
i = infiltration rate of underlying soils (ft/day)
h_f = average height of water above filter bed (ft)
t_f = design filter bed drain time (days)

A "quick check" for sizing the bioretention area is to ignore the infiltration rate and calculate the storage volume capacity of the bioretention area as follows:

A_{inf} = (Area of bioretention area at ponding depth + Bottom area of bioretention area) divided by two = Infiltration area (average area)

The size of the infiltration area is determined by the volume of water necessary to remove as determined by LID/GI criteria, depth of the ponded area (not to exceed 18 inches), infiltration rate of the soil, loading ratio, and, if applicable, any subsurface storage in the amended soil or gravel.

This volume can be considered removed if the bioretention is not underdrained. If the bioretention cell is underdrained, consider the bioretention cell as a detention device with the volume calculated above discharged to a surface water over time t_f.

Verification of meeting volume reduction requirements

The bioretention facility should be sized to accommodate the desired volume reductions towards the required Channel Protection Volume or the required Water Quality Volume, as appropriate.

The volume of a bioretention area can have three components: surface storage volume, soil storage volume, and infiltration bed volume. These three components should be calculated separately and added together. The goal is that this total volume is larger than the required Channel Protection Volume or the Water Quality Volume, as appropriate. If the total volume is less than the required volume, another adjustment may be needed to the bioretention area (e.g., increased filter bed depth).

Total volume calculation:

1. Surface storage volume (ft^3) = Average bed area (ft^2) * Maximum design water depth (ft)
2. Soil storage volume (ft^3) = Infiltration area (ft^2) * Depth of amended soil (ft) * Void ratio of amended soil.
3. Subsurface storage/Infiltration bed volume (ft^3) = Infiltration area (ft^2) * Depth of underdrain material (ft) * Void ratio of storage material

Total bioretention volume = Surface storage volume + Soil storage volume (if applicable) + Infiltration bed volume (if applicable).

Peak rate mitigation

If an underdrain is required, the bioretention also acts as a detention practice with a discharge rate roughly equal to the infiltration rate of the soil multiplied by the average bed area. The discharge rate should be counted towards the site's allowable release rate (if not leading to the site's detention pond) or as inflow to the site's detention pond (if an underdrain would eventually empty into the site's detention pond).

Water Quality Improvement

The reported water quality benefits of bioretention can be expected to remove a high amount of total suspended solids (typically 70-90 percent), a medium amount of total phosphorus (approximately 60 percent), and a medium amount of total nitrogen (often 40-50 percent). In areas with high sediment loading, pretreatment of runoff can significantly reduce the amount of bioretention maintenance required.

Construction Guidelines

The following is a typical construction sequence (Note for all construction steps: Erosion and sediment control methods need to adhere to the construction BMP requirements contained in the Technical Standards document and the latest requirements of IDEM's Soil Erosion and Sedimentation Control (Rule 5) Program).

1. Complete site grading, minimizing compaction as much as possible. If applicable, construct curb cuts or other inflow entrance, but provide protection so that drainage is prohibited from entering the bioretention construction area. Construct pre-treatment devices (filter strips, swales, etc.) if applicable.

2. Subgrade preparation

   a. Existing subgrade in rain gardens should <u>not</u> be compacted or subject to excessive construction equipment traffic. Loads on the subgrade should not exceed four pounds per square inch.
   b. Initial excavation can be performed during rough site grading, but should not be carried to within one foot of the final bottom elevation. Final excavation should not take place until all disturbed areas in the drainage area have been stabilized.
   c. Where erosion of subgrade has caused accumulation of fine materials and/or surface ponding in the graded bottom, this material should be removed with light equipment and the underlying soils scarified to a minimum depth of six inches with a york rake or equivalent by light tractor.
   d. Bring subgrade of bioretention area to line, grade, and elevations indicated. Fill and lightly regrade any areas damaged by erosion, ponding, or traffic compaction. All bioretention areas should be level grade on the bottom.

3. Stabilize grading except within the bioretention area. Bioretention areas may be used as temporary sediment traps, provided the proposed finish elevation of the bed is at least 12 inches lower than the bottom elevation of the sediment trap (if used as such, all accumulated material and at least 12 inches of soil should be removed).

4. Excavate bioretention area to proposed invert depth and scarify the existing soil surfaces. Do not compact soils.

5. Backfill bioretention area with amended soil as shown on plans and specifications.

Overfilling is recommended to account for settling. Light hand tamping is acceptable if necessary.

6. Complete final grading to achieve proposed design elevations, leaving space for upper layer of compost, mulch, or topsoil as specified on plans.

7. Bioretention area/rain garden installation

   a. Upon completing subgrade work, notify the engineer to inspect at his/her discretion before proceeding with bioretention installation.
   b. For the subsurface storage/infiltration bed installation, amended soils should be placed on the bottom to the specified depth.
   c. Planting soil should be placed immediately after approval of subgrade preparation/bed installation. Any accumulation of debris or sediment that takes place after approval of subgrade should be removed prior to installation of planting soil at no extra cost to the owner.
   d. If called for in the design, install approved planting soil in 18-inch maximum lifts and lightly compact (tamp with backhoe bucket or by hand). Keep equipment movement over planting soil to a minimum – **do not over-compact**. Install planting soil to grades indicated on the drawings. Loads on the soil should not exceed four pounds per square inch.
   e. Presoak the planting soil at least 24 hours prior to planting vegetation to aid in settlement.
   f. Plant trees and shrubs according to supplier's recommendations and only from mid-March through the end of June or from mid-September through mid-November.
   g. Install two or three inches of shredded hardwood mulch (minimum age six months) or compost mulch evenly as shown on plans. Do not apply mulch in areas where ground cover is to be grass or where cover will be established by seeding.
   h. Protect rain gardens from sediment at all times during construction. Compost socks, diversion berms, and/or other appropriate measures should be used at the toe of slopes that are adjacent to rain gardens to prevent sediment from washing into these areas during site development.
   i. When the site is fully vegetated and the soil mantle stabilized, notify the plan designer to inspect the rain garden drainage area at his/her discretion before the area is brought online and sediment control devices removed.

8. Mulch and install erosion protection at surface flow entrances where necessary.

Maintenance

Properly designed and installed bioretention areas require some regular maintenance, most within the first year or two of establishment. Less maintenance is required when the native perennial vegetation becomes established.

1. Water vegetation at the end of each day for two weeks after planting is completed. Newly established plants should continue to receive approximately one inch of water per week throughout the first season, or as determined by the landscape architect.

2. While vegetation is being established, pruning and weeding may be required. Weeds should be removed by hand.

3. Organic material may also need to be removed approximately twice per year (typically by hand).

4. Perennial plantings may be cut down at the end of the growing season to enhance root establishment.

5. Mulch should be re-spread when erosion is evident and replenished once every one to two years or until the plants begin to fill in the area and the space between plants is minimized.

6. Bioretention area should be inspected at least two times per year for sediment buildup, erosion, and to evaluate the health of the vegetation. If sediment buildup reaches 25

percent of the ponding depth, it should be removed. If erosion is noticed within the bioretention area, additional soil stabilization measures should be applied. If vegetation appears to be in poor health with no obvious cause, a landscape specialist should be consulted.

7. Bioretention vegetation may require watering, especially during the first year of planting. Ensure the maintenance plan includes a watering schedule for the first year, and in times of extreme drought after plants have been established.

8. Bioretention areas should not be mowed on a regular basis. Trim vegetation as necessary to maintain healthy plant growth.

Winter Considerations

Use salt-tolerant vegetation where significant snow-melt containing deicing chemicals is expected. The use of sand, cinders, and other winter abrasives should be minimized. If abrasives are used, additional maintenance may be required to remove them in the spring. Bioretention soils can be expected to resist freezing and remain functioning for most of the year (although biological pollutant removal processes will be reduced during winter).

Planting Tip

When planting your bioretention area, it is usually helpful to mark the different planting areas. An effective method is using spray paint and flags to mark designated areas. This is especially helpful when utilizing volunteers.

Bioretention areas can even be used for snow storage assuming this will not harm the vegetation. Pipes, inlets, overflow devices, and other stormwa-ter structures associated with bioretention should be designed according to general guidance on cold climate construction.

Cost

Bioretention areas often replace areas that were intensively landscaped and require high maintenance. In addition, bioretention areas can decrease the cost for stormwater conveyance systems on a site. Bioretention areas cost approximately $5-7 per cubic foot of storage to construct.

Designer/Reviewer Checklist for Rain Gardens/Bioretention

Item	Page No.	Yes	No	N/A	Notes
Was Soil infiltration Testing Protocol Appendix followed?*					
Appropriate areas of the site evaluated?	--				
Infiltration rates measured? ➢ *Moderate infiltration rates: > 0.25 inches/hour. If < 0.25 inches/hour, special variations may apply – such as using amended subsoils or underdrains.*	8				
Were the bioretention design guidelines followed?					
Minimum 2-foot separation between the bed bottom and bedrock/ SHWT?	8				
Soil permeability acceptable?	8				
If not, appropriate underdrain provided? ➢ *6-12 inches in diameter; perforated pipes*	10				
Natural, uncompacted soils?	11				
Level infiltration area (bed bottom)?	11				
Excavation in rain garden areas minimized?	--				
Hotspots/pretreatment considered?	1				
Loading ratio below 5:1 (described in infiltration BMP)?	8				
Ponding depth limited to 18 inches?	1,7,8				
Drawdown time less than 48 hours?	1				
Positive overflow from system?	7,8				
Erosion and Sedimentation control?	12				
Feasible construction process and sequence?	11,12				
Entering flow velocities non-erosive or erosion control devices?	7,8				
Acceptable planting soil specified? ➢ *20-40% organic material (compost), 30-50% sand, 20-30% topsoil)*	9				

* In general, the protocol should be followed as much as possible.

➢ *Denotes Minimum Design Considerations*

References

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Minnesota Stormwater Manual, 2006. St. Paul, MN: Minnesota Pollution Control Agency, 2006.

Pennsylvania Stormwater Best Management Practices Manual, 2006. Pennsylvania Department of Environmental Protection. 2006.

Prince George's County Bioretention Manual, 2002. Prince George's County, MD: Department of Environmental Resources, 2002.

Rain Gardens of West Michigan: www.raingardens.org Southeastern Oakland County Water Authority: www.socwa.org/lawn_and_garden.htm *Rain Gardens: A household way to improve water quality in your community*. University of Wisconsin-Extension and Wisconsin Department of Natural Resources, 2002.

Wild Ones Natural Landscapers: www.for-wild.org/

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BMP Fact Sheet

DETENTION BASINS - CONSTRUCTED WETLAND

A constructed wetland is a type of detention basin that is developed as shallow marsh system planted with emergent vegetation designed to treat stormwater runoff.



Figure 1 Photo Courtesy of USDA NRCS

Variations

- Shallow Wetlands
- Extended Detention Shallow Wetlands
- Pocket Wetlands
- Pond/Wetland

Key Design Features

- Storage capacity highly dependent on available site area
- Outlet structure configuration determines peak rate reduction effectiveness
- Regular maintenance of vegetation and sediment removal required
- Natural high groundwater table required
- Relatively impermeable soils or impermeable liner
- Forebay for sediment collection and removal
- Dewatering mechanism required
- Stabilized emergency overflow and energy dissipation at all outlets

Applications	
Residential	Yes
Commercial	Yes
Ultra Urban	No
Industrial	Yes
Retrofit	Yes
Highway/Road	Yes
Recreational	Yes

Stormwater Quantity Functions	
Volume	Low
Groundwater Recharge	None or Low
Peak Rate	High

Stormwater Quality Functions
Varies by type as follows:

Type	TSS	TP	TN	Temperature
Constructed Wetland	High	Medium	Medium	Low/Medium

Site Factors

Type	Basin Bottom Relative to Water Table	Soils	Slope	Potential Hotspots	Max. Drainage Area (acres)	Benefits	Limitations
Const. Wetland	Can be below WT	C or D*	Low	Yes w/ consider-ations	50	Good peak rate & water quality performance, wide applicability, potential aesthetic/ habitat value	Limited volume/GW recharge benefits, high total cost, potentially thermal impact

**C or D soils typically work without modification. A and B soils may require modifications to reduce their permeability.*

Additional Considerations

Cost

- High – Cost must include excavation of basin and enhanced vegetation.
- The cost of each basin is highly dependent on the size of the basin and site characteristics.

Maintenance

Type	Maintenance
Constructed Wetland	Low/Med

Winter Performance

- Med/High

Description and Function

Constructed wetlands are shallow marsh systems planted with emergent vegetation designed to treat stormwater runoff. While they are one of the best BMPs for pollutant removal, constructed wetlands can also mitigate peak rates and even reduce runoff volume to a certain degree. They also can provide considerable aesthetic and wildlife benefits. Constructed wetlands use a relatively large amount of space and may require an adequate source of inflow if a permanent water surface is maintained. (Not all constructed wetlands maintain a water surface year round).

Applications

Constructed wetlands can be used in a wide variety of applications when the necessary space is available. Their use is limited in ultra urban areas and some redevelopment projects simply due to a lack of available space (in these cases underground and/or special detention may be used).

Variations

Constructed wetlands can be designed as either online (within the stormwater system) or offline facilities. They can be used effectively in series with other flow/ sediment reducing BMPs that reduce the sediment load and equalize incoming flows to the constructed wetland. They are a good option for retrofitting existing detention basins and are often organized into the following four groups:

- **Shallow wetlands** are large surface area constructed wetlands that primarily accomplish water quality improvement through displacement of the permanent pool.
- **Extended detention shallow wetlands** are similar to shallow wetlands but use extended detention as another mechanism for water quality and peak rate control.
- **Pocket wetlands** are smaller constructed wetlands that serve drainage areas between approximately 5 and 10 acres and are constructed near the water table.
- **Pond/wetland** systems are a combination of wet ponds and constructed wetlands.

Although discussion of constructed wetlands in this BMP focuses on surface flow as described above, subsurface flow constructed wetlands can also be used to treat stormwater runoff.

While typically used for wastewater treatment, subsurface flow constructed wetlands for

stormwater can offer some advantages over surface flow wetlands, such as improved reduction of total suspended solids and biological oxygen demand. They also can reduce the risk of disease vectors (especially mosquitoes) and safety risks associated with open water. However, nitrogen removal may be deficient (Campbell and Ogden, 1999) if most of the incoming nitrogen is in the form of ammonia. Subsurface flow wetlands are poor converters of ammonia to nitrate (nitrification) but are excellent converters of nitrate to nitrogen gas (denitrification). Perhaps the biggest concern regarding subsurface constructed wetlands is their relatively high cost. They can be two to three times more expensive to construct than surface flow constructed wetlands.

Design Considerations

Hydrology

- Constructed wetlands should be designed to mitigate peak runoff rates for the one-year through 100-year rainfall events.
- Inflow and discharge hydrographs should be calculated for each selected design storm. Hydrographs should be based on the 24-hour rainfall event or that required in the local Stormwater Ordinance. Typically, the NRCS 24-hour Type II rainfall distribution should be utilized to generate hydrographs.
- Constructed wetlands must be able to receive and retain enough flow from rain, runoff, and groundwater to ensure long-term viability. Hydrologic calculations (e.g., a water balance) must be performed to verify this. Shallow marsh areas can become dry at the surface but not for greater than one month, even in the most severe drought. A permanent water surface in the deeper areas of the constructed wetland should be maintained during all but the driest periods. The average target pool depth to maintain emergent wetland vegetation is six to 12 inches. Maximum water depths of three to four feet should not be exceeded for more than 12 hours at a time, for more than a few days out of the year. The deeper the water and the longer it sits, the greater the chances that a wetland vegetation monoculture, such as cattails, will develop. A relatively stable normal water surface elevation reduces the stress on wetland vegetation. A constructed wetland must have a drainage area of at least 10 acres (five acres for “pocket” wetlands) or some means of sustaining constant inflow. Even with a large drainage area, a consaarea, a constant source of inflow can improve the biological health and effectiveness of a constructed wetland. Indiana’s precipitation is generally well distributed throughout the year and is therefore suited for constructed wetlands.

Storage volume, depth, and duration

- Constructed wetlands should be designed to treat the runoff volume produced by the water quality design storm unless additional upstream BMPs are provided.
- If high water table conditions are anticipated, then the design of a constructed wetland should be considered.
- Ponding depths should not exceed 3 – 4 feet for more than 12 hours.
- Detention time is defined as the time from when the maximum storage volume is reached until only 10 percent of that volume remains in the basin. In order to achieve an 80 percent total suspended solids removal rate, a 36-hour detention time is required within an extended detention basin, with no more than 40% of the maximum stored volume released within the first 12 hours.

Basin sizing and configuration

- Constructed wetlands should be shaped to maximize the hydraulic length of the stormwater flow pathway. A minimum length-to width ratio of 3:1 is recommended to maximize sedimentation. If the length-to-width ratio is lower, the flow pathway should be maximized. A wedge-shaped pond with the major inflows on the narrow end can prevent short-circuiting and stagnation.
- Irregularly shaped basins are acceptable and may even be encouraged to improve site aesthetics.
- Distances of flow paths from inflow points to outlets should be maximized.
- If site conditions inhibit construction of a long, narrow basin, baffles consisting of

earthen berms or other materials can be incorporated into the pond design to lengthen the stormwater flow path.
- Constructed wetlands must have one or more sediment forebays or equivalent upstream pretreatment to trap coarse sediment, prevent short circuiting and facilitate maintenance (i.e., sediment removal). The forebay should consist of a separate cell, formed by a structural barrier. The forebay will require periodic sediment removal.
- Permanent access must be provided to the forebay, outlet, and embankment areas. It should be at least nine feet wide, having a maximum slope of 15 percent, and be stabilized for vehicles.
- An emergency outlet or spillway capable of conveying the spillway design flood (SDF) must be included in the design. The SDF is usually equal to the 1.25 times the 100-year design flood.
- Constructed wetlands should be designed so that the 10-year water surface elevation does not exceed the normal water surface elevation by more than three feet. Slopes in and around constructed wetlands should be 4:1 to 5:1 (horizontal:vertical) whenever possible.
- All areas that are deeper than four feet should have two safety benches, each four to six feet wide. One should be situated about one to 1.5 feet above the normal water elevation and the other 2 to 2.5 feet below the water surface.

Embankments

- Vegetated embankments less than or equal to three feet in height are recommended. Embankments should have side slopes no steeper than 3:1 (horizontal to vertical).
- The constructed wetland should have a minimum freeboard of one foot above the SDF elevation to the top of the berm.
- Woody vegetation is generally discouraged in the embankment area because of the risk of compromising the integrity of the embankment.
- Embankments should incorporate measures, such as buried chain link fencing, to prevent or discourage damage caused by tunneling wildlife (e.g., muskrat).

Constructed wetland location

- Constructed wetlands should be located down gradient of disturbed or developed areas on the site. The constructed wetland should collect as much site runoff as possible, especially from the site's impervious surfaces (roads, parking, buildings, etc.), and where other BMPs are not proposed.
- Constructed wetlands should not be constructed on steep slopes, nor should slopes be significantly altered or modified to reduce the steepness of the existing slope, for the purpose of installing a basin.
- Constructed wetlands should not worsen the runoff potential of the existing site by removing trees for the purpose of installing a basin.
- Constructed wetlands should not be constructed within 10 feet of the property line or within 50 feet of a private well or septic system.
- Constructed wetlands should not be constructed in areas with high quality and/or well draining soils, which are adequate for installing BMPs capable of achieving stormwater infiltration and, hence, volume reduction.

Outlet design

- The low-flow orifice should typically be no smaller than 4 inches in diameter. However, the orifice diameter may be reduced to two inches if adequate protection from clogging is provided.
- The hydraulic design of all outlet structures must consider any significant tailwater effects of downstream waterways.
- The primary and low flow outlets should be protected from clogging by an external trash rack or other mechanism.
- Online facilities should have an emergency spillway that can safely pass 1.25 times the 100-year storm with one foot of freeboard. All outflows should be conveyed downstream in a safe and stable manner.
- Outlet control devices should be in open water areas, four to six feet deep, comprising about five percent of the total surface area to prevent clogging and allow the wetland to be drained for maintenance. Outlet devices are

generally multistage structures with pipes, orifices, or weirs for flow control. All outflows should be conveyed downstream in a safe and stable manner.

Inlet structures

- Erosion protection measures should be used to stabilize inflow structures and channels.

Sediment forebay

- Forebays must be in compliance with local Stormwater Management Ordinance and standard detailed drawing requirements.
- Forebays must be incorporated into the basin design. Forebays should be provided at all major inflow points to capture coarse sediment, prevent excessive sediment accumulation in the main basin, and minimize erosion by inflow.
- Forebays should be vegetated to improve filtering of runoff, to reduce runoff velocity, and to stabilize soils against erosion. Forebays should adhere to the following criteria:
  - A minimum length of 10 feet.
  - Storage should be provided to trap sediment over from storms with return periods between 1and 10 years.
  - Forebays should be physically separated from the rest of the pond by a berm, gabion wall, etc.
  - Flows exiting the forebay must be non-erosive to the newly constructed basin.
  - Forebays should be installed with permanent vertical markers that indicate sediment depth.
  - Storage volume of 10 to 15 percent of the total permanent pool volume and is four to six feet deep.
  - All major inflow points to dry detention basins should include sediment forebays sized for 10 percent of the water quality volume.

Vegetation and soils protection

- Underlying soils must be identified and tested. Generally, hydrologic soil groups “C” and “D” are suitable without modification; “A” and “B” soils may require a clay or synthetic liner. Soil permeability must be tested in the proposed constructed wetland location to ensure that excessive infiltration will not cause it to dry out. Field results for permeability should be used in the water balance calculations to confirm suitability. If necessary, constructed wetlands should have highly compacted subsoil or an impermeable liner to minimize infiltration.
- Organic soils should be used for constructed wetlands. Organic soils can serve as a sink for pollutants and generally have high water holding capacities. They will also facilitate plant growth and propagation and may hinder invasion of undesirable species. Care must be taken to ensure that soils used are free of invasive or nuisance plant seed.
- About half of the emergent vegetation zone should be high marsh (up to six inches deep) and half should be low marsh (6 to 18 inches deep). Varying depths throughout the constructed wetland can improve plant diversity and health.
- The open water zone (approx.imately 35 to 40 percent of the total surface area) should be between 18 inches and six feet deep. Allowing a limited five-foot deep area can prevent short-circuiting by encouraging mixing, enhance aeration of water, prevent resuspension, minimize thermal impacts, and limit mosquito growth. Alternating areas of emergent vegetation zone (up to 18 inches deep) and open water zone can also minimize short-circuiting and hinder mosquito propagation.

General Specifications

The following specifications are provided for information purposes only. These specifications include information on acceptable materials for typical applications, but are by no means exclusive or limiting. The designer is responsible for developing detailed specifications for individual design projects in accordance with the project conditions.

Excavation

- The area to be used for the constructed wetland should be excavated to the required depth below the desired bottom elevation to

accommodate any required impermeable liner, organic matter, and/or planting soil.
- The compaction of the subgrade and/or the installation of any impermeable liners will follow immediately.

Subsoil preparation

- Subsoil should be free from hard clods, stiff clay, hardpan, ashes, slag, construction debris, petroleum hydrocarbons, or other undesirable material. Subsoil must not be delivered in a frozen or muddy state.
- Scarify the subsoil to a depth of 8 to 10 inches with a disk, rototiller, or similar equipment.
- Roll the subsoil under optimum moisture conditions to a dense seal layer with four to six passes of a sheepsfoot roller or equivalent. The compacted seal layer should be at least eight inches thick.

Impermeable liner

- If necessary, install impermeable liner in accordance with manufacturer's guidelines.
- Place a minimum 12 inches of subsoil on top of impermeable liner in addition to planting soil.

Earth fill material & placement

- The fill material should be taken from approved designated excavation areas. It should be free of roots, stumps, wood, rubbish, stones greater than six inches, or other objectionable materials. Materials on the outer surface of the embankment must have the capability to support vegetation.
- Areas where fill is to be placed should be scarified prior to placement. Fill materials for the embankment should be placed in maximum eight-inch lifts. The principal spillway must be installed concurrently with fill placement and not excavated into the embankment.
- Control movement of the hauling and spreading equipment over the site.

Embankment core

- The core should be parallel to the centerline of the embankment as shown on the plans. The top width of the core should be at least four feet. The height should extend up to at least the 10-year water elevation or as shown on the plans. The side slopes should be 1:1 or flatter. The core should be compacted with construction equipment, rollers, or hand tampers to assure maximum density and minimum permeability. The core should be placed concurrently with the outer shell of the embankment.
- Construction of the berm should follow specifications by the project's geotechnical engineer.

Structure backfill

- Backfill adjacent to pipes and structures should be of the type and quality conforming to that specified for the adjoining fill material. The fill should be placed in horizontal layers not to exceed eight inches in thickness and compacted by hand tampers or other manually directed compaction equipment. The material should fill completely all spaces under and adjacent to the pipe. At no time during the backfilling operation should driven equipment be allowed to operate closer than four feet to any part of the structure. Equipment should not be driven over any part of a concrete structure or pipe, unless there is a compacted fill of 24 inches or greater over the structure or pipe.
- Backfill content and placement should follow specifications by the project's geotechnical engineer.

Pipe conduits

Corrugated metal pipe – All of the following criteria should apply for corrugated metal pipe:

- Materials – Polymer coated steel pipe, aluminum coated steel pipe, aluminum pipe. This pipe and its appurtenances should conform to the requirements of AASHTO specifications with watertight coupling bands or flanges.

- Coupling bands, anti-seep collars, end sections, etc., must be composed of the same material and coatings as the pipe. Metals must be insulated from dissimilar materials with use of rubber or plastic insulating materials at least 24 mils in thickness.
- Connections – All connections with pipes must be completely watertight. The drain pipe or barrel connection to the riser should be welded all around when the pipe and riser are metal. Anti-seep collars should be connected to the pipe in such a manner as to be completely watertight. Dimple bands are not considered to be watertight.
- Bedding – The pipe should be firmly and uniformly bedded throughout its entire length. Where rock or soft, spongy or other unstable soil is encountered, all such material should be removed and replaced with suitable earth compacted to provide adequate support.
- Backfilling should conform to "structure backfill."
- Other details (anti-seep collars, valves, etc.) should be as shown on drawings.

Reinforced concrete pipe - All of the following criteria should apply for reinforced concrete pipe:

- Materials – Reinforced concrete pipe should have bell and spigot joints with rubber gaskets and should equal or exceed ASTM standards.
- Laying pipe – Bell and spigot pipe should be placed with the bell end upstream. Joints should be made in accordance with recommendations of the manufacturer of the material. After the joints are sealed for the entire line, the bedding should be placed so that all spaces under the pipe are filled. Take care to prevent any deviation from the original line and grade of the pipe.
- Backfilling should conform to "structure backfill."
- Other details (anti-seep collars, valves, etc.) should be as shown on drawings.

Plastic pipe

- Materials – PVC pipe should be PVC-1120 or PVC-1220, conforming to ASTM standards. Corrugated High Density Polyethylene (HDPE) pipe, couplings, and fittings should meet AASHTO specifications.
- Joints and connections to anti-seep collars should be completely watertight.
- Bedding – The pipe should be firmly and uniformly bedded throughout its entire length. Where rock or soft, spongy or other unstable soil is encountered, all such material should be removed and replaced with suitable earth compacted to provide adequate support.
- Backfilling should conform to "structure backfill."
- Other details (anti-seep collars, valves, etc.) should be as shown on drawings.

Drainage diaphragms – When a drainage diaphragm is used, a registered professional engineer must supervise the design and construction inspection.

Planting soil (topsoil)

- See local specifications for general planting soil requirements.
- Use a minimum of 12 inches of topsoil in the emergent vegetation zone (less than 18" inches deep) of the pond. If natural topsoil from the site is to be used it must have at least eight percent organic carbon content (by weight) in the A-horizon for sandy soils, and 12 percent for other soil types.
- If planting soil is imported, it should be made up of equivalent proportions of organic and mineral materials. All soils used should be free of invasive or nuisance seeds.
- Lime should not be added to planting soil unless absolutely necessary, as it may encourage the propagation of invasive species.
- The final elevations and hydrology of the vegetative zones should be evaluated prior to planting to determine if grading or planting changes are required.

Vegetation

- See Recommended Plant List for BMPs Appendix for plant lists for constructed wetlands. Substitutions of specified plants should be subject to prior approval of the designer. Planting locations should be based on the planting plan and directed in the field

by a qualified wetland ecologist. Plant material should be selected based on tolerance to standing water as identified in Recommended Plant List for BMPs Appendix.

- All wetland plant stock should exhibit live buds or shoots. All plant stock should be turgid, firm, and resilient. Internodes of rhizomes may be flexible and not necessarily rigid. Soft or mushy stock should be rejected. The stock should be free of deleterious insect infestation, disease, and defects such as knots, sun-scald, injuries, abrasions, or disfigurement that could adversely affect the survival or performance of the plants.
- All stock should be free from invasive or nuisance plants or seeds.
- During all phases of the work, including transport and onsite handling, the plant materials should be carefully handled and packed to prevent injuries and desiccation. During transit and onsite handling, the plant material should be kept from freezing and be covered, moist, cool, out of the weather, and out of the wind and sun. Plants should be watered to maintain moist soil and/or plant conditions until accepted.
- Plants not meeting these specifications or damaged during handling, loading, and unloading will be rejected.

Outlet control structure

- Outlet control structures should be constructed of non-corrodible material.
- Outlets should be resistant to clogging by debris, sediment, floatables, plant material, or ice.
- Materials should comply with applicable specifications (INDOT or AASHTO, latest edition).
- For maximum flexibility with wetland water levels (if actual depths are uncertain) adjustable water level control structures are recommended (see EPA, 2000 in reference section for design concepts).

Stormwater Functions and Calculations

Volume reduction

Although not typically considered a volume-reducing BMP, constructed wetlands can achieve some volume reduction through infiltration and evapotranspiration, especially during small storms and high temperature periods. The volume stored between the predicted water level (from the water balance calculations) and the lowest outlet elevation will be removed from the storm that occurs under those conditions.

Peak rate mitigation

Inflow and discharge hydrographs must be calculated for each design storm.

Peak rate is primarily controlled in detention facilities through the transient storage above any permanent water surface. The degree to which peak rate is controlled is a function of the transient storage volume provided (i.e., depth and area) and the configuration of the outlet control structure.

Water quality improvement

Constructed wetlands rely on physical, biological, and chemical processes to remove pollutants from influent stormwater runoff. The primary treatment mechanism is settling by gravity of particulates and their associated pollutants, while stormwater is retained in the pond. Another mechanism for the removal of pollutants, especially nutrients, is uptake by algae and aquatic vegetation.

The longer the runoff remains in a constructed wetland, the more settling (and associated pollutant removal) and other treatment can occur. After the particulates reach the bottom, the permanent pool protects them from resuspension when additional runoff enters.

Construction Guidelines

The following guidelines pertain to constructed wetlands:

- Install all temporary erosion and sedimentation controls.
- Separate pond area from contributing drainage area.
- All channels/pipes conveying flows to the pond must be routed away from the pond area until it is completed and stabilized.
- Prior to construction of the pond, the area immediately adjacent to the pond must be stabilized in accordance with the erosion and sediment control methods discussed in the Erosion and Sediment Control Requirements Chapter of the Technical Standards and the latest requirements of IDEM's Soil Erosion and Sedimentation Control (Rule 5) Program.
- Prepare site for excavation and/or embankment construction.
- All existing vegetation should remain, if feasible, and only be removed if necessary for construction.
- Care should be taken to prevent compaction of the basin bottom.
- If excavation is required, clear the area of all vegetation. Remove all tree roots, rocks, and boulders only in the excavation area.
  - Excavate bottom of basin to desired elevation (if necessary).
- Install surrounding embankments and inlet and outlet control structures.
- Grade and prepare subsoil in bottom of basin. Compact bottom of basin in constructed wetlands.
  - Apply and grade planting soil. Matching design grades is crucial especially in constructed wetlands because aquatic plants can be very sensitive to depth.
  - Apply geo-textiles and other erosion-control measures.
- Seed, plant, and mulch according to landscaping plan.
- Install any safety or anti-grazing measures, if necessary.
- Follow required maintenance and monitoring guidelines.

Maintenance

Constructed wetlands must have a maintenance plan and privately owned facilities should have an easement, deed restriction, or other legal measure to prevent neglect or removal.

A basin maintenance plan should be developed which includes the following measures:

- All basin structures should be inspected for clogging and excessive debris and sediment accumulation at least four times per year, as well as after every storm greater than one inch. Structures that should be inspected include basin bottoms, trash racks, outlets structures, riprap or gabion structures, and inlets.
- Sediment should be removed from the forebay before it occupies 50 percent of the forebay, typically every 3 to 10 years. Sediment removal should be conducted when the basin is completely dry.
- Constructed wetlands should be drained prior to sediment removal. Sediment should be disposed of properly and once sediment is removed, disturbed areas need to be immediately stabilized and re-vegetated. Proper disposal of removed material depends on the nature of the drainage area and the intent and function of the detention basin. Material removed from detention basins that treat hot spots such as fueling stations or areas with high pollutant concentrations should be disposed according to IDEM regulations for solid waste. Detention basins that primarily catch sediment from areas such as lawns may redistribute the waste on site.
- The wetland drain should be inspected and tested four times per year.
- The embankment should be inspected for evidence of tunneling or burrowing wildlife at least twice during the growing season. If damage is found, the damage should be repaired and the animals removed.
- Mowing and/or trimming of vegetation should be performed as necessary to sustain the system, but all detritus must be removed from the basin. Embankment should be mowed 1–2 times per year to prevent the establishment of woody vegetation.
- Inspections should assess the vegetation, erosion, flow channelization, bank stability, inlet/outlet conditions, embankment, and sediment/debris accumulation.

- Vegetated areas should be inspected annually for unwanted growth of invasive species.
- Vegetative cover should be maintained at a minimum of 85 percent.

Winter Considerations

One of the biggest problems associated with proper constructed wetland operation during cold weather is the freezing and clogging of inlet and outlet pipes. To avoid these problems, the Center for Watershed Protection (Caraco and Claytor, 1997) made some general design suggestions, which are adapted as follows:

- Inlet pipes should typically not be submerged, since this can result in freezing and upstream damage or flooding.
- Burying all pipes below the frost line can prevent frost heave and pipe freezing. Wind protection can also be an important consideration for pipes above the frost line. In these cases, designs modifications that have pipes "turn the corner" are helpful.
- Incorporate lower winter operating levels as part of the design to introduce available storage for melt events.
- Increase the slope of inlet pipes to a minimum of one percent to prevent standing water in the pipe, reducing the potential for ice formation. This design may be difficult to achieve at sites with flat local slopes.
- If perforated riser pipes are used, the minimum opening diameter should be ½-inch. In addition, the pipe should have a minimum 8-inch diameter.
- When a standard weir is used, the minimum slot width should be 3 inches, especially when the slot is tall.
- Baffle weirs can prevent ice reformation during the spring melt near the outlet by preventing surface ice from blocking the outlet structure.
- In cold climates, riser hoods should be oversized and reverse slope pipes should draw from at least 6 inches below the typical ice layer.
- Alternative outlet designs that have been successful include using a pipe encased in a gravel jacket set at the elevation of the aquatic bench as the control for water-quality events. This practice both avoids stream warming and serves as a non-freezing outlet.
- Trash racks should be installed at a shallow angle to prevent ice formation. Constructed wetland performance can be decreased in spring months when large volumes of runoff occur in a relatively short time carrying the accumulated pollutant load from the winter months. Since constructed wetlands are relatively shallow, freezing of the shallow pool can occur.

Cost

The cost of constructed wetlands varies greatly depending on the configuration, location, site specific conditions, etc. Typical construction costs in 2004 dollars range from approximately $30,000 to $65,000 per acre (USEPA Wetlands Fact Sheet, 1999). Costs are generally most dependent on the amount of earthwork and planting. Annual maintenance costs have been reported to be approximately two to five percent of the capital costs (USEPA, 2000).

Designer/Reviewer Checklist for Constructed Wetlands

ITEM	Page No.	YES	NO	N/A	NOTES
Used in conjunction with other BMPs for groundwater recharge and/or water quality?	1,3				
Adequate drainage area/water supply/groundwater table to maintain permanent water surface? ➢ *Min. drainage area:* ***10 acres, 5 acres for pocket wetland*** ➢ *Max. drainage area:* ***50 acres***	2-4				
Relatively impermeable soils and/or soil modification?	6				
Hydrologic calculations (e.g., water balance) performed?	3,4				
Stable inflow points provided?	6				
Forebay and/or pretreatment provided for sediment removal? ➢ *Min. width of permanent access:* ***9 ft*** ➢ *Max. slope:* ***15%***	4-6				
Adequate length to width ratio?	4				
Appropriate and varying water depths for diverse vegetation? ➢ *Max. water depth:* ***3-4 feet, no longer than 12 hours for more than a few days annually***	4,6				
Sudden water level fluctuations minimized to reduce stress on vegetation?	4				
Acceptable side slopes?	5				
Safety benches provided?	5				
Properly designed outlet structure?	4,5				
Adjustable permanent pool and dewatering mechanism provided?	--				
Trash rack provided to prevent clogging?	5				
Stable emergency overflow and outflow points?	5				
Adequate soils for plantings?	6,7				
Appropriate native plants selected in and around wetland?	9				
25-foot buffer provided?	--				
Erosion and sedimentation control considered?	--				
Maintenance accounted for and plan provided? ➢ *Min. vegetative cover to be maintained:* ***85%***	10,11				

➢ *Denotes Minimum Design Consideration*

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BMP Fact Sheet

DETENTION BASINS - DRY POND

Also called Dry-bottom Detention Ponds, Dry Ponds are earthen structures that provide temporary storage of runoff and release the stored volume of water over time to help reduce flooding.



Figure 1 Photograph Courtesy of US Environmental Protection Agency

Key Design Features

- Storage capacity highly dependent on available site area
- Outlet structure configuration determines peak rate reduction effectiveness
- Regular maintenance of vegetation and sediment removal required
- Relatively impermeable soils or impermeable liner
- Forebay for sediment collection and removal
- Stabilized emergency overflow and energy dissipation at all outlets

Applications	
Residential	Yes
Commercial	Yes
Ultra Urban	No
Industrial	Yes
Retrofit	Yes
Highway/Road	Yes
Recreational	Yes

Stormwater Quantity Functions	
Volume	Low
Groundwater Recharge	None or Low
Peak Rate	High

Stormwater Quality Functions

Type	TSS	TP	TN	Temperature
Dry Pond	Medium	Medium	Low	Low

Site Factors

Type	Basin Bottom Relative to Water Table	Soils	Slope	Potential Hotspots	Max. Drainage Area (acres)	Benefits	Limitations
Dry Pond	Above	N/A	Low/ Med	Yes w/ considerations	50	Good peak rate performance, wide applicability, can be used as temporary sediment basin	Low volume/GW recharge and water quality benefits, must be combined with other BMPs, high total cost

**C or D soils typically work without modification. A and B soils may require modifications to reduce their permeability.*

Additional Considerations

Cost

- High – Cost for above ground basins must include excavation of basin, construction of berm, and installation of storm sewer conveyance system, including pipes and structures.
- The cost of each basin is highly dependent on the size of the basin and site characteristics.

Maintenance

Type	Maintenance
Dry Pond	High/Low - Year round maintenance for vegetation; one time per year sediment removal

Winter Performance

- Med/High

Description and Function

Dry ponds are earthen structures that provide temporary storage of runoff and release the stored volume of water over time to help reduce flooding. They are constructed either by impounding a natural depression or excavating existing soil, and are intended to enhance the settlement process in order to maximize water quality benefits, while achieving reduced runoff volume.

Applications

Dry ponds can be used in a wide variety of applications when the necessary space is available. Their uses are limited in ultra urban areas and some redevelopment projects simply due to a lack of available space (in these cases underground and/or special detention may be used).

Design Considerations

Hydrology

- Dry ponds should be designed to mitigate peak runoff rates for the 1-year through 100-year rainfall events.
- Inflow and discharge hydrographs should be calculated for each selected design storm. Hydrographs should be based on the 24-hour rainfall event or that required in the Stormwater Ordinance. Typically, the local NRCS 24-hour Type II rainfall distribution should be utilized to generate hydrographs.

Storage volume, depth, and duration

- Dry ponds should be designed to treat the runoff volume produced by the water quality design storm unless additional upstream BMPs are provided.
- The lowest elevation within a dry detention basin should be at least two feet above the seasonal high water table. If high water table conditions are anticipated, then the design of a wet pond, constructed wetland, or bioretention facility should be considered.
- The maximum water depth of the basin should not exceed 4 feet.
- Detention time is defined as the time from when the maximum storage volume is reached until only 10 percent of that volume remains in the basin. In order to

achieve an 80 percent total suspended solids removal rate, a 36-hour detention time is required within an extended detention basin, with no more than 40% of the maximum stored volume released within the first 12 hours.

- The low flow orifice should be sized and positioned to detain the calculated water quality runoff volume for at least 24 hours, but no more than 36 hours. The low flow orifice may be allowed as small as 4 inches in diameter if adequate protection from clogging is provided.

Basin sizing and configuration

- Dry ponds should be shaped to maximize the hydraulic length of the stormwater flow pathway. A minimum length-to width ratio of 3:1 is recommended to maximize sedimentation. If the length-to-width ratio is lower, the flow pathway should be maximized. A wedge-shaped pond with the major inflows on the narrow end can prevent short-circuiting and stagnation.
- Irregularly shaped basins are acceptable and may even be encouraged to improve site aesthetics.
- Distances of flow paths from inflow points to outlets should be maximized.
- If site conditions inhibit construction of a long, narrow basin, baffles consisting of earthen berms or other materials can be incorporated into the pond design to lengthen the stormwater flow path.
- Basins must have one or more sediment forebays or equivalent upstream pretreatment to trap coarse sediment, prevent short circuiting and facilitate maintenance (i.e., sediment removal). The forebay should consist of a separate cell, formed by a structural barrier. The forebay will require periodic sediment removal.
- Permanent access must be provided to the forebay, outlet, and embankment areas. It should be at least nine feet wide, having a maximum slope of 15 percent, and be stabilized for vehicles.
- An emergency outlet or spillway capable of conveying the spillway design flood (SDF) must be included in the design. The SDF is usually equal to the 1.25 times the 100-year design flood.

Embankments

- Vegetated embankments less than or equal to three feet in height are recommended. Embankments should have side slopes no steeper than 3:1 (horizontal to vertical).
- The basin should have a minimum freeboard of one foot above the SDF elevation to the top of the berm.
- Woody vegetation is generally discouraged in the embankment area because of the risk of compromising the integrity of the embankment.
- Embankments should incorporate measures, such as buried chain link fencing, to prevent or discourage damage from tunneling wildlife (e.g., muskrat).



Figure 2 Dry Pond: Photo courtesy of Fairfax County VA Department of Public Works and Environmental Services.

Dry pond location

- Dry ponds should be located down gradient of disturbed or developed areas on the site. The pond should collect as much site runoff as possible, especially from the site's impervious surfaces (roads, parking, buildings, etc.), and where other BMPs are not proposed.
- Dry ponds should not be constructed on steep slopes, nor should slopes be significantly altered or modified to reduce the steepness of the existing slope, for the purpose of installing a pond.
- Dry ponds should not worsen the runoff potential of the existing site by removing trees for the purpose of installing a pond.

- Dry ponds should not be constructed within 10 feet of the property line or within 50 feet of a private well or septic system.
- Dry ponds should not be constructed in areas with high quality and/or well draining soils, which are adequate for installing BMPs capable of achieving stormwater infiltration and, hence, volume reduction.

Additional design considerations for extended detention basins (Figure 3)

- Dry detention basins should not be constructed within jurisdictional waters, including wetlands, or their regulated buffers.

Outlet design

- The low-flow orifice should typically be no smaller than 4 inches in diameter. However, the orifice diameter may be reduced to two inches if adequate protection from clogging is provided.
- The hydraulic design of all outlet structures must consider any significant tailwater effects of downstream waterways.
- The primary and low flow outlets should be protected from clogging by an external trash rack or other mechanism.
- Online facilities should have an emergency spillway that can safely pass the 1.25 times the 100-year storm with one foot of freeboard. All outflows should be conveyed downstream in a safe and stable manner.
- When designed to meet discharge criteria for a range of storms, basins should incorporate a multistage outlet structure. Three elements are typically included in

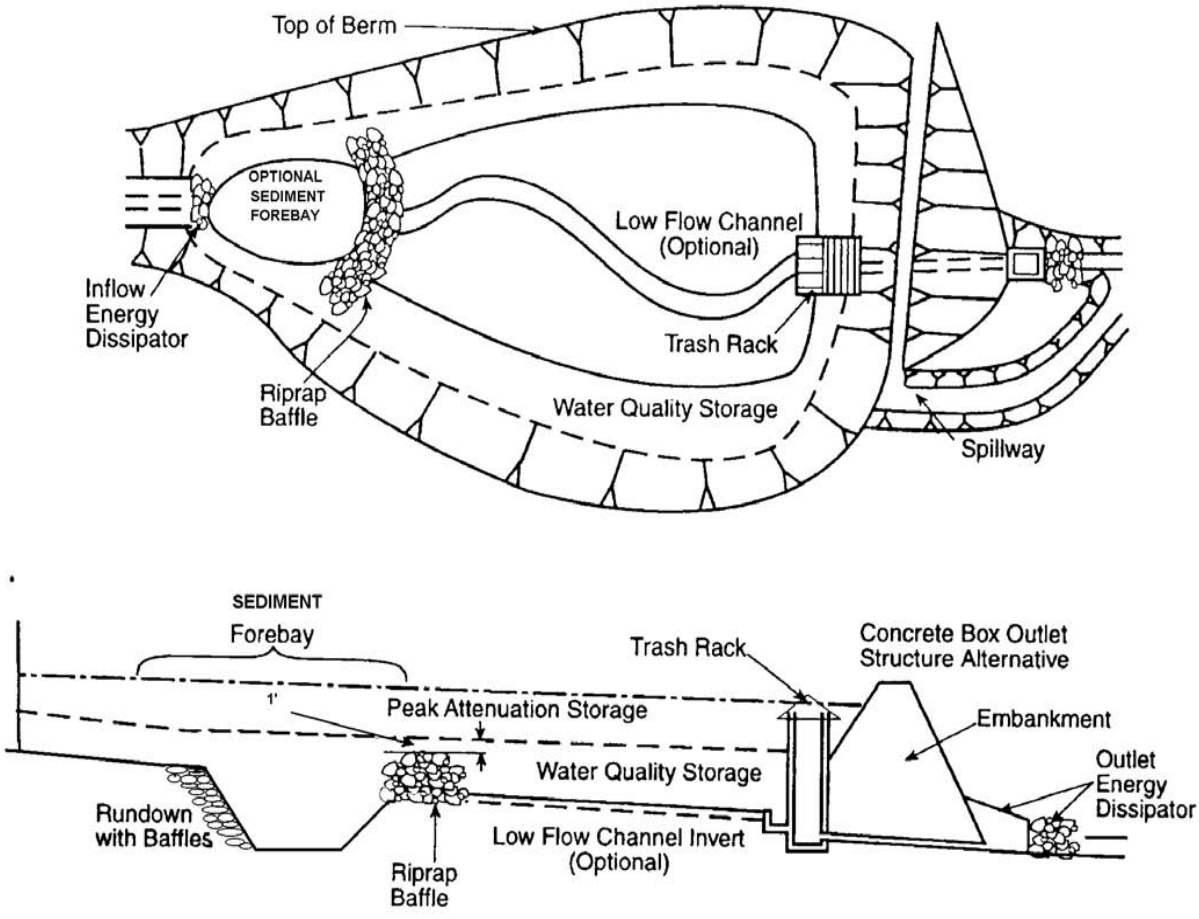


Figure 3 Extended Detention Basin
Source: New Jersey BMP Manual

this design:
  - A low-flow outlet that controls the extended detention and functions to slowly release the water quality or channel protection design storm.
  - A primary outlet that functions to attenuate the peak of larger design storms.
  - An emergency overflow outlet/ spillway. The emergency spillway should be at the top of the berm.
- The primary outlet structure should incorporate weirs, orifices, pipes, or a combination of these to control runoff peak rates for multiple design storms. Water quality storage should be provided below the invert of the primary outlet. When routing basins, the low-flow outlet should be included in the depth-discharge relationship.
- Energy dissipaters should be placed at the end of the primary outlet to prevent erosion. If the basin discharges to a channel with dry weather flow, care should be taken to minimize tree clearing along the downstream channel, and to reestablish a forested riparian zone between the outlet and natural channel. Where feasible, a multiple orifice outlet system is preferred to a single pipe.

Inlet structures

- Erosion protection measures should be used to stabilize inflow structures and channels.

Sediment forebay

- Must be in compliance with local Stormwater Management Ordinance and standard detailed drawing requirements.
- Forebays must be incorporated into the basin design. Forebays should be provided at all major inflow points to capture coarse sediment, prevent excessive sediment accumulation in the main basin, and minimize erosion by inflow.
- Forebays should be vegetated to improve filtering of runoff, to reduce runoff velocity, and to stabilize soils against erosion. Forebays should adhere to the following criteria:
  - A minimum length of 10 feet.
  - Storage should be provided to trap sediment over from storms with return periods between 1 and 10 years.
  - Forebays should be physically separated from the rest of the pond by a berm, gabion wall, etc.
  - Flows exiting the forebay must be non-erosive to the newly constructed basin.
  - Forebays should be installed with permanent vertical markers that indicate sediment depth.
  - Storage volume of 10 to 15 percent of the total permanent pool volume and is four to six feet deep.
  - All major inflow points to dry detention basins should include sediment forebays sized for 10 percent of the water quality volume.

Vegetation and soils protection

- Care should be taken to prevent compaction of soils in the bottom of the extended detention basin in order to promote healthy plant growth and encourage infiltration. If soils compaction is not prevented during construction, soils should be restored as discussed in the Soils Restoration BMP.
- Basin bottoms and side slopes should be vegetated with a diverse native planting mix to reduce maintenance needs, promote natural landscapes, and increase infiltration potential.
- Vegetation may include trees, woody shrubs, and meadow/wetland herbaceous plants.
- Woody vegetation is generally discouraged in the embankment.
- Meadow grasses or other deeply rooted herbaceous vegetation is recommended on the interior slope of embankments.
- Fertilizers and pesticides should not be used.

General Specifications

The following specifications are provided for information purposes only. These specifications include information on acceptable materials for typical applications, but are by no means exclusive or limiting. The designer is responsible for developing detailed specifications for individual design projects in accordance with the project conditions.

Site preparation

All excavation areas, embankments, and structure locations should be cleared and grubbed as necessary, but trees and existing vegetation should be retained and incorporated within the dry detention basin area where possible. Trees should not be removed unless absolutely necessary.

Where feasible, trees and other native vegetation should be protected, even in areas where temporary inundation is expected. A minimum 10-foot radius around the inlet and outlet structures can be cleared to allow room for construction.

Any cleared material should be used as mulch for erosion control or soil stabilization.

Care should be taken to prevent compaction of the bottom of the reservoir. If compaction should occur, soils should be restored and amended.

Earth fill material & placement

- The fill material should be taken from approved designated excavation areas. It should be free of roots, stumps, wood, rubbish, stones greater than six inches, or other objectionable materials. Materials on the outer surface of the embankment must have the capability to support vegetation.

- Areas where fill is to be placed should be scarified prior to placement. Fill materials for the embankment should be placed in maximum eight-inch lifts. The principal spillway must be installed concurrently with fill placement and not excavated into the embankment.

- Control movement of the hauling and spreading equipment over the site.

Embankment core

- The core should be parallel to the centerline of the embankment as shown on the plans. The top width of the core should be at least four feet. The height should extend up to at least the 10-year water elevation or as shown on the plans. The side slopes should be 1:1 or flatter. The core should be compacted with construction equipment, rollers, or hand tampers to assure maximum density and minimum permeability. The core should be placed concurrently with the outer shell of the embankment.

- Construction of the berm should follow specifications by the project's geotechnical engineer.

Structure backfill

- Backfill adjacent to pipes and structures should be of the type and quality conforming to that specified for the adjoining fill material. The fill should be placed in horizontal layers not to exceed eight inches in thickness and compacted by hand tampers or other manually directed compaction equipment. The material should fill completely all spaces under and adjacent to the pipe. At no time during the backfilling operation should driven equipment be allowed to operate closer than four feet to any part of the structure. Equipment should not be driven over any part of a concrete structure or pipe, unless there is a compacted fill of 24 inches or greater over the structure or pipe.

- Backfill content and placement should follow specifications by the project's geotechnical engineer.

Pipe conduits

Corrugated metal pipe – All of the following criteria should apply for corrugated metal pipe:

- Materials – Polymer coated steel pipe, aluminum coated steel pipe, aluminum pipe. This pipe and its appurtenances should conform to the requirements of AASHTO specifications with watertight coupling bands or flanges.
- Coupling bands, anti-seep collars, end sections, etc., must be composed of the same material and coatings as the pipe. Metals must be insulated from dissimilar materials with use of rubber or plastic insulating materials at least 24 mils in thickness.
- Connections – All connections with pipes must be completely watertight. The drain pipe or barrel connection to the riser should be welded all around when the pipe and riser are metal. Anti-seep collars should be connected to the pipe in such a manner as to be completely watertight. Dimple bands are not considered to be watertight.
- Bedding – The pipe should be firmly and uniformly bedded throughout its entire length. Where rock or soft, spongy or other unstable soil is encountered, all such material should be removed and replaced with suitable earth compacted to provide adequate support.
- Backfilling should conform to "structure backfill."
- Other details (anti-seep collars, valves, etc.) should be as shown on drawings.

Reinforced concrete pipe - All of the following criteria should apply for reinforced concrete pipe:

- Materials – Reinforced concrete pipe should have bell and spigot joints with rubber gaskets and should equal or exceed ASTM standards.
- Laying pipe – Bell and spigot pipe should be placed with the bell end upstream. Joints should be made in accordance with recommendations of the manufacturer of the material. After the joints are sealed for the entire line, the bedding should be placed so that all spaces under the pipe are filled. Take care to prevent any deviation from the original line and grade of the pipe.
- Backfilling should conform to "structure backfill."
- Other details (anti-seep collars, valves, etc.) should be as shown on drawings.

Plastic pipe

- Materials – PVC pipe should be PVC-1120 or PVC-1220 conforming to ASTM standards. Corrugated High Density Polyethylene (HDPE) pipe, couplings, and fittings should meet AASHTO specifications.
- Joints and connections to anti-seep collars should be completely watertight.
- Bedding – The pipe should be firmly and uniformly bedded throughout its entire length. Where rock or soft, spongy or other unstable soil is encountered, all such material should be removed and replaced with suitable earth compacted to provide adequate support.
- Backfilling should conform to "structure backfill."
- Other details (anti-seep collars, valves, etc.) should be as shown on drawings.

Drainage diaphragms – When a drainage diaphragm is used, a registered professional engineer must supervise the design and construction inspection.

Rock riprap

Rock riprap should meet the Construction BMP requirements contained in the Technical Standards document and the latest requirements of IDEM's Soil Erosion and Sedimentation Control Program (Rule 5).

Stabilization

All borrow areas should be graded to provide proper drainage and left in a stabilized condition. All exposed surfaces of the embankment, spillway, spoil and borrow areas, and berms should be stabilized by seeding, planting, and mulching in accordance with Construction BMP requirements contained in the Technical Standards document and the latest requirements of IDEM's Soil Erosion and Sedimentation Control Program (Rule 5).

Operation and maintenance

An operation and maintenance plan in accordance with local or state regulations must be prepared for all basins. At a minimum, include a dam and inspection checklist as part of the operation and maintenance plan and perform at least annually.

Stormwater Functions and Calculations

Volume reduction

Dry ponds do not provide an appreciable amount of volume reduction.

Peak rate mitigation

Inflow and discharge hydrographs must be calculated for each design storm.

Peak rate is primarily controlled through the transient storage above the bottom elevation of the dry detention facility. The degree to which peak rate is controlled is a function of the transient storage volume provided (i.e., depth and area) and the configuration of the outlet control structure.

Water quality improvement

The primary treatment mechanism is settling by gravity of particulates and their associated pollutants while stormwater is retained in the pond.

Construction Guidelines

The following guidelines pertain to dry ponds:

- Install all temporary erosion and sedimentation controls.
- Separate pond area from contributing drainage area.
- All channels/pipes conveying flows to the pond must be routed away from the pond area until it is completed and stabilized.
- Prior to construction of the pond, the area immediately adjacent to the pond must be stabilized in accordance with the erosion and sediment control methods discussed in the Erosion and Sediment Control Requirements Chapter of the Technical Standards and the latest requirements of IDEM's Soil Erosion and Sedimentation Control Program (Rule 5).
- Prepare site for excavation and/or embankment construction.
- All existing vegetation should remain, if feasible, and only be removed if necessary for construction.
- Care should be taken to prevent compaction of the basin bottom.
- If excavation is required, clear the area of all vegetation. Remove all tree roots, rocks, and boulders only in excavation area.
  - Excavate bottom of basin to desired elevation (if necessary).
- Install surrounding embankments and inlet and outlet control structures.
- Grade and prepare subsoil in bottom of basin while taking care to prevent compaction. Equipment that will apply pressure to the basin bottom of less than or equal to four pounds per square inch is recommended. Compact only the surrounding embankment areas and around inlet and outlet structures.
  - Apply and grade planting soil.
  - Apply geo-textiles and other erosion-control measures.
- Seed, plant, and mulch according to landscaping plan.
- Install any safety or anti-grazing measures, if necessary.
- Follow required maintenance and monitoring guidelines.

Maintenance

Dry ponds must have a maintenance plan and privately owned facilities should have an easement, deed restriction, or other legal measure to prevent neglect or removal.

A basin maintenance plan should be developed which includes the following measures:

- All basin structures should be inspected for clogging and excessive debris and sediment accumulation at least four times per year, as well as after every storm greater than one inch. Structures that should be inspected include basin bottoms, trash racks, outlets

structures, riprap or gabion structures, and inlets.

- Sediment should be removed from the forebay before it occupies 50 percent of the forebay, typically every three to 10 years. Sediment removal should be conducted when the basin is completely dry.

- Sediment should be disposed of properly and once sediment is removed, disturbed areas need to be immediately stabilized and re-vegetated. Proper disposal of removed material depends on the nature of the drainage area and the intent and function of the detention basin. Material removed from detention basins that treat hot spots, such as fueling stations or areas with high pollutant concentrations, should be disposed according to IDEM regulations for solid waste. Detention basins that primarily catch sediment from areas such as lawns may redistribute the waste on site.

- The pond drain should be inspected and tested four times per year.

- The embankment should be inspected for evidence of tunneling or burrowing wildlife at least twice during the growing season. If damage is found, the damage should be repaired and the animals removed.

- Mowing and/or trimming of vegetation should be performed as necessary to sustain the system, but all detritus must be removed from the basin. Embankment should be mowed 1–2 times per year to prevent the establishment of woody vegetation.

- Inspections should assess the vegetation, erosion, flow channelization, bank stability, inlet/outlet conditions, embankment, and sediment/debris accumulation.

- Vegetated areas should be inspected annually for unwanted growth of invasive species.

- Vegetative cover should be maintained at a minimum of 85 percent.

Winter Considerations

Dry ponds should be inspected and maintained during winter months. Application of sand, ash, cinders, or other anti-skid materials may cause sediment forebays to fill more quickly. Otherwise, dry ponds should function as intended in cold weather.

Cost

The construction costs associated with dry ponds can vary considerably. One study evaluated the cost of all pond systems (Brown and Schueler, 1997). Adjusting for inflation, the cost of dry extended detention ponds can be estimated with the equation:

$$C = 12.4V^{0.760}$$

Where:

C = Construction, design and permitting cost
V = Volume needed to control the 10-year storm (ft^3)

Using this equation, typical construction costs are:

$41,600 for a one acre-foot pond
$239,000 for a 10 acre-foot pond
$1,380,000 for a 100 acre-foot pond

Dry ponds using highly structural design features (riprap for erosion control, etc.) are more costly than natural basins. An installation cost savings is associated with a natural vegetated slope treatment, which is magnified by the additional environmental benefits provided. Long-term maintenance costs for processes such as mowing and fertilizing are reduced when more naturalized approaches are used, due to the ability of native vegetation to adapt to local weather conditions and a reduced need for maintenance.

Annual maintenance costs for dry ponds have been reported to be approximately three to five percent of the capital costs, though there is little data available to support this. Alternatively, a community can estimate the cost of the maintenance activities

outlined in the maintenance section. Ponds are long-lived facilities (typically longer than 20 years). Thus, the initial investment into pond systems may be spread over a relatively long time period.

Designer/Reviewer Checklist for Dry Extended Detention Ponds

ITEM	Page No.	YES	NO	N/A	NOTES
Used in conjunction with other BMPs for groundwater recharge and/or water quality?	1				
Adequate drainage area/water supply/groundwater table to maintain permanent water surface? ➢ *Max. drainage area:* ***50 acres***	2				
Relatively impermeable soils and/or soil modification?	--				
Stable inflow points provided?	6				
Forebay and/or pretreatment provided for sediment removal? ➢ *Min. width of permanent access:* ***9 ft*** ➢ *Max. slope:* ***15%*** ➢ *Min. length of forebay:* ***10 ft***	3,6				
Adequate length to width ratio? ➢ *Min. length-to-width ratio:* ***3:1***	3				
Appropriate and varying water depths? ➢ *Max. water depth:* ***4 feet*** ➢ *Lowest elevation:* ***at least 2 feet above seasonal high water table***	3				
Acceptable side slopes? ➢ *Max. side slopes: 3****:1***	4				
Safety benches provided?	--				
Properly designed outlet structure?	5				
Dewatering mechanism provided?	--				
Trash rack provided to prevent clogging?	5				
Stable emergency overflow and outflow points?	5				
Soil compaction minimized?	6,9				
Adequate soils for plantings?	--				
Appropriate native plants selected in and around pond?	6				
25-foot buffer provided?	--				
Erosion and sedimentation control considered?	8,9				
Maintenance accounted for and plan provided?	10				

➢ *Denotes Minimum Design Consideration*

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BMP Fact Sheet

DETENTION BASINS - WET POND

Wet Ponds, also called wet-bottom detention ponds, are earthen structures that provide temporary storage of runoff and release the stored volume of water over time to help reduce flooding while providing a permanent pool of water for aesthetics, recreation, and settlement of sediments.



Figure 1 Pond collecting runoff from rooftops, sidewalks, and yards, Wilsonville, Oregon (USEPA, picasaweb)

Key Design Features

- Wet Ponds
- Wet Detention Ponds
- Pocket Wet Ponds

Key Design Features

- Storage capacity highly dependent on available site area
- Outlet structure configuration determines peak rate reduction effectiveness
- Regular maintenance of vegetation and sediment removal required
- Natural high groundwater table required
- Relatively impermeable soils or impermeable liner
- Forebay for sediment collection and removal
- Dewatering mechanism required
- Stabilized emergency overflow and energy dissipation at all outlets

Applications	
Residential	Yes
Commercial	Yes
Ultra Urban	No
Industrial	Yes
Retrofit	Yes
Highway/Road	Yes
Recreational	Yes

Stormwater Quantity Functions	
Volume	Low
Groundwater Recharge	None or Low
Peak Rate	High

Stormwater Quality Functions

Type	TSS	TP	TN	Temperature
Wet Pond	High	Medium	Medium	Low/Medium

Site Factors

Type	Basin Bottom Relative to Water Table	Soils	Slope	Potential Hotspots	Max. Drainage Area (acres)	Benefits	Limitations
Wet Pond	Can be below WT	C or D*	Low	Yes w/ considerations	50	Good peak rate & TSS performance, wide applicability, potential aesthetic value; can be used as temporary sediment basin	Low volume/GW recharge benefits, high total cost, potentially thermal impact

*C or D soils typically work without modification. A and B soils may require modifications to reduce their permeability.

Additional Considerations

Cost

- High – Cost for above ground basins must include excavation of basin, construction of berm, and installation of storm sewer conveyance system, including pipes and structures. Additional cost may be added for enhanced vegetation.
- The cost of each basin is highly dependent on the size of the basin and site characteristics.

Maintenance

Type	Maintenance
Wet Pond	Low/Med

Winter Performance

- Med/High

Description and Function

Wet ponds are earthen structures that provide temporary storage of runoff and release the stored volume of water over time to help reduce flooding. They are constructed either by impounding a natural depression or excavating existing soil, and are intended to enhance the settlement process in order to maximize water quality benefits, while achieving reduced runoff volume.

Wet ponds include a permanent pool for water quality treatment and additional capacity above the permanent pool for temporary storage. The pond perimeter should generally be covered by a dense stand of emergent wetland vegetation. While they do not achieve significant groundwater recharge or volume reduction, wet ponds can be effective for pollutant removal and peak rate mitigation.

Wet ponds can also provide aesthetic and wildlife benefits. Wet ponds require an adequate source of inflow to maintain the permanent water surface. Due to the potential to discharge warm water, wet ponds should be used with caution near temperature-sensitive water bodies. Properly designed and maintained wet ponds generally do not support significant mosquito populations.

Applications

Wet ponds can be used in a wide variety of applications when the necessary space is available. Their use is limited in ultra urban areas and some redevelopment projects simply due to a lack of available space (in these cases underground and/or special detention may be used).

Variations

Wet ponds can be designed as either online or offline facilities. They can also be used effectively in series with other sediment-reducing BMPs, such as vegetated filter strips, swales, and filters. Wet ponds may be a good option for retrofitting existing dry detention basins. Wet ponds are often organized into the following three groups:

- **Wet ponds** primarily accomplish water quality improvement through displacement of the permanent pool and are generally only effective for small inflow volumes (often they are placed offline to regulate inflow).
- **Wet detention ponds** are similar to wet ponds but use extended detention as another mechanism for water quality and peak rate control. (Discussion of wet ponds in this BMP section focuses on wet detention ponds as described above because this tends to be the most common and effective design.)
- **Pocket wet ponds** are smaller wet ponds that serve drainage areas between approximately 5 and 10 acres and are constructed near the water table to help maintain the permanent pool. They often include extended detention.

Design Considerations

Hydrology

- Wet ponds should be designed to mitigate peak runoff rates for the 1-year through 100-year rainfall events.
- Inflow and discharge hydrographs should be calculated for each selected design storm. Hydrographs should be based on the 24-hour rainfall event or that required in the local Stormwater Ordinance. Typically, the NRCS 24-hour Type II rainfall distribution should be utilized to generate hydrographs.
- Wet ponds must be able to receive and retain enough flow from rain, runoff, and groundwater to ensure long-term viability. A permanent water surface in the deeper areas of the wet pond should be maintained during all but the driest periods. A relatively stable permanent water surface elevation will reduce the stress on vegetation in an area adjacent to the pond. A wet pond should have a drainage area of at least 10 acres (5 acres for pocket wet ponds) or some means of sustaining constant inflow. Even with a large drainage area, a constant source of inflow can improve the biological health and effectiveness of a wet pond while discouraging mosquito growth.

Storage volume, depth, and duration

- Wet ponds should be designed to treat the runoff volume produced by the water quality design storm unless additional upstream BMPs are provided.
- Detention time is defined as the time from when the maximum storage volume is reached until only 10 percent of that volume remains in the basin. In order to achieve an 80 percent total suspended solids removal rate, a 36-hour detention time is required, with no more than 40% of the maximum stored volume released within the first 12 hours.

Additional design considerations for extended detention (Figure 2)

The low flow orifice should be sized and positioned to detain the calculated water quality runoff volume for at least 24 hours, but no more than 36 hours. The low flow orifice may be allowed as small as 4 inches in diameter if adequate protection from clogging is provided.

Basin sizing and configuration

- Wet ponds should be shaped to maximize the hydraulic length of the stormwater flow pathway. A minimum length-to width ratio of 3:1 is recommended to maximize sedimentation. If the length-to-width ratio is lower, the flow pathway should be maximized. A wedge-shaped pond with the major inflows on the narrow end can prevent short-circuiting and stagnation.
- Irregularly shaped basins are acceptable and may even be encouraged to improve site aesthetics.
- Distances of flow paths from inflow points to outlets should be maximized.
- If site conditions inhibit construction of a long, narrow basin, baffles consisting of earthen berms or other materials can be incorporated into the pond design to lengthen the stormwater flow path.
- Basins must have one or more sediment forebays or equivalent upstream pretreatment to trap coarse sediment, prevent short circuiting and facilitate maintenance (i.e., sediment removal). The forebay should consist of a separate cell, formed by a

structural barrier. The forebay will require periodic sediment removal.

- Permanent access must be provided to the forebay, outlet, and embankment areas. It should be at least nine feet wide, having a maximum slope of 15 percent, and be stabilized for vehicles.
- An emergency outlet or spillway capable of conveying the spillway design flood (SDF) must be included in the design. The SDF is usually equal to the 1.25 times the 100-year design flood.
- The area required for a wet pond is generally one to three percent of its drainage area. Wet ponds should be sized to treat the water quality volume and to mitigate the peak rates for larger events.
- All areas that are deeper than four feet should have two safety benches, totaling 15 feet in width. One should start at the normal water surface and extend up to the pond side slopes at a maximum slope of 10 percent. The other should extend from the water surface into the pond to a maximum depth of 18 inches, also at slopes no greater than 10 percent.
- Slopes in and around wet ponds should be 4:1 to 5:1 (horizontal: vertical) or flatter whenever possible (10:1 max. for safety/aquatic benches).

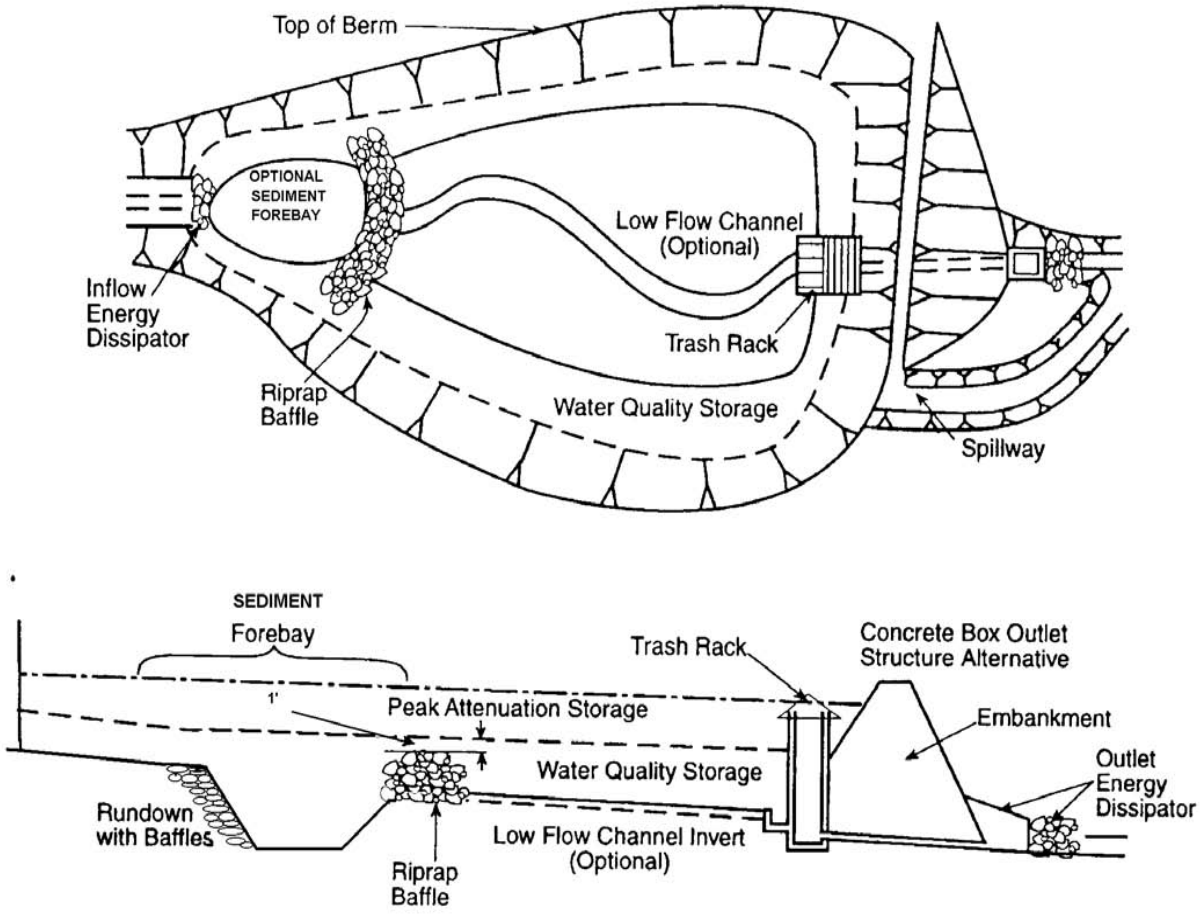


Figure 2 Extended detention basin
Source: New Jersey BMP Manual

Embankments

- Vegetated embankments less than or equal to three feet in height are recommended. Embankments should have side slopes no steeper than 3:1 (horizontal to vertical).
- The basin should have a minimum freeboard of one foot above the SDF elevation to the top of the berm.
- Woody vegetation is generally discouraged in the embankment area because of the risk of compromising the integrity of the embankment.
- Embankments should incorporate measures, such as buried chain link fencing, to prevent or discourage damage from tunneling wildlife (e.g., muskrat).

Wet pond location

- Wet ponds should be located down gradient of disturbed or developed areas on the site. The wet pond should collect as much site runoff as possible, especially from the site's impervious surfaces (roads, parking, buildings, etc.), and where other BMPs are not proposed.
- Wet ponds should not be constructed on steep slopes, nor should slopes be significantly altered or modified to reduce the steepness of the existing slope, for the purpose of installing a wet pond.
- Wet ponds should not worsen the runoff potential of the existing site by removing trees for the purpose of installing a wet pond
- Wet ponds should not be constructed within 10 feet of the property line or within 50 feet of a private well or septic system.
- Wet ponds should not be constructed in areas with high quality and/or well draining soils, which are adequate for installing BMPs capable of achieving stormwater infiltration and, hence, volume reduction.

Additional design considerations for extended detention (Figure 2)

- Wet detention ponds should not be constructed within jurisdictional waters, including wetlands, or their regulated buffers.

Outlet design

- The low-flow orifice should typically be no smaller than 4 inches in diameter. However, the orifice diameter may be reduced to two inches if adequate protection from clogging is provided.
- The hydraulic design of all outlet structures must consider any significant tailwater effects of downstream waterways.
- The primary and low flow outlets should be protected from clogging by an external trash rack or other mechanism.

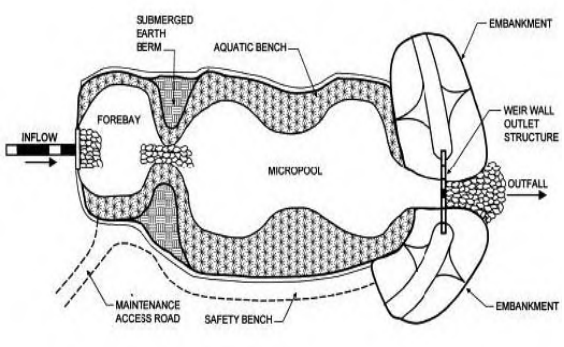


Figure 3 Pocket wet pond
Source: Maryland Stormwater Manual, 2000

- Online facilities should have an emergency spillway that can safely pass the 1.25 times the 100-year storm with one foot of freeboard. All outflows should be conveyed downstream in a safe and stable manner.
- When designed to meet discharge criteria for a range of storms, basins should incorporate a multistage outlet structure. Three elements are typically included in this design:
  - An outlet that controls the extended detention and functions to slowly release the water quality or channel protection design storm.
  - A primary outlet that functions to attenuate the peak of larger design storms.
  - An emergency overflow outlet/spillway. The emergency spillway should be at the top of the berm.
- The primary outlet structure should incorporate weirs, orifices, pipes, or a combination of these to control runoff peak rates for multiple design storms. Water quality storage should be provided above the permanent pool elevation. When routing

basins, the water quality outlet should be included in the depth-discharge relationship.
- Energy dissipaters should be placed at the end of the primary outlet to prevent erosion. If the basin discharges to a channel with dry weather flow, care should be taken to minimize tree clearing along the downstream channel, and to reestablish a forested riparian zone between the outlet and natural channel. Where feasible, a multiple orifice outlet system is preferred to a single pipe.

Inlet structures

- Erosion protection measures should be used to stabilize inflow structures and channels.

Sediment forebay

- Forebays must be in compliance with local Stormwater Management Ordinance and standard detailed drawing requirements.
- Forebays must be incorporated into the basin design. Forebays should be provided at all major inflow points to capture coarse sediment, prevent excessive sediment accumulation in the main basin, and minimize erosion by inflow.
- Forebays should be vegetated to improve filtering of runoff, to reduce runoff velocity, and to stabilize soils against erosion. Forebays should adhere to the following criteria:
  - A minimum length of 10 feet.
  - Storage should be provided to trap sediment over from storms with return periods between 1 and 10 years.
  - Forebays should be physically separated from the rest of the pond by a berm, gabion wall, etc.
  - Flows exiting the forebay must be non-erosive to the newly constructed basin.
  - Forebays should be installed with permanent vertical markers that indicate sediment depth.
  - Storage volume of 10 to 15 percent of the total permanent pool volume and is four to six feet deep.
  - All major inflow points to dry detention basins should include sediment forebays sized for 10 percent of the water quality volume.

Vegetation and soils protection

- Underlying soils must be identified and tested. Generally, hydrologic soil groups “C” and “D” are suitable without modification, though “A” and “B” soils may require modification to reduce their natural permeability. Soil permeability must be tested in the proposed wet pond location to ensure that excessive infiltration will not cause the wet pond to dry out.
- Organic soils should be used for shallow areas within wet ponds. Organic soils can serve as a sink for pollutants and generally have high water holding capacities. They will also facilitate plant growth and propagation and may hinder invasion of undesirable species. Care must be taken to ensure that soils used are free of invasive or nuisance plant seeds.
- To enhance habitat value, visual aesthetics, water temperature, and pond health, a 25-foot buffer should be provided, measured outward from the maximum water surface elevation. The buffer should be planted with trees, shrubs, and native ground covers. Except in maintenance access areas, turf grass should not be used. Existing trees within the buffer should be preserved. If soils in the buffer will become compacted during construction, soil restoration should take place to aid buffer vegetation.

General Specifications

The following specifications are provided for information purposes only. These specifications include information on acceptable materials for typical applications, but are by no means exclusive or limiting. The designer is responsible for developing detailed specifications for individual design projects in accordance with the project conditions.

Excavation

- The area to be used for the wet pond should be excavated to the required depth below the desired bottom elevation to accommodate any required impermeable liner, organic matter, and/or planting soil.

- The compaction of the subgrade and/or the installation of any impermeable liners will follow immediately.

Subsoil preparation

- Subsoil should be free from hard clods, stiff clay, hardpan, ashes, slag, construction debris, petroleum hydrocarbons, or other undesirable material. Subsoil must not be delivered in a frozen or muddy state.
- Scarify the subsoil to a depth of 8 to 10 inches with a disk, rototiller, or similar equipment.
- Roll the subsoil under optimum moisture conditions to a dense seal layer with four to six passes of a sheepsfoot roller or equivalent. The compacted seal layer should be at least eight inches thick.

Impermeable liner

- If necessary, install impermeable liner in accordance with manufacturer's guidelines.
- Place a minimum 12 inches of subsoil on top of impermeable liner in addition to planting soil.
- May be required when in proximity to a public water supply. See Infiltration Considerations (Introduction to Structural BMPs).

Earth fill material & placement

- The fill material should be taken from approved designated excavation areas. It should be free of roots, stumps, wood, rubbish, stones greater than six inches, or other objectionable materials. Materials on the outer surface of the embankment must have the capability to support vegetation.
- Areas where fill is to be placed should be scarified prior to placement. Fill materials for the embankment should be placed in maximum eight-inch lifts. The principal spillway must be installed concurrently with fill placement and not excavated into the embankment.
- Control movement of the hauling and spreading equipment over the site.

Embankment core

- The core should be parallel to the centerline of the embankment as shown on the plans. The top width of the core should be at least four feet. The height should extend up to at least the 10-year water elevation or as shown on the plans. The side slopes should be 1:1 or flatter. The core should be compacted with construction equipment, rollers, or hand tampers to assure maximum density and minimum permeability. The core should be placed concurrently with the outer shell of the embankment.
- Construction of the berm should follow specifications by the project's geotechnical engineer.

Structure backfill

- Backfill adjacent to pipes and structures should be of the type and quality conforming to that specified for the adjoining fill material. The fill should be placed in horizontal layers not to exceed eight inches in thickness and compacted by hand tampers or other manually directed compaction equipment. The material should completely fill all spaces under and adjacent to the pipe. At no time during the backfilling operation should driving equipment be allowed to operate closer than four feet to any part of the structure. Equipment should not be driven over any part of a concrete structure or pipe, unless there is a compacted fill of 24 inches or greater over the structure or pipe.
- Backfill content and placement should follow specifications by the project's geotechnical engineer.

Pipe conduits

Corrugated metal pipe – All of the following criteria should apply for corrugated metal pipe:

- Materials – Polymer coated steel pipe, aluminum coated steel pipe, aluminum pipe. This pipe and its appurtenances should conform to the requirements of AASHTO specifications with watertight coupling bands or flanges.

- Coupling bands, anti-seep collars, end sections, etc., must be composed of the same material and coatings as the pipe. Metals must be insulated from dissimilar materials with use of rubber or plastic insulating materials at least 24 mils in thickness.
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- Laying pipe – Bell and spigot pipe should be placed with the bell end upstream. Joints should be made in accordance with recommendations of the manufacturer of the material. After the joints are sealed for the entire line, the bedding should be placed so that all spaces under the pipe are filled. Take care to prevent any deviation from the original line and grade of the pipe.
- Backfilling should conform to "structure backfill."
- Other details (anti-seep collars, valves, etc.) should be as shown on drawings.

Plastic pipe

- Materials – PVC pipe should be PVC-1120 or PVC-1220 conforming to ASTM standards. Corrugated High Density Polyethylene (HDPE) pipe, couplings, and fittings should meet AASHTO specifications.
- Joints and connections to anti-seep collars should be completely watertight.
- Bedding – The pipe should be firmly and uniformly bedded throughout its entire length. Where rock or soft, spongy or other unstable soil is encountered, all such material should be removed and replaced with suitable earth compacted to provide adequate support.
- Backfilling should conform to "structure backfill."
- Other details (anti-seep collars, valves, etc.) should be as shown on drawings.

Drainage diaphragms – When a drainage diaphragm is used, a registered professional engineer must supervise the design and construction inspection.

Planting soil (topsoil)

- See local specifications for general planting soil requirements.
- Use a minimum of 12 inches of topsoil in the emergent vegetation zone (less than 18" deep) of the pond. If natural topsoil from the site is to be used it must have at least eight percent organic carbon content (by weight) in the A-horizon for sandy soils and 12 percent for other soil types.
- If planting soil is imported, it should be made up of equivalent proportions of organic and mineral materials. All soils used should be free of invasive or nuisance seeds.
- Lime should not be added to planting soil unless absolutely necessary as it may encourage the propagation of invasive species.
- The final elevations and hydrology of the vegetative zones should be evaluated prior to planting to determine if grading or planting changes are required.

Vegetation

- See Recommended Plant List for BMPs Appendix for plant lists for wet ponds. Substitutions of specified plants should be subject to prior approval of the designer. Planting locations should be based on the planting plan and directed in the field by a qualified wetland ecologist. Plant material should be selected based on tolerance to

standing water as identified in Recommended Plant List for BMPs Appendix.

- All wet pond plant stock should exhibit live buds or shoots. All plant stock should be turgid, firm, and resilient. Internodes of rhizomes may be flexible and not necessarily rigid. Soft or mushy stock should be rejected. The stock should be free of deleterious insect infestation, disease, and defects such as knots, sun-scald, injuries, abrasions, or disfigurement that could adversely affect the survival or performance of the plants.
- All stock should be free from invasive or nuisance plants or seeds.
- During all phases of the work, including transport and onsite handling, the plant materials should be carefully handled and packed to prevent injuries and desiccation. During transit and onsite handling, the plant material should be kept from freezing and be covered, moist, cool, out of the weather, and out of the wind and sun. Plants should be watered to maintain moist soil and/or plant conditions until accepted.
- Plants not meeting these specifications or damaged during handling, loading, and unloading will be rejected.

Outlet control structure

- Outlet control structures should be constructed of non-corrodible material.
- Outlets should be resistant to clogging by debris, sediment, floatables, plant material, or ice.
- Materials should comply with applicable specifications (INDOT or AASHTO, latest edition).

Stormwater Functions and Calculations

Volume reduction

Although not typically considered a volume-reducing BMP, wet ponds can achieve some volume reduction through infiltration and evapo-transpiration, especially during small storms and high temperature periods.

According to the International Stormwater BMP Database, wet ponds have an average annual volume reduction of seven percent (Strecker et al., 2004). Hydrologic calculations should be performed to verify that the wet pond will have a viable amount of inflow and to predict the water surface elevation under varying conditions. The volume stored between the predicted water level and the lowest outlet elevation will be removed from the storm that occurs under those conditions.

Peak rate mitigation

Inflow and discharge hydrographs must be calculated for each design storm.

Peak rate is primarily controlled in detention facilities through the transient storage above any permanent water surface. The degree to which peak rate is controlled is a function of the transient storage volume provided (i.e., depth and area) and the configuration of the outlet control structure.

Water quality improvement

Wet ponds rely on physical, biological, and chemical processes to remove pollutants from influent stormwater runoff. The primary treatment mechanism is settling by gravity of particulates and their associated pollutants while stormwater is retained in the pond. Another mechanism for the removal of pollutants, especially nutrients, is uptake by algae and aquatic vegetation.

The longer the runoff remains in a wet pond, the more settling (and associated pollutant removal) and other treatment can occur. After the particulates reach the bottom, the permanent pool protects them from resuspension when additional runoff enters.

The long detention or retention time associated with wet ponds can be problematic in coldwater fisheries due to the potential increase in water temperature. In these situations, detention times should be limited to a maximum of 12 hours, or other treatment alternatives (e.g., infiltration) should be explored.

Construction Guidelines

The following guidelines pertain to wet ponds:

- Install all temporary erosion and sedimentation controls.
- Separate pond area from contributing drainage area.
- All channels/pipes conveying flows to the pond must be routed away from the pond area until it is completed and stabilized.
- The area immediately adjacent to the pond must be stabilized in accordance with the erosion and sediment control methods discussed in the Erosion and Sediment Control Requirements Chapter of the Technical Standards and the latest requirements of IDEM's Soil Erosion and Sedimentation Control Program prior to construction of the pond (Rule 5).
- Prepare site for excavation and/or embankment construction.
- All existing vegetation should remain if feasible and only be removed if necessary for construction.
- Care should be taken to prevent compaction of the basin bottom.
- If excavation is required, clear the area of all vegetation. Remove all tree roots, rocks, and boulders only in excavation area.
- Excavate bottom of basin to desired elevation (if necessary).
- Install surrounding embankments and inlet and outlet control structures.
- Grade and prepare subsoil in bottom of basin. Compact bottom of basin in wet ponds.
  - Apply and grade planting soil. Matching design grades is crucial in wet ponds because aquatic plants can be very sensitive to depth.
  - Apply geo-textiles and other erosion-control measures.
- Seed, plant, and mulch according to landscaping plan.
- Install any safety or anti-grazing measures, if necessary.
- Follow required maintenance and monitoring guidelines.

Maintenance

Wet ponds must have a maintenance plan and privately owned facilities should have an easement, deed restriction, or other legal measure to prevent neglect or removal.

A basin maintenance plan should be developed which includes the following measures:

- All basin structures should be inspected for clogging and excessive debris and sediment accumulation at least four times per year, as well as after every storm greater than one inch. Structures that should be inspected include basin bottoms, trash racks, outlets structures, riprap or gabion structures, and inlets.

- Sediment should be removed from the forebay before it occupies 50 percent of the forebay, typically every 3 to 10 years. Sediment removal should be conducted when the basin is completely dry.

- Wet ponds should be drained prior to sediment removal. Sediment should be disposed of properly and once sediment is removed, disturbed areas need to be immediately stabilized and re-vegetated. Proper disposal of removed material depends on the nature of the drainage area and the intent and function of the detention basin. Material removed from detention basins that treat hot spots, such as fueling stations or areas with high pollutant concentrations, should be disposed according to IDEM regulations for solid waste. Detention basins that primarily catch sediment from areas such as lawns may redistribute the waste on site.

- The pond drain should be inspected and tested four times per year.

- The embankment should be inspected for evidence of tunneling or burrowing wildlife at least twice during the growing season. If damage is found, the damage should be repaired and the animals removed.

- Mowing and/or trimming of vegetation should be performed as necessary to sustain the system, but all detritus must be removed from the basin. Embankment should be

mowed 1–2 times per year to prevent the establishment of woody vegetation.

- Inspections should assess the vegetation, erosion, flow channelization, bank stability, inlet/outlet conditions, embankment, and sediment/debris accumulation.

- Vegetated areas should be inspected annually for unwanted growth of invasive species.

- Vegetative cover should be maintained at a minimum of 85 percent.

Winter Considerations

One of the biggest problems associated with proper wet pond operation during cold weather is the freezing and clogging of inlet and outlet pipes. To avoid these problems, the Center for Watershed Protection (Caraco and Claytor, 1997) made some general design suggestions, which are adapted as follows:

- Inlet pipes should typically not be submerged, since this can result in freezing and upstream damage or flooding.

- Burying all pipes below the frost line can prevent frost heave and pipe freezing. Wind protection can also be an important consideration for pipes above the frost line. In these cases, designs modifications that have pipes "turn the corner" are helpful.

- Incorporate lower winter operating levels as part of the design to introduce available storage for melt events.

- Increase the slope of inlet pipes to a minimum of one percent to prevent standing water in the pipe, reducing the potential for ice formation. This design may be difficult to achieve at sites with flat local slopes.

- If perforated riser pipes are used, the minimum opening diameter should be ½-inch. In addition, the pipe should have a minimum eight-inch diameter.

- When a standard weir is used, the minimum slot width should be three inches, especially when the slot is tall.

- Baffle weirs can prevent ice reformation during the spring melt near the outlet by preventing surface ice from blocking the outlet structure.

- In cold climates, riser hoods should be oversized and reverse slope pipes should draw from at least six inches below the typical ice layer.

- Alternative outlet designs that have been successful include using a pipe encased in a gravel jacket set at the elevation of the aquatic bench as the control for water-quality events. This practice both avoids stream warming and serves as a non-freezing outlet.

- Trash racks should be installed at a shallow angle to prevent ice formation. Constructed wetland performance can be decreased in spring months when large volumes of runoff occur in a relatively short time carrying the accumulated pollutant load from the winter months. Since constructed wetlands are relatively shallow, freezing of the shallow pool can occur.

Cost

The construction cost of wet ponds varies greatly depending on the configuration, location, site specific conditions, etc. Typical construction costs in 2007 dollars range from approximately $30,000 to $60,000 per acre-foot of storage (based on USEPA, 1999). Alternately, the construction cost of a wet pond can be estimated as $6,000 per acre of contributing drainage area. Costs are generally most dependent on the amount of earthwork and the planting.

In addition to the water resource protection benefits of wet ponds, there is some evidence to suggest that they may provide an economic benefit by increasing property values. The results of one study suggest that "pond front" property can increase the selling price of new properties by about 10 percent (USEPA, 1995). Another study reported that the

perceived value (i.e., the value estimated by residents of a community) of homes was increased by about 15 to 25 percent when located near a wet pond (Emmerling-Dinovo, 1995).

Annual maintenance costs for wet ponds have been reported to be approximately three to five percent of the capital costs, though there is little data available to support this. Alternatively, a community can estimate the cost of the maintenance activities outlined in the maintenance section. Ponds are long-lived facilities (typically longer than 20 years). Thus, the initial investment into pond systems may be spread over a relatively long time period.

Designer/Reviewer Checklist for Wet Detention Ponds

ITEM	Page No.	YES	NO	N/A	NOTES
Used in conjunction with other BMPs for water quality and groundwater recharge?	1				
Adequate drainage area/water supply/groundwater table to maintain permanent water surface? ➢ *Min. drainage area:* ***10 acres, 5 acres for pocket wet pond*** ➢ *Max. drainage area:* ***50 acres***	2-4				
Stable inflow points provided?	6				
Forebay and/or pretreatment provided for sediment removal? ➢ *Min. width of permanent access:* ***9 ft*** ➢ *Max. slope:* ***15%*** ➢ *Min. length of forebay:* ***10 ft***	4,6,7				
Adequate length to width ratio? ➢ *Min. length-to-width ratio:* ***3:1***	4				
Total depth limited? ➢ *Max. depth:* ***10 ft***	8				
Acceptable side slopes? ➢ *Max. slope:* ***10%***	4,5				
Safety benches provided?	4				
Properly designed outlet structure?	6				
Trash rack provided to prevent clogging?	6				
Stable emergency overflow and outflow points?	6				
Drawdown time less than 36 hours for 90% of stored volume?	4				
Soil compaction minimized?	--				
Adequate underlying soils?	7,8				
Appropriate native plants selected?	7,8				
25-foot buffer provided?	7				
Erosion and sedimentation control considered?	11				
Maintenance accounted for and plan provided? ➢ *Min. vegetative cover to be maintained:* ***85%***	12				

➢ *Denotes Minimum Design Consideration*

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BMP Fact Sheet

INFILTRATION PRACTICES

Infiltration practices are natural or constructed land areas located in permeable soils that capture, store, and infiltrate the volume of stormwater runoff into surrounding soil.



Figure 1 Infiltration Bed, Seattle, WA (USEPA, picasaweb)

Variations

- **Dry Wells**, also referred to as seepage pits, French drains or Dutch drains, are a subsurface storage facility (structural chambers or excavated pits, backfilled with a coarse stone aggregate) that temporarily store and infiltrate stormwater runoff from rooftop structures. Due to their size, dry wells are typically designed to handle stormwater runoff from smaller drainage areas, less than one acre in size.
- **Infiltration basins** are shallow surface impoundments that temporarily store, capture, and infiltrate runoff over a period of several days on a level and uncompacted surface. Infiltration basins are typically used for drainage areas of 5 to 50 acres with land slopes that are less than 20 percent.
- **Infiltration berms** use a site's topography to manage stormwater and prevent erosion. Berms may function independently in grassy areas or may be incorporated into the design of other stormwater control facilities such as Bioretention and Constructed Wetlands. Berms may also serve various stormwater drainage functions including: creating a barrier to flow, retaining flow for volume control, and directing flows.
- **Infiltration trenches** are linear subsurface infiltration structures typically composed of a stone trench wrapped with geotextile which is designed for both stormwater infiltration and conveyance in drainage areas less than five acres in size.
- **Subsurface infiltration beds** generally consist of a rock storage (or alternative) bed below other surfaces such as parking lots, lawns, and playfields for temporary storage and infiltration of stormwater runoff with a maximum drainage are of 10 acres.
- **Bioretention** can be an infiltration practice and is discussed in the Bioretention BMP.
- **Level spreaders** can be an infiltration practice and are discussed in the Level Spreader BMP.

Key Design Features

- Depth to water table or bedrock
- Pretreatment is often needed to prevent clogging
- Often requires level infiltration surface
- Proximity to buildings, drinking water supplies, karst features, and other sensitive areas
- Soil types
- Provide positive overflow in most uses

Site Factors

- Maximum Site Slope: 20 percent
- Minimum depth to bedrock: Two feet
- Minimum depth to seasonally high water table: Two feet
- Potential Hotspots: Yes, with pretreatment and/or impervious liner
- NRCS Soil Type: A, B, C*, D*

* C & D soils have limited infiltration ability and may require an underdrain

Infiltration BMP	Max. Drainage Area
Berming	5 acres
Dry Well	1 acre
Infiltration Basin	10 acres
Infiltration Trench	2 acres
Subsurface Infiltration Bed	5 acres

Benefits

- Reduces volume of stormwater runoff
- Reduces peak rate runoff
- Increases groundwater recharge
- Provides thermal benefits

Limitations

- Pretreatment requirements to prevent clogging
- Not recommended for areas with steep slopes

Applications

	Residential	Commercial	Ultra Urban	Industrial	Retrofit	Highway/ Road	Recreational
Dry well	Yes	Yes	Yes	Limited	Yes	No	No
Infiltration basin	Yes	Yes	Limited	Yes	Limited	Limited	No
Infiltration berm	Yes	Yes	Limited	Yes	Yes	Yes	No
Infiltration trench	Yes	Yes	Yes	Yes	Yes	Yes	No
Subsurface infiltration bed	Yes	Yes	Yes	Yes	Yes	Limited	No

Stormwater Quantity Functions

	Volume	Groundwater Recharge	Peak Rate
Dry well	Medium	High	Medium
Infiltration basin	High	High	High
Infiltration berm	Low/Medium	Low/Medium	Medium
Infiltration trench	Medium	High	Low/Medium
Subsurface infiltration bed	High	High	High

Stormwater Quality Functions

	TSS	TP	NO3	Temperature
Dry well	High	High/Medium	Medium/Low	High
Infiltration basin	High	Medium/High	Medium	High
Infiltration berm	Medium/High	Medium	TN-Medium	Medium
Infiltration trench	High	High/Medium	Medium/Low	High
Subsurface infiltration bed	High	Medium/High	Low	High

Description and Function

Infiltration practices are designed to store, capture, and infiltrate stormwater runoff into the surrounding soils. During periods of rainfall, infiltrations BMPs reduce the volume of runoff and help to mitigate potential flooding events, downstream erosion, and channel morphology changes. This recharged water serves to provide base-flow to streams and maintain stream water quality.

Infiltration BMPs provide excellent pollutant removal effectiveness because of the combination of a variety of natural functions occurring within the soil mantle, complemented by existing vegetation (where this vegetation is preserved). Soil functions include physical filtering, chemical interactions (e.g., ion exchange, adsorption), as well as a variety of forms of biological processing, conversion, and uptake. The inclusion of appropriate vegetation for some infiltration basins reinforces the work of the soil by reducing velocity and erosive forces, soil anchoring, and further uptake of nonpoint source pollutants. In many cases, even the more difficult-to-remove soluble nitrates can be reduced as well. It should be noted that infiltration BMPs tend to be excellent for removal of many pollutants, especially those that are in particulate form. However, there are limitations to the removal of highly soluble pollutants, such as nitrate, which can be transmitted through the soil.

In addition to the removal of chemical pollutants, infiltration can address thermal pollution. Maintaining natural temperatures in stream systems is recognized as an issue of increasing importance for protection of overall stream ecology. While detention facilities tend to discharge heated runoff flows, the return of runoff to the groundwater through use of infiltration BMPs guarantees that these waters will be returned at natural groundwater temperatures, considerably cooler than ambient air in summer and warmer in winter. As a result, seasonal extreme fluctuations in stream water temperature are minimized. Fish, macro-invertebrates, and a variety of other biota will benefit as the result.

Infiltration Limitations

The use of sediment pretreatment with infiltration BMPs is required for many infiltration BMPs to prevent clogging of the infiltration surface area. Sediment pretreatment can take the form of a water quality filtering device, a settling basin, filter strips, sediment trap, or a combination of these practices upstream of the infiltration practice. Pretreatment practices should be inspected and maintained at least once per year. Before entering an infiltration practice, stormwater should first enter a pretreatment practice sized to treat a minimum volume of 25% of the water quality volume (V_{wq})

Sites that include hot spots, such as gasoline stations, vehicle maintenance areas, and high intensity commercial uses, may need additional pretreatment practices to prevent impairment of groundwater supplies. Infiltration may occur in areas of hot spots provided pretreatment is suitable to address concerns.

Pretreatment devices that operate effectively in conjunction with infiltration include grass swales, vegetated filter strips, bioretention, settling chambers, oil/grit separators, constructed wetlands, sediment sumps, and water quality inserts. Selection of pretreatment practices should be guided by the pollutants of greatest concern, and the extent of the land development under consideration.

Selection of pretreatment techniques will vary depending upon whether the pollutants are of a particulate (sediment, phosphorus, metals, etc.) versus a soluble (nitrogen and others) nature.

Applications

Infiltration systems can be used in a variety of applications, from small areas in residential properties to extensive systems under commercial parking lots or large basins in open space. Industrial, retrofit, highway/ road, and recreational areas can also readily incorporate

infiltration to varying degrees. The use of infiltration basins and berming in ultra-urban and redevelopment settings is limited primarily due to space constraints.

Dry wells have limited applicability in industrial settings as they are designed for runoff from relatively small roof areas (therefore they are also not applicable to transportation corridors). Infiltration basins, subsurface infiltration beds, and berming are also limited for transportation projects due to space constraints and grading requirements (however berming can be used to some degree – especially along the edge of the right of way – to capture runoff).

Variations

Subsurface infiltration

A subsurface infiltration bed generally consists of a rock storage (or alternative) bed below other surfaces such as parking lots, lawns and playfields for temporary storage and infiltration of stormwater runoff. Subsurface storage is enhanced with perforated or open bottom piping. Subsurface infiltration beds can be stepped or terraced down sloping terrain, provided that the base of the bed remains level. Stormwater runoff from nearby impervious areas is conveyed to the subsurface storage media, receives necessary pretreatment and is then distributed via a network of perforated piping.

The storage media for subsurface infiltration beds typically consists of clean-washed, uniformly graded aggregate. However, other storage media alternatives are available. These alternatives are generally variations on plastic cells that can more than double the storage capacity of aggregate beds. Storage media alternatives are ideally suited for sites where potential infiltration area is limited.

If designed, constructed, and maintained using the following guidelines, subsurface infiltration features can stand alone as significant stormwater runoff volume, rate, and quality control practices. These systems can also provide some aquifer recharge, while preserving or creating valuable open space and recreation areas. They have the added benefit of functioning year-round, because the infiltration surface is typically below the frost line.

Various methods can be utilized to connect to subsurface infiltration areas:

Connection of roof leaders

Runoff from nearby roofs can be directly conveyed to subsurface beds via roof leader connections to perforated piping. Roof runoff generally has relatively low sediment levels, making it ideally suited for connection to an infiltration bed.

Connection of inlets

Catch basins, inlets, and area drains may be connected to subsurface infiltration beds. However, sediment, oil and grease, and debris removal must be provided. Storm structures should include sediment trap areas below the inverts of discharge pipes to trap solids and debris. Parking lots and roadways must provide for the removal of oil and grease and other similar constituents through appropriate treatment. In areas of high traffic or excessive generation of sediment, litter, and other similar materials, a water quality insert or other pretreatment device may be required.

Infiltration trench

An infiltration trench is a linear stormwater BMP consisting of a continuously perforated pipe within a sub-surface stone-filled trench wrapped with geotextile. Usually, an infiltration trench is part of a conveyance system and is designed so that large storm events are conveyed through the pipe with some runoff volume reduction. During small storm events, volume reduction may be significant and there may be little or no discharge.

All infiltration trenches should be designed with a positive overflow. Sediment pretreatment of runoff from impervious areas should be considered to prevent clogging within the trench, particularly when conveying runoff from roadways and parking areas.

An infiltration trench differs from an infiltration bed in that it may be constructed in more confined areas. The designer must still consider the impervious area to infiltration area loading rate. It can be located beneath or within

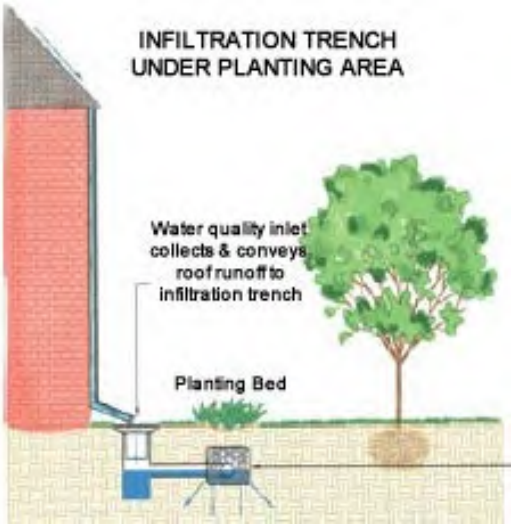


Figure 2 Infiltration trench with continuously perforated pipe for distribution with positive overflow

roadways or impervious areas (**Figure 2**) and can also be located down a mild slope by "stepping" the sections between control structures.

Infiltration basin

Infiltration basins (**Figure 3**) are shallow, impounded areas designed to temporarily store and infiltrate storm-water runoff. The size and shape can vary from one large basin to multiple, smaller basins throughout a site.

Infiltration basins use the existing soil and native vegetation to reduce the volume of stormwater runoff by infiltration and evapotranspiration. Therefore, the use of sediment pretreatment is imperative to prevent clogging of the infiltration surface area within the basin. Sediment pretreatment can take the form of a water quality filtering device, vegetative filter strips, a settling basin, or a sediment trap. The key to promoting infiltration is to provide enough surface area for the volume of runoff to be absorbed within 72 hours.

An engineered overflow structure must be provided for the larger storms and can be designed for peak rate attenuation. With the use of a properly designed outlet structure, infiltration basins can be designed to mitigate volume and water quality for small frequent storms, while managing peak rates for large design storms.

Dry well

A dry well (**Figure 4**) is a subsurface storage facility that temporarily stores and infiltrates stormwater runoff from rooftops. Roof leaders usually connect directly into the dry well, which may be either an excavated pit filled with uniformly graded aggregate wrapped in geotextile or a prefabricated storage chamber or pipe segment. For structures without gutters or

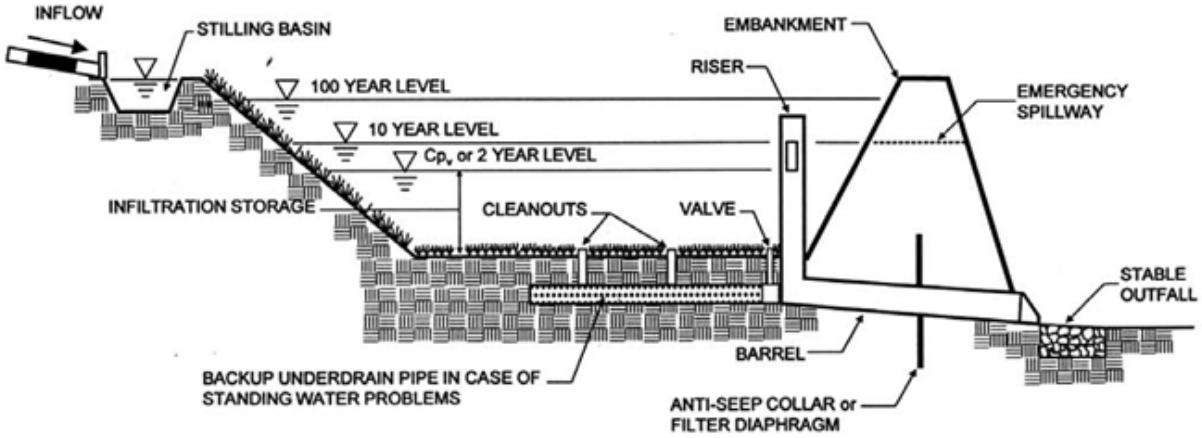


Figure 3 Residential Rain Garden with surface connection to subsurface infiltration bed under garden

downspouts, runoff can be designed to sheet flow off a pitched roof surface and onto a stabilized ground cover that is then directed toward a dry well via stormwater pipes or swales.

Dry wells discharge the stored runoff via infiltration into the surrounding soils. In the event that the dry well is overwhelmed in an intense storm event, an overflow mechanism (e.g., surcharge pipe, connection to larger infiltration area, etc.) will ensure that additional runoff is safely conveyed downstream.

Prefabricated Dry Wells

There are a variety of prefabricated, predominantly plastic subsurface storage chambers on the market today that can replace aggregate dry wells. Since these systems have significantly greater storage capacity than aggregate, space requirements are reduced and associated costs may be defrayed. If the following design guidelines are followed and infiltration is still encouraged, prefabricated chambers can prove just as effective as standard aggregate dry wells.

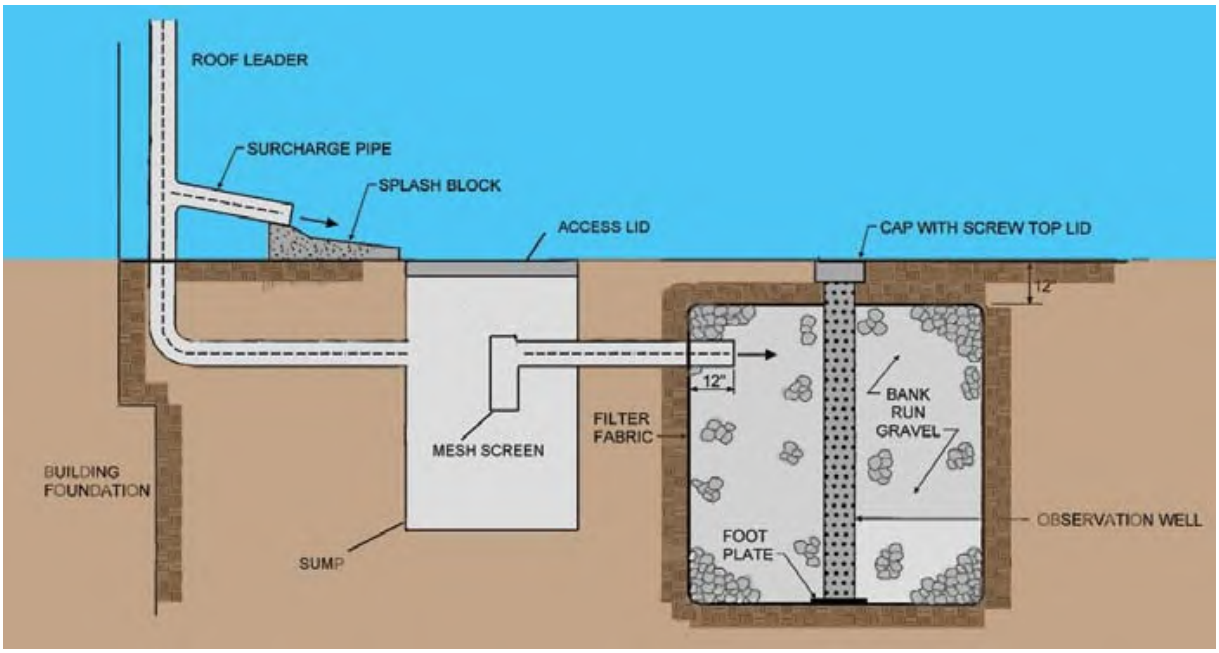


Figure 4 Cross Section of dry well with "sumped" catch basin for sediment pretreatment

Infiltration berm

Infiltration berms are linear vegetation features located along (i.e. parallel to) existing site contours in a moderately sloping area. They are built-up earthen embankments with sloping sides, which function to retain, slow down, or divert stormwater flows. Infiltration berms also have shallow depressions created by generally small earthen embankments that collect and temporarily store stormwater runoff, allowing it to infiltrate into the ground and recharge groundwater.

Infiltration berms can be constructed in various areas on the site, including:

Diversion berms

Diversion berms can be used to protect slopes from erosion and to slow runoff rate. Like swales, berms may divert concentrated discharge from a developed area away from the sloped area. Additionally, berms may be installed in series down the slope to retain

flow and spread it out along multiple, level berms to discourage concentrated flow.

Diversion berms can also be used to direct stormwater flow in order to promote longer flow pathways, thus increasing the time of concentration. For example, berms can be installed such that vegetated stormwater flow pathways are allowed to "meander" so that stormwater travel time is increased.

Meadow/woodland infiltration berms

Woodland infiltration berms can be installed within existing wooded areas for additional stormwater management. Berms in wooded areas can even improve the health of existing vegetation, through enhanced groundwater recharge. Care should be taken during construction to ensure minimum disturbance to existing vegetation, especially tree roots.

Berms are also utilized for a variety of reasons independent of stormwater management, such as to add aesthetic value to a flat landscape, create a noise or wind barrier, separate land uses, screen undesirable views or to enhance or emphasize landscape designs. Berms are often used in conjunction with recreational features, such as pathways through woodlands. In summary, even when used for stormwater management, berms can be designed to serve multifunctional purposes and are easily incorporated into the landscape.

Design Considerations

The following general design considerations are for all BMPs utilizing infiltration. These include site conditions and constraints, as well as general design considerations. Specific design considerations for each BMP follow these same considerations.

Site conditions and constraints for all infiltration BMPs

- **Depth to seasonal high water table**. A four-foot clearance above the seasonally high water table is recommended. A two-foot clearance can be used, but may reduce the performance of the BMP. This reduces the likelihood that temporary groundwater mounding will affect the system, and allows sufficient distance of water movement through the soil to assure adequate pollutant removal. In special circumstances, filter media may be employed to remove pollutants if adequate soil layers do not exist.

- **Depth to bedrock**. A four-foot minimum depth to bedrock is recommended to assure adequate pollutant removal and infiltration. A two-foot depth can be used, but may reduce the performance of the BMP. In special circumstances, filter media may be employed to remove pollutants if adequate soil mantle does not exist.

- **Soil infiltration**. Soils underlying infiltration devices should have infiltration rates between 0.1 and 10 inches per hour, which in most development programs should result in reasonably sized infiltration systems. Where soil permeability is extremely low, infiltration may still be possible, but the surface area required could be large, and other volume reduction methods may be warranted. Undisturbed Hydrologic Soil Groups A, B, and C often fall within this range and cover most of the state. Type D soils may require the use of a double-walled underdrain.

  Soils with rates in excess of six inches per hour may require an additional soil buffer (such as an organic layer over the bed bottom) if the Cation Exchange Capacity (CEC) is less than 10 and pollutant loading is expected to be significant. In carbonate soils, excessively rapid drainage may increase the risk of sinkhole formation, and some compaction or additional measures may be appropriate.

- **Setbacks**. Infiltration BMPs should be sited so that any risk to groundwater quality is minimized and they present no

threat to sub-surface structures such as foundations and septic systems. (**Table 1**)

Setback from	Minimum Distance (feet)
Property Line	10
Building Foundation*	10
Private Well	50
Public Water Supply Well**	50
Septic System Drainfield***	100

Table 1 Setback Distances

* Minimum with slopes directed away from building. 100 feet up gradient from basement foundations.
** At least 200 feet from Type I or IIa wells, 75 feet from Type IIb and III wells
*** 50 feet for septic systems with a design flow of less than 1,000 gallons per day

General design considerations for all infiltration BMPs

- **Do not infiltrate in compacted fill**. Infiltration in native soil without prior fill or disturbance is preferred but not always possible**.** Areas that have experienced historic disturbance or fill are suitable for infiltration provided sufficient time has elapsed and the soil testing indicates the infiltration is feasible. In disturbed areas, it may be necessary to infiltrate at a depth that is beneath soils that have previously been compacted by construction methods or long periods of mowing, often 18 inches or more. If site grading requires placement of an infiltration BMP on fill, compaction should be minimal to prevent excess settlement. The infiltration capacity of the compacted fill should be measured in the field to ensure the design values used are valid.

- **A level infiltration area (one percent or less slope) is preferred**. Bed bottoms should always be graded into the existing soil mantle, with terracing as required to construct flat structures. Sloped bottoms tend to pool and concentrate water in small areas, reducing the overall rate of infiltration and longevity of the BMP. The longitudinal slope may range only from the preferred zero percent up to one percent, and that lateral slopes are held at zero percent. It is highly recommended that the maximum side slopes for an infiltration practice be 1:3 (V: H).

- **The soil mantle should be preserved for surface infiltration BMPs** and excavation should be minimized. Those soils that do not need to be disturbed for the building program should be left undisturbed. Macropores can provide a significant mechanism for water movement in surface infiltration systems, and the extent of macropores often decreases with depth. Maximizing the soil mantle also increases the pollutant removal capacity and reduces concerns about groundwater mounding. Therefore, excessive excavation for the construction of infiltration systems is strongly discouraged.

- **Isolate hot spot areas**. Site plans that include infiltration in hot spots need to be reviewed carefully. Hot spots are most often associated with some industrial uses and high traffic – gasoline stations, vehicle maintenance areas, and high intensity commercial uses (fast food restaurants, convenience stores, etc.). Infiltration may occur in areas of hot spots provided pretreatment is suitable to address concerns.

- **Utilize pretreatment**. Pretreatment should be utilized for most infiltration BMPs, especially for hot spots and areas that produce high sediment loading. Pretreatment devices that operate effectively in conjunction with infiltration include grass swales, vegetated filter strips, settling chambers, oil/grit separators, constructed wetlands, sediment sumps, and water quality inserts. Selection of pretreatment should be guided by the pollutants of greatest concern, site by site, depending upon the nature and extent of the land development under consideration. Selection of pretreatment techniques will vary depending upon whether the pollutants are of a particulate (sediment, phosphorus, metals, etc.) versus soluble (nitrogen and others) nature. Types of

pretreatment (i.e., filters) should be matched with the nature of the pollutants expected to be generated.

- **The loading ratio of impervious area to bed bottom area must be considered**. One of the more common reasons for infiltration system failure is the design of a system that attempts to infiltrate a substantial volume of water in a very small area. Infiltration systems work best when the water is "spread out". The loading ratio describes the ratio of imperious drainage area to infiltration area, or the ratio of total drainage area to infiltration area. In general, the following loading ratios are recommended (some situations, such as highly permeable soils, may allow for higher loading ratios):

    - Maximum impervious loading ratio of 5:1 relating impervious drainage area to infiltration area.
    - Maximum total loading ratio of 8:1 relating total drainage area to infiltration area.

- **The hydraulic head or depth of water should be limited**. The total effective depth of water within the infiltration BMP should generally not be greater than two feet to avoid excessive pressure and potential sealing of the bed bottom. Typically, the water depth is limited by the loading ratio and drawdown time and is not an issue.

- **Drawdown time must be considered**. In general, infiltration BMPs should be designed so that they completely empty within a 72-hour period in most situations (a 48-hour period is preferred).

- **All infiltration BMPs should be designed with a positive overflow** that discharges excess volume in a non-erosive manner, and allows for controlled discharge during extreme rainfall events or frozen bed conditions. Infiltration BMPs should never be closed systems dependent entirely upon infiltration in all storm frequency situations.

- **Geotextiles should be incorporated into the design as necessary**. Infiltration BMPs that are subject to soil movement into the stone medium or excessive sediment deposition must be constructed with suitably permeable non-woven geotextiles to prevent the movement of fines and sediment into the infiltration system. The designer is encouraged to err on the side of caution and use geotextiles as necessary within the BMP structure.

- **Aggregates used in construction should be washed**. In general, bank run material will contain fines that will wash off and clog the infiltration surface.

- **Infiltration utilizing vegetation**. Adequate soil cover (generally 12 to 18 inches) must be maintained above the infiltration bed to allow for a healthy vegetative cover. Vegetation over infiltration beds can be native grasses, meadow mix, or other low-growing, dense species (see Recommended Plant List for BMPs Appendix). These plants have longer roots than traditional grass and will likely benefit from the moisture in the infiltration bed, improving the growth of these plantings and, potentially increasing evapotranspiration.

- **Using underdrains in poor draining soils**. Double-walled underdrains can be used in infiltration BMPs where in situ soils are expected to cause ponding lasting longer than 48 hours. If used, underdrains are typically small diameter (6 to 12 inches) perforated pipes in a clean gravel trench wrapped in geotextile fabric (or in the storage/infiltration bed). Underdrains should have a flow capacity greater than the total planting soil infiltration rate and should have at least 18 inches of soil/gravel cover. They can daylight to the surface or connect to another stormwater system. A method to inspect and clean

underdrains should be provided (via cleanouts, inlet, overflow structure, etc.)

Infiltration practices are prohibited in proximity to public water supply wells.

excessive ponding behind the berm. Greater heights may be used where berms are being used to divert flow or to create "meandering" or lengthened flow pathways. In these cases, stormwater is designed to flow adjacent to (parallel

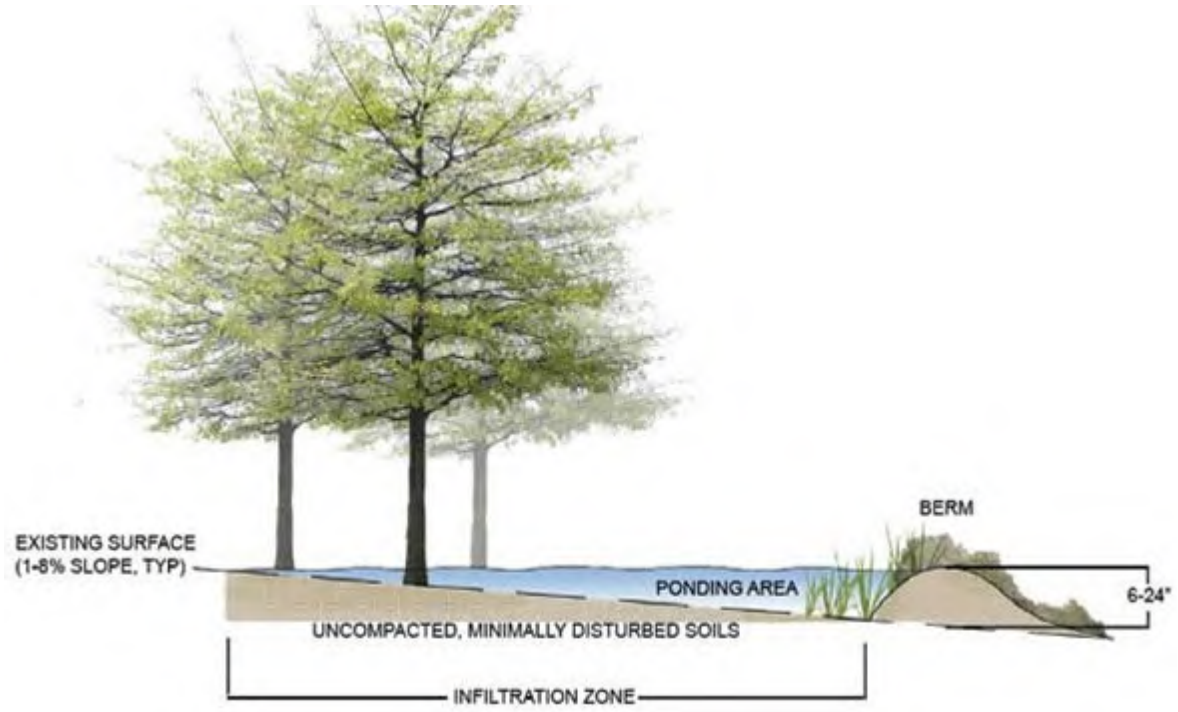


Figure 5 Typical Components of a Berm

- **Freeboard**. It is recommended that two feet of freeboard be provided from the 100-year flood elevation of the infiltration practice to the lowest basement floor elevation of residential, commercial, industrial, and institutional buildings located adjacent to the BMP, unless local requirements recommend or stipulate otherwise.

Additional design considerations for infiltration berms

- Sizing criteria (**Figure 5**) are dependent on berm function, location, and storage volume requirements.
  - Low **berm height** (less than or equal to 24 inches) is recommended to encourage maximum infiltration and to prevent

to), rather than over the crest of the berm. Generally, more berms of smaller size are preferable to fewer berms of larger size.

- **Berm length** is dependent on functional need and site size. Berms installed along the contours should be level and located across the slope. Maximum length will depend on width of the slope.

- Infiltration berms should be constructed along (parallel to) contours at a **constant level elevation**.

- **Soil**. The top one foot of a berm needs to consist of high quality topsoil, with well-drained, stable fill material making up the remainder of the berm. A berm may also

consist entirely of high quality topsoil, but this is the more expensive option.

The use of gravel is not recommended in the layers directly underneath the topsoil because of the tendency of the soil to wash through the gravel. In some cases, the use of clay may be required due to its cohesive qualities (especially where the berm height is high or relatively steeply sloped). However, well-compacted soil is usually sufficient provided that the angle of repose, the angle at which the soil will rest and not be subject to slope failure (discussed below), is adequate for the soil medium used.

- The **angle of repose** of any soil will vary with the texture, water content, compaction, and vegetative cover. Typical angles of repose are given below:

    - Non-compacted clay: 5 to 20 percent
    - Dry Sand: 33 percent
    - Loam: 35 to 40 percent
    - Compacted clay: 50 to 80 percent

- **Slope** The angle of repose for the soil used in the berm should determine the maximum slope of the berm with additional consideration being given to aesthetic, drainage, and maintenance needs. If a berm is to be mowed, the slope should not exceed a 4:1 ratio (horizontal to vertical) in order to avoid "scalping" by mower blades. If trees are to be planted on berms, the slope should not exceed a 5:1 to 7:1 ratio. Other herbaceous plants which do not require mowing can tolerate slopes of 3:1, though this slope ratio may promote increased runoff rate and erosive conditions. Berm side slopes should never exceed a 2:1 ratio.

- **Plant materials**. It is important to consider the function and form of the berm when selecting plant materials. When using native trees and shrubs, plant them in a pattern that appears natural and accentuates the form of the berm. Consider native species from a rolling prairie or upland forest habitat. If turf will be combined with woody and herbaceous plants, the turf should be placed to allow for easy maneuverability while mowing. Low maintenance native plantings, such as trees and meadow plants, rather than turf and formal landscaping, are encouraged and can be found in Recommended Plant List for BMPs Appendix. Additionally, plant material should be selected based on tolerance to standing water as identified in Recommended Plant List for BMPs Appendix.

- **Infiltration trench option**. Soil testing is required for infiltration berms that will utilize a subsurface infiltration trench. Infiltration trenches are not recommended in existing woodland areas, because excavation and installation of subsurface trenches could damage tree root systems. See the infiltration trench section for information on infiltration trench design.

- **Aesthetics**. To the extent possible, berms should reflect the surrounding landscape. Berms should be graded so that the top of the berm is smoothly convex and the toes of the berms are smoothly concave. Natural, asymmetrical berms are usually more effective and attractive than symmetrical berms, which tend to look more artificial. The crest of the berm should be located near one end of the berm rather than in the middle.

- **Pretreatment**. The small depression created by an infiltration berm can act as a sediment forebay prior to stormwater entering a down slope BMP, such as a bioretention basin, a subsurface infiltration bed, or another such facility. Sediment forebays must be in compliance with local Stormwater Management Ordinance and standard detailed drawing requirements.

Additional design considerations for dry wells

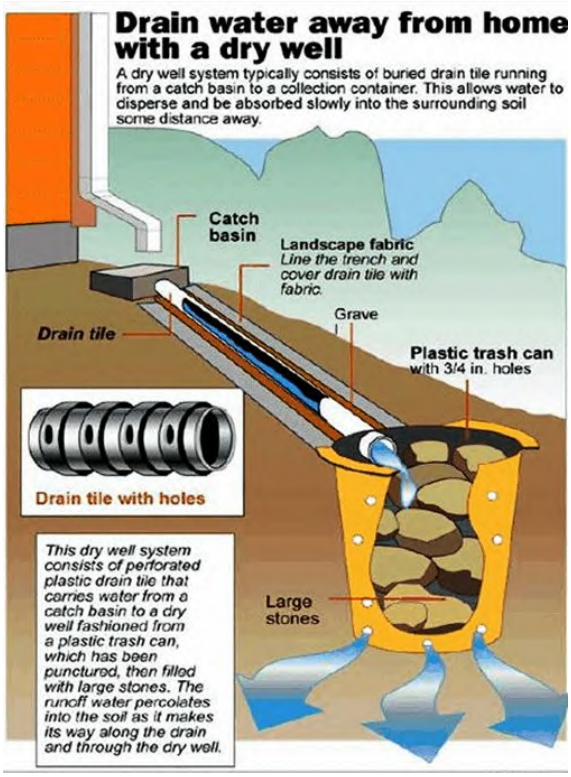


Figure 6 Residential dry well
Source - AP/Stan Kohler

- Dry wells typically consist of 18 to 48 inches of clean washed, uniformly graded aggregate with 40 percent void capacity (AASHTO No. 3, or similar). Dry well aggregate is wrapped in a nonwoven geotextile, which provides separation between the aggregate and the surrounding soil. Typically, dry wells will be covered in at least 12 inches of soil or six inches of gravel or riverstone. An alternative form of dry well is a subsurface, prefabricated chamber, a number of which are currently available on the market.

- All dry wells must be able to convey system overflows to downstream drainage systems. System overflows can be incorporated either as surcharge (or overflow) pipes extending from roof leaders or via connections from the dry well itself.

- The design depth of a dry well should take into account frost depth to prevent frost heave.

- A removable filter with a screened bottom should be installed in the roof leader below the surcharge pipe in order to screen out leaves and other debris.

- Inspection and maintenance access to the dry well should be provided. Observation wells not only provide the necessary access to the dry well, but they also provide a conduit through which pumping of stored runoff can be accomplished in case of slowed infiltration.

- Though roofs are generally not a significant source of runoff pollution, they can still be a source of particulates and organic matter, as well as sediment and debris during construction. Measures such as roof gutter guards, roof leader clean-outs with sump, or an intermediate sump box can provide pretreatment for dry wells by minimizing the amount of sediment and other particulates that enter it.

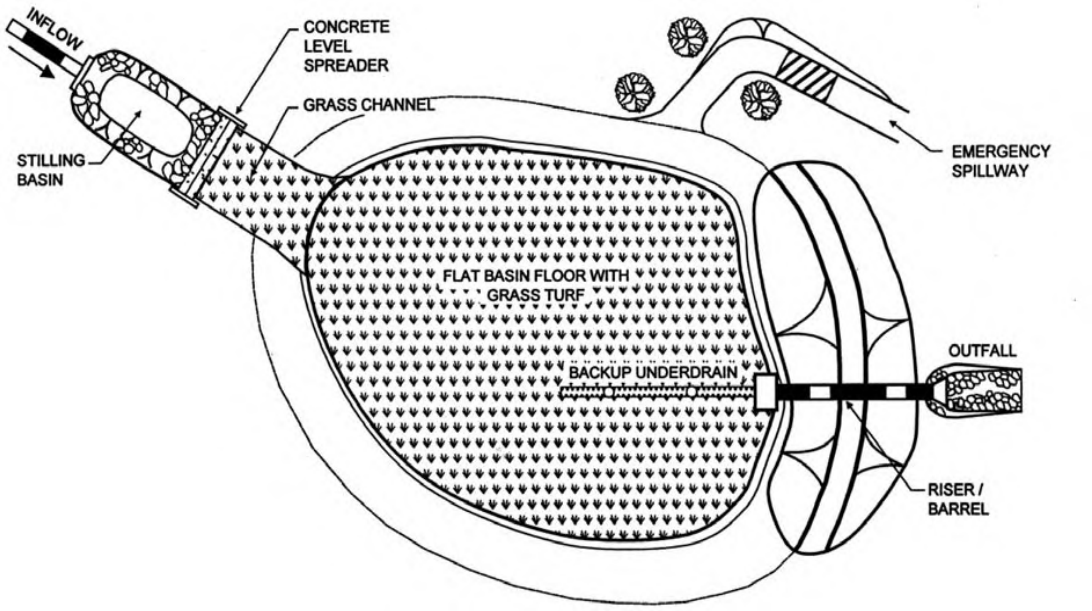


Figure 7 Infiltration Basin Sketch

Additional Design Considerations for Infiltration Basins

- Infiltration basins are typically used for drainage areas of 5 to 50 acres with land slopes that are less than 20 percent.

- A six-inch layer of sand must be placed on the bottom of an infiltration basin (**Figure 7**). This sand layer can intercept silt, sediment, and debris that could otherwise clog the top layer of the soil below the basin.

- An infiltration basin does not normally have a structural outlet to discharge runoff from the stormwater quality design storm. Instead, outflow from an infiltration basin is through the surrounding soil. An infiltration basin may also be combined with an extended detention basin to provide additional runoff storage for both stormwater quality and quantity management. A structural outlet or emergency spillway is provided for storms that exceed the design of the infiltration basin.

- The berms surrounding the basin should be compacted earth with a slope of not less than 3:1, and a top width of at least two feet.

- The overflow from the infiltration basin must be properly designed for anticipated flows. Large infiltration basins may require multiple outlet control devices to effectively allow for overflow water during the larger storms. Emergency overflow systems can be constructed to direct large storm overflows.

- The sediment pre-treatment structure should be designed to provide for access and maintenance.

- In some cases, basins may be constructed where impermeable soils on the surface are removed and where more permeable underlying soils are used for the basin bottom. Care should be taken in the excavation process to make sure that soil compaction does not occur.

- The inlets into the basin should have erosion protection.

- Use of a backup, double-walled underdrain or low-flow orifice may be considered in the event that the water in the basin does not drain within 72 hours.

Additional design considerations for infiltration trenches

- The infiltration trench (**Figure 8)** is typically comprised of a section of uniformly graded aggregate, such as AASHTO No. 3, which ranges one to two inches in gradation. Depending on local aggregate availability, both larger and smaller size aggregate may be used. The critical requirements are that the aggregate be uniformly-graded, clean-washed, and contain at least 40 percent void space. The depth of the trench is a function of stormwater storage requirements, frost depth considerations, and site grading.

lot and road runoff should never be directly discharged underground.

- Cleanouts, observation wells, or inlets must be installed at both ends of the infiltration trench and at appropriate intervals to allow access to the perforated pipe.

- When designed as part of a storm sewer system, a continuously perforated pipe that extends the length of the trench and has a positive flow connection may be included to allow high flows to be conveyed through the infiltration trench. Depending on size, these pipes may provide additional storage volume.

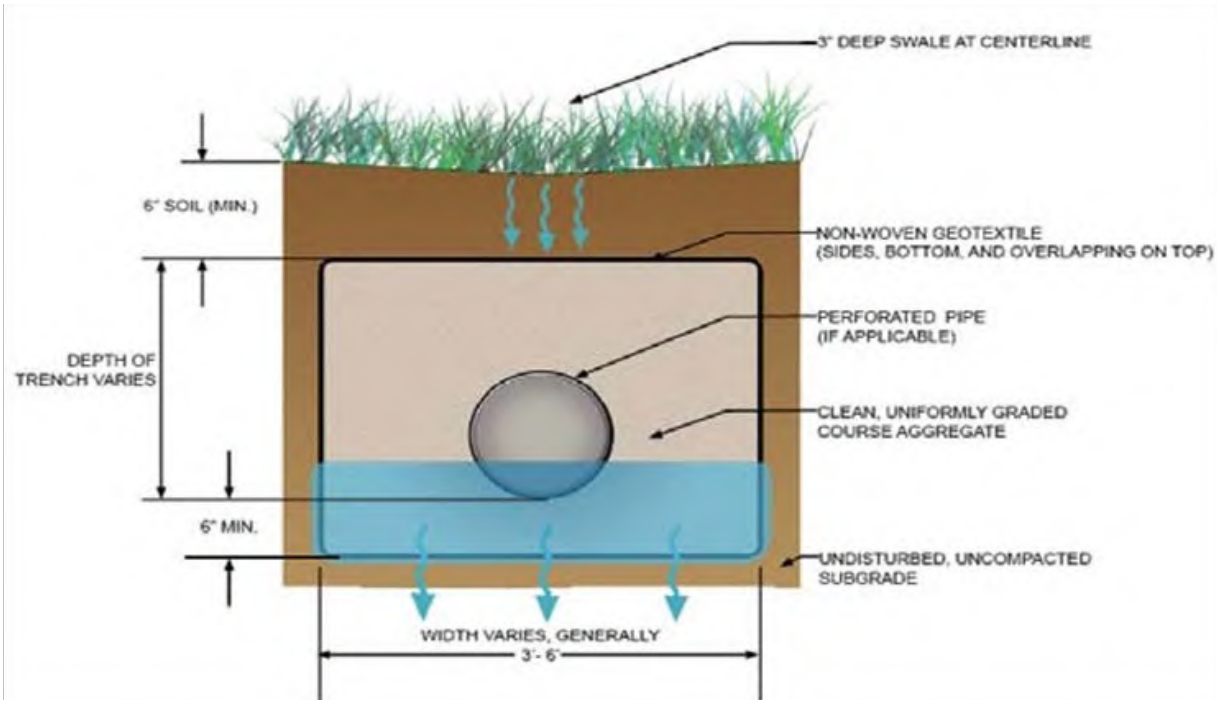


Figure 8 Infiltration trench cross section

- Water quality inlets or catch basins with sumps are required for all surface inlets to prevent clogging of the infiltration trench with sediment and debris. Parking lot and street runoff must be treated by vegetated filter strips, bioretention, or water quality inlets capable of removing oil and grease and similar pollutants. Untreated parking

- Trees may be planted over the infiltration trench provided that adequate soil media is provided above the trench (a minimum of three feet).

- While most infiltration trenches areas consist of an aggregate storage bed, alternative subsurface storage products may also be employed. These include a

variety of proprietary, interlocking plastic units that contain much greater storage capacity than aggregate, though at an increased cost.

Additional design considerations for subsurface infiltration beds

- The infiltration bed must be wrapped in nonwoven geotextile filter fabric to prevent migration of the subsoils into the stone voids (bottom, top, and sides).

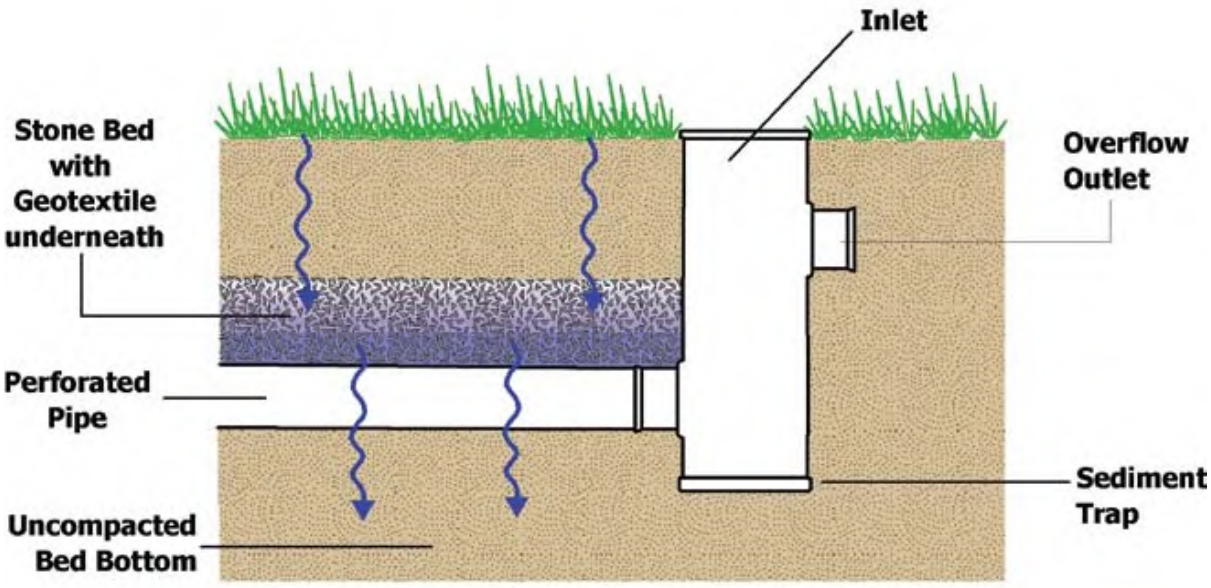


Figure 9 Schematic of subsurface infiltration bed cross section

- The subsurface infiltration bed (**Figure 9**) is typically comprised of a 12 to 36-inch section of aggregate, such as AASHTO No.3, which ranges from one to two inches in gradation. Depending on local aggregate availability, both larger and smaller size aggregate has been used. The critical requirements are that the aggregate be uniformly-graded, clean-washed, and contain at least 40 percent void space. The depth of the bed is a function of stormwater storage requirements, frost depth considerations, and site grading. Infiltration beds are typically sized to mitigate the increased runoff volume from a two-year design storm.

- A water quality inlet or catch basin with sump is required for all surface inlets to avoid standing water for periods greater than 72 hours.

- Perforated pipes along the bottom of the bed can be used to evenly distribute runoff over the entire bed bottom. Continuously perforated pipes should connect structures (such as cleanouts and inlet boxes). Pipes should lay flat along the bed bottom to provide for uniform distribution of water. Depending on size, these pipes may provide additional storage volume.

- Cleanouts or inlets should be installed at a few locations within the bed at appropriate intervals to allow access to the perforated piping network and storage media.

- Grading of adjacent contributing areas should be mildly sloped between one percent and three percent to facilitate drainage.

- In areas with poorly-draining soils, subsurface infiltration areas may be designed to slowly discharge to adjacent wetlands or bioretention areas.

- The subsurface bed and overflow may be designed and evaluated in the same manner as a detention basin to demonstrate the mitigation of peak flow rates. In this manner, detention basins may be eliminated or significantly reduced in size.

- During construction, the excavated bed may serve as a temporary sediment basin or trap, which can reduce overall site disturbance. The bed should be excavated to at least one foot above the final bed bottom elevation for use as a temporary sediment trap or basin. Following construction and site stabilization, sediment should be removed and final grades established.

Incorporating a Safety Factor into Infiltration BMP Design

For the purposes of site suitability, areas with tested soil infiltration rates as low as 0.1 inches per hour may be used for infiltration BMPs. However, in the design of these BMPs and the sizing of the BMP, the designer should incorporate a safety factor. Safety factors between 1 (no adjustment) and 10 have been used in the design of stormwater infiltration systems, with a factor of two being used in most cases. Therefore a measured infiltration rate of 0.5 inches per hour should generally be considered as a rate of 0.25 inches per hour in design. See the Soil Infiltration Protocol Index in Appendix E for guidance on performing infiltration tests.

Modeling Infiltration Systems

Infiltration systems can be modeled similarly to traditional detention basins. The marked difference with modeling infiltration systems is the inclusion of the infiltration rate, which can be considered as another outlet. For modeling purposes, it is sometimes useful to develop infiltration rates that vary (based on the infiltration area provided as the system fills with runoff) for inclusion in the stage-storage-discharge table.

	Volume	Peak Rate	Water Quality
Infiltration Berms	Can be used to reduce the volume of runoff and provide infiltration in accordance with LID/GI stormwater goals. The volume reduction potential of berms is a function of the storage provided (surface and subsurface, if applicable) and the infiltration that will occur.	Can be used at mitigating peak rates for larger storms through two mechanisms: providing storage for detention (and on-going infiltration) behind them and, in some cases, elongating the flow path through a site, thereby extending the time of concentration.	Can be expected to achieve pollutant removals between 30% - 70% and in the upper ranges especially for smaller storms.
Infiltration Basins	Provides an excellent means of capturing and infiltrating runoff. Provides runoff volume storage during storm events, while the undisturbed vegetated surface allows infiltration of runoff into the underlying soil mantle. Can be sized to meet the entire channel protection volume recommended by LID/GI criteria or sized smaller and used in conjunction with other LID/GI practices.	Provides effective management of peak rates to meet the LID/GI design criteria. The basin acts as a storage reservoir during large storm events, even while runoff infiltrates. Outlet structures can be designed to manage peak rates with the use of weir and orifice controls and systems can be designed to manage peak rates for storms up to and including the 100-year storm.	Effective in reducing total suspended solids, nutrients, metals, and oil and grease. Both the vegetative surface and the underlying soils allow pollutant filtration. When designed to capture and infiltrate runoff volumes from small storm events, they provide very high pollutant reductions.
Infiltration Trenches	Provides an excellent means of capturing and infiltrating runoff from small storms. The trench provides runoff volume storage and infiltration during small storm events, while the perforated pipe allows runoff conveyance during large design storms or more extreme events.	Provides limited management of peak rates. The trench may provide more peak rate benefit for small frequent storms, rather than large design storms. Because infiltration trenches help to provide a decentralized approach to stormwater management, they may benefit peak rate mitigation by contributing to increased stormwater travel time.	Effective in reducing total suspended solids, metals, and oil and grease. They provide very high pollutant reductions when designed to capture the volume from small storms because there is little, if any, discharge of runoff carrying the highest pollutant loads. Provide limited treatment of dissolved pollutants, such as nitrates.
Dry Wells	Dry wells are typically designed to capture and infiltrate runoff volumes from small storm events from roof area.	Provides limited management of peak rates. Provides some peak rate benefit by reducing direct connections of impervious area to storm sewer collection systems, and by contributing to increased stormwater travel time.	Effective at capturing and infiltrating the water quality volume or “first flush”. Provides very high pollutant reductions because there is little, if any, discharge of “first flush” runoff, which carries the highest pollutant loads.
Subsurface Infiltration	Provides effective management of volume. A well-designed system is capable of infiltrating the majority of small frequent storms on an annual basis.	Can be designed to manage peak rates by utilizing the stormwater storage bed, including simple rate controls such as weirs and orifices in the overflow control structure. Capable of infiltrating the majority of small frequent storms, while managing peak rates for designs storms up to the 100-year frequency storm.	Very effective at reducing total suspended solids, phosphorus, metals, and oil and grease. Because many systems are designed to capture and infiltrate small, frequent storms, they provide effective water quality control by reducing pollutants associated with the “first-flush”.

Table 2 Stormwater Functions by Infiltration BMP Type

Stormwater Functions and Calculations

Infiltration practices can provide excellent benefits for managing volume and water quality protection. While some BMPs are better than others in managing peak rates, all infiltration BMPs provide some peak rate benefit by removing direct connections from impervious surfaces and increasing time of travel. **Table 2** provides a summary of the stormwater functions by BMP type.

Calculations for Infiltration BMPs

Infiltration area

The minimum infiltration area should be based on the following (according to the loading ratio):
Minimum Surface Infiltration Area = [Contributing impervious area] / 5[#]

- May be increased depending on soil infiltration capacity (e.g., where soils are Type A or rapidly draining). For carbonate, geologic areas may be decreased to three.

This actual infiltration area (**Table 3**) should be greater than the minimum infiltration area.

Protecting Groundwater Quality

The protection of groundwater quality is of utmost importance in any Indiana watershed. The potential to contaminate groundwater by infiltrating stormwater in properly designed and constructed BMPs with proper pretreatment is low.

Numerous studies have shown that stormwater infiltration BMPs have a minor risk of contaminating either groundwater or soil. The U.S. Environmental Protection Agency summarized in "Potential Groundwater Contamination from Intentional and Non-intentional Stormwater Infiltration" (Pitt et al., 1994) the potential of pollutants to contaminate groundwater as either low, low/moderate, moderate, or high. Of the 25 physical pollutants listed, one has a "high" potential (chloride), and two have "moderate" potential (fluoranthene and pyrene) for polluting groundwater through the use of shallow infiltration systems with some sediment pretreatment.

While chloride can be found in significant quantities due to winter salting, relatively high concentrations are generally safe for both humans and aquatic biota. Pentachlorophenol, cadmium, zinc, chromium, lead, and all the pesticides listed are classified as having a "low" contamination potential. Even nitrate, which is soluble and mobile, is only given a "low/moderate" potential.

BMP	Infiltration Area Definition
Infiltration Berms	Total Infiltration Area (Ponding Area) = Length of Berm * Average Width of ponding behind berm.
Infiltration Basin	The Infiltration Area is the bottom area of the basin. This is the area to be considered when evaluating the Loading Ratio to the Infiltration basin.
Infiltration Trench	The Infiltration Area is the bottom area of the trench. This is the area to be considered when evaluating the Loading Rate to the Infiltration basin. [Length of Trench] x [Width of Trench] = Infiltration Area (Bottom Area) Some runoff reduction recognition can be taken for the side area that is frequently inundated, as appropriate.
Dry Well	A dry well may consider both bottom and side (lateral) infiltration according to design.
Subsurface Infiltration	The Infiltration Area is the bottom area of the bed. Some runoff reduction recognition can be taken for the side area that is frequently inundated as appropriate.

Table 3 Definition of Infiltration Area for Infiltration BMPs

Volume reduction

Infiltration BMPs can be used to reduce the volume of runoff and provide infiltration in accordance with LID/GI stormwater goals. The volume reduction potential is a function of the storage provided (surface and subsurface, if applicable) and the infiltration that will occur. If a perforated pipe or double-walled underdrain is used in the design that discharges directly to surface water, the volume of water discharged must be subtracted from the volume reduction calculation.

Total Volume Reduced = Surface Storage Volume (if applicable) + Subsurface Volume (if applicable) + Infiltration Volume

Where,

Surface storage volume (ft^3) = Average bed area * (ft^2) x maximum design water depth (ft)

Subsurface storage/Infiltration bed volume (ft^3) = Infiltration area (ft^2) * Depth of underdrain material (ft) * Void ratio of storage material

**Depth is the depth of the water stored during a storm event, depending on the drainage area, conveyance to the bed, and outlet control.*

Estimated Infiltration Volume (CF) = [Bed bottom area (ft^2)] * [Infiltration design rate (in/hr)] * [Infiltration period# (hr)] / 12 inches/ft.

 # - Infiltration Period is the time during the storm event when bed is receiving runoff and capable of infiltration at the design rate (typically 6 to 12 hours).

Peak rate mitigation

The amount of peak rate control provided by infiltration practices is dependent on the cumulative runoff volume removed by all the infiltration practices applied to a site. Where sufficient infiltration is provided to control the runoff volume from any size storm, the corresponding peak runoff rate will also be restored and the peak runoff rate from larger, less frequent storms will be reduced. Where possible, reducing peak rate of runoff through volume control is generally more effective than fixed rate controls.

Some infiltration BMPs (e.g., infiltration basins) can manage peak rates better than others (e.g., infiltration berms). However, all infiltration BMPs provide some peak rate benefit (e.g., by removing direct connections from impervious surfaces and increasing time of travel).

Water quality improvement

Infiltration practices are effective in reducing pollutants such as total suspended solids, nutrients, metals, oil and grease. The vegetative surface and the underlying soils allow pollutant filtration and studies have shown that pollutants typically are bound to the soils and do not migrate deeply below the surface (i.e. greater than 30-inches). Infiltration practices should be used as part of a treatment train when capturing runoff from storm-water hot spots, such as industrial parking lots, due to the increased level of pollutants. Typical ranges of pollutant reduction efficiencies for infiltration practices are based on available literature data and listed below:

TSS – 75 to 90 percent
TP – 60 to 75 percent
TN – 55 to 70 percent
NO_3 – 30 percent

Construction Guidelines

The following guidelines apply for all infiltration BMPs.

- **Do not compact soil infiltration beds during construction**. Prohibit all heavy equipment from the infiltration area and absolutely minimize all other traffic. Equipment should be limited to vehicles that will cause the least compaction, such as low ground pressure (maximum four pounds per square inch) tracked vehicles. Areas for Infiltration should be clearly marked before any site work begins to avoid soil disturbance and compaction during construction.

- Protect the infiltration area from sediment by ensuring erosion and sediment control practices are implemented until the surrounding site is completely stabilized. Methods to prevent sediment from washing into BMPs should be clearly shown on plans. Where geotextile is used as a bed bottom liner, this should be extended several feet beyond the bed and folded over the edge to protect from sediment wash into the bed during construction, and then trimmed.

- Runoff from construction areas should never be allowed to drain to infiltration BMPs. This can usually be accomplished by diversion berms and immediate vegetative stabilization. The infiltration area may be used as a temporary sediment trap or basin during earlier stages of construction. However, if an infiltration area is also to be utilized as a temporary sediment basin, excavation should be limited to within one foot of the final bottom invert of the infiltration BMP to prevent clogging and compacting the soil horizon, and final grade removed when the contributing site is fully stabilized.

- All infiltration BMPs should be finalized at the end of the construction process, when upstream soil areas have a dense vegetative cover. In addition, do not remove inlet protection or other erosion and sediment control measures until site is fully stabilized. Any sediment which enters inlets during construction is to be removed within 24 hours.

- Provide thorough construction oversight. Long-term performance of infiltration BMPs is dependent on the care taken during construction. Generally, plans and specifications must be followed precisely. The designer is encouraged to meet with the contractor to review the plans and construction sequence prior to construction, and to inspect the construction at regular intervals and prior to final acceptance of the BMP.

- Provide quality control of materials. As with all BMPs, the final product is only as good as the materials and workmanship that went into it. The designer is encouraged to review and approve materials and workmanship, especially as related to aggregates, geotextiles, soil and topsoil, and vegetative materials.

Additional Construction Guidelines for Infiltration Berms

The following is a typical construction sequence for an infiltration berm without a subsurface infiltration trench, though alterations will be necessary depending on design variations.

- Lightly scarify (by hand) the soil in the area of the proposed berm before delivering soil to site (if required). Heavy equipment should not be used within the berm area.

- Bring in fill material to make up the major portion of the berm (as necessary) as soon as subgrade preparation is complete in order to avoid accumulation of debris. Soil should be added in eight-inch lifts and compacted after each addition according to design specifications. The slope and shape of the berm should be graded out as soil is added.

- Protect the surface ponding area at the base of the berm from compaction. If compaction of this area does occur, scarify soil to a depth of at least 8 inches.

- After allowing for settlement, complete final grading within two inches of proposed design elevations. Tamp soil down lightly and smooth sides of the berm. The crest and base of the berm should be level along the contour.

- Seed and plant berm with turf, meadow plants, shrubs or trees, as desired. Water vegetation at the end of each day for two weeks after planting is completed. (Recommended Plant List for BMPs Appendix).

- Mulch planted and disturbed areas with compost to prevent erosion while plants become established.

Additional Construction Guidelines for Subsurface Infiltration

- Where erosion of subgrade has caused accumulation of fine materials and/or surface ponding, this material should be removed with light equipment and the underlying soils scarified to a minimum depth of six inches with a York rake (or equivalent) and light tractor. All fine grading should be done by hand. All bed bottoms are to be at level grade.

- Earthen berms (if used) between infiltration beds should be left in place during excavation.

- Geotextile and bed aggregate should be placed immediately after approval of subgrade preparation and installation of structures. Adjacent strips of geotextile should overlap a minimum of 18 inches, and should also be secured at least four feet outside of the bed to prevent any runoff or sediment from entering the storage bed. This edge strip should remain in place until storage media is placed in the bed.

- Clean washed, uniformly graded aggregate should be placed in the bed in maximum eight-inch lifts. Each layer should be lightly compacted, with construction equipment kept off the bed bottom as much as possible.

- Once bed aggregate has been installed, geotextile can be folded over the top of the aggregate bed. Additional geotextile should be placed as needed to provide a minimum overlap of 18 inches between adjacent geotextile strips.

- Place approved engineered soil media over infiltration bed in maximum six-inch lifts.

- Seed and stabilize topsoil.

Additional Construction Guidelines for Infiltration Trenches

- Excavate infiltration trench bottom to a uniform, level uncompacted subgrade free from rocks and debris. Do NOT compact subgrade.

- Place nonwoven geotextile along bottom and sides of trench. Nonwoven geotextile rolls should overlap by a minimum of 16 inches within the trench. Fold back and secure excess geotextile during stone placement.

- Install upstream and downstream control structures, cleanouts, observation wells, etc.

- Place uniformly graded, clean-washed aggregate in 8-inch lifts, lightly compacting between lifts.

- Install continuously perforated pipe as indicated on plans. Backfill with uniformly graded, clean-washed aggregate in 8-inch lifts, lightly compacting between lifts.

- Fold and secure nonwoven geotextile over infiltration trench, with minimum overlap of 16 inches.

- If vegetated, place a minimum six-inch lift of approved topsoil over infiltration trench, as indicated on plans.

- Seed and stabilize topsoil.

Additional Construction Guidelines for Infiltration Basins

- If necessary, excavate infiltration basin bottom to provide a level and uncompacted subgrade free from rocks and debris. Never compact subgrade.

- Install outlet control structures.

- Seed and stabilize topsoil (Planting with native species is preferred).

Causes of Infiltration BMP Failure

With respect to stormwater infiltration BMPs, the result of "failure" is a reduction in the volume of runoff anticipated or the discharge of stormwater with excessive levels of some pollutants. Where the system includes built structures, such as porous pavements, failure may include loss of structural integrity for the wearing surface, whereas the infiltration function may continue uncompromised. For infiltration systems with vegetated surfaces, such as play fields or rain gardens, failure may include the inability so support surface vegetation, caused by too much or too little water.

The primary causes of reduced performances are:

- Poor construction techniques, especially soil compaction/smearing, which results in significantly reduced infiltration rates.
- A lack of site soil stabilization prior to the BMP receiving runoff, which greatly increases the potential for sediment clogging from contiguous land surfaces.
- Inadequate pretreatment, especially of sediment-laden runoff, which can cause a gradual reduction of infiltration rates.
- Lack of proper maintenance (erosion repair, re-vegetation, removal of detritus, catch basin cleaning, vacuuming of pervious pavement, etc.), which can reduce the longevity of infiltration BMPs.
- Inadequate design
- Inappropriate use of geotextile

Infiltration systems should always be designed such that failure of the infiltration component does not completely eliminate the peak rate attenuation capability of the BMP. Because infiltration BMPs are designed to infiltrate small, frequent storms, the loss or reduction of this capability may not significantly impact the storage and peak rate mitigation of the BMP during extreme events.

Additional Construction Guidelines for Dry Wells

- Excavate dry well bottom to a uniform, level uncompacted subgrade, free from rocks and debris. Do NOT compact subgrade. To the greatest extent possible, excavation should be performed with the lightest practical equipment. Excavation equipment should be placed outside the limits of the dry well.

- Completely wrap dry well with nonwoven geotextile. If sediment and/or debris have accumulated in dry well bottom, remove prior to geotextile placement. Geotextile rolls should overlap by a minimum of 18-24 inches within the trench. Fold back and secure excess geotextile during stone placement.

- Install continuously perforated pipe, observation wells, and all other dry well structures. Connect roof leaders to structures as indicated on plans.

- Place uniformly graded, clean-washed aggregate in 6-inch lifts, between lifts.

- Fold and secure nonwoven geotextile over trench, with minimum overlap of 12 inches.

- Place 12-inch lift of approved topsoil over trench, as indicated on plans.

- Seed and stabilize topsoil.

- Connect surcharge pipe to roof leader and position over splashboard.

Maintenance

There are a few general maintenance practices that should be followed for all infiltration BMPs. These include:

- All catch basins and inlets should be inspected and cleaned at least twice per year.

- The overlying vegetation of subsurface infiltration features should be maintained in good condition, and any bare spots re-vegetated as soon as possible.

- Vehicular access on subsurface infiltration areas should be prohibited (unless designed to allow vehicles), and care should be taken to avoid excessive compaction by mowers.

Additional Maintenance Information for Infiltration Berms

Infiltration berms have low to moderate maintenance requirements, depending on the design. Unless otherwise noted, the following maintenance actions are recommended on an as-needed basis.

Infiltration berms

- Regularly inspect to ensure they are infiltrating; monitor drawdown time after major storm events (total drawdown of the system should not exceed 72 hours; surface drawdown should not exceed 48 hours).

- Inspect any structural components, such as inlet structures, to ensure proper functionality

- If planted in turf grass, maintain by mowing (maintain two to four-inch height); other vegetation will require less maintenance; trees and shrubs may require annual mulching, while meadow planting requires annual mowing and clippings removal

- Avoid running heavy equipment over the infiltration area at the base of the berms; the crest of the berm may be used as access for heavy equipment when necessary to limit disturbance.

- Do not apply pesticides or fertilizers in and around infiltration structures

- Routinely remove accumulated trash and debris

- Remove invasive plants as needed

- Inspect for signs of flow channelization and/or erosion; restore level spreading immediately after deficiencies are observed (monthly)

Diversion berms

- Regularly inspect for erosion or other failures (monthly)

- Regularly inspect structural components to ensure functionality

- Maintain turf grass and other vegetation by mowing and re-mulching

- Do not apply pesticides or fertilizers where stormwater will be conveyed

- Remove invasive plants as needed

- Routinely remove accumulated trash and debris

Additional Maintenance Information for Infiltration Basins

- Inspect the basin after major storm events and make sure that runoff drains down within 72 hours. Mosquitoes should not be a problem if the water drains in 72 hours. Mosquitoes require a considerably long breeding period with relatively static water levels.

- Inspect for accumulation of sediment, damage to outlet control structures, erosion control measures, signs of water contamination/spills, and slope stability in the berms.

- Mow only as appropriate for vegetative cover species.

- Remove accumulated sediment from the sediment pretreatment device/forebay as

needed. Inspect pretreatment forebay at least one time per year.

- If the infiltration basin bottom becomes clogged, scrape bottom and remove sediment and restore original cross section. Properly dispose of sediment.

Additional Maintenance Information for Dry Wells

- Inspect dry wells at least four times a year, as well as after every storm exceeding one inch.
- Remove sediment, debris/trash, and any other waste material from the dry well and dispose of at a suitable disposal/recycling site and in compliance with local, state, and federal waste regulations.
- Evaluate the drain-down time of the dry well to ensure the maximum time of 72 hours is not being exceeded. If drain down time exceeds the maximum, drain the dry well via pumping and clean out perforated piping, if included. If slow drainage persists, the system may need replacing.
- Regularly clean out gutters and ensure proper connections which will facilitate the effectiveness of the dry well.
- Replace filter screen that intercepts roof runoff as necessary.
- If an intermediate sump box exists, clean it out at least once per year.

Winter Considerations

Most infiltration practices are typically located below the frost line and continue to function effectively throughout the winter. It is imperative to prevent salt, sand, cinder, and any other deicers from clogging the surface area of infiltration practices by avoiding piling snow in these areas. Sand and cinder deicers could clog infiltration devices and soluble deicers, such as salt, can damage the health of vegetation.

Cost

The construction cost of many infiltration BMPs can vary greatly depending on the configuration, location, site conditions, etc. Following is a summary of both construction and maintenance costs. This information should be strictly as guidance. More detailed cost information should be discerned for the specific site before assessing the applicability of the BMP.

	Construction Costs	Maintenance Costs
Dry well*	$4-9/ft^3	5-10% of capital costs
Infiltration basin	Varies depending on excavation, plantings, and pipe configuration.	Disposal costs
Infiltration trench**	$20-30/ ft^3	5-10% of capital costs
Subsurface infiltration bed	$13/ ft^3	

Table 4 Summary of construction and maintenance costs for infiltration BMPs
**2003 dollars.*
***City of Portland. 2006 dollars.*

Designer/Reviewer Checklist for Infiltration Berms

ITEM	Page No.	YES	NO	N/A	NOTES
Was the Soil Infiltration Testing Protocol followed?*	--				
Appropriate areas of the site evaluated?	--				
Infiltration rates measured? ➢ *Min. infiltration rate:* ***0.1 in/hr*** ➢ *Max. infiltration rate:* ***10in/hr***	8				
Was the Infiltration BMP followed?	--				
Two-foot separation from bedrock/SHWT?	2,8				
Soil permeability acceptable?	8				
Natural, uncompacted soils?	9				
Excavation in berm areas minimized?	9				
Loading ratio considered? ➢ *Max. impervious drainage area loading ratio –* ***5:1*** ➢ *Max. total drainage area loading ratio –* ***8:1*** ➢ *Max. drainage area:* ***5 acres***	10				
Drawdown time less than 72 hours?	10				
Erosion and Sedimentation control?	--				
Feasible construction process and sequence?	20				
Entering flow velocities non-erosive?	--				
Berm height 6 to 24 inches?	11				
Berm designed for stability (temporary and permanent)?	--				
Acceptable berm side slopes? ➢ *Max. side slopes:* ***2:1 (H:V)***	12				
Are berm materials resistant to erosion?	--				
Located level, along contour?	11				
Acceptable soil for plants specified?	11				
Appropriate plants selected?	10				
Maintenance accounted for and plan provided?	23				

➢ *Denotes Minimum Design Considerations*

* *In general, the protocol should be followed as much as possible (although there is more flexibility for berms than for other BMPs such as pervious pavement and subsurface infiltration that rely almost entirely on infiltration).*

Designer/Reviewer Checklist for Infiltration Trenches, Infiltration Basins, Dry Wells, and Subsurface Infiltration Beds

ITEM	Page No.	YES	NO	N/A	NOTES
Was the Soil Infiltration Testing Protocol followed?	--				
Appropriate areas of the site evaluated?	--				
Infiltration rates measured? ➢ *Min. infiltration rate:* ***0.1 in/hr*** ➢ *Max. infiltration rate:* ***10in/hr***	8				
Was the Infiltration BMP followed?	--				
Two-foot separation between the bed bottom and bedrock/ SHWT?	2,8				
Soil permeability acceptable?	8				
If not, appropriate underdrain provided?	--				
Adequate separations from wells, structures, etc.? ➢ *Minimum Setback Distances:* o *Property Line –* ***10 feet*** o *Building Foundation –* ***10 feet*** o *Private Well –* ***50 feet*** o *Public Water Supply Well –* ***50 feet*** o *Septic System Drainfield –* ***100 feet***	9				
Natural, uncompacted soils?	9				
Level infiltration area (e.g., trench bottom, bed bottom)?	9				
Excavation in infiltration area minimized?	9				
Hotspots/pretreatment considered?	9				
Loading ratio below 5:1? ➢ *Max. impervious drainage area loading ratio –* ***5:1*** ➢ *Max. total drainage area loading ratio –* ***8:1*** ➢ *Max. drainage area:* o *Dry Well –* ***1 acre*** o *Infiltration Basin –* ***10 acres*** o *Infiltration Trench –* ***2 acres*** o *Subsurface Infiltration Bed –* ***5 acres***	10				
Storage depth limited to two feet?	10				
Drawdown time less than 72 hours?	10,24				
Positive overflow from system?	2,7, 10,12				
Erosion and sedimentation control?	14,19, 20				
Feasible construction process and sequence?	21,22				
Geotextile specified?	10,12				

(continued on next page)

ITEM	Page No.	YES	NO	N/A	NOTES
Pretreatment provided?	4,9				
Clean, washed, open-graded aggregate specified?	10,12, 15				
Stable inflows provided (infiltration basin)?	--				
Appropriate perforated pipe, if applicable?	--				
Appropriate plants selected, if applicable?	10				
Observation well/clean out provided, if applicable?	13,15				
Maintenance accounted for and plan provided?	22-24				
Additional Design Considerations ➢ *Infiltration Basins* o *Berms – Min. 3:1 slope; Min. top width – 2 feet* ➢ *Infiltration Trenches* o *Soil media for tree planting – Min. of 3 feet*	--				

➢ Denotes Minimum Design Considerations

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BMP Fact Sheet

PERVIOUS PAVEMENT WITH INFILTRATION

Pervious pavement is an infiltration technique that combines stormwater infiltration, storage, and structural pavement consisting of a permeable surface underlain by a storage reservoir. Pervious pavement is well suited for parking lots, walking paths, sidewalks, playgrounds, plazas, tennis courts, and other similar uses.

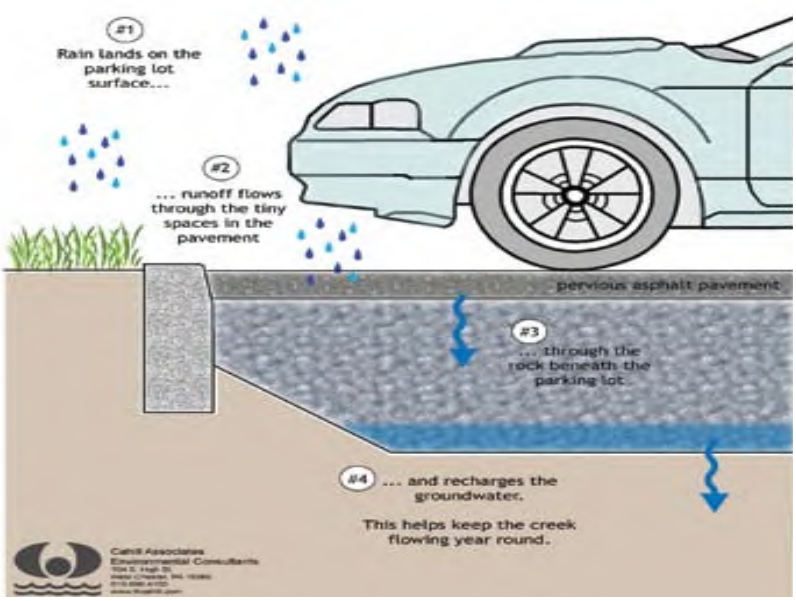


Figure 1 Pervious Pavement with infiltration schematic

Applications		Stormwater Quantity Functions	
Residential	Yes*	Volume	High
Commercial	Yes	Groundwater Recharge	High
Ultra Urban	Yes	Peak Rate	Med/High
Industrial	Yes*	Stormwater Quality Functions	
Retrofit	Yes*	TSS	High**
Highway/Road	Limited	TP	Med/High
Recreational	Yes	TN	Medium
		Temperature	High

Additional Considerations	
Cost	Medium
Maintenance	High
Winter Performance	Medium

*Applicable with special design considerations.

**Pretreatment for TSS is recommended.

Variations

- Porous asphalt
- Pervious concrete
- Permeable paver blocks
- Reinforced turf/gravel

Key Design Features

- Follow soil infiltration testing protocol (Appendix D4) and infiltration BMP guidelines
- Do not infiltrate on compacted soil
- Level storage bed bottoms
- Provide positive stormwater overflow from bed
- Surface permeability > 20”/hour

Site Factors

- Water table/Bedrock separation: two-foot minimum***
- Feasibility on steeper slopes: Low
- Potential hotspots: Not without design of pretreatment system

Benefits

- Volume control and groundwater recharge, moderate peak rate control
- Dual use for pavement structure and stormwater management

Limitations

- Pervious pavement not suitable for all uses
- High maintenance needs

*** Four feet recommended, if possible

Description and Function

A pervious pavement system consists of a porous surface course underlain by a storage reservoir placed on uncompacted subgrade to facilitate stormwater infiltration (**Figure 2)**. The storage reservoir may consist of a stone bed of uniformly graded, clean, and washed course aggregate with a void space of approximately 40 percent or other pre-manufactured structural storage units (see Infiltration BMP for detailed information on the use of structural storage units). The pervious pavement may consist of porous asphalt, pervious concrete, permeable paver blocks, or reinforced turf/gravel.

Stormwater drains through the surface course where it is temporarily held in the voids of the stone bed, and then slowly infiltrates into the underlying, uncompacted soil mantle (in some extreme cases, minimal compaction of the soil may be required). The stone bed can be designed with an overflow control structure so that during large storm events peak rates are controlled. At no time does the water level rise to the pavement level.

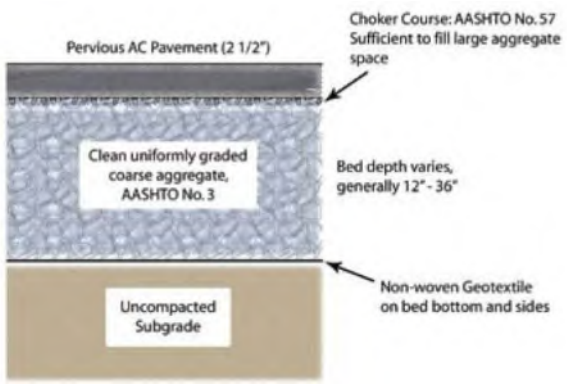


Figure 2 Example cross-section of porous asphalt system

A layer of nonwoven geotextile filter fabric separates the aggregate from the underlying soil, preventing the migration of fines into the bed. The bed bottoms should be level and uncompacted to allow for even and distributed stormwater infiltration.

If new fill is required, it should consist of additional stone and not compacted soil. It is recommended that a failsafe be built into the system in the event that the pervious surface is adversely affected and suffers reduced performance. Many designs incorporate a riverstone/rock edge treatment or inlets which are directly tied to the bed so that the stormwater system will continue to function despite the performance of the pervious pavement surface.

Pervious pavement is well suited for parking lots, walking paths, sidewalks, playgrounds, plazas, tennis courts, and other similar uses. Pervious pavement can be used in driveways if the homeowner is aware of the stormwater functions of the pavement. Pervious pavement roadways have seen wider application in Europe and Japan than in the U.S., although at least one U.S. system has been constructed successfully. (In Japan and the U.S., applying an open-graded asphalt pavement of one inch or less on roadways has been used to provide lateral surface drainage and prevent hydroplaning, but these are applied over impervious pavement on compacted subgrade. This application is not considered a stormwater BMP.)

Properly installed and maintained pervious pavement has a significant life span. For example, existing systems that are more than 20 years old continue to function successfully. Because water drains through the surface course and into the subsurface bed, freeze-thaw cycles do not tend to adversely affect pervious pavement.

Pervious pavement is most susceptible to failure difficulties during construction and, therefore, it is important that construction be undertaken in such a way as to prevent:

- Compacted underlying soil (except in certain limited conditions),
- Contaminated stone subbase with sediment and fines,
- Tracking of sediment or any temporary storage of soil on the pavement surface, and
- Drainage of sediment-laden waters onto pervious surface or into constructed bed.

Staging, construction practices, and erosion and sediment control must all be considered when using pervious pavements.

When properly designed, pervious pavement systems provide effective management of stormwater volume and peak rates. The storage reservoir below the pavement surface can be sized

to manage both direct runoff and runoff generated by adjacent areas, such as rooftops. Because the stone bed provides storage, outlet structures can be designed to manage peak rates with the use of weir and orifice controls. A well-designed system can infiltrate the majority of frequent small storms on an annual basis while providing peak rate control for storms up to and including the 100-year frequency storm event.

Studies have shown that pervious systems have been very effective in reducing contaminants such as total suspended solids, metals, and oil and grease. Because pervious pavement systems often have zero net discharge of stormwater for small frequent storms, they provide effective water quality control. The pervious surface and underlying soils below the storage bed allow filtration of most pollutants.

However, care must be taken to prevent infiltration in areas where toxic/contaminated materials are present in the underlying soils or within the stormwater itself. When designed, constructed, and maintained according to the following guidelines, pervious pavement with underlying infiltration systems can dramatically reduce both the rate and volume of runoff, recharge the groundwater, and improve water quality.

In northern climates, pervious pavements have less of a tendency to form black ice and often require less plowing. Sand and other abrasives should never be used on pervious pavements, although salt may be used on pervious asphalt as long as it does not contain significant non-soluble particles. Commercial deicers may be used on pervious concrete. Pervious pavement surfaces often provide better traction for walking paths in rain or snow conditions.

Variations

Porous asphalt

Early work on porous asphalt pavement was conducted in the early 1970s by the Franklin Institute in Philadelphia. It consists of standard bituminous asphalt in which the fines have been screened and reduced, allowing water to pass through small voids. Pervious asphalt is typically placed directly on the stone subbase in a single 3½ to 4-inch lift that is lightly rolled to a finished thickness of 2½ to 3 inches.

Because porous asphalt is standard asphalt with reduced fines, it is similar in appearance to standard asphalt. Newer open-graded mixes for highway application give improved performance through the use of additives and higher-grade binders. Porous asphalt is suitable for use in any climate where standard asphalt is appropriate.

Pervious concrete

Pervious Portland Cement Concrete, or pervious concrete, was developed by the Florida Concrete Association. Like pervious asphalt, pervious concrete is produced by substantially reducing the number of fines in the mix in order to establish voids for drainage. In northern and mid-Atlantic climates such as Indiana, pervious concrete should always be underlain by a stone subbase designed for stormwater management and should never be placed directly onto a soil subbase.

While porous asphalt is very similar in appearance to standard asphalt, pervious concrete has a coarser appearance than conventional concrete. A clean, swept finish cannot be achieved. Care must be taken during placement to avoid working the surface and creating an impervious layer. Placement should be done by a contractor experienced with pervious concrete. Appropriately installed pervious concrete has proven to be an effective stormwater management BMP. Additional information pertaining to pervious concrete, including specifications, is available from the Indiana Ready Mixed Concrete Association (www.irmca.com).

Permeable paver blocks

Permeable paver blocks consist of interlocking units (often concrete) that provide some portion of surface area that may be filled with a pervious material such as gravel. These units are often very attractive and are especially well suited to plazas, patios, parking areas, and low-speed streets. As new products are always being developed, the designer is encouraged to evaluate the benefits of various products with respect to the specific application.

Reinforced turf/gravel

Reinforced turf consists of interlocking structural units that contain voids or areas for turf grass growth or gravel and suitable for traffic loads and parking. Reinforced turf units may consist of concrete or plastic and are underlain by a stone and/or sand drainage system for stormwater management.

Reinforced turf/gravel applications are excellent for fire access lanes, overflow parking, and occasional-use parking (such as at religious and athletic facilities). Reinforced turf is also an excellent application to reduce the required standard pavement width of paths and driveways that must occasionally provide for emergency vehicle access.

Other

There are other proprietary products similar to pervious asphalt and concrete, but they use clear binders so that the beauty of the natural stone is visible. Material strength varies, so some of these products are not suitable for vehicular traffic Typical applications include tree pits, walkways, plazas, and playgrounds. There are also pervious pavements made using recycled tires.

Applications

Pervious pavements have been widely applied in retrofit situations when existing standard pavements are being replaced. Care must be taken when using pervious pavements in industrial and commercial applications where pavement areas are used for material storage or the potential for surface clogging is high due to pavement use. They have also been used extensively for parking areas, walkways, playgrounds, basketball courts, tennis courts, streets, and alleys.

Pervious pavement systems are often used to provide total site stormwater management where rooftops and other impervious surfaces are tied into the infiltration bed below the pavement surface. This can be an effective means to manage stormwater for a development site, while reducing land disturbance for stormwater BMPs.

If pervious pavement systems receive runoff from adjacent areas, proper sediment pretreatment for that runoff must be considered to prevent clogging of the storage bed. Typical pretreatment can be achieved by the use of properly maintained cleanouts, inlet sediment traps, and water quality inserts or filter devices.

It is recommended that direct surface sheet flow conveyance of large impervious areas to the pervious pavement surface be avoided. High sheet flow loading to pervious pavement surfaces can lead to premature clogging of the pavement surface. To avoid this, it is recommended that adjacent impervious areas be drained and conveyed to the infiltration bed via inlets and trench drains with proper sediment pretreatment.

Design Considerations

While evaluating the following design considerations, there are also several additional resources to consider when implementing pervious pavement. These include the Soil Infiltration Testing Protocol (Soil Infiltration Testing Protocol Appendix), the Recommendations for Materials are specific to porous asphalt and porous concrete (Recommended Materials Appendix), and additional steps set forth in the Introduction to Structural BMPs segment of this Appendix.

Siting

1. The overall site should be evaluated for potential pervious pavement/infiltration areas *early* in the design process because effective pervious pavement design requires consideration of grading.

2. A four foot clearance above the seasonally high water table and bedrock is recommended. A two foot clearance can be used but may reduce the performance of the infiltration BMP used.

3. Orientation of the parking bays along the existing contours will significantly reduce the need for cut and fill.

4. Pervious pavement and infiltration beds **should not be placed on areas of recent fill** or compacted fill. If fill is unavoidable, permeable stone subbase material should be used wherever possible (and applicable infiltration rates should be used in the design). Areas of historical fill (>5 years) may also be considered for pervious pavement.

5. In those areas where the threat of spills and groundwater contamination is likely, pretreatment systems, such as filters and wetlands, may be required before any infiltration occurs. In hot spot areas, such as truck stops and fueling stations, the appropriateness of pervious pavement must be carefully considered. A stone infiltration bed located beneath standard pavement, preceded by spill control and water quality treatment, may be more appropriate.

6. The use of pervious pavement must be carefully considered in areas where the pavement may be seal-coated or paved over due to lack of awareness, such as individual home driveways. In those situations, a system that is not easily altered by the property owner may be more appropriate. An example would include an infiltration system constructed under a conventional driveway. Educational signage at pervious pavement installations can encourage proper maintenance and is recommended.

7. In areas with poorly draining soils, infiltration beds below pervious pavement may be designed to slowly discharge to adjacent swales, wetlands, or bioretention areas. Only in extreme cases (e.g., industrial sites with contaminated soils) will the aggregate bed need to be lined to prevent infiltration.

Design

1. Bed bottoms must be level (0 percent slope) or nearly level. Sloping bed bottoms will lead to areas of ponding and reduced stormwater distribution within the bed. However, beds may be placed on a slope by benching or terracing parking bays (**Figure 3**). Orienting parking bays along existing contours will reduce site disturbance and cut/fill requirements.



Figure 3 Slope stepping with berms Source: Andropogon

2. All systems should be designed with an overflow system. Water within the subsurface stone bed should typically never rise to the level of the pavement surface. Inlet boxes can be used for cost-effective overflow structures. All beds should empty within 72 hours, preferably within 48 hours.

3. While infiltration beds are typically sized to handle the increased volume from a two-year design storm, they must also be able to convey and mitigate the peak of the less-frequent, more-intense storms, such as the 100-year storm. Control in the beds is usually provided in the form of an outlet control structure. A modified inlet box with an internal weir and low-flow orifice is a common type of control structure (**Figure 4**). The specific design of these structures may vary, depending on factors such as rate and storage requirements, but it always must include positive overflow from the system to prevent surface ponding.

4. A weir plate or weir within an inlet or overflow control structure may be used to maximize the water level in the stone bed while providing sufficient cover for overflow pipes (Figure 3).

5. The subsurface bed and overflow may be designed and evaluated in the same manner as a detention basin to demonstrate the

mitigation of peak flow rates. In this manner, the need for a detention basin may be eliminated or significantly reduced in size.

8. Perforated pipes can also be used as underdrains where necessary. Underdrains must be double-walled. Underdrains can ultimately discharge to daylight or to another stormwater system. They should be accessible for inspection and maintenance via cleanouts, overflow devices (Figure 4), or other structures.

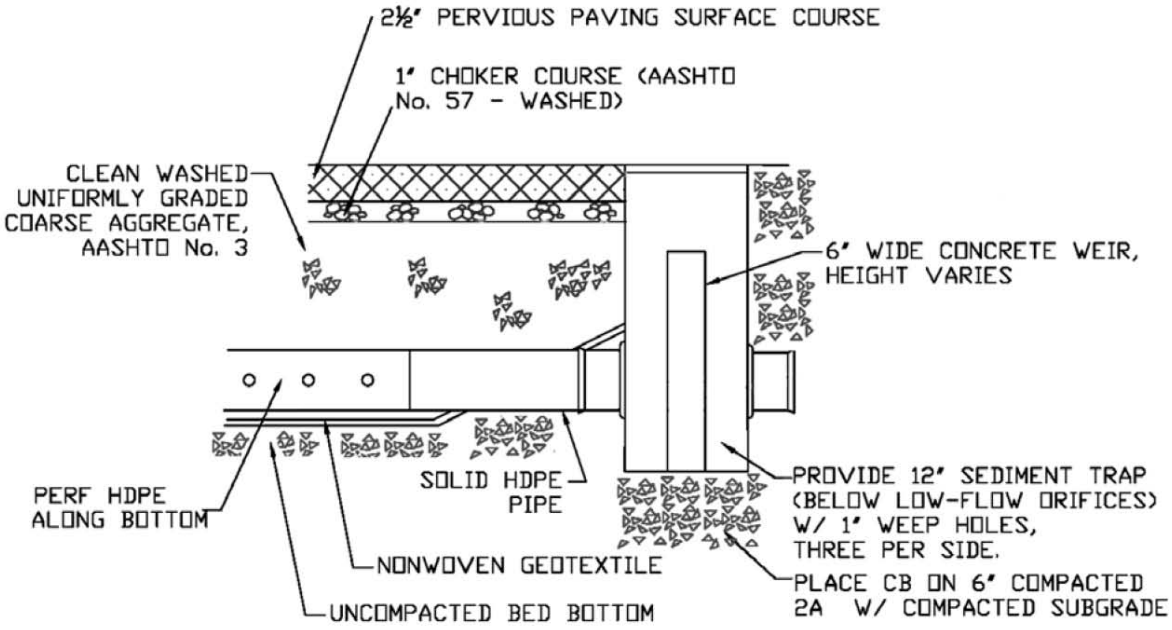


Figure 4 Example cross-section of porous asphalt system

6. Pervious pavement installations should have a backup method for water to enter the stone storage bed in the event that the pavement fails or is altered. In uncurbed lots, this backup drainage may consist of an unpaved one-to-two foot wide stone edge drain connected directly to the bed. In curbed lots, inlets with sediment traps may be used at low spots. Backup drainage elements will ensure the functionality of the infiltration system if the pervious pavement is compromised.

7. Perforated pipes along the bottom of the bed may be used to evenly distribute runoff over the entire bed bottom (especially if runoff from adjacent areas is being brought into the bed). Continuously perforated pipes should connect structures (such as cleanouts and inlet boxes). Pipes may lay flat along the bed bottom and connect to the overflow structure (**Figure 4**). Depending on size, these pipes may provide additional storage volume.

9. Sediment transport to pervious systems should be minimized as much as possible to reduce maintenance requirements and extend the life of these systems. If roof leaders and area inlets convey water from adjacent areas to the bed, then native vegetation, water quality inserts, and/ or sumped inlets should be used to prevent the conveyance of sediment and debris into the bed. Areas of impervious pavement draining directly onto pervious pavements should also be minimized as they can lead to clogging near the impervious-pervious boundary.

10. Infiltration areas should be located within the immediate project area in order to control runoff at its source. Expected use and traffic demands should also be considered in pervious pavement placement. An impervious

water stop should be placed along infiltration bed edges where pervious pavement meets standard impervious pavements.

11. The underlying infiltration bed is typically 8 to 36 inches deep and comprised of clean, uniformly graded aggregate with approximately 40 percent void space. Local aggregate availability typically dictates the size of the aggregate used. The critical requirements are that the aggregate be uniformly graded, clean washed, and contain a significant void content. The depth of the bed is a function of stormwater storage requirements, frost depth considerations, site grading, and structural needs.

12. Proper pervious pavement applications are resistant to freeze-thaw problems because of their permeable and open-graded components (the pavement surface should not be saturated and the base has a high void content which allows for expansion). In somewhat frost-susceptible soils, it may be necessary to increase the minimum bed depth to 14-22 inches (depending on loading and specific soil conditions). In extremely susceptible soils, the bed and/or improved soils can be placed down to the full frost depth (Smith, 2006).

13. While most pervious pavement installations are underlain by an aggregate bed, alternative subsurface storage products may also be used. These include a variety of proprietary plastic units that contain much greater storage capacity than aggregate, at an increased cost.

Stormwater Functions and Calculations

Infiltration area

The infiltration area is defined as the plan area of the storage reservoir under the pervious pavement. The minimum infiltration area should be based on the following equation:

Minimum infiltration area = Contributing impervious area (including pervious pavement) / 5[#]

[#] - May be increased depending on soil infiltration capacity (where soils are Type A or rapidly draining).

Volume reduction

Pervious pavements with infiltration provide an excellent means of capturing and infiltrating runoff. The storage bed below the pavement provides runoff volume storage during storm events, while the undisturbed subgrade allows infiltration of runoff into the underlying soil mantle. The total volume reduction can be estimated by summing the storage and infiltration volumes described below.

Storage volume = Depth[#] (ft) * Area (ft^2) * Void space (i.e., 0.40 for aggregate)

[#] - Depth is the depth of the water stored during a storm event, depending on the drainage area, conveyance to the bed, and outlet control.

Infiltration volume = Bed bottom area (ft^2) * Infiltration design rate (in/hr) * Infiltration period[#] (hr) * (1/12)

[#] - Infiltration period is the time when bed is receiving runoff and capable of infiltrating at the design rate. Not to exceed 72 hours.

Peak rate mitigation

Properly designed pervious pavement systems provide effective management of peak rates. The infiltration bed below the pavement acts as a storage reservoir during large storm events, even while runoff exfiltrates through the soil mantle through the process of infiltration. Outlet structures can be designed to manage peak rates with the use of weir and orifice controls and carefully designed systems may be able to manage peak rates for storms up to and including the 100-year storm.

Water quality improvement

Pervious pavement systems are effective in reducing pollutants such as total suspended solids, metals, and oil and grease. Both the pervious pavement surface and the underlying soils below the infiltration bed allow pollutant filtration.

When pervious pavement systems are designed to capture and infiltrate runoff volumes from small

storm events, they provide very high pollutant reductions because there is little if any discharge of runoff carrying the highest pollutant loads. Pervious pavement systems require pretreatment of TSS when adjacent areas drain to them, resulting in a high reduction of TSS and other particulates. However, pervious pavement systems will provide limited treatment of dissolved pollutants, such as nitrates. Typical ranges of pollutant reduction efficiencies for pervious pavements are listed as follows:

- TSS[#] – 65-100%
- TP – 30-90%
- NO_3 – 30%

[#] - Pretreatment for TSS is recommended if adjacent areas drain to pervious pavement

Construction Guidelines

1. Follow the Recommendations for Materials that are specific to porous asphalt and porous concrete in Recommended Materials Appendix.

2. Due to the nature of construction sites, pervious pavement and other infiltration measures should be installed toward the end of the construction period, if possible. Infiltration beds under pervious pavement may be used as temporary sediment basins or traps provided that they are not excavated to within 12 inches of the designated bed bottom elevation. Once the site is stabilized and sediment storage is no longer required, the bed is excavated to its final grade and the pervious pavement system is installed.

3. The existing subgrade under the bed areas should **not** be compacted or subject to excessive construction equipment traffic prior to geotextile and stone bed placement. (Minor areas of unavoidable compaction can be partially remediated by scarifying the soil; see below.)

   Where erosion of subgrade has caused accumulation of fine materials and/or surface ponding, this material should be removed with light equipment and the underlying soils scarified to a minimum depth of six inches with a York rake (or equivalent) and light tractor. All fine grading should be done by hand. All bed bottoms are level grade.

4. Earthen berms (if used) between infiltration beds may be left in place during excavation. These berms do not require compaction if proven stable during construction.

5. Geotextile and bed aggregate should be placed immediately after approval of subgrade preparation. Geotextile is to be placed in accordance with manufacturer's standards and recommendations.

   Adjacent strips of geotextile should overlap a minimum of 18 inches. It should also be secured at least four feet outside of bed in order to prevent any runoff or sediment from entering the storage bed. This edge strip should remain in place until all bare soils contiguous to beds are stabilized and vegetated. As the site is fully stabilized, excess geotextile along bed edges can be cut back to bed edge.

6. Clean (washed) uniformly graded aggregate is placed in the bed in eight-inch lifts. Each layer should be lightly compacted, with construction equipment kept off the bed bottom. Once bed aggregate is installed to the desired grade, approximately one inch of choker base course crushed aggregate should be installed uniformly over the surface in order to provide an even surface for paving (if required).

7. Cement mix time: Mixtures should be produced in central mixers or in truck mixers. When concrete is delivered in agitating or non-agitating units, the concrete should be mixed in the central mixer for a minimum of 1.5 minutes or until a homogenous mix is achieved. Concrete mixed in truck mixers should be mixed at the speed designated as mixing speed by the manufacturer for 75-100 revolutions.

8. The Portland Cement aggregate mixture may be transported or mixed onsite and should be

used within one hour of the introduction of mix water, unless otherwise approved by an engineer. This time can be increased to 90 minutes when using the specified hydration stabilizer. Each truck should not haul more than two loads before being cycled to another type concrete. Prior to placing concrete, the subbase should be moistened and in a wet condition. Failure to provide a moist subbase will result in reduced strength of the pavement.

9. A minimum of 30 revolutions at the manufacturer's designated mixing speed is required following any water added to the mix. Discharge should be a continuous operation and completed as quickly as possible. Concrete should be deposited as close to its final position as practicable and such that fresh concrete enters the mass of previously placed concrete.

10. Placing and finishing concrete equipment: The contractor should provide mechanical equipment of either slipform or form riding with a following compactive unit that will provide a minimum of 10 psi vertical force. The pervious concrete pavement will be placed to the required cross section and should not deviate more than +/- 3/8 inch in 10 feet from profile grade.

    Placement should be continuous and spreading and strikeoff should be rapid. It is recommended to strike off about ½ to ¾ inch above the forms to allow for compaction. This can be accomplished by attaching a temporary wood strip above the top of the form to bring it to the desired height. After strikeoff, the strips are removed and the concrete is consolidated to the height of the forms.

11. Consolidation should be accomplished by rolling over the concrete with a steel roller, and compacting the concrete to the height of the forms. Consolidation should be completed within 10 minutes of placement to prevent problems associated with rapid hardening and evaporation. After mechanical or other approved strike-off and compaction operation, no other finishing operation is needed. The contractor will be restricted to pavement placement widths of a maximum of 15 feet.

12. Jointing: Control (contraction) joints should be installed at maximum 20-foot intervals. They should be installed at a depth of ¼ the thickness of the pavement. These joints can be installed in the plastic concrete or saw cut. However, installing in the plastic concrete is recommended. Joints installed in the plastic concrete should be constructed using a small roller (salt or joint roller) to which a beveled fin with a minimum depth of ¼ the thickness of the slab has been welded around the circumference of a steel roller. When this option is used it should be performed immediately after roller compaction and prior to curing. If saw cut, the procedure should begin as soon as the pavement has hardened sufficiently to prevent raveling and uncontrolled cracking (normally just after curing).

    Transverse construction joints should be installed whenever placing is suspended a sufficient length of time that concrete may begin to harden. In order to assure aggregate bond at construction joints, a bonding agent suitable for bonding fresh concrete should be brushed, tolled, or sprayed on the existing pavement surface edge. Isolation (expansion) joints will not be used except when pavement is abutting slabs or other adjoining structures.

13. Curing procedures should begin within 15 minutes after placement. The pavement surface should be covered with a minimum six millimeter thick polyethylene sheet or other approved covering material. Prior to covering, a fog or light mist should be sprayed on the surface. The cover should overlap all exposed edges and should be completely secured (without using dirt) to prevent dislocation due to winds or adjacent traffic conditions.

14. Porous asphalt should not be installed on wet surfaces or when the ambient air temperature is below 50 degrees Fahrenheit. The

temperature of the bituminous mix should be determined by the results of the Draindown test (ASTM D6390), but typically ranges between 275 degrees Fahrenheit and 325 degrees Fahrenheit (as determined by the testing and recommendations of the asphalt supplier).

Pervious pavement should be laid in one lift directly over the storage bed and stone base course to a 2.5- to 3-inch finished thickness. Compaction of the surface course should take place when the surface is cool enough to resist a 10-ton roller. One or two passes is all that is required for proper compaction. More rolling could cause a reduction in the surface course porosity.

15. Do not place Portland Cement pervious pavement mixtures when the ambient temperature is 40 degrees Fahrenheit or lower, unless otherwise permitted in writing by the engineer.

16. Mixing, placement, jointing, finishing, and curing do not apply to permeable paver systems. A manual on Permeable Interlocking Concrete Pavements from the Interlocking Concrete Pavement Institute (Smith, 2006) offers detailed guidance on the design and construction of permeable paver systems.

17. After final pervious asphalt or concrete installation, no vehicular traffic of any kind should be permitted on the pavement surface until cooling and hardening or curing has taken place, and not within the first 72 hours (many permeable paver systems can be used right away). The full permeability of the pavement surface should be tested by applying clean water at the rate of at least five gallons per minute over the surface using a hose or other distribution device. All water should infiltrate directly without puddle formation or surface runoff.

Maintenance

The primary goal of pervious pavement maintenance is to prevent the pavement surface and/or underlying infiltration bed from being clogged with fine sediments. To keep the system clean and prolong its life span, the pavement surface should be vacuumed twice per year with a commercial cleaning unit. Pavement washing systems or compressed air units are generally not recommended but may be acceptable for certain types of pavement. All inlet structures within or draining to the infiltration beds should also be cleaned out twice a year.

Planted areas adjacent to pervious pavement should be well maintained to prevent soil washout onto the pavement. If any washout does occur, immediately clean it off the pavement to prevent further clogging of the pores. Furthermore, if any bare spots or eroded areas are observed within the planted areas, they should be replanted and/or stabilized at once. Planted areas should be inspected twice a year. All trash and other litter should be removed during these inspections.

Superficial dirt does not necessarily clog the pavement voids. However, dirt that is ground in repeatedly by tires can lead to clogging. Therefore, trucks or other heavy vehicles should be prevented from tracking or spilling dirt onto the pavement. Furthermore, all construction or hazardous materials carriers should be prohibited from entering a pervious pavement lot.

Potholes in pervious pavement are unlikely, though settling might occur if a soft spot in the subgrade is not removed during construction. For damaged areas of less than 50 square feet, a depression could be patched by any means suitable with standard pavement, with the loss of porosity of that area being insignificant. The depression can also be filled with pervious mix.

If an area greater than 50 sq. ft. is in need of repair, approval of patch type must be sought from either the engineer or owner. If feasible, permeable pavers can be taken up and then simply re-installed (replacing damaged pavers if necessary). **Under no circumstance should the pavement surface ever be seal-coated**. Any required repair of drainage structures should be done promptly to ensure continued proper functioning of the system.

Pervious pavement maintenance considerations are summarized below:

Prevent clogging of pavement surface with sediment

- Vacuum pavement twice a year,
- Maintain planted areas adjacent to pavement,
- Immediately clean any soil deposited on pavement,
- Do not allow construction staging, soil/mulch storage, etc., on unprotected pavement surface, and
- Clean inlets draining to the subsurface bed twice a year.

Snow/Ice removal

- Pervious pavement systems generally perform better and require less treatment than standard pavements,
- Do not apply abrasives such as sand or cinders on or adjacent to pervious pavement,
- Snow plowing is fine but should be done carefully (i.e., set the blade slightly higher than usual), and
- Salt application is acceptable, although alternative deicers are preferable.

Repairs

- Surface should never be seal-coated,
- Damaged areas less than 50 sq. ft. can be patched with pervious or standard pavement,
- Larger areas should be patched with an approved pervious pavement,
- Permeable pavers should be repaired/replaced with similar permeable paver block material, and
- Permeable pavers and gravel pavers may require the addition of aggregate on an annual basis or as needed, in order to replenish material used to fill in the open areas of the pavers. Turf pavers may require reseeding if bare areas appear.

Winter Considerations

Pervious pavement systems should perform equally well in the winter, provided that infiltration bed design considers the soil frost line, and proper snow removal and deicing procedures are followed. Winter maintenance for pervious pavement may be necessary but is sometimes less intensive than that required for a standard pavement (especially for pervious asphalt). The underlying stone bed tends to absorb and retain heat so that freezing rain and snow melt faster on pervious pavement. Therefore, ice and light snow accumulation are generally not as problematic. However, snow will accumulate during heavier storms.

Abrasives such as sand or cinders should not be applied on or adjacent to the pervious pavement. Snow plowing is fine, provided it is done carefully (i.e., by setting the blade slightly higher than usual, about an inch). Salt with low non-soluble solids content is acceptable for use as a deicer on the pervious pavement. Non-toxic, organic deicers applied either as blended, magnesium chloride-based liquid products or as pretreated salt, are preferred.

Cost

The majority of added cost of a pervious pavement/infiltration system lies in the underlying stone bed, which is generally deeper than a conventional subbase and wrapped in geotextile. Costs may also be higher in areas where experienced contractors are not readily available. However, these additional costs are often offset by the significant reduction in the required number of inlets and pipes. Also, since pervious pavement areas are often incorporated into the natural topography of a site, there is generally less earthwork and/or deep excavations involved. Furthermore, pervious pavement areas with subsurface infiltration beds often eliminate the need (and associated costs, space, etc.) for detention basins. When all of these factors are considered, pervious pavement with infiltration has often proven itself less expensive than impervious pavement with associated stormwater management.

- Porous asphalt, with additives, is generally 15 percent to 25 percent higher in cost than standard asphalt on a unit area basis. Unit costs for pervious asphalt (without infiltration bed) range from about $\$4/ft^2$ to $\$5/ft^2$.

- Pervious concrete as a material is generally more expensive than asphalt and requires more labor and expertise to install. Unit cost of a six-inch-thick pervious concrete (without infiltration bed) section is about $\$4/ ft^2$ to $\$6/ ft^2$.

- Permeable paver blocks vary in cost depending on type and manufacturer.

- NOTE: The data provided are based on average market costs. For greater accuracy, a site and market-specific cost estimate should be developed.

Designer/Reviewer Checklist for Pervious Pavement with Infiltration Bed

Type of pervious pavement(s) proposed: _____

Source of mix design or material source: _____

ITEM	Page No.	YES	NO	N/A	NOTES
Appropriate application of pervious pavement (e.g., use, traffic loading, slopes)?	--				
Was the Soil Infiltration Testing Protocol followed?	1				
Appropriate areas of the site evaluated?	--				
Infiltration rates measured?	1				
Was the Infiltration BMP followed?	1				
Two-foot minimum separation between the bed bottom and bedrock/SHWT?	1				
Soil permeability acceptable?	--				
If not, appropriate underdrain provided?	--				
Adequate separations from wells, structures, etc.?	--				
Natural, uncompacted soils?	2				
Level infiltration area (bed bottom)?	1,5				
Excavation in pervious pavement areas minimized?	-				
Hotspots/pretreatment considered?	1,5				
Loading ratio below 5:1?	--				
Storage depth limited to two feet?	--				
Drawdown time less than 48 hours?	5				
Positive overflow from system?	1,5				
Erosion and Sedimentation control?	2				
Feasible construction process and sequence?	--				
Geotextile specified?	2				
Clean, washed, open-graded aggregate specified?	2				
Properly designed/specified pervious pavement surface?	8-10				
Maintenance accounted for and plan provided?	10				
Signage provided?	5				

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BMP Fact Sheet

PLANTER BOXES

Planter boxes receive runoff from multiple impervious surfaces, which is used for irrigation of the vegetation in the planter box preventing stormwater from directly draining into nearby sewers. They also play an important role in urban areas by minimizing stormwater runoff, reducing water pollution, and creating a greener and healthier appearance of the built environment by providing space for plants and trees near buildings and along streets. There are three main types of planter boxes which can be used on sidewalks, plazas, rooftops, and other impervious areas: contained, infiltration, and flow-through.



Figure 1 Streetside planter, Portland, OR (city of Portland, Bureau of Environmental Services)

Variations

- Contained
- Infiltration
- Flow-through

Key Design Features

- May be designed as pretreatment
- May be designed to infiltrate
- Captures runoff to drain out in three to four hours after a storm event
- Receives less than 15,000 ft^2 of impervious area runoff
- Planters should be made of stone, concrete, brick, or pressure-treated wood

Benefits

- Enhances the area where they are placed
- Potential air quality and climate benefits
- Can be used in a wide range of areas, including ultra-urban

Limitations

- Limited stormwater quantity/ quality benefits
- Relatively high cost due to structural components

Potential Applications		Stormwater Quantity Functions	
Residential	Yes	Volume	Low/Med
Commercial	Yes	Groundwater Recharge	Low/Med
Ultra Urban	Yes	Peak Rate	Low
Industrial	Limited	Stormwater Quality Functions	
Retrofit	Yes	TSS	Medium
Highway/Road	No	TP	Medium
Recreational	Yes	TN	Low/Med
		Temperature	Low/Med

Additional Considerations	
Cost	High
Maintenance	Medium
Winter Performance	Medium

Description and Function

Planter boxes receive runoff from multiple impervious surfaces, including rooftops, sidewalks, and parking lots. Runoff is used for irrigation purposes, and the vegetation in the planter box absorbs stormwater and releases it back into the atmosphere through evapotranspiration. Boxes can take any form and can be made out of a variety of materials, although many are constructed from wood.

Construction specifications are critical to ensure that an appropriate volume of runoff from smaller storms "feeds" the carefully selected vegetation types in the boxes; however, consistent watering is necessary during dry periods.

In general, planter boxes must be carefully designed to accommodate the desired amount of runoff. In addition, plantings must be carefully selected, and boxes must be carefully maintained, to accomplish stormwater objectives, and perhaps, most importantly, to succeed in a landowner's overall landscaping objectives.

Stormwater benefits of planter boxes include reduction in runoff volumes and some reduction in peak rates of runoff. Boxes which overflow also effectively reduce peak rates of runoff. Depending on the type of box selected, evapotranspiration will increase along with infiltration and groundwater recharge. Water quality may benefit, depending upon how much runoff is directed into the ground and prevented from worsening erosive stream flows.

When well designed, installed, and maintained, planter boxes are extremely attractive additions to homes, commercial businesses, and office buildings. In fact, an essential objective in developing planter boxes is to enhance overall landscape aesthetics. Boxes are ideal for buffers around structures, foundation plantings, providing seat walls, and for defining walkways, patios, terraces, drives, and courtyards.

Variations

Of all the BMPs listed in this manual, planter boxes are probably the most adaptable to all types of sites with all types of site constraints. The infiltration variation is influenced by all factors which are limiting to any infiltration-oriented BMP (i.e., bedrock/seasonal high water table at or close to the surface, very poorly draining soils, etc., all of which are described in the Infiltration BMP of this manual). However, both the contained and flow-through variations can be used on virtually every type of site - large or small, front yard or backyard, flat or sloping, shady or sunny.

Contained

Contained planter boxes (**Figure 2**) are generally traditional planters that have weep holes to drain excess water from the planter. They effectively reduce impervious area by retaining rainwater which slows stormwater runoff from draining into sewers. Contained planters are used for planting trees, shrubs, perennials, and annuals. The planter is either prefabricated or permanently constructed in a variety of shapes and sizes. Planters are typically placed on impervious surfaces like sidewalks, plazas, and rooftops. Contained planters may drain onto impervious surfaces through their base or into an overflow structure.

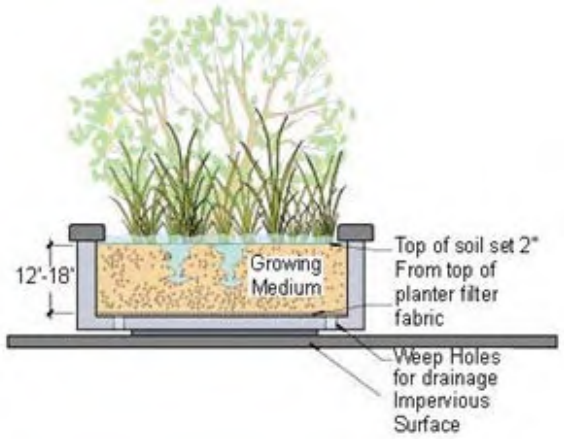


Figure 2 Schematic of Contained Planter Box
Source: City of Portland, OR, Bureau of Environmental Services

Native vegetation should be used in contained planter boxes (Recommended Plant List for BMPs

Appendix). They are hardy and self-sustaining with little need for fertilizers or pesticides. Irrigation needs to be monitored, since plants will need to be watered during dry periods. Sensors can help to regulate moisture in the planter box, ensuring consistent moisture. Smaller trees are highly encouraged because of the canopy and shade they will provide, reducing the urban heat island effect. Planters should be constructed of stone, concrete, brick, wood, or any other suitable material.

This type of planter box can be installed to retrofit an existing urban streetscape or large area of pavement, such as at an entryway to a building.

Infiltration

An infiltration planter box (**Figure 3**) is designed to allow runoff to filter through the planter soils (thus capturing pollutants) and then infiltrate into native soils below the planter. These planters are generally constructed to be flush with surrounding paved areas. The planter is sized to accept runoff and temporarily store the water in a reservoir on top of the soil. Different design variations are encouraged, but should allow a minimum delay in stormwater runoff capture of three to four hours after a wet weather event.

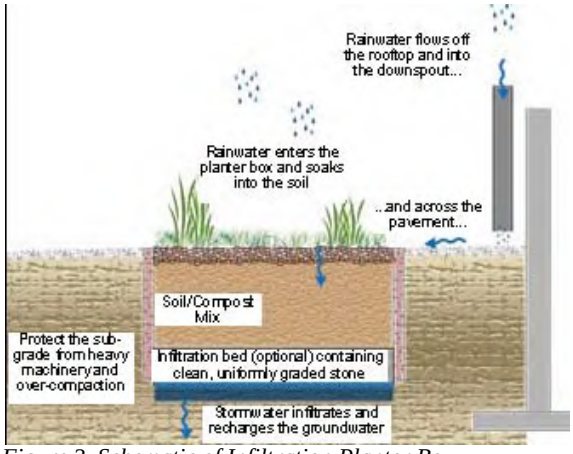


Figure 3 Schematic of Infiltration Planter Box

Recommended vegetation includes native rushes, reeds, sedges, irises, dogwoods, and currants. Also, the dimensions of the sand/gravel area used in these designs should be determined by an engineer and designed to receive less than approximately 15,000 square feet of impervious area runoff. The minimum planter width is typically 30 inches with no minimum length or required shape.

Suggested structural elements of infiltration planter boxes are stone, concrete, brick, or pressure-treated wood. In general, infiltration facilities should be greater than 10 feet from structures and at least five feet from an adjoining property line or as required by local ordinances.

Flow-through

The flow-through planter box (**Figure 4**) is completely contained and drains to a stormwater system. These planters are designed with an impervious bottom or are placed on an impervious surface. Pollutant reduction is achieved as the water filters through the soil/ growing medium. Flow control is obtained by ponding runoff above the soil and in a gravel layer beneath it. In most storm events, runoff flows through the soil into the gravel layer and is slowly discharged via the perforated pipe. In more extreme events, inflow may exceed the capacity of the soil and some runoff may be discharged through surface overflow. This type of planter can be used adjacent to a building if the planter box and/or building are adequately waterproofed to allow for saturated soil and temporary ponded runoff next to the building.

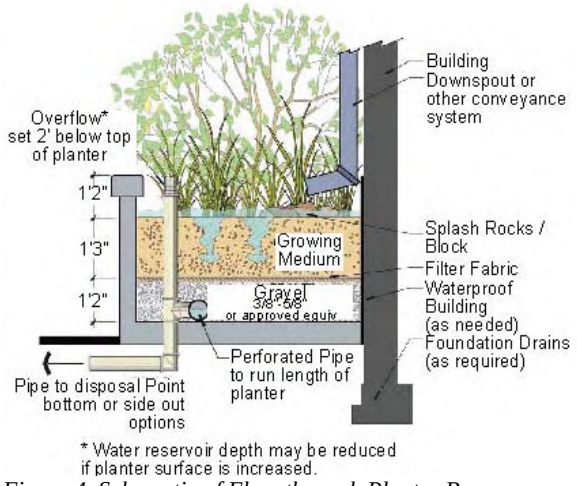


Figure 4 Schematic of Flow-through Planter Box

Flow-through planter boxes should be designed to retain water for no more than three to four hours after an average storm event. Recommended vegetation includes native rushes, reeds, sedges, irises, dogwoods, and currants. The minimum planter width is typically 18 inches with no minimum length or required shape. Planters should be designed to receive less than 15,000 square-feet of impervious area runoff.

Potential Applications

Planter boxes can be used in urbanized areas of high pollutant loads. They are especially applicable where there is limited area for construction of other BMPs. Planter boxes may be used as a pretreatment BMP for other BMPs such as wet ponds or infiltration systems. Areas that would benefit from using a planter box include:

- Parking garage
- Office building
- Residential building
- Other building use (commercial, light industrial, institutional, etc.)
- Transportation facilities
- Urban streetscapes

Design Considerations

- **Suggested structural elements** of planters are stone, concrete, brick, or pressure-treated wood. Flow-through planters are completely contained and, therefore, not designed to drain directly into the ground. Pipes can also be designed to transport water to an approved disposal point. It is recommended that planter boxes have setback distances of 10 feet from structures and five feet from property lines, unless the planter height is less than 30 inches or as required by local ordinances.

- The **flow entrance/inflow** must be designed to prevent erosion in the planter box. Some alternatives include gravel, splash blocks, perforated pipe, and erosion control mats.

- A **positive overflow system** should be designed to safely convey away excess runoff. The overflow can be routed to the surface in a non-erosive manner or to another stormwater system. Some alternatives include domed risers, inlet structures, weirs, and openings in the planter box wall.

- **Planting soil** should be capable of supporting a healthy vegetative cover and should generally be between 12 and 36 inches deep. Planting soil should be approximately four inches deeper than the bottom of the largest root ball.

- A subsurface **gravel layer**, if used, should be at least six inches thick and constructed of clean gravel with a significant void space for runoff storage (typically 40 percent) and wrapped in geotextile (filter) fabric.

- If used, **underdrains** are typically small diameter (412 inches) perforated pipes in a clean gravel trench wrapped in geotextile fabric (or in the gravel layer). Underdrains should be double-walled and have a flow capacity capable of draining the planter box system in approximately 12 hours. They can daylight to the surface or connect to another stormwater system. A way to inspect and clean underdrains should be provided (via cleanouts, inlet, overflow structure, etc.)

- **Native trees and shrubs** may require irrigation during dryer summer months to remain healthy. Monitoring vegetation in planter boxes is critical to the health of the plants, as they may need supplemental watering, in addition to the water received from storms.

- **Many planter box styles and sizes** are used to improve site aesthetics and stormwater management. Incorporating smaller planter boxes over the site adds visual appeal and a greater surface area.

Design variations:

- **Contained boxes**

  Plants should be relatively self-sustaining, with little need for fertilizers or pesticides. Irrigation is optional, although plant viability should be maintained. Trees are encouraged and will receive added runoff reduction recognitions for the canopy that will extend beyond the planter walls. Structural elements of the planters should be stone, concrete, brick, wood, or other durable material. Treated wood that may leach out any toxic chemicals should not be used.

- **Infiltration**

  Allow captured runoff to drain out in three to four hours after a storm event. The sand/ gravel area width, depth, and length are to be determined by an engineer or a dry well may be required for complete onsite infiltration. Planters should be designed to receive less than 15,000 square-feet of impervious area runoff. Minimum planter width is 30 inches; there is no minimum length or required shape. The structural elements of the planters should be stone, concrete, brick, or pressure-treated wood. Treated wood that may leach out any toxic chemicals should not be used.

- **Flow-through**

  Allow captured runoff to drain out in three to four hours after a storm event. Minimum planter width is 18 inches; there is no minimum length or required shape. Planters should be designed to receive less than 15,000 square-feet of impervious area runoff. Structural elements of the planters should be stone, concrete, brick, or pressure-treated wood. Treated wood that may leach out any toxic chemicals should not be used. The flow-through planter box is contained and, thus, not designed to drain into the ground near a building. Irrigation is optional, although plant viability should be maintained.

- The plants within the **perimeter planter boxes** are designed to accept stormwater runoff from adjacent impervious areas. Plants and vegetation absorb most of the water volume. Overflow gradually drains to the surface, which slows the peak rates.

- Review the materials list in Recommended Materials Appendix for recommended planter box specifications.

- **Landscaping requirements**

  The following quantities are recommended per 100 square feet of planter box area:

  - Four large shrubs/small trees in three-gallon containers or equivalent.
  - Six shrubs/large grass-like plants in one-gallon containers or equivalent
  - Ground cover plants (perennials/ annuals) one per 12 inches on center, triangular spacing. Minimum container: four-inch pot. Spacing may vary according to plant type.

- Plantings can include rushes, reeds, sedges, iris, dogwood, currants, and numerous other shrubs, trees, and herbs/grasses (Recommended Plant List for BMPS Appendix).

- Container planting requires that plants be **supplied with nutrients** that they would otherwise receive from being part of an ecosystem. Since they are cut off from these processes, they must be cared for accordingly.

- **Tree planting** in planters is encouraged where practical. Tree planting is also encouraged near planters.

- Generally, plants requiring **moist-wet conditions** are preferred for flow-through planters.

Stormwater Functions and Calculations

Volume reduction

If a planter box is designed to infiltrate, the volume reduction is a function of the area of the filter and the infiltration rate. There is generally less volume reduction for planter boxes that are not designed to infiltrate.

Infiltration Volume[#] = Bottom Area (ft^2) * Infiltration Rate (in/hr) * Drawdown time[##] (hr)

[#] - For filters with infiltration only
[##] - Not to exceed 3-4 hours

Peak rate mitigation

Planter boxes generally provide little, if any, peak rate reduction. However, if the planter box is designed to infiltrate, then a modest level of peak rate attenuation can be expected.

Water Quality Improvement

Planter boxes are considered a moderate stormwater treatment practice with the primary pollutant removal mechanism being filtration and settling. Less significant processes can include evaporation, infiltration (if applicable), transpiration, biological and microbiological uptake, and soil adsorption. The extent to which planter boxes remove pollutants in runoff is primarily a function of their design, configuration, plant species/ density, and soil type.

For planter boxes that are also designed to infiltrate, see the water quality summary in the Subsurface Infiltration Bed section of Infiltration Practices Fact Sheet, or in the other infiltration BMP sections. For manufactured planters, see the manufacturer's information, as well as findings from independent studies.

Construction Guidelines

Constructing or retrofitting planter boxes varies in difficulty at each site. Boxes may be ideal for inclusion in patio or walkway design and integrate easily with roof downspouts. In most cases, a landscape architect is essential, especially if the more complex infiltration and flow through variation is being constructed, and as the size/scale of the planter box grows larger.

1. Areas for planter boxes, especially the infiltration type, should be clearly marked before any site work begins to avoid soil disturbance and compaction during construction.

2. Planter boxes should generally be installed after the site is stabilized. Excessive sediment generated during construction can clog the planter and prevent or reduce the anticipated post-construction water quality benefits. Stabilize all contributing areas before runoff enters the filters.

3. Structures such as inlet boxes, reinforced concrete boxes, etc. should be installed in accordance with the guidance of the manufacturers or design engineer.

4. Infiltration planter boxes should be excavated in such a manner as to avoid compaction of the subbase. Structures may be set on a layer of clean, lightly compacted gravel (such as AASHTO #57).

5. Infiltration planter boxes should be underlain by a layer of permeable, nonwoven-geotextile.

6. Place underlying gravel/stone in minimum six-inch lifts and lightly compact. Place underdrain pipes in gravel.

7. Wrap and secure nonwoven geotextile to prevent gravel/stone from clogging with sediments.

8. Install planting soil per the recommendations of the landscape architect. Do not compact.

9. Install native vegetation (trees, shrubs, etc.) per the recommendations of the landscape architect.

Maintenance

Planter boxes are relatively high maintenance, as is the case with any containerized garden. Property owners should be especially prepared for maintaining the vegetation itself, which will vary depending upon planting. In many cases, planter boxes may need additional watering during extremely dry periods. Selection of planter box construction material is also important (e.g., masonry construction is easier to maintain than wood construction).

Generally speaking, stormwater facilities need an adequate amount of space for proper maintenance. The minimum required width for maintenance is typically eight feet and the maximum slope is 10 percent. If structural surfaces need to support maintenance vehicles, access routes should be constructed of gravel or other permeable paving surface.

Winter Considerations

Indiana's winter temperatures can go below freezing for a few months every year and surface filtration may not take place in the winter. Winterizing becomes an important issue in plant species selection, especially for larger, hardy or nearly hardy species intended to winter over. In these cases, planter boxes must be designed and dimensioned so that plantings are adequately protected.

Depending on the composition of the planting soil, it may hold water, freeze, and become impervious on the surface. Design options that allow directly for subsurface discharge into the underlying infiltration bed, if applicable, during cold weather may overcome this condition, but at the possible expense of surface filtration.

Cost

Costs for planter boxes are quite modest. However, based on unit cost of cubic foot or gallons of runoff being managed, costs tend to be rather high. Because of the extreme variability of design and construction, costs will range based on the goals of the designer. Smaller boxes with smaller-scale vegetation will be less expensive than larger boxes with more mature vegetation.

Designer/Reviewer Checklist for Planter Boxes

ITEM	Page No.	YES	NO	N/A	NOTES
For infiltration planters, was Soil Testing Infiltration Protocol (Soil Infiltration Testing Protocol Appendix) followed?*	--				
Appropriate areas of the site evaluated?	--				
Infiltration rates measured?	--				
For infiltration planters, was the Infiltration BMP followed?*	--				
Two-foot separation between the bed bottom and bedrock/ seasonally high water table?	--				
Soil permeability acceptable?	--				
If not, appropriate underdrain provided?	-				
Natural, uncompacted soils?	--				
Excavation in infiltration areas minimized?	--				
Drawdown time less than 48 hours?	--				
Erosion and sedimentation control?	--				
Adequately stable inflow point(s)?	4				
Positive overflow from system?	4				
Waterproofing provided, as necessary?	3				
Acceptable soil/growing medium specified? ➢ *Between 12 and 36 inches deep; 4 inches deeper than bottom of largest root ball*	4				
Gravel layer specified properly? ➢ *At least 6 inches thick*	4				
Underdrain positioned and sized? ➢ *Should be sized to drain within approximately 12 hours*	4				
Appropriate native plants selected?	3-5				
Feasible construction process and sequence?	--				
Maintenance accounted for and plan provided? ➢ *Minimum maintenance width = **8 ft**; max. width = **10 ft***	6,7				

➢ *Denotes Minimum Design Considerations*

** In general, the Protocol and Infiltration BMP should be followed as much as possible (although there is more flexibility for infiltration planters than for other BMPs such as pervious pavement and subsurface infiltration that rely almost entirely on infiltration).*

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Stormwater Management Guidance Manual, Version 2.0. Office of Watersheds, Philadelphia, PA: Water Department. www.phillyriverinfo.org/programs/SubProgramMain.aspx?Id=StormwaterManual

Stormwater Management Manual, Revision 3. Portland, OR: Environmental Services, Clean River Works, September 2004. www.portlandonline.com

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BMP Fact Sheet

RAINWATER HARVESTING

Structures designed to intercept and store runoff from rooftops allow for its reuse, reducing volume and overall water quality impairment. Stormwater is contained in the structures and typically reused for irrigation or other water needs.



Figure 1 Above ground cistern, Chicago, IL (USEPA, picasaweb)

Applications		Stormwater Quantity Functions	
Residential	Yes	Volume	High
Commercial	Yes	Groundwater Recharge	Low
Ultra Urban	Yes	Peak Rate	Low*
Industrial	Yes	Stormwater Quality Functions	
Retrofit	Yes	TSS	Med
Highway/Road	No	TP	Med
Recreational	Yes	NO_3	Med
		Temperature	Med

* *Depends on site design*

Additional Considerations	
Cost • Rain Barrel • Cistern • Manufactured product	 Low Med Varies
Maintenance	Med
Winter Performance	Med

Variations

- Rain barrels
- Cisterns, both underground and above ground
- Tanks
- Storage beneath a surface (using manufactured products)

Key Design Features

- Small storm events are captured with most structures
- Provide overflow for large storm events
- Discharge water before next storm event
- Consider site topography, placing structure up-gradient in order to eliminate pumping needs

Site Factors

- Water table to bedrock depth: N/A (although must be considered for subsurface systems
- Soils: N/A
- Slope: N/A
- Potential hotspots: Yes, with treatment
- Maximum drainage area: N/A

Benefits

- Provides supplemental water supply
- Wide applicability
- Reduces potable water use
- Related cost savings and environmental benefits

Limitations

- Manages only relatively small storm events which requires additional management and use for the stored water

Description and Function

Rainwater harvesting is the practice of collecting rainwater in a container and reusing it in the future. Other terms for this BMP include *storage/reuse, rainwater harvesting, and rainwater catchment system.*

This structural BMP reduces potable water needs while simultaneously reducing stormwater discharges. When rain barrels or cisterns are full, rooftop runoff should be directed to drywells, planters, or bioretention areas where it will be infiltrated.

Variations

Rain Barrels

Commonly, rooftop downspouts are connected to a rain barrel that collects runoff and stores water until needed for a specific use. Rain barrels are often used at individual homes where water is reused for garden irrigation, including landscaped beds, trees, or other vegetated areas. Other uses include commercial and institutional facilities where the capacity of stormwater can be captured in smaller volume rain barrels.

Cisterns

A cistern is a container or tank that has a greater storage capacity than a rain barrel. Typically, cisterns are used to supplement greywater needs (i.e., toilet flushing, or some other sanitary sewer use), though they can also be used for irrigation. Cisterns may be comprised of fiberglass, concrete, plastic, brick, or other materials and can be located either above or below ground. The storage capacity of cisterns can range from 200 gallons to 10,000 gallons. Very large cisterns, essentially constructed like an underground parking level, can also be used. **Figure 2** highlights the typical components of a cistern.



Figure 2 Typical cistern components
Source: This image generously provided by www.rainkeeper.us

Figure Description:

1. Filter/screening mechanism to filter runoff
2. Inflow into cistern
3. Intake for water use
4. Cistern overflow
5. Subsequent stormwater system (infiltration system in this case) for cistern overflow
6. Optional level gauge

Vertical storage

A vertical storage container is a structure designed to hold a large volume of stormwater drained from a large impervious area and is the largest of the capture reuse containers. The use of these structures is a function of drainage area and water needs. Vertical structures are best used for intensive irrigation needs or even fire suppression requirements, and should be designed by a licensed professional. These storage systems can be integrated into commercial sites where water needs may be high.

Storage beneath structure

Stormwater runoff can be stored below ground under pavement and landscaped surfaces through the use of structural plastic storage units and can supplement onsite irrigation needs. These structures can provide large storage volumes without the need for additional structural support from the building.

Designing a rainwater harvesting system in which the storage unit is underground is best used in institutional or commercial settings. This type of subsurface storage is larger, more elaborate, typically designed by a licensed professional, and requires pumps to connect to the irrigation system.

Applications

Rainwater harvesting containers can be used in urbanized areas where the need for supplemental onsite irrigation or other high water use exists. Areas that would benefit from using a capture reuse container include:

- Parking garage,
- Office building,
- Residential home or building, and
- Other building use (commercial, light industrial, institutional, etc.).

Design Considerations

Design and installation procedures for capture reuse containers can vary from simple residential rain barrels to highly engineered underground systems in ultra-urban areas. **Table 1** provides general information on cistern holding capacity. The following procedures should be considered when designing sites with capture reuse containers.

Height (feet)	6-foot Diameter	12-foot Diameter	18-foot Diameter
6	1,269	5,076	11,421
8	1,692	6,768	15,227
10	2,115	8,460	19,034
12	2,538	10,152	22,841
14	2,961	11,844	26,648
16	3,384	13,535	30,455
18	3,807	15,227	34,262
20	4,230	16,919	38,069

Table 1 Round Cistern Capacity (Gallons)
Source: The Texas Manual on Rainwater Harvesting

1. Identify opportunities where water can be reused for irrigation or indoor greywater reuse and then calculate the water need for the intended uses. For example, if a 2,000 square foot landscaped area requires irrigation for four months in the summer at a rate of one inch per week, the designer must determine how much water will be needed to achieve this goal (1,250 gallons per week, approximately 22,000 gallons for the season), and how often the storage unit will be refilled with precipitation. The usage requirements and the expected rainfall volume and frequency must be determined.

2. Rain barrels and cisterns should be positioned to receive rooftop runoff.

3. If cisterns are used to supplement greywater needs, a parallel conveyance system must be installed to separate greywater from other potable water piping systems. Do not connect to domestic or commercial potable water system.

4. Consider household water demands (**Table 2**) when sizing a system to supplement residential greywater use.

Fixture	Use	Flow Rate
Toilet	# flushes per person per day	1.6 gallons per flush (new toilet)
Shower	# minutes per person per day (5 minutes suggested max.)	2.75 gallons per minute (restricted flow head)
Bath	# baths per person per day	50 gallons per bath (average)
Faucets	Bathroom and kitchen sinks	10 gallons per day
Washing Machine	# loads per day	50 gallons per load (average)
Dishwasher	# loads per day	9.5 gallons per load

Table 2 Household Water Demand Chart
Source: Philadelphia Stormwater Manual

5. Discharge points and storage units should be clearly marked "Caution: Untreated Rainwater, Do Not Drink."

6. Screens should be used to filter debris from runoff flowing into the storage units. Screens should be made of a durable, non-corrodible material and be easily maintainable.

7. Protect storage elements from direct sunlight by positioning and landscaping. Limit light into devices to minimize algae growth.

8. The proximity to building foundations should be considered for overflow conditions. The minimum setback distance for capture and reuse systems is 10 feet.

9. If the capture and reuse system or any elements of the system are exposed to freezing temperatures, then it should be emptied during the winter months to prevent ice damage.

10. Cisterns should be watertight (joints sealed with nontoxic waterproof material) with a smooth interior surface.

11. Covers and lids should have a tight fit to keep out surface water, insects (mosquitoes), animals, dust, and light.

12. Release stored water between storm events for the necessary storage volume to be available.

13. Positive outlet for overflow should be provided a few inches from the top of the cistern and sized to safely discharge the appropriate design storms when the cistern is full.

14. Rain barrels require a release mechanism in order to drain empty between storm events. Connect a soaker hose to slowly release stored water to a landscaped area.

15. Observation risers should be at least six inches above grade for buried cisterns.

16. Reuse may require pressurization. Water stored has a pressure of 0.43 psi per foot of water elevation. A 10-foot tank when full would have a pressure of 4.3 psi (0.43*10). Most irrigation systems require at least 15 psi. To add pressure, a pump, pressure tank, and fine mesh filter can be used. While this adds to the cost of the system, it makes the system more versatile and therefore practical.

17. Capture/reuse can also be achieved using a subsurface storage reservoir which provides temporary storage of stormwater runoff for reuse. The stormwater storage reservoir may consist of clean uniformly graded aggregate and a waterproof liner or pre-manufactured structural stormwater storage units.

Stormwater Functions and Calculations

Volume reduction

In order to keep storage costs to a minimum, it makes sense to size the storage tank so that it does not greatly exceed the water need. Where this is done, especially where a high-volume demand greatly exceeds runoff (e.g., irrigation or industrial makeup water), runoff volume

reduction for a particular storm can be assumed to equal the total volume of storage.

Where the captured water is the sole source for a particular operation (e.g., flushing toilets), the user does not want the stored water to be depleted before the next runoff event that replenishes it. In that case, the appropriate volume to store will be relatively easy to calculate based on the daily water need. After water need is determined, choose which structure will be large enough to contain the amount of water needed. The amount replenished by a particular storm is equal to the volume reduction.

Additional Volume Reduction Considerations

For storage vessels that are not drained down completely before the next runoff event, the volume available to be filled by a particular storm may be difficult to calculate. Typical LID/GI sizing criteria is based on the volume that goes to storage during a particular storm. That volume can be subtracted from the runoff volume, and the designer/developer can size the storage unit to achieve the targeted volume reduction. But sizing criteria under these capture and reuse circumstances may become need based. The designer/builder may estimate the volume removal for a particular storm, but estimates should be realistic given the use rate and storm runoff frequency. The estimate can be based on an average available storage capacity or preferably on a water balance analysis based on actual rainfall statistics.

Available Volume for Capture (gallons) = Runoff Coefficient (unitless) * Precipitation (inches) * Area (ft^2) * 1 foot/12 inches * 7.4805 gallons/ft^3

OR

$$V = 0.62 * C * P * A$$

Where:

V = available volume for capture (gallons)
0.62 = unit conversion (gal/in./ft^2)
C = volumetric runoff coefficient (unitless), typically 0.9 to 0.95 for impervious areas
P = precipitation amount (inches)
A = drainage area to cistern (ft^2)

Sizing the tank is a mathematical exercise that balances the available collection (roof) area, annual rainfall, intended use of rainwater and cost. In other words, balance what can be collected against how the rainwater will be used and the financial and spatial costs of storing it. In most areas of the country, it's possible to collect 80 percent of the rain that falls on the available roof area. The 20 percent reduction accounts for loss due to mist and heavy storms that release more rain than the tank can accommodate. That level of capture would yield approximately 500 gallons per inch of rain per 1000 SF of capture area. **Table 3** includes available capture volumes based on drainage area and annual rainfall.

Note: Although utilization of Rainwater Harvesting BMP is beneficial and encouraged, no runoff reduction recognition is considered for this practice towards the required Channel Protection Volume or Water Quality Volume discussed under LID/GI Approach in Post-Construction Stormwater Quality Management Chapter of Technical Standards.

Peak rate mitigation

Overall, capture and reuse takes a volume of water out of site runoff and puts it back into the ground. This reduction in volume will translate to a lower overall peak rate for the site.

Water quality improvement

Pollutant removal takes place through filtration of recycled primary storage, and/or natural filtration through soil and vegetation for overflow discharge. Quantifying pollutant removal will depend on design. Sedimentation will depend on the area below the outlet that is designed for sediment accumulation, time in

Annual Rainfall Yield in Gallons for Various Impervious Surface Sizes and Rainfall Amounts								
Impervious Surface Area (ft^2)	Rainfall (inches)							
	26	28	30	32	34	36	38	40
200	3,079	3,316	3,553	3,790	4,027	4,264	4,501	4,738
400	6,159	6,633	7,106	7,580	8,054	8,528	9,002	9,475
600	9,238	9,949	10,660	11,370	12,081	12,792	13,502	14,213
800	12,318	13,265	14,213	15,160	16,108	17,056	18,003	18,951
1,000	15,397	16,582	17,766	18,951	20,135	21,319	22,504	23,688
1,200	18,477	19,898	21,319	22,741	24,162	25,583	27,005	28,426
1,400	21,556	23,214	24,873	26,531	28,189	29,847	31,505	33,164
1,600	24,636	26,531	28,426	30,321	32,216	34,111	36,006	37,901
1,800	27,715	29,847	31,979	34,111	36,243	38,375	40,507	42,639
2,000	30,795	33,164	35,532	37,901	40,270	42,639	45,008	47,377
2,200	33,874	36,480	39,086	41,691	44,297	46,903	49,508	52,114
2,400	36,954	39,796	42,639	45,481	48,324	51,167	54,009	56,852
2,600	40,033	43,113	46,192	49,272	52,351	55,431	58,510	61,589
2,800	43,113	46,429	49,745	53,062	56,378	59,694	63,011	66,327
3,000	46,192	49,745	53,299	56,852	60,405	63,958	67,512	71,065
3,200	49,272	53,062	56,852	60,642	64,432	68,222	72,012	75,802
3,400	52,351	56,378	60,405	64,432	68,459	72,486	76,513	80,540
3,600	55,431	59,694	63,958	68,222	72,486	76,750	81,014	85,278
3,800	58,510	63,011	67,512	72,012	76,513	81,014	85,515	90,015

Table 3 Annual Rainfall yield (in gallons) for impervious surfaces

storage, and maintenance frequency. Filtration through soil will depend on flow draining to an area of soil, the type of soil (infiltration capacity), and design specifics (stone bed, etc.).

Maintenance

Rain Barrels

- Inspect rain barrels four times per year, and after major storm events.
- Remove debris from screen as needed.
- Replace screens, spigots, downspouts, and leaders as needed.
- To avoid damage, drain container prior to winter, so that water is not allowed to freeze in devices.

Cisterns

- Flush cisterns annually to remove sediment.
- Brush the inside surfaces and thoroughly disinfect twice per year.
- To avoid damage, drain container prior to winter, so that water is not allowed to freeze in devices.

Cost

Both rain barrels and cisterns are assumed to have a life span of 25 years. **Table 4** shows typical costs for rain barrels and cisterns.

	Capacity	Cost Range
Rain barrel	40-75 gal.	$100-$250
Cistern	200-10,000 gal.	Varies by manufacturer and material
Vertical storage	64-12,000 gal	$100-$11,000

Table 4 Capacities and Costs

Designer/Reviewer Checklist for Rainwater Harvesting

Type and size (gallons) of storage system provided: _____

ITEM*	Page No.	YES	NO	N/A	NOTES
Capture area defined and calculations performed?	5,6				
Pretreatment provided to prevent debris/sediment from entering storage system?	4				
Water use identified and calculations performed?	3,4				
If the use is seasonal, has off-season operation been considered?	--				
Draw-down time considered?	--				
Is storage system located optimally for the use?	--				
Is a pump required?	--				
If so, has an adequate pump system been developed?	4				
Acceptable overflow provided?	4				
Winter operation (protection from freezing) considered?	4,6				
Observation/clean-out port provided?	4				
Maintenance accounted for and plan provided?	6				

** These items primarily relate to larger systems, not residential rain barrels*

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BMP Fact Sheet

VEGETATED FILTER STRIP

A vegetated filter strip is a permanent, maintained strip of vegetation designed to slow runoff velocities and filter out sediment and other pollutants from urban stormwater. Filter strips require the presence of sheet flow across the strip, which can be achieved through the use of level spreaders. Frequently, filter strips are designed where runoff is directed from a parking lot into a stone trench, a grass strip, and a longer naturally vegetative strip.



Figure 1 Gravel trench directing water from the parking lot to a vegetated filter strip in Springfield, OR (Springfield Environmental Services Division)

Applications		Stormwater Quantity Functions	
Residential	Yes	Volume	Low
Commercial	Yes	Groundwater Recharge	Low
Ultra Urban	Limited*	Peak Rate	Low
Industrial	Limited*	Stormwater Quality Functions	
Retrofit	Yes	TSS	Med/High
Highway/Road	Yes	TP	Med/High
Recreational	Yes	NO_3	Med/High
		Temperature	Med/High

* According to site characteristics

Additional Considerations	
Cost	Low
Maintenance	Low/Medium Varies dependent on type of vegetation
Winter Performance	High

Variations

- Turf grasses
- Prairie grasses, shrubs, and groundcover vegetation, including trees
- Indigenous woods and dense vegetation

Key Design Features

- Use with level spreaders to promote sheet flow across strips
- Longitudinal slope from 1-6 percent
- Maintain dense vegetation

Site Factors

- Water table to bedrock depth: N/A
- Soils: N/A for permeability
- Slope: 2-5 percent preferred (1-10 percent if soils/vegetation allow)
- Potential hotspots: Yes with special design considerations
- Max. drainage area: 100 feet impervious or 150 feet pervious up gradient

Benefits

- Low cost
- Good water quality performance
- Aesthetic and habitat benefits

Limitations

- Generally should be coupled with other BMPs for comprehensive stormwater management

Description and Function

Filter strips (**Figure 2**) are gently sloping areas that combine a grass strip and dense vegetation to filter, slow, and infiltrate sheet flowing stormwater. Filter strips are best used to treat runoff from roads and highways, roof downspouts, small parking lots, and other impervious surfaces. They are generally not recommended as stand-alone features, but as pretreatment systems for other BMPs, such as infiltration trenches or bioretention areas. Therefore, filter strips generally should be combined with other BMPs as part of a treatment train so that water quality and quantity benefits are sufficient to meet recommended site design criteria.

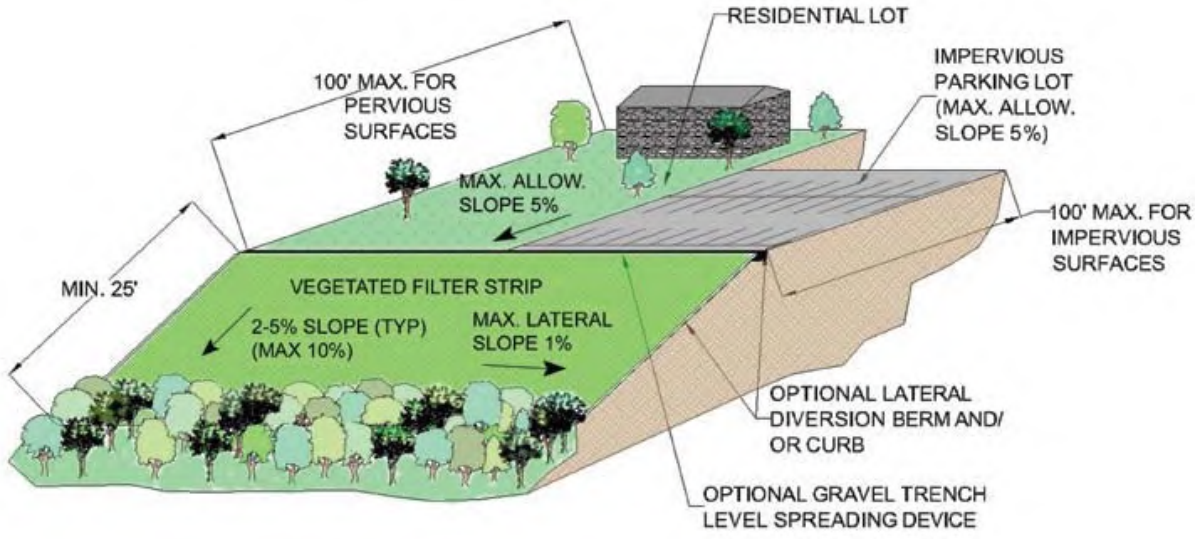


Figure 2 Diagram showing elements of a vegetated filter strip

Maintaining a dense growth pattern that includes turf-forming grasses and vegetation on a filter strip is critical for maximizing pollutant removal efficiency and erosion prevention.

The grass portion of the filter strip provides a pretreatment of the stormwater before it reaches the densely vegetated, or wooded area. In addition, a stone drop can be located at the edge of the impervious surface to prevent sediment from depositing at this critical entry point.

In addition to a stone drop, a pervious berm can reduce runoff velocity and increase volume reduction by providing a temporary, shallow ponded area for the runoff. The berm should have a height of not more than six to 12 inches and be constructed of sand, gravel, and sandy loam to encourage growth of a vegetative cover.

An outlet pipe(s) or an overflow weir may be provided and sized to ensure that the area drains within 24 hours or to allow larger storm events to pass. The berm must be erosion resistant under the full range of storm events. Likewise, the ponded area should be planted with vegetation that is resistant to frequent inundation.

Filter strips are primarily designed to reduce total suspended solids (TSS) levels. However, pollutants such as hydrocarbons, heavy metals, and nutrients may also be reduced. Pollutant removal mechanisms include sedimentation, filtration, absorption, infiltration, biological uptake, and microbial activity. Depending on soil properties, vegetative cover type, slope, and length of the filter strip, a reduction in runoff volume may also be achieved by infiltration.

Applications

Vegetated filter strips can be used in a wide variety of applications from residential/commercial developments to industrial sites and even transportation projects where the required space is available. Lack of available space limits use in ultra urban areas and some redevelopment projects.

Design Considerations

1. The design of vegetated filter strips is determined by existing drainage area conditions including drainage area size, length, and slope. In addition, the filter strip soil groups, proposed cover type, and slope needs to be determined. This information is used to determine the length of the filter strip using the appropriate graph (Figures 5 through 9).

2. Level spreading devices are highly recommended to provide uniform sheet flow conditions at the interface of the adjacent site area and the filter strip (see Level Spreader BMP Fact Sheet for detailed information). Concentrated flows should not be allowed to flow onto filter strips, as they can lead to erosion and, thus, failure of the system. Examples of level spreaders include:

    a. A gravel-filled trench (**Figure 3**), installed along the entire up-gradient edge of the strip. The gravel in the trenches may range from pea gravel (ASTM D 448 size no. 6, 1/8" to 3/8") for most cases to shoulder ballast for roadways. Trenches are typically 12" wide, 24-36" deep, and lined with a nonwoven geotextile. When placed directly adjacent to an impervious surface, a drop (between the pavement edge and the trench) of 1-2" is recommended, in order to inhibit the formation of the initial deposition barrier.

    b. A concrete curb stop with cutouts (**Figure 4**) can be used to provide uniform sheet flow across a vegetated filter strip.

    c. Concrete sill (or lip).

    d. An earthen berm (**Figure 5)** with optional perforated pipe.

3. Where possible, natural spreader designs and materials, such as earthen berms, are generally recommended, though they can be more susceptible to failure due to irregularities in berm elevation and density of vegetation. When it is desired to treat runoff from roofs or curbed impervious areas, a more structural approach, such as a gravel trench, is required. In this case, runoff should be directly conveyed, via pipe from downspout or inlet, into the subsurface gravel and uniformly distributed by a perforated pipe along the trench bottom.

4. The upstream edge of a filter strip should be level and directly abut the contributing drainage area.

5. In areas where the soil infiltration rate has been compromised (e.g., by excessive compaction), the filter strip should be tilled prior to establishing vegetation. However, tilling will only have an effect on the top 12-18 inches of the soil layer. Therefore, other measures, such as planting trees and shrubs,

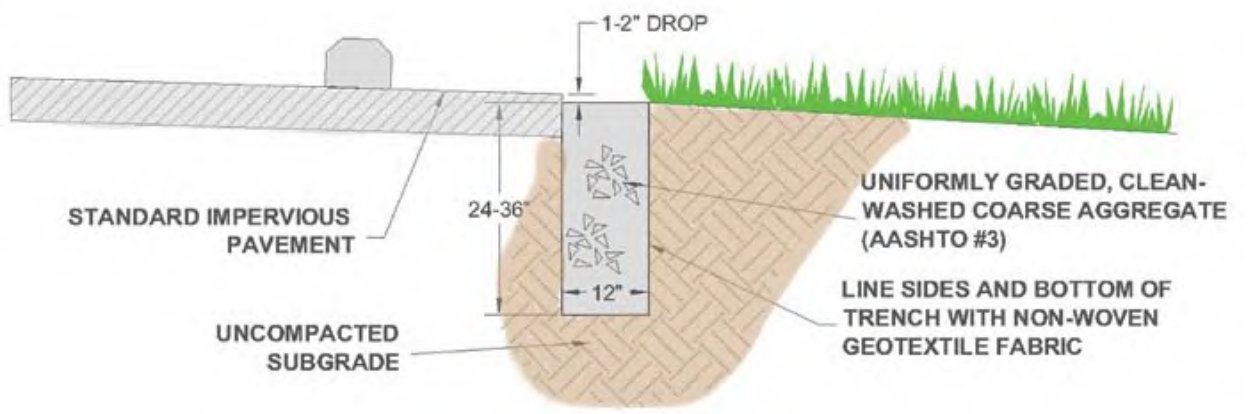


Figure 3 A Level Spreader device (gravel-filled trench)

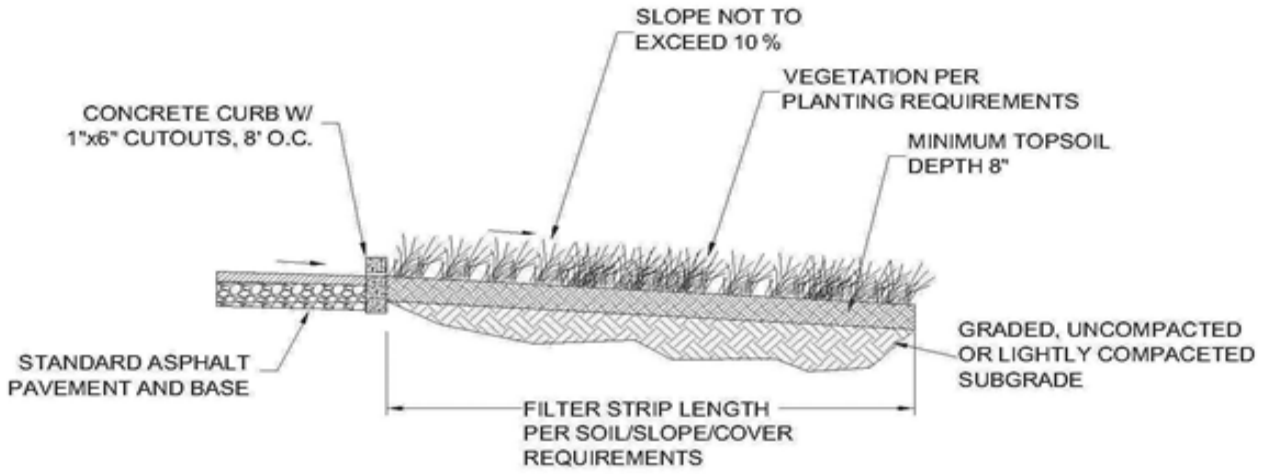


Figure 4 Concrete curb stop schematic

may be needed to provide deeper aeration. Deep root penetration will promote greater absorptive capacity of the soil.

6. The ratio of contributing drainage area to filter strip area should never exceed 6:1.

7. The filter strip area should be densely vegetated with a mix of salt-tolerant, drought-tolerant, and erosion-resistant plant species. Filter strip vegetation, whether planted or existing, may range from turf and native grasses to herbaceous and woody vegetation. The optimal vegetation strategy consists of plants with dense growth patterns, a fibrous root system for stability, good regrowth ability (following dormancy and cutting), and adaptability to local soil and climatic conditions. Native vegetation is always preferred. (See Recommended Plant List for BMPs Appendix for vegetation recommendations.)

8. Natural areas, such as forests and meadows, should never be unduly disturbed when creating a filter strip. If these areas are not already functional as natural filters, they may be enhanced by restorative methods or by constructing a level spreader.

9. The maximum lateral slope of a filter strip is one percent.

10. To prohibit runoff from laterally bypassing a strip, berms and/or curbs can be installed along the sides of the strip, parallel to the direction of flow.

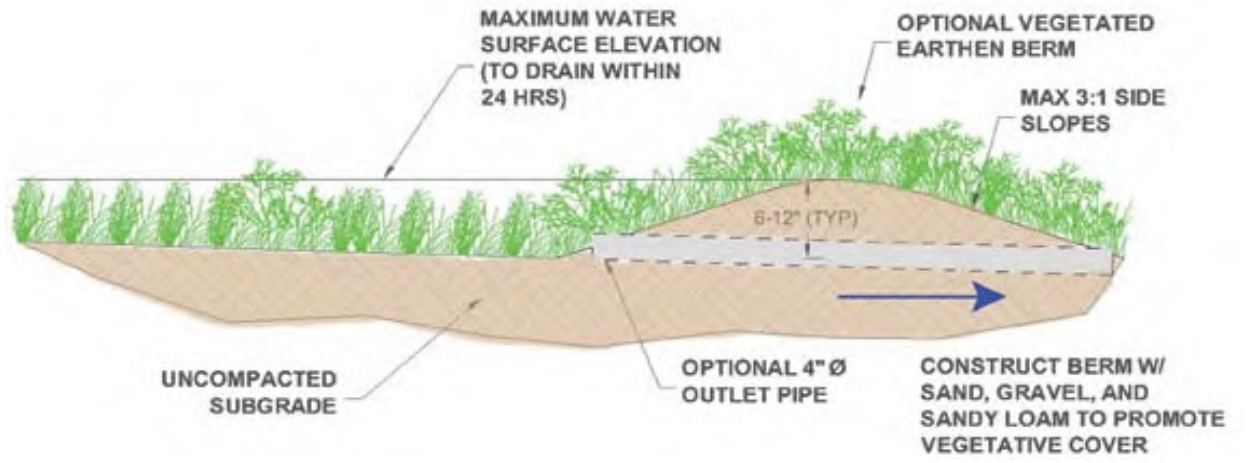


Figure 5 Optional earthen berm at bottom of vegetated filter strip

As shown in **Figure 6, Figure 7, Figure 8, Figure 9,** and **Figure 10**, the recommended filter strip length varies depending on the type of soil, the type of vegetation, and the filter strip slope. Generally, the more permeable the soil and/or the lower the slope, the shorter the filter strip may be for equivalent storm-water benefits.

County Regulated Drain Considerations: Special attention needs to be paid regarding the type and density of vegetation used if the natural drainage pathway or stream is or will be a part of the County Regulated Drain system. The County may require that at least one side of the regulated drain is clear of woody vegetation, with continuous access provided for reconstruction and maintenance. Pre-coordination with the County Surveyor's Office is highly recommended.

		Maximum Filter Strip Slope (Percent)	
Filter Strip Soil Type	**Hydrologic Soil Group**	**Turf Grass, Native Grasses and Meadows**	**Planted and Indigenous Woods**
Sand	A	7	5
Sandy Loam	B	8	7
Loam, Silt Loam	B	8	8
Sandy Clay Loam	C	8	8
Clay Loam, Silty Clay, Clay	D	8	8

Table 1 Recommended Length as a Function of Slope, Soil Cover

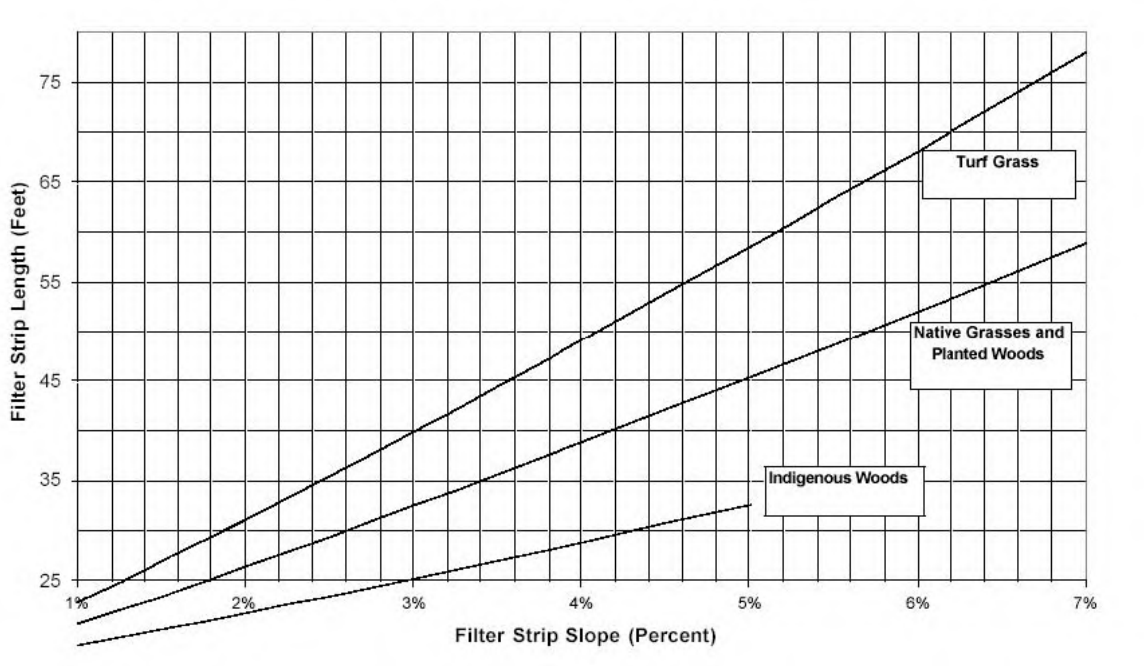


Figure 6 Sandy Soils with Hydrologic Soil Group A
Source: New Jersey Stormwater Best Management Practices Manual, February 2004

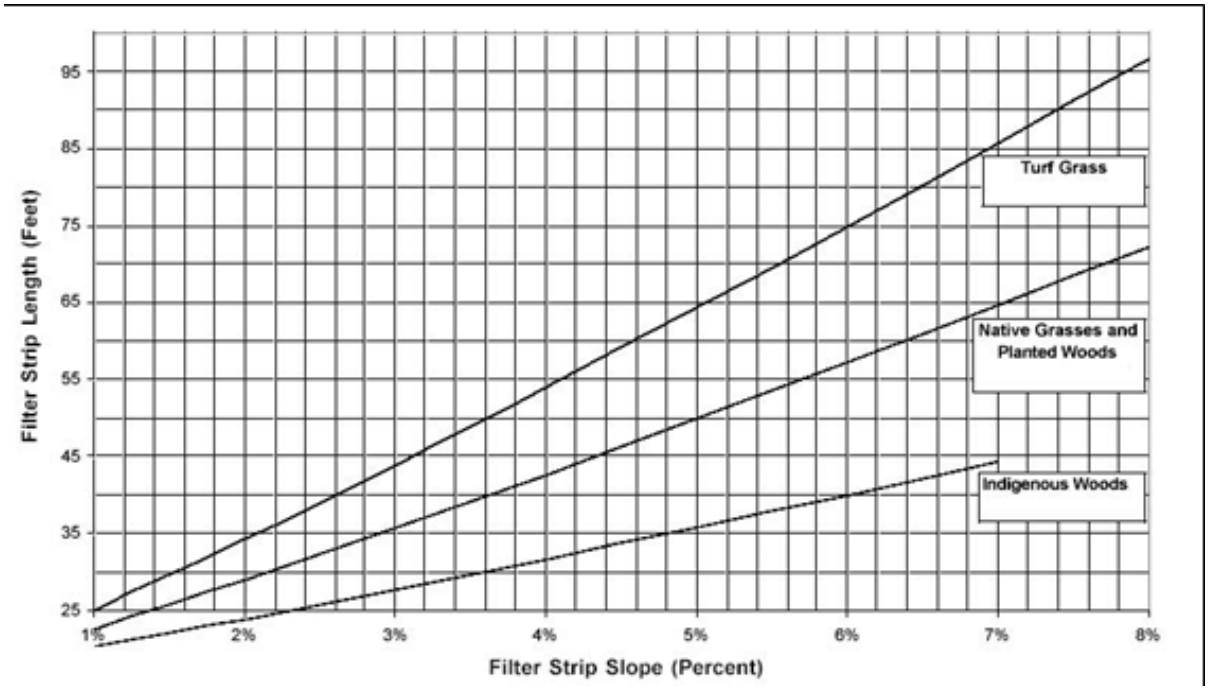


Figure 7 Sandy Loam Soils with Hydrologic Soil Group B
Source: New Jersey Stormwater Best Management Practices Manual, February 2004

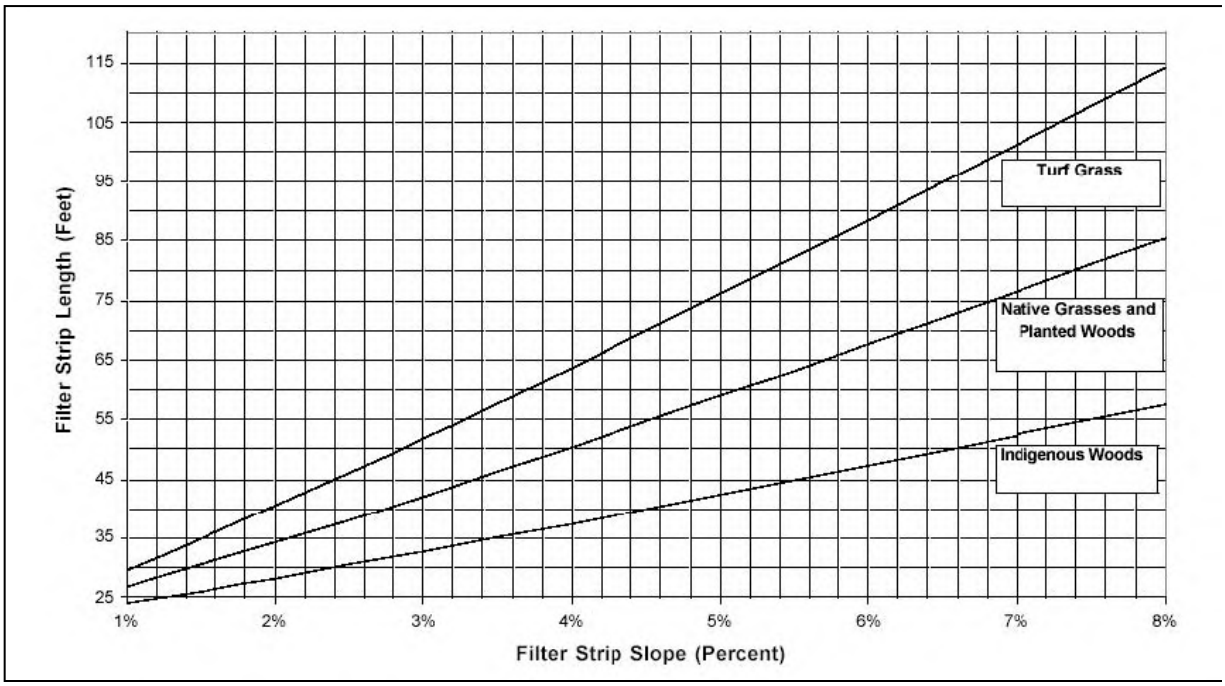


Figure 8 Loam, Silt-Loam Soils Hydrologic Soil Group B
Source: New Jersey Stormwater Best Management Practices Manual, February 2004

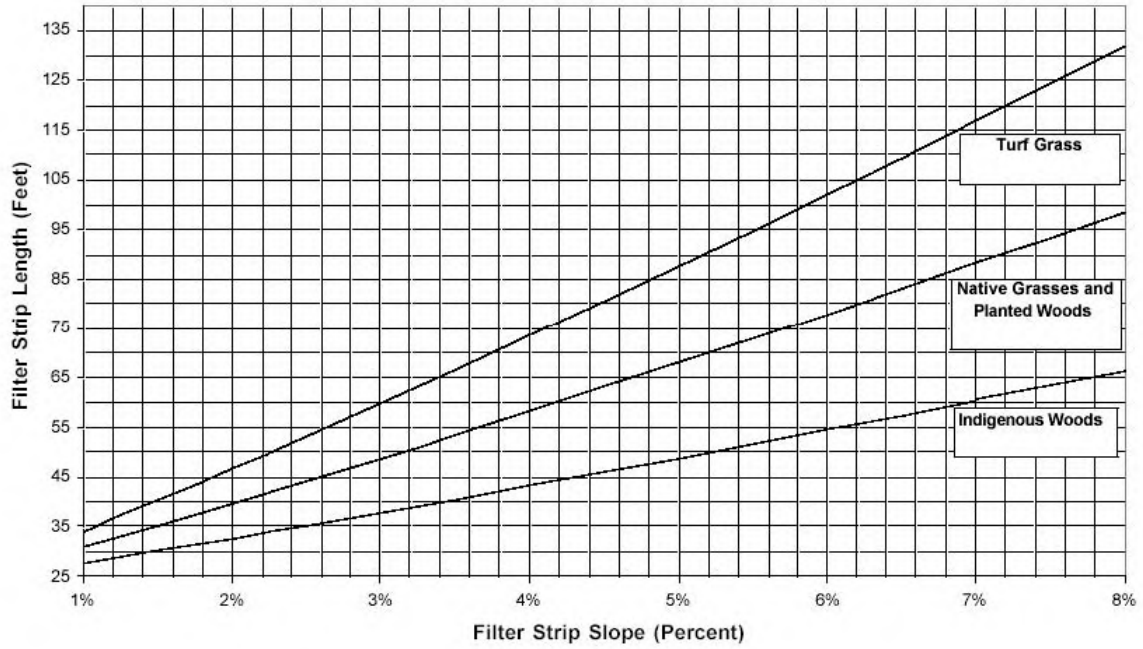


Figure 9 Sandy Clay Loam Soils with Hydrologic Soil Group C
Source: New Jersey Stormwater Best Management Practices Manual, February 2004

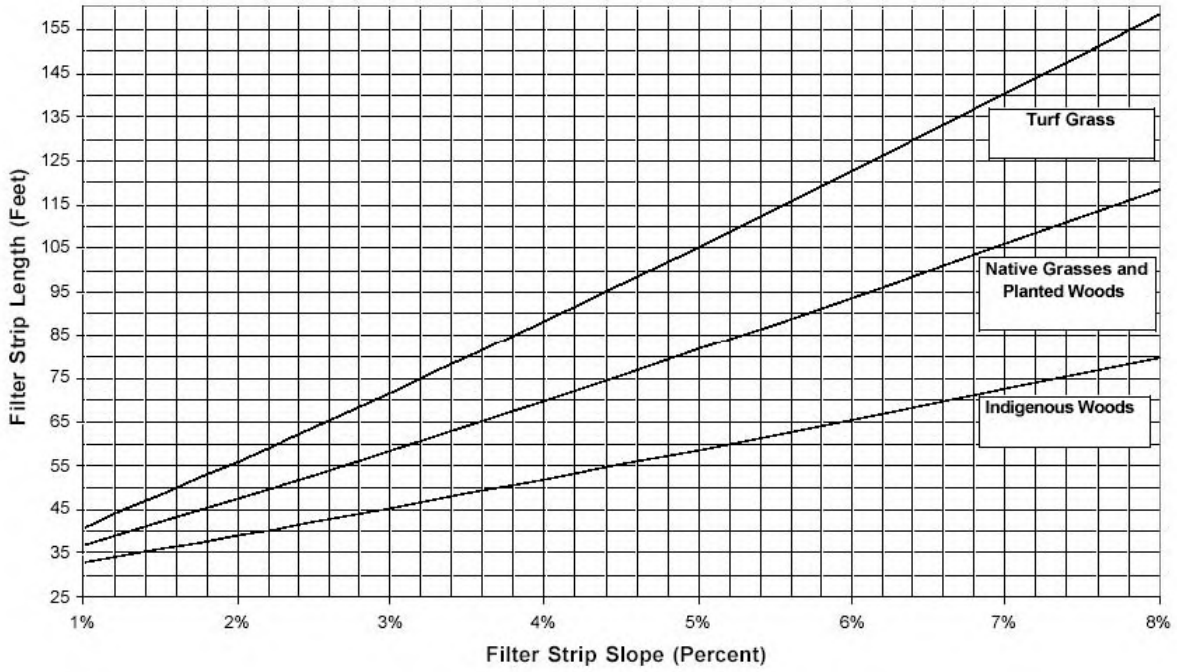


Figure 10 Clay Loam, Silty Clay or Clay Soils with Hydrologic Soil Group D
Source: New Jersey Stormwater Best Management Practices Manual, February 2004

Stormwater Functions and Calculations

Volume reduction

Although not typically considered a volume-reducing BMP, vegetated filter strips can achieve some volume reduction through infiltration and evapotranspiration, especially during small storms (storms less than approximately one inch). The volume reduction benefit of a filter strip can be estimated through hydrologic calculations. Two recommended methods are weighting the curve number of the drainage area with that of the filter strip or routing the runoff from the drainage area onto the filter strip area as inflow in addition to incident precipitation.

Large areas with dense vegetation may absorb unconcentrated flows that result from small storms, while areas covered by turf grass will absorb limited runoff. If a berm is constructed at the down-gradient end of the filter strip, an additional volume will be detained and may infiltrate the underlying soil.

Peak rate mitigation

Vegetated filter strips do not substantially reduce the peak rate of discharge. However, if a volume reduction is achieved through infiltration and evapotranspiration, a related reduction in peak rate will occur. If a berm is constructed at the down-gradient end of the filter strip, the rate of release of the detained volume may be controlled by an outlet structure.

Water quality improvement

Water quality benefits of vegetated filter strips are medium to high. The amount of benefit is based on flow characteristics and nutrient, sediment, and pollutant loadings of the runoff, as well as the length, slope, type, and density of vegetation in the filter strip.

Studies have shown 85 to 90 percent reductions in TSS and 40 to 65 percent reductions in nitrates (NO_2) from runoff being treated by vegetated filter strips. In these studies, the vegetated filter strips were between 25 and 29 feet wide with mild (0.7 percent to 1.7 percent) slopes, with grass and mixed vegetation.

Other studies have shown that suspended solids and metals are reduced to steady state amounts within several meters of the edge of the filter strip. (Note: If a filter strip is used for temporary sediment control, it should be regraded and reseeded immediately after construction and stabilization has occurred.)

Construction Guidelines

1. Follow the recommendations for materials in Recommended Materials Appendix.

2. Begin filter strip construction only when the up-gradient site has been sufficiently stabilized and temporary erosion and sediment control measures are in place. The strip should be installed at a time of the year when successful establishment without irrigation is most likely. However, temporary irrigation may be needed in periods of little rain or drought.

3. For non-indigenous filter strips, clear and grade site as needed. Care should be taken to disturb as little existing vegetation as possible, whether in the designated filter strip area or in adjacent areas, and to avoid soil compaction. Grading a level slope may require removing existing vegetation.

4. Grade the filter strip area, including the berm at the toe of the slope. Pressure applied by construction equipment should be limited to four pounds per square inch to avoid excessive compaction or land disturbance.

5. Construct level spreader device at the upgradient edge of the filter strip. For gravel trenches, do not compact the subgrade. (Follow construction sequence for Infiltration Trench.)

6. Fine grade the filter strip area. Accurate grading is crucial for filter strips. Even the smallest irregularities may compromise sheet flow conditions.

7. Seed, sod, or plant more substantial vegetation, as proposed. If sod is proposed,

place tiles tightly to avoid gaps, and stagger the ends to prevent channelization along the strip. Use a roller on sod to prevent air pockets from forming between the sod and soil.

8. Stabilize seeded filter strips with appropriate permanent soil stabilization methods, such as erosion control matting or blankets. Erosion control for seeded filter strips should be required for at least the first 75 days following the first storm event of the season.

9. Once the filter strip is sufficiently stabilized after one full growing season, remove temporary erosion and sediment controls.

Maintenance

As with other vegetated BMPs, filter strips must be properly maintained to ensure their effectiveness. In particular, it is critical that sheet flow conditions are sustained throughout the life of the filter strip. Field observations of strips in urban settings show that their effectiveness can deteriorate due to lack of maintenance, inadequate design or location, and poor vegetative cover. Compared with other vegetated BMPs, filter strips require only minimal maintenance efforts, many of which may overlap with standard landscaping demands.

- Inspect sediment devices quarterly for clogging, excessive accumulations, and channelization for the first two years following installation, and then twice a year thereafter. Inspections should also be made after every storm event greater than one inch during the establishment period.

- Sediment and debris should be removed when buildup exceeds two inches in depth in either the filter strip or the level spreader. Improve the level spreader if erosion is observed. Rills and gullies observed along the strip may be filled with topsoil, stabilized with erosion control matting, and either seeded or sodded. For channels less than 12 inches wide, filling with crushed gravel, which allows grass to creep in over time, is acceptable. For wider channels (greater than 12 inches), regrading and reseeding may be necessary. Small bare areas may only require overseeding. Regrading may also be required when pools of standing water are observed along the slope. In no case should standing water be tolerated for longer than 48 to 72 hours.

- If check dams are proposed, inspect for cracks, rot, structural damage, obstructions, or any other factors that cause altered flow patterns or channelization. Inlets or sediment sumps that drain to filter strips should be cleaned periodically or as needed.

- Remove sediment when the filter strip is thoroughly dry. Dispose of sediment and debris at a suitable disposal or recycling site that complies with applicable local, state, and federal waste regulations.

- When a filter strip is used for sediment control, it should be regraded and reseeded immediately after construction.

- Guidance information, usually in written manual form, for operating and maintaining filter strips, should be provided to all facility owners and tenants. Facility owners are encouraged to keep an inspection log, for recording all inspection dates, observations, and maintenance activities.

- Grass cover should be mowed to maintain a height of 4-6 inches.

- Invasive plants should be removed on an annual basis. Vegetative cover should be sustained at 85 percent and reestablished if damage greater than 50 percent is observed.

- If a filter strip exhibits signs of poor drainage, periodic soil aeration or liming may help to improve infiltration.

Winter Considerations

Filter strips often make convenient areas for snow storage. Thus, vegetation should be salt-tolerant and the maintenance schedule should include removing sand buildup at the toe of the slope.

The bottom of the gravel trench (if used as the level spreader) should be placed below the frost line to prohibit water from freezing in the trench. The perforated pipe in the trench should be at least eight inches in diameter to further discourage freezing. Other water quality options may be explored to provide backup to filter strips during the winter, when pollutant removal ability is reduced.

Cost

The cost of constructing filter strips includes grading, sodding (when applicable), installing vegetation, constructing a level spreader, and constructing a pervious berm, if proposed. Depending on whether seed or sod is applied, enhanced vegetation use or design variations such as check dams, construction costs may range anywhere from no cost (assuming the area was to be grassed regardless of use as treatment) to $50,000 per acre. The annual cost of maintaining filter strips (mowing, weeding, inspecting, litter removal, etc.) generally runs from $100 to $1,400 per acre and may overlap with standard landscape maintenance costs. Maintenance costs are highly variable, as they are a function of frequency and local labor rates.

Designer/Reviewer Checklist for Vegetated Filter Strips

Soil type and HSG category: _____

ITEM	Page No.	YES	NO	N/A	NOTES
Sheet flow provided?	1				
Recommended slope ranges followed? ➢ *Max. lateral slope = **1%***	1				
Appropriate length for soil, vegetation, and slope?	4-7				
Slope of drainage area below five percent?	--				
If not, is energy dissipation provided?	3				
Length/area of incoming drainage appropriately limited?	3				
Receiving vegetation considered?	2				
Located in undisturbed virgin soil?	4				
If not, will soil be properly compacted and stabilized?	8,9				
Appropriate vegetation selected for stabilization?	3,4				
Feasible construction process and sequence?	8,9				
Soil compaction avoided or mitigated?	--				
Erosion and sedimentation control provided to protect filter strip during construction?	8,9				
Maintenance accounted for and plan provided? ➢ *Remove sediment/debris if > 2 inches deep.* ➢ *Standing water should not exceed 48-72 hours* ➢ *Grass cover: 4-6 inches high*	9				
Additional Design Considerations ➢ *Underlying soils should have low permeability, ranging between 0.06 and 0.6 inches/hour* ➢ *Hydraulic residence time no less than 5 minutes; optimal time is 9 minutes* ➢ *Average velocity no greater than 0.9 feet/second* ➢ *Average depth of flow no more than 0.5 inches*	--				

➢ *Denotes Minimum Design Considerations*

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BMP Fact Sheet

VEGETATED ROOF

Vegetated roofs, or green roofs, are conventional rooftops that include a thin covering of vegetation allowing the roof to function more like a vegetated surface. The overall thickness of the vegetated roof may range from 2 to 6 inches, typically containing multiple layers consisting of waterproofing, synthetic insulation, non-soil engineered growth media, fabrics, synthetic components, and foliage.

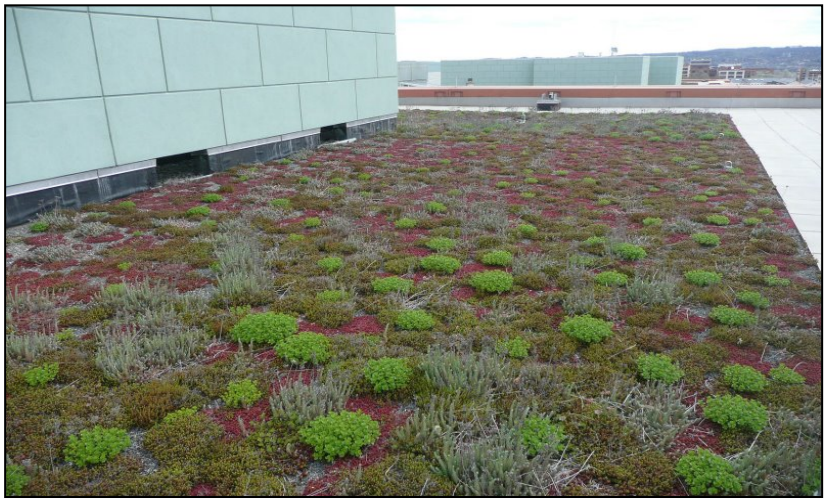


Figure 1 Vegetated Roof, Washington, DC (USEPA, picasaweb)

Variations

- Intensive
- Semi-intensive
- Extensive

Key Design Features

- Extensive roofs are most commonly used for rainfall runoff migration
- Roofs with pitches steeper than 2:12 (9.5 degrees) must incorporate supplemental measures

Benefits

- Good stormwater volume control
- Heating and cooling energy benefits
- Increased lifespan of roof
- Heat island reduction
- Enhance habitat value

Limitations

- Cost (intensive systems)
- Careful design and construction required
- Maintenance requirements until plants established
- Can't store or treat stormwater from other parts of the property

Applications		Stormwater Quantity Functions	
Residential	Limited	Volume	Med/High
Commercial	Yes	Groundwater Recharge	Low*
Ultra Urban	Yes	Peak Rate	Medium
Industrial	Yes	Stormwater Quality Functions	
Retrofit	Yes	TSS	Medium
Highway/Road	N/A	TP	Medium
Recreational	Yes	TN	Medium
		Temperature	High

Additional Considerations	
Cost	High
Maintenance	Medium
Winter Performance	Medium

* Although vegetated roofs can be used very successfully in combination with infiltration systems.

Description and Function

Vegetated roofs involve growing plants on rooftops, thus replacing the vegetated footprint that was removed when the building was constructed. Vegetated roof covers are an "at source" measure for reducing the rate and volume of runoff released during rainfall events. The water retention and detention properties of vegetated roof covers can be enhanced through selection of the engineered media and plants. Depending on the plant material and planned usage for the roof area, modern vegetated roofs can be categorized as systems that are intensive, semi-intensive, or extensive (**Table 1**).

Intensive vegetated roofs utilize a wide variety of plant species that may include trees and shrubs, require deeper substrate layers (usually greater than four inches), are generally limited to flat roofs, require 'intense' maintenance, and are often park-like areas accessible to the general public.

Extensive vegetated roofs are limited to herbs, grasses, mosses, and drought tolerant succulents such as sedum, can be sustained in a shallow substrate layer (less than four inches), require minimal maintenance once established, and are generally not designed for access by the public. These vegetated roofs are typically intended to achieve a specific environmental benefit, such as rainfall runoff mitigation. Extensive roofs are well suited to rooftops with little load bearing capacity and sites which are not meant to be used as roof gardens. The mineral substrate layer, containing little nutrients, is not very deep but suitable for less demanding and low-growing plant communities.

Semi-intensive vegetated roofs fall between intensive and extensive vegetated roof systems. More maintenance, higher costs and more weight are the characteristics for this intermediate system compared to that of the extensive vegetated roof.

Vegetated system layers

A proprietary system provides a growing environment on the roof which adequately compensates for the plant's natural environment. It ensures reliable technical and ecological functionality for decades. Vegetated roof systems contain the following functional layers (from bottom to top):

Root barrier: The root barrier protects the roof construction from being damaged by roots. If the waterproofing is not root resistant a separate root barrier has to be installed.

Waterproof membrane: This layer protects the roof structure from moisture and can include a unique root-resistant compound to prevent roots from penetrating.

Protection layer: A specially designed perforation resistant protection mat prevents mechanical damage of the root barrier and roof construction during the installation phase. Depending on the thickness and the material the protection layer can also retain water and nutrients.

Drainage Layer: The drainage layer allows for excess water to run-off into the water outlets. Depending on the design and the material the drainage layer has additional functions such as water storage, enlargement of the root zone, space for aeration of the system and protection for the layers below it. Due to the weight constraints of the roof, the drainage layer is made of light-weight materials. Molded drainage elements made of rubber or plastic are used quite often. Other drainage layers are made of gravel, lava, expanded clay or clay tiles.

Filter layer: The filter layer separates the plant and substrate layers from the drainage layer below. Especially small particles, humic and organic materials, are retained by the filter sheet and are therefore available for the plants. In addition, the filter sheet ensures that the drainage layer and the water outlet are not clogged with silt. Filter layers are preferably made of geo-textiles such as fleece or other woven materials.

Growing medium: The growing medium is the basis of the vegetated roof. A sufficient depth for the root zone has to be ensured as well as an adequate nutrient supply and a well balanced water-air relation. Depending on the type of vegetated roof and the construction requirements, a variety of different system substrates are available.

	Extensive Vegetated Roof	Semi-Intensive Vegetated Roof	Intensive Vegetated Roof
Maintenance	Low	Periodically	High
Irrigation (after plants are established)*	No	Periodically	Regularly
Plant Communities	Moss, Sedum, Herbs, and Grasses	Grass, Herbs, and Shrubs	Perennials, Shrubs, and Trees
System build-up height	60-200 mm	120-250 mm	150-400 mm Underground garages = > 1000 mm
Weight	60 - 150 kg/m^2 13-30 lbs/ ft^2	120 - 200 kg/m^2 25-40 lbs/ ft^2	180 - 500 kg/m^2 35-100 lbs/ft^2
Construction costs	Low	Medium	High
Desired use	Ecological protection layer	Designed vegetated roof	Park-like garden

Table 1 Vegetated roof types
**Irrigation is required regularly to establish plant communities, especially during the first season.*
Source: Adapted from International Green Roof Association

Light-weight mineral materials, with high water retention capacity and good water permeability, such as lava, pumice, expanded clay, expanded schist, and clay tiles, have proven to be reliable for many years. Untreated organic material and top soil have disadvantages in terms of weight and drainage function; they are only used as additions to mineral substrates.

Plant level: The plant selection depends on the growing medium as well as local conditions, available maintenance and the desired appearance. Low maintenance, durable and drought resistant plants are used for extensive vegetated roofs, versus, a nearly limitless plant selection for intensive vegetated roofs.

Variations

Some specialized vegetated roof companies offer installation using vegetated blankets/mats or trays. Pre-vegetated blankets/mats are grown off-site and brought to the site for installation (similar to the concept of sod for grass). They can provide an immediate vegetative coverage which can prevent erosion, reduce installation times, and reduce maintenance during what would otherwise be the establishment period for vegetation.

Modular systems are manufactured trays filled with various vegetated roof layers (often pre-vegetated as well) that are delivered to the site and installed on a prepared roof. Manufacturers of these systems claim that benefits include faster installation and easier access to the roof if maintenance or leak repairs are necessary (in addition to the potential benefits of a pre-vegetated system). Others argue that these benefits are not significant and that trays can have drawbacks such as increased cost, poor aesthetics (module edges being visible), and reduced performance (wet and dry spots resulting from the barriers between modules in the system).

Extensive vegetated roofs

Extensive vegetated roofs are the most commonly used systems due to their higher mitigation of stormwater runoff as well as their lower cost compared to the other systems. Extensive systems have three variations of assemblies that can be considered in design.

Single media assemblies

Single media assemblies (**Figure 2**) are commonly used for pitched roof applications and for thin and lightweight installations. These systems typically incorporate very drought tolerant plants and utilize coarse engineered media with high permeability. A typical profile would include the following layers:

1. Waterproofing membrane
2. Protection layer
3. Root barrier (optional, depending on the root-fastness of the waterproofing)
4. Drainage layer
5. Filter layer
6. Growth media
7. Vegetation

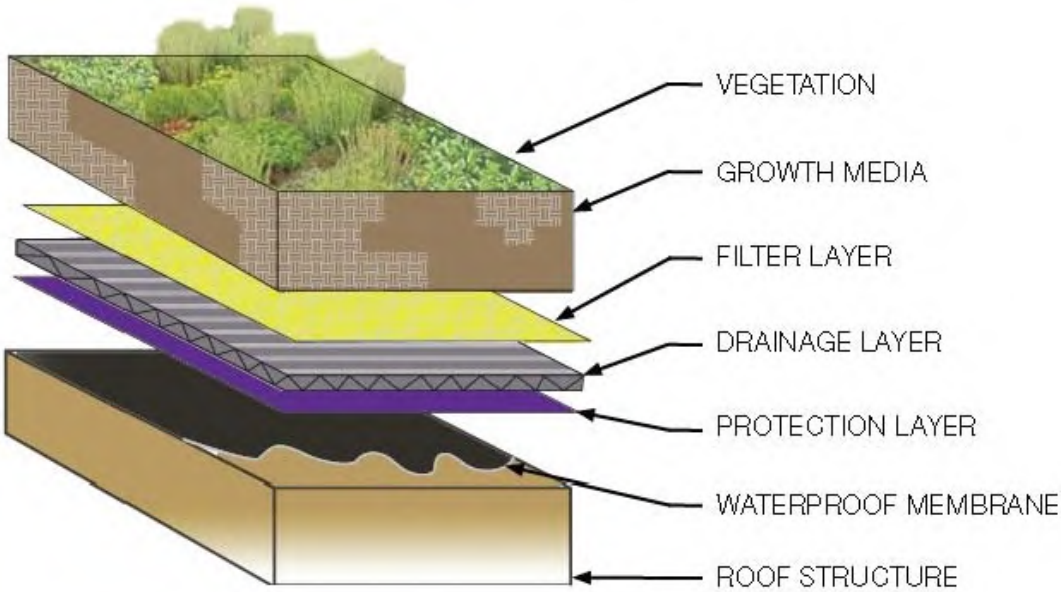


Figure 2 Single media assembly

Pitched roof applications may require the addition of slope bars, rigid slope stabilization panels, cribbing, reinforcing mesh, or similar method of preventing sliding instability.

Flat roof applications with mats as foundations typically require a network of perforated internal drainage conduit to enhance drainage of percolated rainfall to the deck drains or scuppers.

Dual media assemblies

Dual media (**Figure 3**) assemblies utilize two types of non-soil growth media. In this case a finer-grained media with some organic content is placed over a base layer of coarse lightweight mineral aggregate. They do not include a geocomposite drain.

The objective is to improve drought resistance by replicating a natural alpine growing environment in which sandy topsoil overlies gravelly subsoil. These assemblies are typically 4 to 6 inches thick and include the following layers:

1. Waterproofing membrane
2. Root barrier/ protection layer
3. Coarse-grained drainage media
4. Filter layer
5. Growth media
6. Vegetation

These assemblies are suitable for roofs with pitches less than, or equal to about 1.5:12 (7.1 degrees). Large vegetated covers will generally incorporate a network of perforated internal drainage conduit located within the coarse grained drainage layer.

Dual media with synthetic retention/detention layer

These assemblies introduce impervious plastic panels with cup-like receptacles on their upper surface (i.e., a modified geocomposite drain sheet). The panels are in-filled with coarse lightweight mineral aggregate. The cups trap and retain water. They also introduce an air layer at the bottom of the assembly. A typical profile would include:

1. Waterproof membrane
2. Protection layer
3. Retention/detention panel
4. Coarse-grained drainage media
5. Filter layer
6. Growth media
7. Vegetation

These assemblies are suitable on roof with pitches less than or equal to 1:12 (4.8 degrees). Due to their complexity, these systems are usually a minimum of five inches deep. If required, irrigation can be provided via surface spray or mid-level drip.

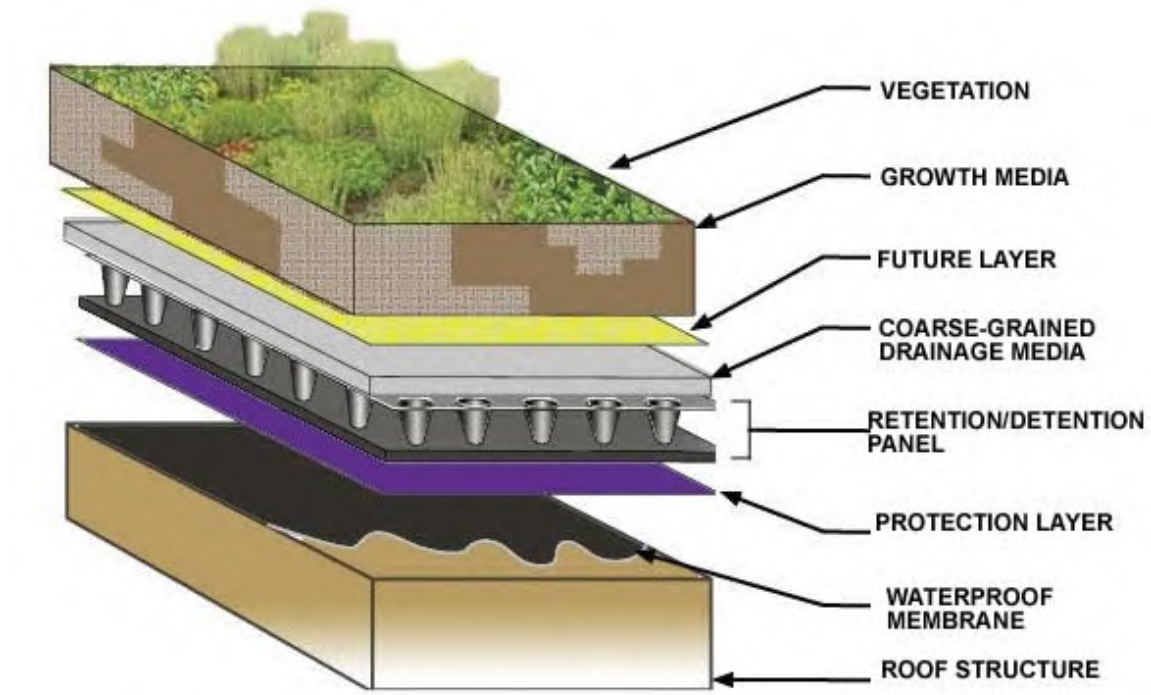


Figure 3 Dual media assembly

Treatment Train

Vegetated roof covers are frequently combined with ground infiltration measures. This combination can be extremely effective for stormwater management and is one of the best ways to replicate the natural hydrologic cycle. Vegetated roofs evapotranspirate a significant fraction of annual rainfall and typically discharge larger storm events relatively slowly. If overflow is directed to an infiltration system, the discharge can be infiltrated efficiently as the system has more time to absorb water as it is slowly released from the roof. Vegetated roof covers improve the efficiency of infiltration devices by:

- Reducing the peak runoff rate,
- Prolonging the runoff, and
- Filtering runoff to produce a cleaner effluent.

Benefits

Establishing plant material on rooftops provides numerous ecological and economic benefits including stormwater management, energy conservation, mitigation of the urban heat island effect, increased longevity of roofing membranes, as well as providing a more aesthetically pleasing environment to work and live. A major benefit of green roofs is their ability to absorb stormwater and release it slowly over a period of several hours, retaining 60-100 percent of the stormwater they receive, depending on the duration and the intensity of the storm.

In addition, green roofs have a longer life-span than standard roofs because they are protected from ultraviolet radiation and the extreme fluctuations in temperature that cause roof membranes to deteriorate. A vegetated roof has a life expectancy of 60 years — three times as long as a traditional roof.

As pervious surfaces are replaced with impervious surfaces due to urban development, the need to recover green space is becoming increasingly critical for the health of our environment. Vegetated roof covers have been used to create functional meadows and wetlands to mitigate the development of open space. This can be accomplished with assemblies as thin as six inches.

Design Considerations

Roof substructure

Wooden constructions, metal sheeting as well as reinforced concrete decks can be considered as

appropriate roof substructures. The base for the vegetated roof is a waterproof roof construction with appropriate load bearing capacity.

Root barrier

Root barriers should be thermoplastic membranes with a thickness of at least 30 mils. Thermoplastic sheets can be bonded using hot-air fusion methods, rendering the seams safe from root penetration. Membranes that have been certified for use as root-barriers are recommended.

Over a period of time roots can damage the water-proofing and roof construction if there have been no corresponding protection measures taken. The root resistance of the waterproofing is determined from the "Procedure for investigating resistance to root penetration at green-roof sites" by the FLL (The Landscaping and Landscape Development Research Society). Over 70 different waterproofing products meet the requirements of this test. If the waterproofing is not root resistant, an additional root barrier has to be installed. Aside from the roof surface, the upstands, perimeters, joints and roof edges also have to be protected against root penetration.

Growth media

Growth media should be a soil-like mixture containing not more than 15 percent organic content. The appropriate grain-size distribution is essential for achieving the proper moisture content, permeability, nutrient management, and non-capillary porosity, and 'soil' structure. The grain-size guidelines vary for single and dual media vegetated cover assemblies.

Separation fabric

Separation fabric should be readily penetrated by roots, but provide a durable separation between the drainage and growth media layers. (Only lightweight nonwoven geotextiles are recommended for this function.)

Roof penetrations

For vegetated roofs, the following upstand and perimeter heights have to be considered:

- Upstand height for adjacent building parts and penetrations: minimum of 6 inches.
- Upstand height for roof edges: minimum of 4 inches.

Important: The upstand height is always measured from the upper surface of the vegetated roof system build up or gravel strip. Clamping profiles guarantee reliable protection and a tight connection of the upstand areas. Roof penetrations (e.g. water connections, building parts for the usage of the roof area, etc.), when possible, should be grouped in order to keep roof penetration to a minimum.

> Even though it is possible to build pitched green roofs with a slope of 45º, it is not recommended to exceed 10 º due to significant limited accessibility for upkeep and maintenance.

Roof slope

Using modern technologies it is possible to install a reliable vegetated roof system not only on conventional flat roofs, but also on saddle roofs, shed roofs and barrel roofs. Special technical precautions for the mitigation of existing shear forces and erosion are only necessary for a roof slope over 10°.

Roofs with a slope of more than 45° are normally not suitable for a vegetated roof system. Roofs with a slope of less than two percent are special roof constructions on which puddles often develop.

In order to avoid damage to extensive vegetated roofs by water retention, specific arrangements for the roof drainage are necessary. In contrast, it can be beneficial for intensive vegetated roofs to design the roof construction without slope to allow for dam up irrigation.

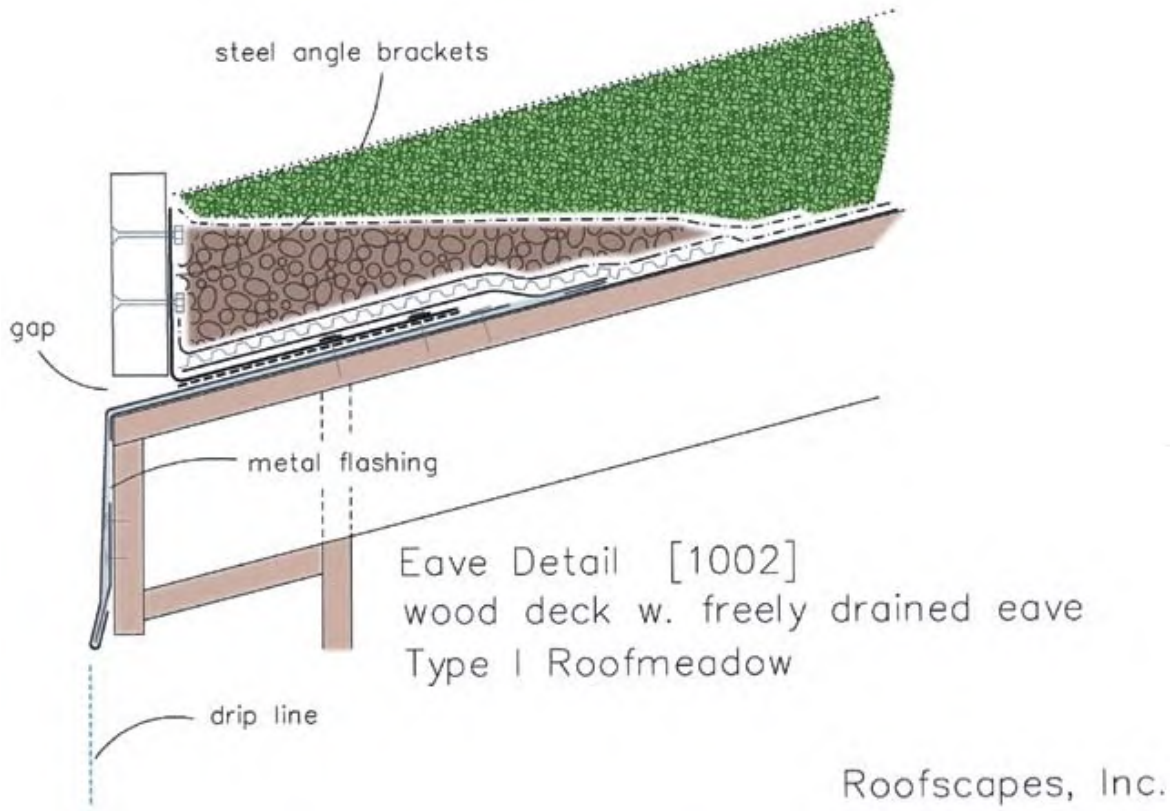


Figure 4 Example Eave Detail for sloped roof
Source: Roofscapes, Inc.

Load calculations

The maximum load bearing capacity of the roof construction must be considered when installing vegetated roofs. Therefore, the water saturated weight of the green roof system, including vegetation must be calculated as permanent load. Extensive vegetated roofs weigh between 60-150 kg/m^2 (13.0-30.0 lb/ft^2.) depending on the thickness of the vegetated roof system build-up. Trees, shrubs, and construction elements such as pergolas and walkways cause high point loads and, therefore, have to be calculated accordingly.

Wind uplift

A vegetated roof must be tight to the roof, especially in cases of strong wind. When designing and installing the vegetated roof, safety measures against wind uplift are to be considered.

This is especially important when the vegetated roof provides the load for a loose laid waterproofing and root barrier. The actual influence from the wind depends on the local wind zone, height of the building, roof type, slope, and area (whether corner, middle or edge) and the substructure.

Roof drainage

Vegetated roof systems store a major part of the annual precipitation and release it to the atmosphere by transpiration. Depending on the thickness of the vegetated roof system build-up and rain intensity, surplus water may accumulate at certain times and must be drained off the roof area. The number of roof outlets and the penetrability factor, or more precisely, the water retaining capacity of the vegetated roof system build-up, has to be adjusted to the average local precipitation.

Roof outlets are to be kept free of substrate and vegetation and have to be controllable at all times. For this purpose “inspection chambers” are installed over the roof outlets. Due to safety precautions, roof areas with inlayed drainage must always have two drainage outlets or one outlet and one safety overflow. For facades and roof areas, gravel strips,

Pitched Vegetated Roofs

Technical Requirements

Root resistant waterproofing is necessary for pitched vegetated roofs; installing an additional root barrier, requires much effort and increases the risk of slippage. Stable abutments have to be installed on the eaves edges to transfer shear forces from the vegetated roof system build-up and the additional snow load into the roof construction. Additional shear barriers may be necessary to transfer the shear forces depending on the roof slope and the roof length. It is recommended the design for the shear barriers and the eaves profiles be done by a structural engineer. With increasing slope, the vegetated roof system build-up is more complicated and the substrate has to be protected from erosion; plastic grid elements can be used for this purpose.

Plant Selection

The success of the landscaping on pitched roofs depends on the plants. Fast surface coverage is the highest priority. A dense planting of root ball plants or pre-cultivated vegetation mats are used in cases of steep slopes and allow for rapid coverage. It is also important to consider the exposure of the roof area, the slope and the location of the building when selecting plants. Perennials and grasses can be used whereas Sedum is the most suitable for pitched roofs, due to the species' high water retention capacity and erosion protection. The water run-off is much faster on pitched roofs compared to a flat roof. It is advisable to plan for an additional irrigation system to provide water during dry periods. The irrigation can be provided either by drip irrigation or by sprinkler systems.

gullies and grids provide fast drainage of rainwater into the drainage system.

Irrigation

Extensive vegetated roofs with drought resistant plant species have to be irrigated only during planting and installation maintenance over the first two years. After its establishment, the annual rainfall is sufficient to sustain the vegetation. In contrast, the requirements are more involved for intensive vegetated roofs with lawn, shrubs, or trees. An adequate number of precisely dimensioned hoses with automatic irrigation units make plant maintenance during drought periods more manageable. The water supply for roof gardens with no slope can be increased through additional dam-up irrigation. Vegetated roofs can also be irrigated with cistern water.

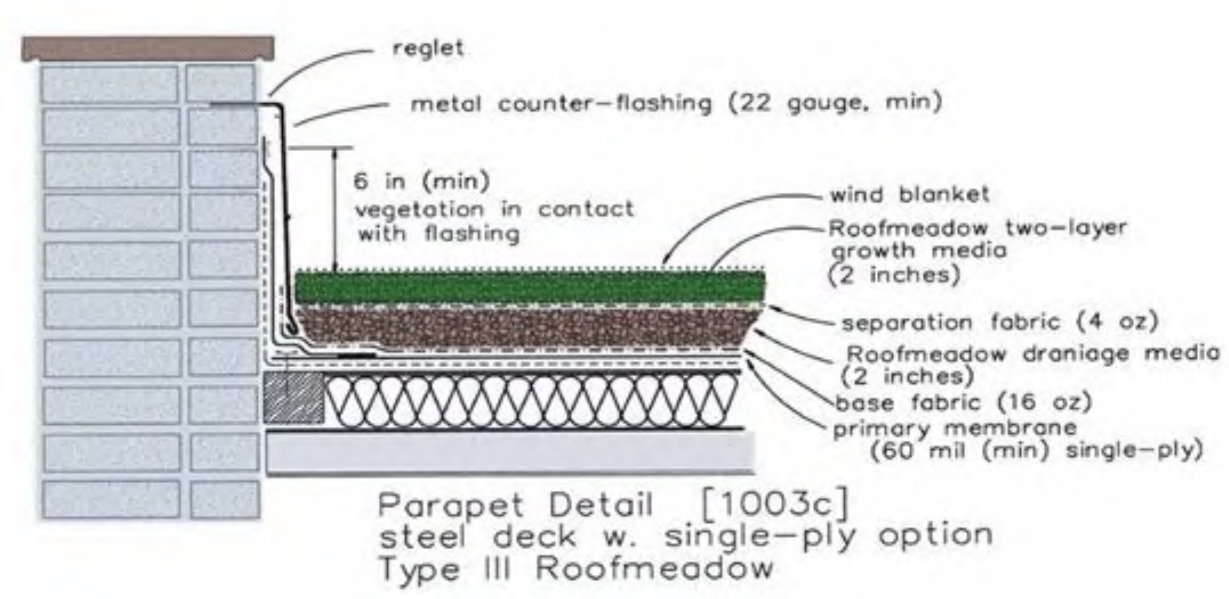


Figure 5 Example parapet flashing detail for flat roof
Source: Roofscapes, Inc.

Fire prevention

As a part of the "hard roof" classification, intensive vegetated roofs provide preventative fire protection in the case of sparks and radiating heat. The criteria that extensive vegetated roofs must meet in order to be considered fire resistant, are already met by most vegetated roof systems that are offered by suppliers. Openings within the vegetated roof (e.g. skylights) need to be installed with a vegetation free zone (approximately 20 inches). On larger roof areas a vegetation free zone (e.g. gravel strip or concrete slabs) are to be installed at least every 130 feet.

Vegetation Considerations

Extensive vegetated roofs

Plants for extensive vegetated roofs have to survive intense solar radiation, wind exposure, drought, low nutrient supply, freezing temperatures and limited root area. Suitable plant varieties are those growing in severe locations with little moisture and nutrient supply, such as dry meadows. The main varieties are sedum, and delosperma. The plants are able to store high amounts of water in the leaves, are stress resistant and recover easily from periods of drought. Other varieties such as dianthus species, asteraceae and ornamental grasses are also suitable for these conditions.

Intensive green roofs

Having an appropriate vegetated roof system and sufficient growing medium (with higher root penetration volume, nutrients and water supply) growth of sophisticated plant varieties on the roof is possible. The selected plants need to be resistant to intense solar radiation and strong winds. Vegetation with various plant varieties such as perennials, herbs, grasses and trees allow for a natural character on the roof. Having a broader plant community increases the amount of maintenance required.

Stormwater Functions and Calculations

The performance of vegetated roof covers as stormwater best management practices cannot be represented by simple algebraic expressions used for surface runoff. In the analysis of vegetated roof covers, the water that is discharged from the roof is not surface runoff, but rather underflow, i.e., percolated water. The rate and quantity of water released during a particular storm can be predicted based on knowledge of key physical properties, including:

- Maximum media water retention
- Field capacity
- Plant cover type
- Saturated hydraulic conductivity
- Non-capillary porosity

The maximum media water retention is the maximum quantity of water that can be held against gravity under drained conditions. Standards that have been developed specifically for measuring this quantity in roof media are available from FLL and ASTM (E2399).

Conventional runoff coefficients, such as the NRCS runoff curve number, CN, can be back-calculated from computer simulation or measurements of vegetated roof cover assemblies. However, these coefficients will only apply for the specific design storm for which they have been determined.

Volume reduction

All vegetated roof covers have both retention and detention volume components. Benchmarks for these volumes can be developed from the physical properties described above.

Peak rate mitigation

Vegetated roof covers can exert a large influence on peak rate, especially in less extreme storms such as the 1-, 2-, and 5-year storms. Because volume is reduced, there is some peak rate reduction achieved for all storms. An evaluation of peak runoff rates requires either computer simulation or measurements made using prototype assemblies.

A general rule for vegetated roof covers is that rate of runoff from the covered roof surface will be less than or equal to that of open space (i.e., NRCS curve number of about 65) for storm events with total rainfall volumes up to three times the maximum media water retention of the assembly. For example, a representative vegetated roof cover with maximum moisture retention of one inch will

react like open space for storms up to and including the three-inch magnitude storm.

Using computer simulations, municipalities could generate a table of CN values for specific design storms and green roof types. The table would relate maximum moisture capacity to the CN coefficients.

Water quality improvement

Direct runoff from roofs is a contributor to pollutants in stormwater runoff. Vegetated roof covers can significantly reduce this source of pollution. Assemblies intended to produce water quality benefits will employ engineered media with almost 100 percent mineral content. Furthermore, following the plant establishment period (usually about 18 months), on-going fertilization of the cover is no longer needed. Experience indicates that it may take five or more years for a water quality vegetated cover to attain its maximum pollutant removal efficiency.

Maintenance

- Irrigation will be required as necessary during the plant establishment period and in times of drought.
- During the plant establishment period, three to four visits to conduct basic weeding, fertilization, and infill planting is recommended.
- The soluble nitrogen content (nitrate plus ammonium ion) of the soil should be adjusted to between one and five parts per million, based on soil test.
- Once plants are established, it is crucial to maintain the roof once or twice a year. Weeds and other unwanted plants on the entire roof, at the perimeters and at the upstands need to be removed. For grass and herb vegetation the organic buildup has to be removed once a year. Intensive vegetated roofs require higher maintenance and service throughout the year.

Winter Considerations

Applicable snow load must be considered in the design of the roof structure.

Dam-up Irrigation in Vegetated Roof

Intensive Vegetated Roofs depend mainly on additional irrigation. To install an irrigation system which does not use fresh water, a water dam-up irrigation unit is recommended.

Requirements of a dam-up irrigation unit:

- flat roof
- dam-up elements above roof outlets
- an appropriate drainage layer with the necessary height

In case of heavy rain the reservoir is filled primarily and any excess water is collected in the cistern. During dry periods the water on the roof is used first, then water is pumped from the cistern onto the roof and supplied to the plants.

This process can be carried out either manually or electronically. The water in the cistern can also be used for other purposes, provided the reservoir is big enough.

Cost

The construction cost of vegetated roof covers varies greatly, depending on factors such as:

- Height of the building
- Accessibility to the structure by large equipment such as cranes and trailers
- Depth and complexity of the assembly
- Remoteness of the project from sources of material supply
- Size of the project

However, under 2007 market conditions, extensive vegetated covers for roof will typically range between $8 and $16 per square foot, including design, installation, and warranty service (not including waterproofing). Basic maintenance for extensive vegetated covers typically requires about 2-3 person-hours per 1,000 square feet, annually.

Although vegetated roofs are relatively expensive compared to other BMPs in terms of stormwater management, they can have other significant benefits which serve to reduce their life-cycle costs. For example, the longevity of the roof system may be greatly increased. In addition, heating and cooling costs can be significantly reduced.

Designer/Reviewer Checklist for Vegetated Roofs

Type of vegetated roof(s) proposed: _____

ITEM	Page No.	YES	NO	N/A	NOTES
Load and structural capacity analyzed?	7				
Waterproofing layer and protection adequate?	2,6				
Leak protection system provided?	3-5				
Internal drainage capacity for large storms?	--				
Appropriate growing medium?	3,6				
Appropriate drainage media and/or layer?	2				
Geotextile/filter fabric specified?	2,6				
Good detailing (flashings, penetrations, drains, gravel edges, etc.)?	--				
Slope stability provided, if necessary?	4,8				
Appropriate vegetation selected?	3,6, 8,9				
Plant establishment (temporary irrigation/fertilization) procedures provided?	8				
Erosion control / wind protection provided?	7				
Maintenance accounted for and plan provided?	10				

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BMP Fact Sheet

VEGETATED SWALE

A vegetated swale (or bioswale) is a shallow stormwater channel that is densely planted with a variety of grasses, shrubs, and/or trees designed to slow, filter, and infiltrate stormwater runoff. Check dams can be used to improve performance and maximize infiltration, especially in steeper areas.



Figure 1 Vegetated Swale, Philadelphia, PA (USEPA, picasaweb)

Applications		Stormwater Quantity Functions	
Residential	Yes	Volume	Low/Med
Commercial	Yes	Groundwater Recharge	Low/Med
Ultra Urban	Limited	Peak Rate	Low/Med
Industrial	Yes	Stormwater Quality Functions	
Retrofit	Limited	TSS	Med/High
Highway/Road	Yes	TP	Low/High
Recreational	Yes	TN	Medium
		Temperature	Medium

Additional Considerations	
Cost	Low/Med
Maintenance	Low/Med
Winter Performance	Medium

Variations

- Vegetated swale with infiltration trench
- Linear wetland swale
- Grass swale

Key Design Features

- Handles the 10-year storm event with some freeboard
- Two-year storm flows do not cause erosion
- Maximum size is five acres
- Bottom width of two to eight feet
- Side slopes from 3:1 (H:V) to 5:1
- Longitudinal slope from one to six percent
- Check dams can provide additional storage and infiltration

Site Factors

- Water table to bedrock depth: 2 foot minimum.*
- Soils: A, B preferred; C & D may require an underdrain (see infiltration BMP)
- Slope: 1 to 6 percent. (less than one percent can be used w/ infiltration)
- Potential hotspots: No
- Maximum drainage area: 5 acres

Benefits

- Can replace curb and gutter for site drainage and provide significant cost savings
- Water quality
- Peak and volume control with

Limitations

- Limited application in areas where space is a concern
- Unless designed for infiltration, there is limited peak and volume control

* four feet recommended, if possible

Description and Function

Vegetated swales are broad, shallow, earthen channels designed to slow runoff, promote infiltration, and filter pollutants and sediments in the process of conveying runoff. Water is filtered through the soil to under drains and the swale is quickly dewatered, preventing standing water. Vegetated swales are an excellent alternative to conventional curb and gutter conveyance systems, because they provide pretreatment and can distribute stormwater flows to subsequent BMPs.

Vegetated swales are sometimes used as pretreatment devices for other structural BMPs, especially from roadway runoff. While swales themselves are intended to effectively treat runoff from highly impervious surfaces, pretreatment measures are recommended to enhance swale performance. Pretreatment can dramatically extend the functional life of any BMP, as well as increase its pollutant removal efficiency by settling out some of the coarser sediments. This treatment volume is typically obtained by installing check dams at pipe inlets and/or driveway crossings. Other

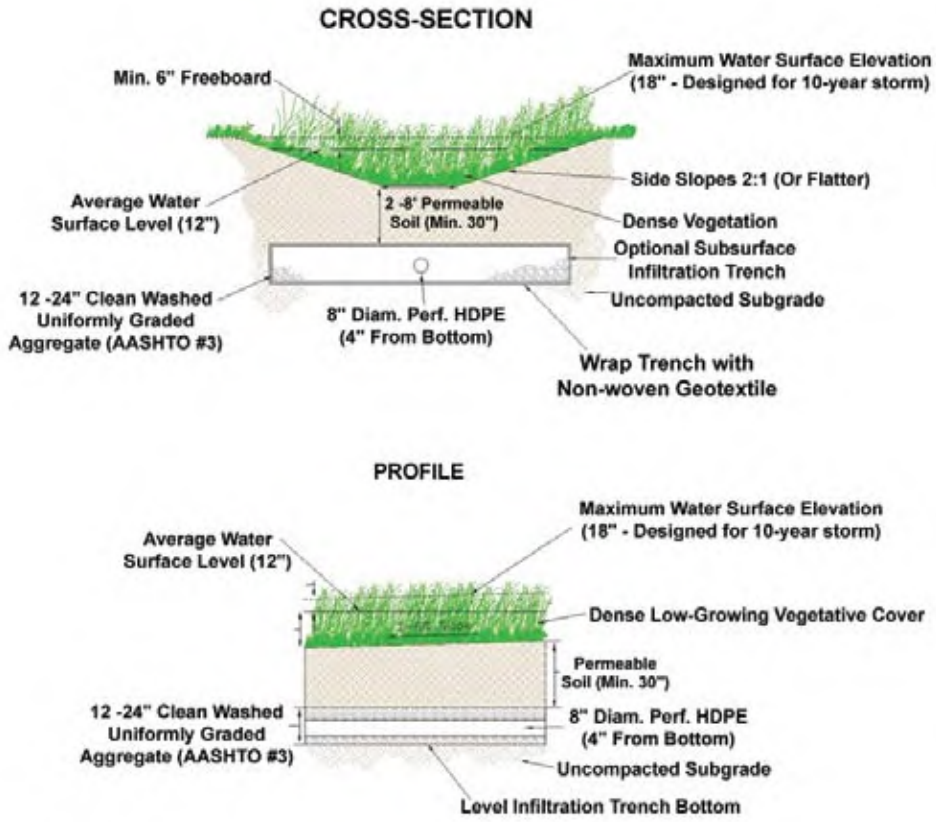


Figure 2 Schematics of Vegetated Swale with an underlying aggregate layer
Source: Pennsylvania Stormwater BMP Manual, 2006

A vegetated swale typically consists of a band of dense vegetation, underlain by at least 12 inches of permeable soil (greater than 0.5 inches/hour). Swales constructed with an underlying aggregate layer (**Figure 2)** can provide significant volume and peak rate reductions. The permeable soil media should have a minimum infiltration rate of 0.5 inches per hour.

pretreatment options include a vegetated filter strip, a sediment fore-bay (or plunge pool) for concentrated flows, or a pea gravel diaphragm (or alternative) with a six-inch drop where parking lot sheet flow is directed into a swale.

Check dams made of wood, stone, or concrete are often employed to enhance infiltration capacity, decrease runoff volume, rate, and velocity. They

also promote additional filtering and settling of nutrients and other pollutants. Check-dams create a series of small, temporary pools along the length of the swale, which drain down within a maximum of 48 hours.

Weep holes may be added to a wood or concrete check dam to allow the retained volume to slowly drain out. Care should be taken to ensure that the weep holes are not subject to clogging. For stone check dams, allow lower flows (two-year storm) to drain through the stone, while allowing higher flows (10-year storm) to drain through a lower section in the center (thereby reducing the potential erosion from water flowing around the sides of the check dam). Flows through a stone check dam are a function of stone size, flow depth, flow width, and flow path length through the dam.

Conveyance

It is highly recommended that a flow splitter or diversion structure be provided to allow larger flows to bypass this practice as needed. Contributing drainage areas should be limited to five acres and an overflow should be provided within the practice to pass the excess flows to a stabilized water course or storm drain. Weirs are common overflow systems with media filters and can control velocities so that they are non-erosive at the outlet point to prevent downstream erosion.

Media filters should be equipped with a minimum eight-inch diameter underdrain in a one-foot gravel bed. Increasing the size of the underdrain makes freezing less likely. The porous gravel bed prevents standing water in the system by promoting drainage. Gravel is also less susceptible to frost heaving than finer grained media. It is also highly recommended that a permeable filter fabric be placed between the underdrain and gravel layer but not extend laterally from the pipe more than two feet on either side (**Figure 2**).

Variations

Vegetated swale with infiltration trench

This option includes a six to 24-inch aggregate bed or trench, wrapped in a nonwoven geotextile (See Infiltration BMP for further design guidelines). The addition of an aggregate bed or trench can substantially increase volume control and water quality performance although cost is also increased.

Vegetated swales with infiltration trenches are best fitted for milder sloped swales (< 1 percent) or poorly-drained soils where the addition of the aggregated bed system can help to make sure that the maximum allowable ponding time of 48 hours is not exceeded. Ideally, the subsurface system should be designed like an infiltration trench (see Infiltration Practices BMP Fact Sheet). The subsurface trench should be comprised of terraced levels, though sloping trench bottoms may also be acceptable. The storage capacity of the infiltration trench may be added to the surface storage volume to achieve the desired storage.

Grass swale

Grass swales are essentially conventional drainage ditches. They typically have milder side and longitudinal slopes than their vegetated counterparts. Grass swales are usually less expensive than vegetated swales. However, they provide far less infiltration and pollutant removal opportunities and should be used only as pretreatment for other structural BMPs. Grassed swales, where appropriate, are preferred over catch basins and pipes because of their ability to increase travel time and reduce peak flow rates from a site.

Linear wetland swale

Wetland swales occur when the water table is located very close to the surface, incorporating long, shallow, permanent pools or marshy conditions that can sustain wetland vegetation. Like the dry swale, the entire water quality treatment volume is stored within a series of cells created by check dams.

Potential Applications

1. Residential – Swales can be used along road rights of ways and for side yard and backyard drainage.

2. Commercial/Industrial – Swales can provide site drainage around a site, within a site and can help take/slow discharge from other BMPs that outlet to the swale (**Figure 3**).

3. Ultra-urban – There may be some opportunity for swales in ultra-urban areas. However, swales are usually no less than two feet deep. With horizontal to vertical side slopes between 3:1 to 5:1(horizontal to vertical), the top width of the swale can prohibit its use in this setting where space is usually at a premium.

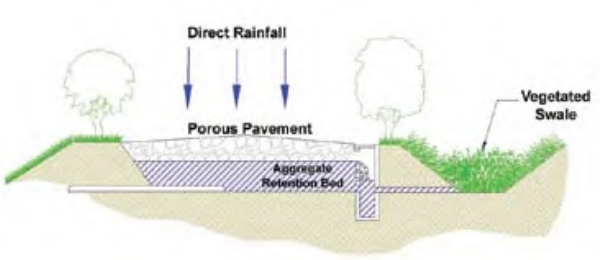


Figure 3 Slow discharge from porous pavement bed to vegetated swale
Source: Pennsylvania Stormwater BMP Manual 2006

4. Retrofit – Potential application in retrofit situations will depend strongly on space and topographic limitations. On sites with little to no slope, swales may not be the best drainage option. In these areas, swales may end up not moving water fast enough or may be prone to long periods of flooding or inundation in areas meant to be mostly dry.

5. Highway/Road – Vegetated swales are an excellent alternative to curb and gutter systems. Appropriately sized roadside swales should be able to handle all the runoff from the roadway and may also be able to handle runoff from areas outside the road surface.

Design Considerations

1. Sizing

   a. Convey the calculated peak discharge from a 10- year storm event. Use Manning's equation to calculate the velocity associated with the flow to make sure velocities are not larger than permissible velocities.

   b. Temporarily store and infiltrate the one-inch storm event, while providing capacity for up to the 10-year storm with 12 inches of freeboard.

   c. Flows for up to the 2-year storm should be conveyed without causing erosion.

   d. Maintain a maximum ponding depth of 18 inches at the end point of the channel, with a 12-inch average maintained throughout.

   e. The maximum ponding time should be 24 hours. It is critical that swale vegetation not be submerged during smaller storms, because it could cause the vegetation to bend over with the flow. This leads to reduced roughness of the swale, higher flow velocities, and reduced contact filtering opportunities.

   f. Bottom widths typically range from two to eight feet. The maximum bottom width to depth ratio for a trapezoidal swale should be 12:1.

2. Longitudinal slopes between one and six percent are recommended.

3. Swale side slopes are best within a range of 3:1 to 5:1 and should never be greater than 2:1 for ease of maintenance and side inflow from sheet flow.

4. Check dams

   a. Recommended for vegetated swales with longitudinal slopes greater than three percent or when additional detention or infiltration is desired.

   b. Should be constructed to a height of six to 18 inches and regularly spaced.

   c. Should be keyed into the bottom and sides of the swale, usually at least one to two feet on all sides. The height of the key on both sides should exceed the water surface elevation of the 10-year event by at least six inches.

   d. The middle of the check dam crest should be below the sides of the check dam to help

focus flow over the check dam and away from the channel sides.

5. Maximum drainage area is five acres.

6. Soil testing is required when infiltration is planned (Soil Infiltration Testing Protocol Appendix).

7. Runoff can be directed as concentrated flows or as lateral sheet flow drainage. Both are acceptable provided sufficient stabilization or energy dissipation is included. If flow is to be directed into a swale via curb cuts, provide a two- to three-inch drop at the interface of pavement and swale. Curb cuts should be at least 12 inches wide to prevent clogging and should be spaced appropriately to minimize the number of cuts but maximize area drained.

8. Soil should be at least 12 inches of loamy or sand with an infiltration rate of at least 0.5 inches per hour.

9. Inundation time is 24 hours. Rototill and replant swale if draw down time is more than 24 hours.

10. Prior to establishment of vegetation, a swale is particularly vulnerable to scour and erosion and therefore its seed bed must be protected with temporary erosion control, such as straw matting, straw-coconut matting, compost blankets, or fiberglass roving. Most vendors will provide information about the Manning's 'n' value and will specify the maximum permissible velocity. It is critical that the selected erosion control measure is adequate to prevent scour.

Stormwater Functions and Calculations

Utilize Manning's equation to calculate the velocity associated with the flow from the peak discharge of the 10 year storm or local standard. Maintain velocity of the 10 year and water quality criteria at non-erosive rates.

Manning's Equation

$$Q = VA = \frac{1.49}{n}\left(\frac{A}{WP}\right)^{2/3} S^{1/2}$$

Where;

Q = Flow in cfs
V = Velocity in ft/sec
A = Area in ft^2
n = Manning's roughness coefficient
WP = Wetted Perimeter in ft
S = Slope in ft/ft

Manning's roughness coefficient, or 'n' value in the equation, varies with the type of vegetative cover and design flow depth. Therefore, more conservative, lower numbers in the "n" value guidance tables should be used in design to determine flow velocities.

If driveways or roads cross a swale, culvert capacity may supersede Manning's equation for determination of design flow depth. In these cases, use standard culvert calculations to establish that the backwater elevation would not exceed the banks of the swale. If the maximum permissible velocity is exceeded at the culvert outlet, energy dissipation measures must be implemented.

Volume calculations (as it relates to the use of check dams)

The volume stored behind each check-dam (**Figure 4**) can be approximated from the following equation:

Storage Volume = 0.5 * (Length of Swale Impoundment Area per Check Dam) * (Depth of Check Dam) * [(Top Width of Check Dam) + (Bottom Width of Check Dam)] / 2

Active infiltration during the storm should also be accounted for when appropriate according to guidance provided in the Infiltration Practices BMP Fact Sheet.

Peak rate mitigation

Vegetated swales can help reduce peak flows by

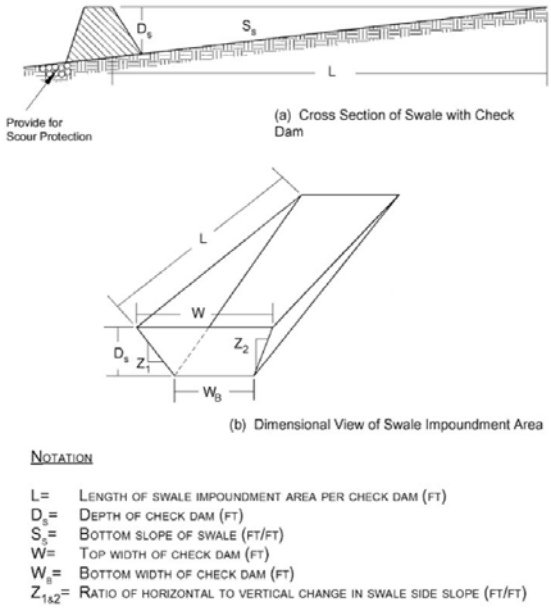


Figure 4 Storage behind check dam
Source: Northern Virginia Planning District Commission, 1992

increasing travel times, reducing volume through infiltration, and storing runoff behind check dams, culverts, etc.

Water quality improvement

Although the reported water quality benefits of vegetated swales vary widely, they can be expected to remove a high amount of total suspended solids (typically 70 percent to 90 percent), a low-to-medium amount of total phosphorus (approximately 10 percent to 50 percent), and a medium amount of total nitrogen (often 40 percent to 75 percent). There is some research to suggest that longer swales provide additional treatment. Vegetated swales can be used effectively for pretreatment prior to discharge to other BMPs.

Construction Guidelines

1. Begin vegetated swale construction only when the upgradient site has been sufficiently stabilized and temporary erosion and sediment control measures are in place. Vegetated swales should be constructed and stabilized very early in the construction schedule, preferably before mass earthwork and paving increase the rate and volume of runoff.

2. Rough grade the vegetated swale. Equipment should avoid excessive compaction and/or land disturbance. Excavating equipment should operate from the side of the swale and never on the bottom. If excavation leads to substantial compaction of the subgrade (where an infiltration trench is not proposed), the compacted soils should be removed and replaced with a blend of topsoil and sand to promote infiltration and biological growth. At the very least, topsoil should be thoroughly deep plowed into the subgrade in order to penetrate the compacted zone and promote aeration and the formation of macropores. Following this, the area should be disked prior to final grading of topsoil.

3. After rough grading, fine grade the vegetated swale. Accurate grading is crucial for swales. Even the smallest non-conformities may compromise flow capacity or soil stability.

4. Vegetation should consist of a dense and diverse selection of close-growing, water-tolerant plants (See Recommended Plant List for BMPs Appendix for complete list). Common species used in vegetated swales in Michigan include Canada Bluejoint (Calamagrostis canadensis), Virginia Wild Rye (Elymus virginicus), Switch Grass (Panicum virgatum), and Prairie Cord Grass (Spartina pectinata). Additionally, a cover crop of seed oats (Avena sativa) and Annual Rye (Lolium multiforum) should be used for quick germination and stability.

Maintenance

1. Irrigation will be necessary during plant establishment and may be needed in periods of little rain or drought. Vegetation should be established as soon as possible to prevent erosion and scour.

2. Stabilize freshly seeded swales with appropriate temporary or permanent soil stabilization methods, such as erosion control matting or blankets. Erosion control for

seeded swales should be required for at least the first 75 days following the first storm event after planting. If runoff velocities are high, consider sodding the swale or diverting runoff until vegetation is fully established.

3. Annually inspect and correct erosion problems, damage to vegetation, and sediment and debris accumulation (address when greater than three inches at any spot or covering vegetation).

4. Annually mow and trim vegetation to ensure safety, aesthetics, proper swale operation, or to suppress weeds and invasive vegetation. Dispose of cuttings in a local composting facility; mow only when swale is dry to avoid rutting.

5. Annually inspect for uniformity in cross-section and longitudinal slope; correct as needed.

6. Inspect and correctly check dams when signs of altered water flow (channelization, obstructions, etc.) are identified.

Winter Considerations

Plowing snow into swales will help insulate the bottom of the swale. However, snow that has accumulated salt or sand from de-icing operations should not be pushed into swales. Winter conditions also necessitate additional maintenance concerns, which include the following:

1. Inspect swale immediately after the spring melt, remove residuals (e.g., sand) and replace damaged vegetation without disturbing remaining vegetation.

2. If roadside or parking lot runoff is directed to the swale, mulching and/or soil aeration/manipulation may be required in the spring to restore soil structure and moisture capacity and to reduce the impacts of de-icing agents.

3. Use nontoxic, organic de-icing agents, applied either as blended, magnesium chloride-based liquid products or as pretreated salt.

4. Consider the use of salt-tolerant vegetation in swales.

Cost

Vegetated swales provide a cost-effective alternative to traditional curbs and gutters, including associated underground storm sewers (**Table 1**). The following table compares the cost of a typical vegetated swale (15-foot top width) with the cost of traditional conveyance elements.

It is important to note that the costs listed are strictly estimates and should be used for rough estimating purposes only. Also, these costs do not include the cost of activities such as clearing, grubbing, leveling, filling, and sodding of vegetated swale (if required). When all construction, operation, and maintenance activities are considered, the cost of vegetated swale installation and maintenance is far less than that of traditional conveyance elements.

	SWALE	Underground Pipe	Curb & Gutter
Construction Cost (per linear foot)	$4.50 - $8.50 (from seed)	$2 per foot per inch of diameter	$13 - $15
	$15 - $20 (from sod)	(e.g. a 12” pipe would cost $24 per linear foot)	
Annual O & M Cost (per linear foot)	$0.75	No Data	No Data
Total annual cost (per linear foot)	$1 from seed $2 from sod	No Data	No Data
Lifetime (years)	50		20

Table 1 Cost Comparison showing vegetated swale to pipe, curb, and gutter
Source: Bay Area Stormwater Management Agencies Association, June 1997

Designer/Reviewer Checklist for Vegetated Swales

Type of vegetated swale proposed: _____

ITEM	Page No.	YES	NO	N/A	NOTES
Can the swale safely (with freeboard) convey the 10-year event? ➢ *Max. drainage area:* ***5 acres*** ➢ *Max. ponding depth at end point of channel:* ***18 inches*** ➢ *Max. ponding time:* ***24 hours***	1,4				
Are bottom slopes between one percent and six percent?	1,4				
Are check dams provided for slopes > 3%?	4				
Are check dams adequately keyed into swale bottom and sides?	4				
Are two-year and ten-year flows non-erosive?	1,4,5				
Will the swale completely drain in 48 hours?	2,3				
Are side slopes between 3:1 and 5:1 H:V?	1,3,4				
Are swale soils loam, loamy sand or sandy loam? ➢ *Min. infiltration rate of permeable media:* ***0.5 in/hr***	5				
Underdrain provided for infiltration swales? ➢ *Min. diameter of underdrain:* ***8 inches***	2,3				
Vegetation and Manning's coefficient selected?	4,6				
Non-erosive inflow condition(s)?	--				
Erosion control provided during construction?	5				
Maintenance accounted for and plan provided?	6,7				
Additional Design Factors ➢ *Hydraulic residence time no less than 5 minutes; optimal time is 9 minutes* ➢ *Average velocity no greater than 0.9 ft/sec*					

➢ *Denotes Minimum Design Considerations*

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BMP Fact Sheet

NATIVE REVEGETATION

Native revegetation includes the restoration of forest savanna (scattered trees among prairie plants), and/or prairie. Revegetation should primarily use native vegetation due to the numerous benefits, including reduced maintenance needs.



Figure 1 Native vegetation utilized in a wetland, Philadelphia, PA (USEPA, picasaweb)

Potential Applications		Stormwater Quantity Functions	
Residential	Yes	Volume	Low/Med/High
Commercial	Yes	Groundwater Recharge	Low/Med/High
Ultra Urban	Limited	Peak Rate	Low/Med
Industrial	Yes	Stormwater Quality Functions	
Retrofit	Yes	TSS	High
Highway/Road	Limited	TP	High
Recreational	Yes	TN	Med/High
		Temperature	Med

Additional Considerations	
Cost	Low/Med
Maintenance	Low
Winter Performance	Medium

Variations

- Prairie
- No-mow lawn area
- Woodland
- Constructed wetlands
- Buffer areas
- Replacement lawn areas

Key Design Features

- Minimize traditional turf lawn area
- Develop landscape plan using native materials, determining the most appropriate
- Protect areas during construction
- Use integrated pest management (IPM) approach

Site Factors

- Water table to bedrock depth: N/A
- Soils: Vegetation should match soil types
- Slope: Applicable on most slopes (up to 1H:1V)
- Potential hotspots: No
- Maximum drainage area: Optimal is five times (maximum 20 times) the revegetated area

Benefits

- Low long-term maintenance needs
- Improves water quality
- Reduces volume

Limitations

- Establishment period requires more intensive maintenance, such as weeding and watering

Description and Function

Using native plants to vegetate an area is an effective method of improving the quality and reducing the volume of site runoff. Native plants significantly change the soil medium by adding carbon, decreasing bulk density, and increasing infiltration rates by as much as a factor of 10 or more even in clay soils (see Bharati, et.al, 2002 and Fuentes, et.al, 2004).

Native species are generally described as those existing in a given geographic area prior to European settlement. Over time, native vegetation does not typically require significant chemical maintenance by fertilizers and pesticides. This results in additional water quality benefits. Native species are typically more tolerant and resistant to pest, drought, and other local conditions than non-native species. Landscape architects and ecologists specializing in native plant species are usually able to identify a wide variety of plants that meet these criteria anywhere in the state. Recommended Plan List for BMPs Appendix provides lists of commercially available native species. Additional information relating to native species and their use in landscaping is available from the Indiana Native Plant and Wildflower Society at www.inpaws.org.

In addition to chemical applications, minimum maintenance also means minimal mowing and irrigation in established areas. Native grasses and other herbaceous materials that do not require mowing or intensive maintenance are preferred. Because selecting such materials begins at the concept design stage, this BMP can generally result in a site with reduced runoff volume and rate, as well as significant nonpoint source load reduction/ prevention.

A complete elimination of traditional lawns as a site design element can be a difficult BMP to implement, given the extent to which the lawn as an essential landscape design feature is embedded in current national culture. Instead, the landscape design should strategically incorporate areas of native plantings – surrounding limited turf grass areas – to act as buffers that will capture and filter stormwater flowing off of turf grasses or pavements.

Native species, being strong growers with denser root and stem systems than turf grass (**Figure 2**), result in:

- A greater volume of water uptake (evapotranspiration)
- Improved soil conditions through organic material and macropore formation
- Carbon sequestration
- Enhanced infiltration

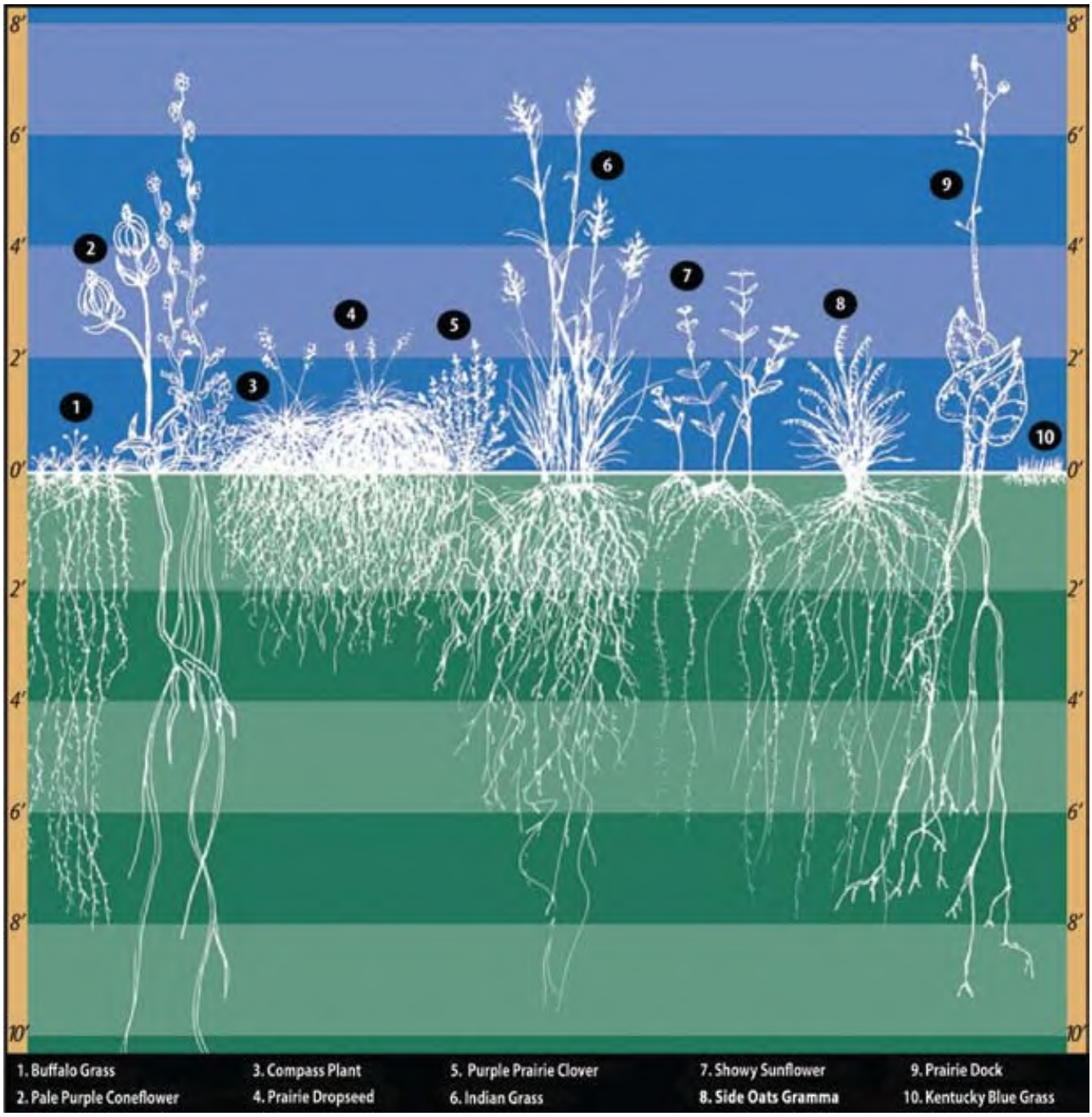


Figure 2 Native meadow species compared to turf grass
Source: JF New

If the objective is to revegetate an area with woodland species, the longer-term effect is a significant reduction in runoff volumes when contrasted with a conventional lawn planting. This decrease in runoff is caused by increases in interception, infiltration, evapotranspiration, and recharge. Peak runoff rate reduction also is achieved. Similarly, prairie reestablishment is also more beneficial than a conventional lawn planting. Again, these benefits are long term in nature and will not be apparent until the species have an opportunity to grow and mature (one advantage of the prairie planting is that this maturation process requires considerably less time than a woodland area).

In general, seeded prairie plantings grow roots in the first two years of planting, and by the third year, start to show substantial top growth. Therefore, a prairie planting may not be aesthetically pleasing during the first several years. Aesthetic expectations should therefore be adjusted accordingly. Posting

signs explaining this fact to passersby can increase understanding and alleviate concerns about the look of the new planting. The signs can also explain the environmental benefits of planting native grasses.

Variations

Most newly-created native landscapes in Indiana fall under the category of either woodlands or prairies. Woodlands will provide shade, vertical structure, and a high level of rainfall interception in the long term. However, woodlands typically require a significant amount of time to mature. Prairies, on the other hand, have a tendency to establish and regain function rather quickly (3-10 years), and can provide lower-growing vegetation with highly attractive native grasses and wildflowers.

Species selection for any native landscape should be based on function, availability, and level of appropriateness for site conditions. Native species plantings can achieve variation in landscape across a variety of characteristics, such as texture, color, and habitat potential. Plant material should be selected based on tolerance to standing water as identified in Recommended Plant List for BMPs Appendix.

Properly selected mixes of flowering prairie species can provide seasonal color; native grasses offer seasonal variation in texture. Seed production is a food source for wildlife and reinforces habitat. In all cases, selection of native species should strive to achieve species variety and balance, avoiding creation of single-species or limited species "monocultures" which pose multiple problems. In sum, many different aspects of native species planting reinforce the value of native landscape restoration, typically increasing in their functional value as species grow and mature over time. Examples include:

- Prairie – Install forb/grass matrix that bears similarities to historic Indiana prairies and savannas

- No-mow lawn area – Install low-growing native grasses that are used as a substitute for lawn or cool-season grass plantings.

- Woodland – Install a balance of native trees, shrubs, forbs, grasses, and sedges that would historically be represented in Indiana woodlands.

- Constructed wetlands – Historic drained wetlands or existing artificial low areas may be planted with wetland species that will thrive in standing water or saturated conditions.

- Buffer areas – Bands of re-established native vegetation occurring between impermeable surfaces, lawns, or other non-native land uses and existing natural areas.

- Replacement lawn areas – Existing turf lawns may be converted to native prairies, wetlands, or woodlands to minimize maintenance while increasing stormwater benefits and wildlife habitat.

Applications

- Residential – Native landscapes can be incorporated into common areas of residential developments. Additionally, individual homeowners may incorporate native landscapes into their own properties. Native revegetation should also be used to provide buffers around any existing natural areas that are undisturbed within the residential development.

- Commercial – Common areas and open spaces within commercial developments may be planted with native species, as well as any created detention/retention basins or artificial waterways. Native revegetation should also be used to provide buffers around any existing natural areas that are undisturbed within the commercial development.

- Ultra Urban – Use of native revegetation in limited in ultra-urban settings because of the lack of available green space. Wherever possible, however, native species should be incorporated.

- Industrial – Use of native revegetation in industrial settings is very similar to that in commercial settings.

- Retrofit – Established turf grass may be converted into prairie, woodland, or wetland.

- Highway/Road – Native plants may be established in rights-of-way to minimize long-term maintenance while establishing linear habitat corridors.

Design Considerations

The basis for native revegetation design scheme begins with assessing the site for:

- Existing native vegetation,
- Soil,
- Hydrologic regimes,
- Sun exposure, and
- Aesthetics

Existing native vegetation is a good starting point for determining what can thrive on a given site. However, the designer must also consider and balance various factors in developing a successful plant list. The hydrologic patterns set the stage for where the moisture continuum plants will be most successful (easily found in native plant resource guides). The amount of sun and shade that a given species tolerates is also critical in successful plant selection (and is easy to find as well). Soil texture and pH (less often found in resources guides) will further narrow the plant choices. If soils are strongly acidic or basic, the pH will greatly influence and reduce plant choices. Once the potential plant list has run through the sieves of moisture, sun/shade, and soil characteristics, the designer will hopefully have a suite of loosely associated native plants that grow in similar conditions.

Besides the plants' physical requirements, there is the cultural issue of aesthetics to consider. Common issues that people have with native landscapes are the potential height and lack of cultivated appearance (tall and thin, smaller flowers, looser look, etc.). If the designed areas are highly visible, then these aesthetic issues can be addressed with good design principles and a solid understanding of native plants.

1. **Analyze site's physical conditions**

   The most important physical condition of the site is the topography, hydrology, and soil, each of which will guide protection activities and plant selection. Evaluate the soil using the USDA soil survey to determine important soil characteristics such as flooding potential, seasonal high water table, soil pH, soil moisture, and other characteristics. Evaluate the topography based on USGS maps or a topographical survey of the site.

2. **Analyze site's vegetative features**

   Existing vegetation present at the site should be examined to determine the overall strategy for vegetation restoration and establishment. Strategies will differ whether pre-existing conditions are pasture, overgrown abandoned field, mid-succession forest, or another type of setting. An effort to inventory existing vegetation for protection and to determine type of pre-settlement vegetation should be made to guide efforts.

   a. *Identify desirable species*: Use native tree and shrub species that thrive in local habitat. These species should be identified in the restoration site and protected. Several native vines and shrubs can provide an effective ground cover during establishment of the area, though they should be controlled to prevent herbaceous competition.

   b. *Identify undesirable species*: Control invasive plants prior to planting new vegetation.

   c. *Identify sensitive species*: Because many areas are rich in wildlife habitat and could potentially harbor wetland plant species, be aware of any rare, threatened, or endangered plant or animal species. Take

care to protect sensitive species during restoration activities.

3. *Map the site*: Prepare an existing conditions sketch of the site that denotes important features, including stream width, length, stream bank condition, adjacent land uses, stream activities, desired width of buffer, discharge pipes, obstructions, etc.

4. *Create a design that meets multiple stakeholder objectives*

    a. *Landowner objectives*: Consider the current use of the existing vegetation, especially if the area will be protected by the landowner in perpetuity. Determine how the revegetated area will complement or conflict with existing and probable future uses of the property.

    b. *Local objectives:* Consider linking the revegetated area to an existing or planned green infrastructure system, which may include trails, parks, preserves, and wildlife habitat corridors. Evaluate how the new vegetation could help achieve local outdoor recreation goals.

    c. *Watershed objectives:* Examine the local watershed plan to identify goals related to establishing native plants. Have goals related to water quality been emphasized, or is wildlife habitat of primary concern? If no watershed plan has been prepared, examine other regional resource or recreation plans for reference to native plantings.

5. *Amend soil*: In those sites where soils have been disturbed, restore compromised soils by subsoiling and/or adding a soil amendment, such as compost. This will help in reestablishing its long-term capacity for infiltration and pollution removal.

6. Limit the development footprint as much as possible, preserving natural site features, such as vegetation and topography. In contrast to turf, "natural forest soils with similar overall slopes can store up to 50 times more precipitation than neatly graded turf." (Arendt, *Growing Greener*, pg. 81) If lawns are desired in certain areas of a site, they should be confined to those areas with slopes less than six percent.

7. Prairie restoration can reduce turf or create a buffer between turf and forest. Meadow buffers along forests help reduce off-trail trampling and direct pedestrian traffic in order to avoid "desire-lines" which can further concentrate stormwater.

    Prepare the site for a prairie planting by weeding well before planting and during the first year. Perennial weeds may require year-long smothering, repeated sprayings with herbicides, or repeated tillage with equipment that can uproot and kill perennial weeds.

    The site should be sunny, open, and well-ventilated, as prairie plants require at least a half a day of full sun.

    Erosion prone sites should be planted with a nurse crop (such as annual rye or seed oats) for quick vegetation establishment to prevent seed and soil loss. Steep slopes (25 percent or steeper) and areas subject to water flow should be stabilized with erosion blankets, selected to mitigate expected runoff volumes and velocities. Hydro-seeding is generally not recommended for native species. There is tremendous variation among seed suppliers; choose seeds with a minimum percent of non-seed plant parts. Native seed should also be PLS (Pure Live Seed) tested by a third party to gauge seed viability.

8. Converting turf grass areas to prairie requires that all turf be killed or removed before planting, and care taken to control weeds prior to planting.

9. Forest restoration includes planting of tree species, 12-18 inches in height, and shrubs at 18-24 inches, with quick establishment of an appropriate ground cover to stabilize the soil and prevent colonization of invasive species. Trees and shrubs should be planted on eight-

foot centers, with a total of approximately 430 trees per acre.

Reforestation can be combined with other volume control BMPs such as retentive berming, vegetated filter strips and swales. Plant selection should mimic the surrounding native vegetation and expand on the native species already found on the site. A mixture of native trees and shrubs is recommended and should be planted once a ground cover is established.

10. Ensure adequate stabilization, since native grasses, meadow flowers, and woodlands establish more slowly than turf. Stabilization can be achieved for forest restoration by establishing a ground cover before planting of trees and shrubs. When creating meadows, it may be necessary to plant a fast growing nurse crop with meadow seeds for quick stabilization. Annual rye can be planted in the fall or spring with meadow seeds and will establish quickly and usually will not present a competitive problem. Erosion prone sites should be planted with a nurse crop and covered with weed-free straw mulch, while steep slopes and areas subject to runoff should be stabilized with erosion control blankets suitable for the expected volume and velocity of runoff.

11. Prepare a landscape maintenance plan that identifies weeding plans, mowing goals, irrigation needs, and trimming of herbaceous perennials or key tree specimens, as needed.

Maintenance

Local land conservancies are excellent resources when considering the long-term stewardship of the area. If a site has critical value, a local conservancy may be interested in holding a conservation easement on the area, or may be able to provide stewardship services and assistance. Wild Ones (www.for-wild.org/) is a national organization with local chapters which may also provide stewardship resources.

Applying a carefully selected herbicide (Roundup® or similar glyphosate herbicide) around the protective tree shelters/tubes may be necessary, reinforced by selective cutting/manual removal, if necessary. This initial maintenance routine is often necessary for the first two to three years of growth and may be needed for up to five years until tree growth and tree canopy form, naturally inhibiting weed growth. Once shading is adequate, growth of invasive species and other weeds will be naturally prevented, and the woodland becomes self-maintaining. Survey the new woodland intermittently to determine if replacement trees should be provided. Keep in mind that some modest rate of planting failure is usual.

Prairie management is somewhat more straightforward. A seasonal mowing or burning may be required, although care must be taken to make sure that any management is coordinated with essential reseeding and other important aspects of meadow reestablishment. In addition, burning needs to be coordinated with the local fire department and follow local regulations. In the first year, weeds must be carefully controlled and consistently mowed back to four to six inches tall when they reach 12-18 inches in height.

In the second year, continue to monitor and mow weeds and hand-treat perennial or rhizomatous weeds with herbicide. Weeds should not be sprayed with herbicide if the drift from the spray may kill large patches of desirable plants; weeds will most likely move in to these new open areas. If necessary, controlled spot herbicide applications may be used to treat invasive plants if the treatments can be completed without damage to off-target vegetation.

A prescribed burn should be conducted at the end of the second or beginning of the third growing season. If burning is not possible, the prairie should be mowed very closely to the ground instead. If possible or practical, the mowed material should be removed from the site to expose the soil to the sun. This helps encourage rapid soil warming which favors the establishment of “warm season” plants over “cool season” weeds. Long-term maintenance should incorporate burning or mowing on a two to five year cycle to minimize woody species growth while encouraging development of the native prairie species.

Stormwater Functions and Calculations

Volume and peak rate

Native revegetation will lower runoff volume and peak rates by lowering the runoff coefficient (i.e., curve number). Designers can receive runoff reduction recognition based on the square feet of trees or shrubs being added. Proposed trees and shrubs to be planted under the requirements of these BMPs can be assigned a curve number (CN) reflecting a woodlot in "good" condition associated with the pre-development underlying soil layer for an area of 200 square feet per tree or the estimated tree canopy, whichever is greater. For shrubs, the area should be 25 square feet per shrub.

Water quality improvement

Landscape restoration using native species (which includes minimizing disturbance and maintenance), improves water quality preventively by minimizing the application of fertilizers and pesticides. Avoiding this nonpoint pollutant source is an important water quality objective.

Cost

Cost estimates for various aspects of native landscaping, including material and installation costs, are the following:

- $1,000-$2,500/acre for prairie installation or woodland understory installation
- $1,800-$2,600/acre for bare-root tree installation (10-foot spacing)
- $10-$20/plant for gallon-potted native perennial
- $2.50-$3.50/plant for plug-sized native perennial
- $250-$400/tree for balled-and-burlap tree installation

Costs for meadow re-establishment are lower than those for woodland, largely due to the need for tree installation. Again, such costs can be expected to be greater than installing a conventional lawn (seeding and mulching), although installation cost differences diminish when conventional lawn seeding is redefined in terms of conventional planting beds.

Cost differentials grow greater when longer term operating and maintenance costs are taken into consideration. If lawn mowing can be eliminated, or even reduced significantly to a once per year requirement, substantial maintenance cost savings result, often in excess of $2,000-$3,000 per acre per year.

If chemical application (fertilization, pesticides, etc.) can be eliminated, substantial additional savings result with use of native species. These reductions in annual maintenance costs resulting from a native landscape re-establishment quickly outweigh any increased installation costs that are required at project initiation. The aesthetic, water quality, and environmental protection benefits of native landscaping are clear. Nonetheless, implementation is often hindered because parties paying the higher up-front costs (usually the developer) are different than the parties reaping the benefits of reduced maintenance costs. Overcoming this impediment involves recognizing that native landscaping is another part of the "infrastructure" that communities must build into design in order to achieve the desired outcome of appearance and water quality protection.

Criteria to receive runoff reduction recognition for Native Revegetation

To receive runoff reduction recognition for native revegetation under a location regulation, the following criteria must be met:

- ☐ Area is protected by clearly showing the limits of disturbance on all construction drawings and delineated in the field.
- ☐ Area to receive runoff reduction recognition for trees is 200 square feet per tree or the estimated tree canopy, whichever is greater.
- ☐ Area to receive runoff reduction recognition for shrubs is 25 square feet per shrub.
- ☐ Area is located on the development project.
- ☐ Area has a maintenance plan that includes weeding and watering requirements from initial installation through ongoing maintenance.

Designer/Reviewer Checklist for Native Revegetation

ITEM	Page No.	YES	NO	N/A	NOTES
Avoidance of stormwater concentration as much as practical?	--				
Soil erodibility considered?	6				
Slope considered and appropriate?	6				
Existing and surrounding vegetation assessed, including desirable, sensitive, and non-native species?	5				
Site mapped?	5				
Does the design meet all stakeholder objectives, including storm-water, habitat, aesthetics, and timeframe for establishment?	5				
Does the soil require amendment?	6				
Erosion control matting, compost blankets, etc. provided as needed?	6				
Feasible construction process and sequence?	--				
Short and long-term maintenance accounted for and plan provided?	7				

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BMP Fact Sheet

RIPARIAN BUFFER RESTORATION

A riparian buffer is the area of land that exists between low, aquatic areas such as rivers, streams, lakes, and wetlands, and higher, dry upland areas such as forests, farms, cities, and suburbs. Unaltered riparian buffers may exist as various types of floodplain forest or wetland ecosystems. The Michigan Natural Features Inventory (MNFI) has identified multiple types of distinct natural communities which may occur in Michigan's riparian areas, such as southern floodplain forest, southern wet meadow, emergent marsh, and hardwood conifer swamp.



Figure 1 Suburban riparian buffer - Edward Drain, West Bloomfield, MI
Source: JF New

Applications		Stormwater Quantity Functions	
Residential	Yes	Volume	Low/Med
Commercial	Yes	Groundwater Recharge	Low/Med
Ultra Urban	Yes	Peak Rate	Low/Med
Industrial	Yes	Stormwater Quality Functions	
Retrofit	Yes	TSS	Med/High
Highway/Road	Limited	TP	Med/High
Recreational	Yes	NO_3	Med/High
		Temperature	Med/High

Additional Considerations	
Cost	Low/Med
Maintenance	Low
Winter Performance	High

Key Design Features

Riparian buffers consist of three distinct zones:

- **Zone 1**: Streamside zone extends a minimum distance of 25 feet
- **Zone 2**: Middle zone extends immediately from the outer edge of Zone 1 for a minimum of 55 feet
- **Zone 3**: Outer zone extends a minimum of 20 feet immediately from outer edge of Zone 2

Site Factors

- Water table to bedrock depth: N/A
- Soils: Match vegetation to soils to maximize long-term viability of plantings
- Slope: N/A
- Potential hotspots: No
- Maximum drainage area: 5 to 20 times the buffer area

Benefits

- Water Quality
- Ecological and aesthetic value
- Low cost

Limitations

- Reduced volume and peak conrtol

Description and Function

A riparian buffer is a permanent restoration area of trees, shrubs, and herbaceous vegetation adjacent to a waterbody that serves to protect water quality and provide critical wildlife habitat. A riparian buffer can be designed to intercept surface runoff and subsurface flow from upland sources for the purpose of removing or buffering the effects of associated nutrients, sediment, organic matter, pesticides, or other pollutants prior to entry into surface waters and groundwater recharge areas.

The riparian buffer is most effective when used as a component of a sound land management system including nutrient management and runoff and sediment and erosion control practices. Use of this practice without other runoff and sediment and erosion control practices can result in adverse impacts on riparian buffer vegetation and hydraulics including high maintenance costs, the need for periodic replanting, and the flow of excess nutrients and sediment through the buffer.

Riparian buffer restoration areas consist of three distinct zones and can be designed to filter surface runoff as sheet flow and down-slope subsurface flow, which occurs as shallow groundwater. For the purposes of these buffer strips, shallow groundwater is defined as saturated conditions which occur near or within the root zone of trees and other woody vegetation and at relatively shallow depths where bacteria, low oxygen concentrations, and soil temperature contribute to denitrification. Riparian buffers are designed to encourage sheet flow and infiltration and impede concentrated flow.

Buffer widths and vegetation types

When developing specific widths for riparian buffers (**Figure 2**), keep site specific factors in mind, and use exact measurements as a guide for each site. Various buffer widths and vegetation types may be appropriate depending on:

- Project goals,
- The natural features of the river valley, wetlands, lake, and floodplain, and
- Wildlife habitat requirements.

Buffer averaging and minimum distances

Buffer ordinances that set specific and minimum buffer dimensions allow the local government to accept buffer averaging in order to accommodate variability in terrain or development plans. For example, a wetland normally entitled by ordinance to a 75-foot minimum buffer may be able to tolerate a 50-foot buffer over part of its margin if a wider buffer is provided along another part. This depends upon such issues as water flow, topography, habitat, and species needs, and other factors that can best be assessed on a case-by-case basis.

Port Townsend, Washington allows buffer averaging if the applicant demonstrates that the averaging will not adversely affect wetland functions and values, that the aggregate area within the buffer is not reduced, and that the buffer is not reduced in any location by more than 50 percent or to less than 25 feet.

Woodbury, Minnesota allows buffer averaging where averaging will provide additional protection to the wetland resource or to environmentally valuable adjacent uplands, provided that the total amount of buffer remains the same.

Source: Environmental Law Institute

Zone 1: Also termed the "streamside zone," begins at the edge of the stream bank of the active channel and extends a minimum distance of 25 feet; this is measured horizontally on a line perpendicular to the water body.

Undisturbed vegetated area helps protect the physical and ecological integrity of the stream ecosystem. The vegetative target for the streamside zone is undisturbed native woody species with native plants forming canopy, understory, and duff layer where such forest does not grow naturally; then native vegetative cover appropriate for the area (such as grasses, forbs, or shrubs) is the vegetative target. (*HRWC Model Ordinance*, p. 8)

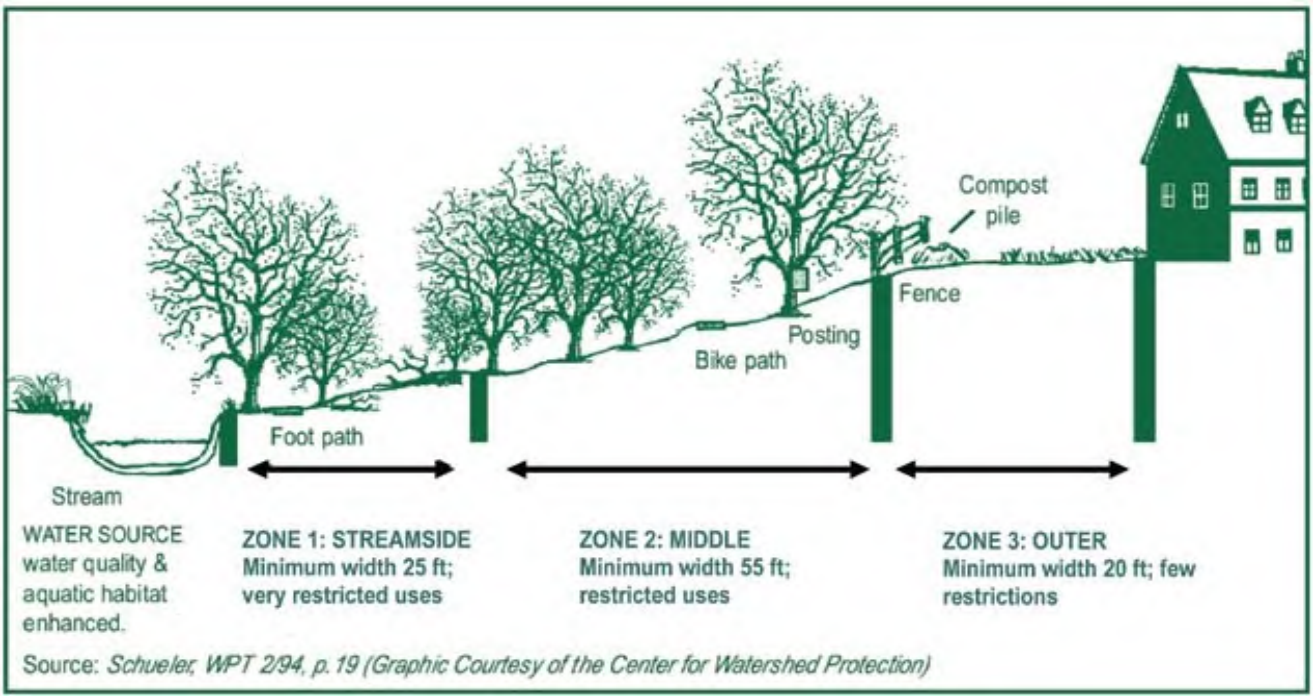


Figure 2 Source of a three-zone buffer
Source: Schueler, Watershed Protection Techniques, 1994 (Graphic courtesy of the Center for Watershed Protection)

Zone 2: Also termed the "middle zone," extends immediately from the outer edge of Zone 1 for a minimum distance of 55 feet.

This managed area of native vegetation protects key components of the stream ecosystem and provides distance between upland development and the stream-side zone. The vegetative target for the middle zone is either undisturbed or managed native woody species or, in its absence, native vegetative cover of shrubs, grasses, or forbs. Undisturbed forest, as in Zone 1, is strongly encouraged to protect further water quality and the stream ecosystem. (*HRWC Model Ordinance* p. 8)

Zone 3: Also termed the "outer zone," it extends a minimum of 20 feet immediately from outer edge of Zone 2.

This zone prevents encroachment into the riparian buffer area, filters runoff from adjacent land, and encourages sheet flow of runoff into the buffer. The vegetative target for the outer zone is native woody and herbaceous vegetation to increase the total width of the buffer. Native grasses and forbs are acceptable. (*HRWC Model Ordinance* p. 8)

To maximize wildlife habitat, restoration buffers should reflect the type of riparian vegetation that was found at the site before alteration. If water quality protection is the primary goal, greater emphasis may be placed on installing vegetation that enhances soil stability and absorbs pollutants. If the riparian area is very wet, wetland vegetation may be required.

In addition to installing vegetation, riparian buffer restoration may require physical restoration of soils, topography, or hydrology to achieve the desired result. Geographic factors such as the presence of steep slopes may necessitate an expanded buffer to achieve soil stability. If a river valley is very narrow, the buffer may be adjusted accordingly.

Applications

Riparian buffers are used adjacent to any wetland and bodies of water, such as lakes, streams, swales, and detention ponds. They are not typically applicable in upland areas where water bodies are not present. While riparian buffers provide

significant water quality and ecological benefits, they have only very little benefit for volume control, unless they have some ability to trap and rapidly infiltrate water. Therefore, they should be used with other BMPs that will fulfill any volume control requirements.

Restoring riparian buffers can be applied in many settings:

1. Adjacent to permanent or intermittent streams,
2. At the margins of lakes or ponds,
3. At the margin of intermittent or permanently flooded, environmentally sensitive, open water wetlands,
4. On karst formations at the margin of sinkholes and other small groundwater recharge areas, and
5. Between manicured lawns, cultivated areas or hardscape and swales, streams or rivers to help dissipate and treat runoff and help stabilize the tops of channel banks.

Design Considerations

Restoring riparian buffer areas requires a plan to ensure long-term success. Below is a summary of the steps that groups, designers, engineers, or volunteers should undertake during the planning stages of a riparian buffer project.

1. **Confirm suitability for restoration**

   If stream banks are extensively eroded, consider an alternative location for preparing the riparian buffer, or consider stream bank restoration first. Rapidly eroding stream banks may undermine top of bank restoration efforts.

2. **Analyze site's physical conditions**

   Consider site specific factors to determine the particular width of the individual zones:

   - Watershed condition,
   - Slope,
   - Stream order,
   - Soil depth and erodibility,
   - Hydrology,
   - Seasonal high water table,
   - Floodplains,
   - Wetlands,
   - Streambanks,
   - Soil type,
   - Vegetation type, and
   - Stormwater systems.

3. **Analyze site's vegetative features** Existing vegetation at the restoration site should be examined to determine the overall strategy for buffer protection and establishment. Strategies will differ whether pre-restoration conditions are pasture, overgrown abandoned field, mid-succession forest, predominantly invasive vegetation, or another type of setting. An effort to inventory existing vegetation for protection and to determine type of pre-settlement vegetation should be made to guide efforts.

   - *Identify desirable species*: Native tree and shrub species that thrive in riparian habitats in Michigan should be used. These species should be identified in the restoration site and protected. Several native vines and shrubs can provide an effective ground cover when establishing the buffer, though they should be controlled to prevent herbaceous competition.

   - *Identify non-native and invasive species*: Consider using undesirable species for shading during buffer establishment. Control invasive plants prior to buffer planting may be necessary.

   - *Identify sensitive species*: Because riparian zones are rich in wildlife habitat and wetland plant species, be aware of any rare, threatened, or endangered plant or animal species. Be sure to protect sensitive species during riparian buffer restoration.

4. **Map the site**

   Prepare an existing conditions sketch of the site noting important features such as stream width, length, stream bank condition, adjacent land uses, stream activities, desired width of buffer, discharge pipes, obstructions, etc.

5. **Create a design that accomplishes multiple stakeholder objectives**

   Ideally, the three-zone system should be incorporated into the design to meet landowner, community, and watershed objectives:

   - *Landowner objectives*: Consider the current use of the buffer by the landowner, especially if the buffer will be protected by the landowner in perpetuity. How will the riparian buffer complement or conflict with existing and probable future uses of the property?

   - *Community objectives*: Consider linking the buffer to an existing or planned green infrastructure system, which may include trails, parks, preserves, and wildlife habitat buffers. How can a buffer help achieve local recreation and green space goals?

   - *Watershed objectives*: Examine the local watershed plan to identify goals related to riparian buffers. Have goals related to water quality been emphasized. Is wildlife habitat a primary concern?

6. **Design measures**

   The following elements represent a menu of design measures for riparian and natural resource protection that communities may choose to encourage or require developers to incorporate during the site plan review process.

   - *Stream size* – A majority of Indiana's stream system is comprised of small streams (first, second, and third order). It is important to reduce nutrient inputs to these streams.

   - *Availability of areas for continuous buffers* – Establishing continuous riparian buffers on the landscape should be given a priority over establishing fragmented buffers. Continuous buffers provide better shading and water quality protection as well as buffers for the wildlife movement.

   - *Degrees of degradation* – Urban streams have often been buried or piped as a result of previous development. Streams in areas without forestation may benefit the most from buffer restoration.

   - *Loading rates* - The potential for removing pollutants is generally highest where nutrient and sediment loading are the highest.

   - *Land uses* – Land uses adjacent to the riparian buffer may influence the required buffer width and vegetation types. While the three-zone riparian buffers described herein are ideal, the full widths of each zone may not always be feasible to establish, especially in urban areas.

   - *Habitats* – Establishing a buffer for habitat enhancement requires additional strategies beyond installing a buffer for increased water quality.

7. **Determine the appropriate buffer width**

   Riparian buffer areas need not have a fixed linear boundary, but may vary in shape, width, and vegetative type and character, depending on the goals of the restoration and the natural geography of the water body and riparian area. The desired function of the buffer (habitat, water quality, etc.) determines buffer width. Many factors, including slope, soil type, adjacent land uses, floodplain, vegetative type, and watershed condition influence the design of the buffer. A rule of thumb is "the bigger, the better." Buffer widths for water quality and habitat maintenance should generally be 35 to100 feet. Buffers less than 35 feet generally do not protect aquatic resources in the long term.

   - *Streamside buffers*

The minimum width of streamside buffer areas can be determined by a number of methods suitable to the geographic area.

Based on soil hydrologic groups as shown in the soil survey report, the width of Zone 2 should be increased to occupy any soils designated as Hydrologic Group D and those soils of Hydrologic Group C that are subject to frequent flooding. If soils of Hydrologic Groups A or B occur adjacent to intermittent or perennial streams, the combined width of Zones 1 and 2 may be limited to the 80-foot minimum.

Based on area, the width of Zone 2 should be increased to provide a combined width of Zones 1 and 2 equal to one-third of the slope distance from the stream bank to the top of the pollutant source area. The effect is to create a buffer strip between field and stream that occupies approximately one-third of the source area.

- Pond and lake-side buffers

  The area of pond or lake-side buffer strips should be at least one-fifth the drainage area of the cropland and pastureland source area. The width of the buffer strip is determined by creating a uniform width buffer of the required area between field and pond. Hydrologic group determining width remains the same as for streamside buffers. Minimum widths apply in all cases.

8. **Vegetation selection**

   Zone 1 and 2 vegetation should consist of native streamside species on soils of Hydrologic Groups C and D and native upland species on soils of Hydrologic Groups A and B.

   Deciduous species are important in Zone 2 due to the production of carbon leachate from leaf litter, which drives bacterial processes that remove nitrogen and sequester nutrients in growth processes. In warmer climates, evergreens are also important due to the potential for nutrient uptake during the winter months. In both cases, a variety of species is important to meet the habitat needs of insects important to the aquatic food chain.

   Zone 3 vegetation should consist of perennial grasses and forbs. Species recommendations for restoring riparian buffers depend on the geographic location of the buffer. Suggested species lists can be developed in collaboration with appropriate state and federal forestry agencies, the Natural Resources Conservation Service, and the USDA Fish and Wildlife Service. Species lists should include trees, shrubs, grasses, legumes, and forbs, as well as site preparation techniques. Please refer to the plant list in Recommended Plant List for BMPs Appendix for a recommended list of native trees and shrubs.

   The choice of planting stock (seeds, container seedling, bare-root seedlings, plugs, etc.) is often determined by cost. Larger plants usually cost more, though will generally establish more rapidly.

   Many factors threaten the long-term viability of riparian plant protection or establishment. With proper foresight, these problems can be minimized. The following items should be considered during the planning stage:

Green Development Standards

In 2007, the U.S. Green Building Council finalized pilot rating standards for the new Leadership in Energy and Environmental Design – Neighborhood Development (LEED –ND) certification program, which set standards for environmentally superior development practices. Developers can earn certification credit for preserving a buffer around all wetlands and water bodies located on site in perpetuity. Local governments that adopt buffer ordinances encourage LEED-ND developments.

Source: Environmental Law Institute

- Deer control
    - Look for signs of high deer densities, including an overgrazed understory with a browse line five to six feet above the ground.
    - Select plants that deer do not prefer (e.g., paper birch, beech, common elderberry)
    - Apply homemade deer repellants
    - Install tree shelters
- Tree shelters
    - Tree shelters, such as plastic tubes that fit over newly planted trees, are extremely successful in protecting seedlings. They may be secured with a wooden stake and netting may be placed over top of the tree tube. They are recommended for riparian plantings where deer or human intrusion may be a problem. Tree shelters should be removed two to three years after the saplings emerge. ° Tree shelters protect trees from accidental strikes from mowing or trimming.
    - Tree shelters create favorable microclimate for seedlings.
    - Tree shelters should be inspected at least four times per year. The following maintenance should be performed as necessary:
        - Repair broken stakes
        - Tighten stake lines
        - Straighten leaning tubes
        - Clean debris from tube
        - Remove netting as tree grows
        - Remove when tree trunk is approximately two inches wide
- Stream buffer fencing
    - Farm animals may cause great damage to stream banks. Consider permanent fencing such as high-tensile smooth wire fencing or barbed fencing.
    - The least expensive fencing is eight-foot plastic fencing, which is also effective against deer and is easily repaired
- Vegetation
    - Consider using plants that are able to survive frequent or prolonged flooding conditions. Plant trees that can withstand high water table conditions.
    - Soil disturbance can allow for unanticipated infestation by invasive plants.
- Accidental or purposeful destruction by landowners
    - Signage, posts, fencing, boulders, etc., may be required to alert adjacent landowners to the location, purpose, and management aims of riparian buffers. This is particularly important where actively managed landscaped areas abut native plant buffers. Signs that stress no mow/no pesticide and fertilizer zones may need to be in several languages, e.g., English and Spanish.

9. **Restoration design within your budget** The planting design (density and types) must ultimately conform to the financial constraints of the project. See discussion below for estimating direct costs of planting and maintenance.

10. **Draw a restoration planting plan**

- Planting layout: The planting plan should be based on the plant types and density. The plan must show the site with areas denoted for trees and shrub species and plant spacing and buffer width.
- Planting density: Trees should be planted at a density sufficient to provide 320 trees per

Spacing (feet)	Trees (number)	Spacing (feet)	Trees (number)	Spacing (feet)	Trees (number)
2x2	10,890	7x9	691	12x15	242
3x3	4,840	7x10	622	12x18	202
4x4	2,722	7x12	519	12x20	182
4x5	2,178	7x15	415	12x25	145
4x6	1,815	8x8	681	13x13	258
4x7	1,556	8x9	605	13x15	223
4x8	1,361	8x10	544	13x20	168
4x9	1,210	8x12	454	13x25	134
4x10	1,089	8x15	363	14x14	222
5x5	1,742	8x25	218	14x15	207
5x6	1,452	9x9	538	14x20	156
5x7	1,245	9x10	484	14x25	124
5x8	1,089	9x12	403	15x15	194
5x9	968	9x15	323	15x20	145
5x10	871	10x10	436	15x25	116
6x6	1,210	10x12	363	16x16	170
6x7	1,037	10x15	290	16x20	136
6x8	908	10x18	242	16x25	109
6x9	807	11x11	360	18x18	134
6x10	726	11x12	330	18x20	121
6x12	605	11x15	264	18x25	97
6x15	484	11x20	198	20x20	109
7x7	889	11x25	158	20x25	87
7x8	778	12x12	302	25x25	70

Table 1 Tree spacing per acre

acre at maturity. To achieve this density, approximately 436 (10 x 10 feet spacing) to 681 (8 x 8 feet spacing) trees per acre should be planted initially. Some rules of thumb for tree spacing and density based on plant size at installation follow:

- Seedlings 6 to 10 feet spacing (~700 seedlings/acre)
- Bare root stock 4 to 16 feet spacing (~200 plants/acre)
- Larger & Container 16 to 18 feet spacing (~150 plants/acre)

Formula for estimating number of trees and shrubs:

Number of Plants = length x width of buffer (feet) / 50 ft^2

This formula assumes each tree will occupy an average of 50 square feet, random placement of plants approximately 10 feet apart, and a mortality rate of up to 40 percent.

Alternatively, the table below can be used to estimate the number of trees per acre needed for various methods of spacing.

11. **Prepare site for restoration** Existing site conditions determine the degree of preparation needed prior to planting. Invasive plant infestation and vegetative competition are variable and must be considered in the planning stages. Site preparation should begin in the fall prior to planting. Determine whether the use of herbicides is necessary.

    Mark the site with flags, or marking paint, so that the plants are placed in the correct locations.

County Regulated Drain Considerations: Special attention needs to be paid regarding the type and density of vegetation used if the natural drainage pathway or stream is or will be a part of the County Regulated Drain system. The County may require

that at least one side of the regulated drain is clear of woody vegetation, with continuous access provided for reconstruction and maintenance. Pre-coordination with the local County Surveyor's Office is highly recommended.

Stormwater Functions and Calculations

Volume and peak rate

Restoration of the riparian buffer will lower runoff volume and peak rates through lowering the runoff coefficient (i.e., curve number). Designers can receive runoff reduction recognition based on the square feet of trees or shrubs being added. Proposed trees and shrubs to be planted under the requirements of these BMPs can be assigned a curve number (CN) reflecting a woodlot in "good" condition associated with the pre-development underlying soil layer for an area of 200 square feet per tree or the estimated tree canopy, whichever is greater. For shrubs, calculate based an area of 25 square feet per shrub.

Water quality improvement

Water quality benefits of restoring riparian buffers are medium to high. The amount of benefit is based on flow characteristics and nutrient, sediment, and pollutant loadings of the runoff as well as the length, slope, type, and density of vegetation in the riparian buffer.

Runoff entering Zone 3 filters sediment, begins nutrient uptake, and converts concentrated flow to uniform, shallow sheet flow. Zone 2 provides contact time and carbon energy sources in which buffering processes can take place. It also provides long-term sequestering of nutrients. Zone 1 provides additional soil and water contact area to further facilitate nutrient buffering processes, provides shade to moderate and stabilize water temperature, and encourages production of beneficial algae.

Maintenance

An effective riparian buffer restoration project should include stewardship guidelines to manage and maintain the site in perpetuity. The most critical period of riparian buffer establishment is canopy closure, which is typically the first three to five years after saplings are planted. Buffer boundaries should be well defined with clear signs or markers. During this time, the riparian buffer should be monitored four times annually (February, May, August, and November are recommended) and inspected after any severe storm. Maintenance measures that should be performed regularly include:

1. **Watering**

   - Plantings need deep, regular watering during the first growing season, either natural watering via rainfall, or planned watering via caretaker.
   - Planting in the fall increases the likelihood of sufficient rain during planting establishment.

2. **Mulching**

   - Mulch provides moisture retention in the root zone of plantings, or potentially impacted vegetation from construction, moderate soil temperature, and some weed suppression.

   - Use coarse, organic mulch that is slow to decompose in order to reduce the need for repeat application.

   - Apply a two to four-inch layer, leaving air space around tree trunk to prevent fungus growth.

   - Use a combination of woodchips, leaves, and twigs that have been stockpiled for six months to a year.

3. **Weed and invasive plan control**

   - Invasive plants can overrun even a well-designed planting. It is essential that there is a plan in place to monitor and remove invasive vegetation as the planting matures. Use the Nature Conservancy's Global Invasive Species Team Web page as a resource for management techniques.

(http://tncweeds.ucdavis.edu/esadocs.html) Non-chemical weed control methods are preferred since chemicals can easily be washed into the stream.

- *Herbicides*

  Using herbicides is a short-term maintenance technique (two to three years) that is generally considered less expensive and more flexible than mowing and will result in a quicker establishment of the buffer. Consider and evaluate the proximity of herbicide use to water features.

- *Mowing*

  Mowing controls the height of the existing grasses, yet increases nutrient uptake. Therefore, competition for nutrients will persist until the canopy closure shades out lower layers of growth. A planting layout similar to a grid format will facilitate ease of mowing, but will yield an unnaturally spaced community. Mowing may result in strikes to tree trunks unless protective measures are used. Mowing should occur twice each growing season. Mower height should be set between eight and12 inches.

- *Weed mats*

  Weed mats are geo-textile fabrics used to suppress weed growth around newly planted vegetation by blocking sunlight and preventing seed deposition. Weed mats are installed after planting, and should be removed once the trees have developed a canopy that will naturally shade out weeds.

4. **Stable debris**

   As Zone 1 reaches 60 years of age or is hit with pests or disease, it will begin to produce large debris. Large debris, such as logs, create small dams which trap and hold debris for processing by aquatic insects, thus adding energy to the stream ecosystem, strengthening the food chain, and improving aquatic habitat. Wherever possible, stable debris should be conserved.

   - Where debris dams must be removed, try to retain useful, stable portions which can provide storage. (A state permit may be required). For guidance on evaluating debris impacts on streams and methods for managing debris jams, refer to the "Primer on Large Woody Debris Management" developed by the City of Rochester Hills (see References).

   Deposit removed material a sufficient distance from the stream so that it will not be refloated by high water.

5. **Resources for assistance** Local land conservancies are excellent resources when considering the long-term stewardship of the area. If a site has critical value, a local conservancy may be interested in holding a conservation easement on the area, or may be able to provide stewardship services and assistance. Wild Ones (www.for-wild.org/) is a national organization with local chapters which may also provide stewardship resources.

Winter Considerations

Volume reduction, peak rate mitigation, and water quality benefits are not as pronounced in winter months compared to the rest of the year in riparian buffers because infiltration rates are generally lower during prolonged cold weather periods. In addition, evapotranspiration rates are lower in winter months because most vegetation is dormant. However, riparian buffers still provide stormwater management benefits even in winter.

Cost

Installing a riparian buffer involves site preparation, planting, second year reinforcement planting, and additional maintenance. Costs may fluctuate based on numerous variables including whether or not volunteer labor is used, and whether plantings and other supplies are donated or provided at a reduced cost. The following table presents an estimate of typical costs for riparian buffer restoration.

Criteria to receive runoff reduction recognition for Riparian Buffer Restoration

To receive runoff reduction recognition for riparian buffer restoration under a location regulation, the following criteria must be met:

- ☐ Area is protected by having the limits of disturbance clearly shown on all construction drawings and delineated in the field. 3
- ☐ Area to receive runoff reduction recognition for trees is 200 square feet per tree or the estimated tree canopy, whichever is greater.
- ☐ Area to receive runoff reduction recognition for shrubs is 25 square feet per shrub.
- ☐ Area is located on the development project.
- ☐ Area has a maintenance plan that includes weeding and watering requirements from initial installation throughout ongoing maintenance.

Designer/Reviewer Checklist for Riparian Buffer Restoration

ITEM	Page No.	YES	NO	N/A	NOTES
Avoidance of stormwater concentration as much as practical?	--				
Appropriate buffer widths designed? ➢ *Ideal minimum widths* o *Zone 1: **25 feet*** o *Zone 2: **55 feet*** o *Zone 3: **20 feet*** o *Water Quality/Habitat Maintenance: **35-100 feet*** ➢ *Various Buffer widths and vegetation types may be appropriate depending on:* o *Project Goals* o *Natural features of river valley, wetlands, lake, and floodplain* o *Wildlife Habitat Requirements*	1-3, 5,6				
Soil erodibility considered?	4				
Slope considered and appropriate?	--				
Appropriate vegetation selected based on soils, hydrology, and ecoregion?	6,7				
Appropriate vegetation selected based on budget and aesthetics?	--				
Appropriate plant spacing designed?	7,8				
Appropriate balance of woody to herbaceous species?	6				
Seasonality of planting/construction considered?	8				
Erosion and sedimentation control provided?	--				
Maintenance accounted for and plan provided?	9,10				

➢ *Denotes Minimum Design Considerations*

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BMP Fact Sheet

SOIL RESTORATION

Soil is a key ingredient in effective stormwater and water quality management, making proper care of soils a key component of low impact development.

Soil restoration is a technique used to enhance and restore soils by physical treatment and/or mixture with additives – such as compost – in areas where soil has been compacted. Soil media restoration increases the water retention capacity of soil, reduces erosion, improves soil structure, immobilizes and degrades pollutants (depending on soil media makeup), supplies nutrients to plants, and provides organic matter. Soil restoration is also used to reestablish the soil's long term capacity for infiltration and to enhance the vitality of the soil as it hosts all manner of microbes and plant root systems in complex, symbiotic relationships.



Table 1 Soil restoration, Colerain Park, OH (Colerain Township)

Applications		Stormwater Quantity Functions	
Residential	Yes	Volume	Medium
Commercial	Yes	Groundwater Recharge	Low
Ultra Urban	Yes	Peak Rate	Medium
Industrial	Yes	Stormwater Quality Functions	
Retrofit	Limited	TSS	High*
Highway/Road	Yes	TP	High*
Recreational	Yes	NO_3	Medium
		Temperature	Medium

Additional Considerations	
Cost	Medium
Maintenance	Low
Winter Performance	High

Key Design Features

- Follow nonstructural BMP to minimize soil compaction
- Evaluate existing soil conditions using methods referenced in Soil Infiltration Testing Protocol (found in AppendixD4) before creating a soil restoration strategy
- Soil media used in restoration is either organic or inorganic (man-made) and is mixed into existing soil

Benefits

- Widely applicable
- Relatively low cost
- Additional benefits such as improved plant health and reduced erosion

Limitations

- Relatively limited stormwater benefits on a unit area basis

*Newly amended soils are susceptible to erosion and release of TSS and phosphorus until stabilized with mulch, erosion blanket, sod, or some other covering.

Description and Function

Soil can be restored after construction to partially recondition that which has been degraded by compaction. Bulk density field tests measure soil compaction and can be used to help determine if soil restoration is necessary. Restoring the soil improves its structure and function, increases infiltration potential, and supports healthy vegetative communities.

A healthy soil (**Figure 1**) provides a number of vital functions including water storage and nutrient storage, regulate the flow of water, and immobilize and degrade pollutants. Healthy soil contains a diverse community of beneficial microorganisms, a sufficient amount of plant nutrients (nitrogen and phosphorous), some trace elements (e.g., calcium and magnesium), and organic matter (generally 5 to 10 percent). Healthy soil typically has a neutral or slightly acidic pH and good structure which includes various sizes of pores to support water movement, oxygenation, and a variety of other soil processes.

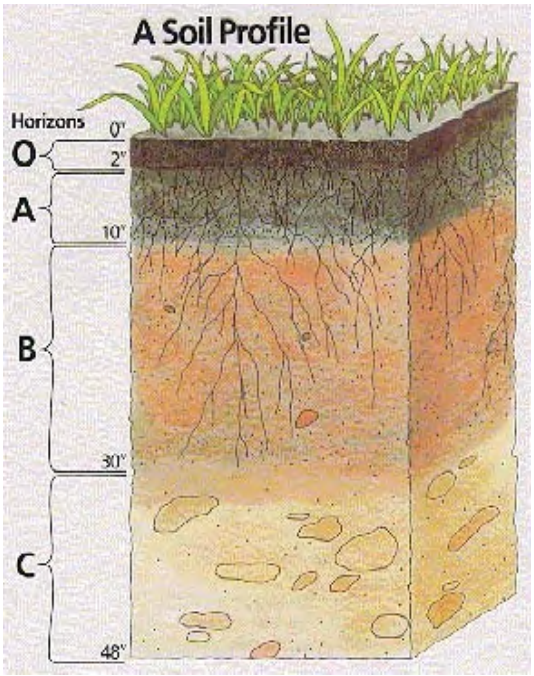


Figure 1 Healthy Soil Profile
Source: USDA NRCS

Caring for soil is also a critical component of water management, especially during development activities, such as construction grading, which often result in erosion, sedimentation, and soil compaction. Proper protection and restoration of soil is a critical BMP to combat these issues. Soil restoration prevents and controls erosion by enhancing the soil surface to prevent the initial detachment and transport of soil particles.

Soil compaction

Soil compaction is the enemy of water quality protection. Soil compaction occurs when soil particles are pressed together, reducing the pore space necessary to allow for the movement of air and water throughout the soil (**Figure 2**). This decrease in porosity causes an increase in bulk density (weight of solids per unit volume of soil). The greater the bulk density of the soil, the lower the infiltration and therefore, the larger the volume of runoff.

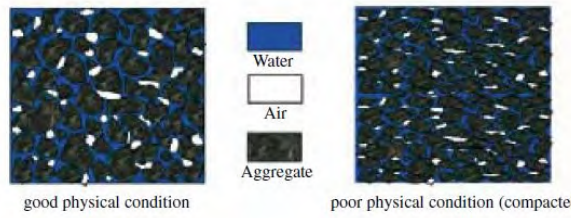


Figure 2 Compacted soil constrains movement of air and water
Source: USDA NRCS

Compaction limits vegetative root growth, restricting the health of plants as well as the biological diversity of the soil. Compaction also affects the infiltrating and water quality capacity of soils. Soil compaction can lead to increased erosion and stormwater runoff, low infiltration rates, increased flooding, and decreased water quality from polluted runoff. After compaction, a typical soil has strength of about 6,000 kilopascals (kPa), while studies have shown that root growth is not possible beyond 3,000 kPa. There are two types of compaction, minor and major, each of which requires a particular restoration technique(s) or method:

- **Minor compaction** – Surface compaction within 8-12 inches due to contact pressure and axle load greater than 20 tons can compact through root zone up to one-foot deep. Soil restoration activities can include: subsoiling, organic matter amendment, and

native landscaping. Tilling/scarifying is an option as long as it is deep enough (i.e., 8-12 inches) and the right equipment is used (should not be performed with common tillage tools such as a disk or chisel plow because they are too shallow and can compact the soil just beneath the tillage depth).

- **Major compaction** – Deep compaction, contact pressure and axle load greater than 20 tons can compact up to two-feet deep (usually large areas are compacted to increase strength for paving and foundation with overlap to "lawn" areas). Soil restoration activities can include: deep tillage, organic matter amendment, and native landscaping.

To evaluate the level of compaction in soils, bulk density field tests are conducted. **Table 2** shows the ideal bulk densities for various textures of soils.

Amending media

Compacted soil can be amended by first tilling the soil, breaking apart the compaction, and then applying various soil media. For minor soil compaction, six inches of soil media (18.5 cubic yards per 1,000 square feet of soil) should be applied, and then tilled into the existing soil up to eight inches. For major soil compaction, 10 inches of soil media (31 cubic yards per 1,000 square feet of soil) should be applied and then tilled into the existing soil up to 20 inches.

Soil media used for amendment may be comprised of either organic or inorganic material. Organic media can increase soil organic matter content, which improves soil aeration, water infiltration, water and nutrient holding capacity, and is an important energy source for bacteria, fungi, and earthworms.

Organic media:

- Compost,*
- Aged manure,*
- Biosolids* (must be a Grade 1 biosolid),
- Sawdust, (can tie up nitrogen and cause deficiency in plants),
- Wood ash (can be high in pH or salt),
- Wood chips (can tie up nitrogen and cause deficiency in plants),
- Grass clippings,
- Straw, and
- Sphagnum peat (low pH;).

**Materials containing animal wastes can cause phosphorus to be exported from the amended soils.*

Inorganic media:

- Vermiculite,
- Perlite,
- Pea gravel, and
- Sand.

Soil Texture	Ideal Bulk densities, g/cm$_3$	Bulk densities that may affect root growth, g/cm$_3$	Bulk densities that restrict root growth, g/cm$_3$
Sands, loamy sands	<1.60	1.69	1.8
Sandy loams, loams	<1.40	1.63	1.8
Sandy clay loams, loams, clay loams	<1.40	1.6	1.75
Silt, silt loams	<1.30	1.6	1.75
Silt loams, silty clay loams	<1.10	1.55	1.65
Sandy clays, silty clays, some clay loams (35-45% clay)	<1.10	1.49	1.58
Clays (>45% clay)	<1.10	1.39	1.47

Table 2 Bulk Densities for Soil Textures
Source: Protecting Urban Soil Quality, USDA-NRCS

Applications

Soil restoration can occur anywhere to alleviate soil compaction. It can be specifically addressed in the following examples:

- **New development (residential, commercial, industrial)** – Heavily compacted soils can be restored prior to lawn establishment and/or landscaping to increase the porosity of the soils and aid in plant establishment.

- **Detention basin retrofits** – The inside face of detention basins are usually heavily compacted, and tilling the soil mantle will encourage infiltration to take place and aid in establishing vegetative cover.

- **Golf courses** – Using compost as part of landscaping upkeep on the greens has been shown to alleviate soil compaction, erosion, and turf disease problems.

Design Considerations

1. Tilling the soil (also referred to as scarification, ripping, or subsoiling)

   a. Effective when performed on dry soils.

   b. Should be performed where subsoil has become compacted by equipment operation, dried out, and crusted, or where necessary to obliterate erosion rills.

   c. Should be performed using a solid-shank ripper and to a depth of 20 inches, (eight inches for minor compaction).

   d. Should be performed before amending media is applied and after any excavation is completed.

   e. Should not be performed within the drip line of any existing trees, over underground utility installations within 30 inches of the surface, where trenching/drainage lines are installed, where compaction is by design, and on inaccessible slopes.

   f. The final pass should be parallel to slope contours to reduce runoff and erosion.

   g. Tilled areas should be loosened to less than 1,400 kPa (200 psi) to a depth of 20 inches below final topsoil grade.

   h. The subsoil should be in a loose, friable condition to a depth of 20 inches below final topsoil grade and there should be no erosion rills or washouts in the subsoil surface exceeding three inches in depth.

   i. Tilling should form a two-directional grid. Channels should be created by a commercially available, multi-shanked, parallelogram implement (solid-shank ripper), capable of exerting a penetration force necessary for the site.

   j. No disc cultivators, chisel plows, or spring-loaded equipment should be used for tilling. The grid channels should be spaced a minimum of 12 inches to a maximum of 36 inches apart, depending on equipment, site conditions, and the soil management plan.

   k. The channel depth should be a minimum of 20 inches or as specified in the soil management plan. If soils are saturated, delay operations until the soil, except for clay, will not hold a ball when squeezed.

   l. Only one pass should be performed on erodible slopes greater than one vertical to three horizontal.

2. Applying soil media for amendment

   a. Soil media should not be used on slopes greater than 30 percent. In these areas, deep-rooted vegetation can be used to increase stability.

   b. Soil restoration should not take place within the critical root zone of a tree to avoid damaging the root system. (Where one inch of tree trunk diameter at breast height (DBH) is equal to one foot of soil area on the ground away from the tree trunk.)

c. Onsite soils with an organic content of at least five percent can be stockpiled and reused to amend compacted soils, saving costs. Note: These soils must be properly stockpiled to maintain organic content.

d. Soils should generally be amended at about a 2:1 ratio of native soil to media. If a proprietary product is used, follow the manufacturer's instructions for the mixing and application rate.

e. Add six inches compost or other media and till up to eight inches for minor compaction. (Six inches of compost equates to 18.5 yd^3 per 1,000 ft^2 of soil.)

f. Add 10 inches compost or other amendment and till up to 20 inches for major compaction. 10 inches of compost equates to approx. 30.9 yd^3 per 1,000 ft^2.

g. Compost can be amended with bulking agents, such as aged crumb rubber from used tires, or wood chips. This can be a cost-effective alternative that reuses waste materials while increasing permeability of the soil.

Stormwater Functions and Calculations

Volume and peak rate reduction

Restored soils result in increased infiltration, decreased volume of runoff, and significantly delayed runoff.

Soil restoration will lower runoff volume and peak rates by lowering the runoff coefficient (i.e., curve number). Designers can receive runoff reduction recognition based on areas (acres) complying with the requirements of these BMPs. These areas can be assigned a curve number (CN) reflecting the pre-development hydrologic soil group of underlying soil layer type instead of what is required for other disturbed pervious areas..

Water quality improvement

Although either organic or inorganic materials may be used as soil media, only organic matter can improve water quality by increasing the nutrient holding capacity of soils. Soils rich in organic matter contain microorganisms that immobilize or degrade pollutants.

Organic materials that include fecal matter or animal renderings should not be used where water may infiltrate though the soil and carry nutrients, primarily phosphorus, to surface waters (Hunt and Lord, 2006).

Maintenance

Soil restoration may need to be repeated over time, due to compaction by use and/or settling. Taking soil core samples will help to determine the degree of soil compaction and if additional media application is necessary.

Winter Considerations

Since soil restoration is performed in conjunction with plantings, this BMP should be undertaken in spring or autumn and during dry weather, so that plantings can establish.

Cost

Cost information has been compiled by Cahill Associates and reflects 2007 conditions:

- Tilling costs range from $800/acre to $1,000/acre
- Compost costs range from $860/acre to $1,000/acre. Costs of other soil media would vary greatly depending on their individual material costs and the amounts used.

Criteria to receive runoff reduction recognition for Soil Restoration

To receive runoff reduction recognition for soil restoration under a location regulation, the following criteria must be met:

- ☐ Area is clearly shown on all construction drawings and delineated in the field.
- ☐ Tilling the soil is required if subsoil is compacted; needs to occur before amending media is applied.
- ☐ Area is not located on slopes greater than 30 percent.
- ☐ Area is not within the critical root zone of any tree.
- ☐ Amendment consists of six inches for minor compaction; 10 inches of amendment for major compaction.
- ☐ Area is located on the development project.

Designer/Reviewer Checklist for Soil Restoration

Type of soil amendment(s) proposed: _____

Amount of amendments(s) to be used: _____

ITEM	Page No.	YES	NO	N/A	NOTES
Appropriate soil amendment(s) for the site conditions? ➢ *Do not use soil media on slopes > 30%*	3,4				
Adequate amount of amendment materials?	3				
Bulk density/degree of compaction considered?	2,3				
Appropriate decompaction techniques and equipment?	4				
Appropriate construction sequencing?	--				
Sensitive areas (e.g., near existing trees, shallow utilities, and steep slopes) accounted for?	4				
Appropriate vegetation selected?	--				
Seasonality of planting/construction considered?	5				
Erosion and sedimentation control provided?	--				
Maintenance accounted for and plan provided?	5				

➢ *Denotes Minimum Design Considerations*

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Appendix F

Stormwater Drainage Manual

Research Engineers
Christopher B. Burke
Thomas T. Burke

December 2015

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Indiana LTAP Stormwater Drainage Manual - Revised December 2015

Acknowledgements

The original HERPICC “County Storm Drainage Manual” was completed by Dr. Christopher Burke in 1981. Jean Hittle, Director of HERPICC, was very supportive of the manual and provided leadership. Norma Gray worked endlessly to type and format the equations, graphs and tables prior to the assistance of Word Processing. Several revisions have taken place over the last 27 years. The latest revision, now titled Indiana LTAP "Stormwater Drainage Manual" encompasses the basis and theory that constitutes stormwater drainage contained in the original manual and expands many areas where current information or methods have made available new data and techniques. The previous version, updated in 1995, added a study performed at Purdue University developed new Intensity-Duration-Frequency (I-D-F) curves for several cities throughout Indiana by using hourly rainfall data from July 1948 to June 1991 (Purdue, et al., 1992). These curves are presented for determining rainfall data for the various cities in the state where the data was analyzed. Now, in 2008, the rainfall information has been updated to include NOAA Atlas 14 data in Chapter 2. We have provided a website link so that users may easily find rainfall for their specific area of interest. In Chapter 3, StreamStats has been incorporated utilizing new regression equations developed by Knipe and Rao. Another example of encompassing current techniques is the addition of Chapter 8, which focuses on computer programs that are widely used for stormwater drainage computations, and the references throughout the manual to various programs available for simplifying detailed hand calculations. All programs in Chapter 8 are now Windows based. Chapter 9, has been added to address the relation between stormwater and water quality as the rules for meeting water quality standards continue to evolve.

In the process of updating the manual, there have been several people who have contributed a great deal of their time and efforts. Thomas Burke and David McCormick worked as graduate students, School of Civil Engineering, in 1995. Dr. A. R. Rao assisted throughout the 1995 revisions by reviewing and suggesting new methods that were implemented in the manual. In 2007 and 2008, Megan Burke worked as a graduate student, School of Civil Engineering, correcting some errors and added Best Management Practices. Luke Sherry, Christopher B. Burke Engineering, Ltd., updated Chapters 2 and 3, wrote the original version of Chapter 9 and updated the models in Chapter 8 to Windows based programs. Dr. G.S. Rao provided guidance and suggestions throughout the latest update.

Special thanks is also due to the HERPICC Advisory Board for their financial support throughout the revisions. In particular, Dr. C. F. Scholer, Past - Chairman HERPICC Advisory Board, for his excellent guidance and support. Most especially, we are appreciative of the continued support of John Habermann at Indiana LTAP for the last 10 plus years as we have updated the manual and continue to put on one of the most beneficial conferences LTAP has to offer!

Historical Acknowledgement

Purdue University

ENGINEERING EXPERIMENT STATION
LAFAYETTE, INDIANA 47907

ADDRESS REPLY TO –
HIGHWAY EXTENSION AND RESEARCH PROJECT
FOR INDIANA COUNTIES
CIVIL ENGINEERING BUILDING

To: Manual Readers and Users

From: Jean E. Hittle – HERPIC

Subject: HERPIC County Storm Drainage Manual
Credits and Acknowledgements

HERPIC was indeed fortunate to have the talent and dedication of Mr. Chris Burke available to pursue the subject of "storm drainage".

Chris, now completing his doctor's degree in the School of Civil Engineering at Purdue, has produced an excellent treatise on drainage criteria and their application to the design of drainage works and facilities. The manual will serve, in a very positive way, the needs of all engineers (county, city, state, federal) in the analysis of drainage needs to protect property against damage and to assure the delivery of public services.

The initial HERPIC effort to develop such a manual was undertaken, some years back, by Mr. Phillip Dick, graduate student, and later by Professor John Spooner. HERPIC therefore recognizes their endeavors and contributions to the initial manual effort.

Using the limited work of Mr. Dick and Professor Spooner as a starting point, Chris Burke expanded the scope of the manual, developed a detailed outline and format, and provided a series of example problems that illustrate the application of the drainage criteria and analysis. The finished product is the "HERPIC County Storm Drainage Manual", which is indeed an outstanding contribution to Purdue University's HERPIC program of extension and research to county highways.

Special credit and acknowledgement is also due Dr. Donald Gray, Professor of Hydromechanics, School of Civil Engineering, for his review, comments and suggestions on the manuscript for the Manual. Likewise, a special note of thanks and appreciation to Mrs. Frank (Norma) Gray, of the HERPIC staff, for her excellent typing service and untiring patience to producing the final typed version of the manual manuscript.

Jean E. Hittle - HERPIC

INDIANA LTAP

STORMWATER DRAINAGE MANUAL

Chapter 1	Introduction
Chapter 2	Rainfall
Chapter 3	Runoff and Its Estimation
Chapter 4	Open Channels
Chapter 5	Flow in Gutters and Inlets
Chapter 6	Stormwater Storage
Chapter 7	Storm Sewer System Design
Chapter 8	Computer Applications
Chapter 9	Stormwater Quality
Chapter 10	Green Infrastructure
Appendix A	Statistical Analysis
Appendix B	Fundamentals of Hydraulics
Appendix C	Regulatory Agencies for Drainage Projects

Chapter 1 - INTRODUCTION

Indiana, like many parts of the country, is experiencing a rapid change in the character of its land use. Areas which were once predominantly rural are now being developed for urban or suburban use. The consequence of covering once pervious soils with concrete, asphalt and buildings is a decrease in the rainfall quantity which may infiltrate and a subsequent increase in the runoff volume. In addition, components of the drainage system such as sewers, gutters and streets, convey this increased volume to the point of disposal much quicker than in the rural condition. The result of this "urbanization" is a significant increase in the volume of stormwater and a conveyance rate higher than that experienced in the undeveloped rural state.

Engineers, surveyors or others involved with storm drainage design are faced with the task of designing drainage systems that are economical and at the same time provide adequate protection to minimize the loss of property or life. This manual has been compiled to provide the designer with resource materials which will help in meeting this challenge.

The information presented in this manual is not necessarily original or unique. It is a comprehensive catalog of procedures, design methods and criteria, and general background information which will enable the designer to quickly learn or review the basic principles and applications of storm drainage design. This information is currently dispersed in many other texts and manuals and is not readily available as a single source.

The manual presents nine chapters along with three appendices. Each chapter presents an introduction and background information about the subject(s) discussed. Following the introduction is a presentation of the appropriate equations, graphs, charts or tables for the methods which are employed in drainage design. Each chapter includes example problems which illustrate the application of the material presented. References at the end of the chapter provide the reader with additional sources of information.

Chapter 2 presents the precipitation and hydrologic cycle which is the starting point of any drainage design. The processes involved in the formation of rainfall are presented, along with a discussion of the temporal and areal distribution of rainfall. A discussion of the collection and analysis of precipitation follows, along with a statistical analysis and hydraulic risk. Depth and intensity-duration-frequency equations for several cities throughout Indiana are presented. NOAA Atlas 14 information is provided to obtain temporal distributions for specific time periods and storm type. The chapter includes example problems illustrating the application of the Huff curves to generate a time distribution of rainfall, use of the statistical analysis and hydraulic risk associated with rainfall, and determination of a rainfall intensity using an intensity-duration-curve. The final example compares the rainfall intensities obtained from the intensity-duration-frequency curves.

Chapter 3 presents the phenomenon of runoff and its estimation, which is the most important aspect of drainage design. The various components which affect runoff are presented along with seven methods for estimating the amount of runoff from a rainfall event. The first method is the popular Rational Method which computes a peak runoff rate only. The second procedure

outlined is the Soil Conservation Curve Number Method which computes a volume of runoff. The third procedure outlined is the use of hydrographs. This includes unit hydrographs, dimensionless unit hydrographs and storm hydrographs. The fourth method provided is the Water Resource Council Method which evaluates a series of discharge data to obtain the flowrate corresponding to a desired period. Statistical analysis of peak discharges, which is very similar to the techniques used in analyzing rainfall data, is the fifth method. The sixth procedure is the coordinated discharges method for selected streams in Indiana. The last procedure demonstrates flows obtained from Flood Insurance Studies. Example problems illustrate applications of all of these procedures.

Since open channels are the primary conveyances employed in storm drainage design, they are discussed in Chapter 4. The chapter presents a discussion of channel geometry, flow classification and applications of the energy equation. Next, the appropriate equations for computing uniform flow, specific energy, critical flow and flow in a floodplain are presented. Design criteria used in the selection of location, channel cross-section, roughness coefficients and lining are then given. The text portion of the chapter concludes with the analysis of gradually-varied flow and its application to backwater curves. The example problems illustrate most of the methods presented. A brief introduction to the computer program HY-8 is given and an example is provided.

Regardless of a drainage systems' capacity, it must have inlets which will allow the stormwater into the system. Chapter 5 discusses the methods used for sizing inlets and gutters. The chapter begins with a discussion of flow in gutters and methods used in properly estimating gutter capacity. The estimation of inlet capacity for gutter, curb, slotted drain and combined inlets for continuous grades and sump conditions is presented. The text portion of the chapter concludes with design criteria for inlet design, including inlet spacing using the Rational Method. Example problems illustrate methods used in computing flow in gutters, gutter inlets and curb inlets for both a continuous grade and sump condition and a slotted drain inlet for sump condition. The last example problem illustrates the spacing of a gutter inlet using the Rational Method.

One important element of drainage design is stormwater storage; this topic is presented in Chapter 6. The chapter starts with a discussion of all the types of storage facilities which may be employed and follows with a discussion of two methods (outlined in Chapter 3) which can be used to compute the volume of storage needed: the Rational Method and the SCS Hydrograph Method. The text portion of the chapter designates the criteria used in designing retention and detention ponds, and parking lot, rooftop and infiltration facilities. A discussion of devices used for regulating outflow is also presented. Example problems present applications of the Rational Method, Curve Number Method and Hydrograph Method. The fourth problem illustrates methods for sizing a multi-component facility site.

The design of a storm sewer system is presented in Chapter 7, using the methods and procedures presented in all the previous chapters (2-6). A general introduction is followed by a discussion of the methods used in the sizing of storm sewers, including the rational method and a computer program method. A brief introduction to the hydraulics of culverts is presented,

along with design criteria for designing storm sewer systems. The chapter concludes with a presentation of the types of pipe material which may be used for storm sewers. Example problems illustrate the application of the rational method to hypothetical drainage basin and to an actual subdivision.

Chapter 8 presents three computer applications for computing watershed runoff. This chapter incorporates the concepts and procedures in Chapter 2 and 3 and simplifies the calculations by using the computer programs Win TR-20 and HEC-HMS. A description of the program is followed by three example problems. The same example problems are used for both applications. The Win TR-55 has also been added along with an example problem. This chapter previously contained the DOS version of TR-20 and HEC-1.

A description of multiple stormwater quality Best Management Practices (BMPs) is provided in Chapter 9. The chapter classifies these BMPs into two categories: (1) construction (temporary) BMPs, and (2) post-construction (permanent) BMPs. Finally, Chapter 10 presents a discussion of various site-level green infrastructure systems. The chapter provides a description of the most common types of green infrastructure and demonstrates how these practices reduce the volume and rate of site runoff, both through descriptive text and example problems.

The manual concludes with three appendices. Appendices A and B present background material for statistical analysis and the fundamentals of hydraulics. Appendix C outlines regulatory agencies and governmental bodies which may have jurisdiction over drainage projects.

The basics of statistical analysis included in Appendix A, consists of the general concepts of the mean, standard deviation and probability. The Gumbel and Log-Pearson Type III distributions are presented, along with an example showing the various aspects of the material presented. Also, rainfall depth curves for the continental United States are provided.

The fundamentals of hydraulics in Appendix B presents a general review of hydraulic principles needed by the drainage engineer. This includes the law of conservation of mass, continuity equation, and the concepts of pressure and energy. A discussion of pipes flowing under pressure is presented, along with the Darcy-Weisbach and Hazen-Williams equations and a discussion of minor losses and flow in series and parallel pipe networks. A summary of some of the elements of open channel flow (Chapter 4) concludes Appendix B.

Appendix C contains a list of regulatory agencies which may have jurisdiction over drainage projects. A discussion of the local, state, and Federal organizations from which the designer may need to get approval is presented, along with citations to applicable statutes and regulations.

A separate manual is now available to provide documentation for the HERPICC Stormwater Drainage Manual disk. This disk provides spreadsheets for many of the example problems that were calculated using spreadsheets in Chapters 2 through 7. The disk can be obtained through the HERPICC office and is available in Quattro Pro for DOS, Quattro Pro for Windows, and Excel for Windows formats.

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LIST OF PARAMETERS

c, d, α, β	Regional coefficients for use in rainfall intensity-duration-frequency equations
C_s	Skewness coefficient
ΔD	Increment of depth (inches)
i	Rainfall intensity
J	Risk
K	Gumbel frequency factor
K_t	Log Pearson Type III frequency factor
m	Ranking of event
n	Total number of events or years
p	Rainfall depth
$\bar{p}$	Arithmetic average of rainfall data
p_i	Individual extreme rainfall value
P	Frequency of an event being equalled or exceeded
s	Standard deviation
s_y	Standard deviation of y
t	Storm duration (hours)
Δt	Increment of time (hours)
T_r	Return Period
$\bar{y}$	Arithmetic average of the logarithm of rainfall data
y_i	Logarithm of individual extreme rainfall value

Chapter 2 - RAINFALL

2.1 - INTRODUCTION

In the design of drainage systems, rainfall provides the input to deterministic methods or models. This input may be from statistical analysis of data or from an actual rainfall event. The basic aspects of the hydrologic cycle, of rainfall data and the precipitation process, and how precipitation varies in space and time, as well as a discussion of the collection of rainfall data and the statistical analysis of this data, are included.

2.1.1 - Hydrologic Cycle

Precipitation is a part of the hydrologic cycle. The hydrologic system is illustrated in Figure 2.1.1.

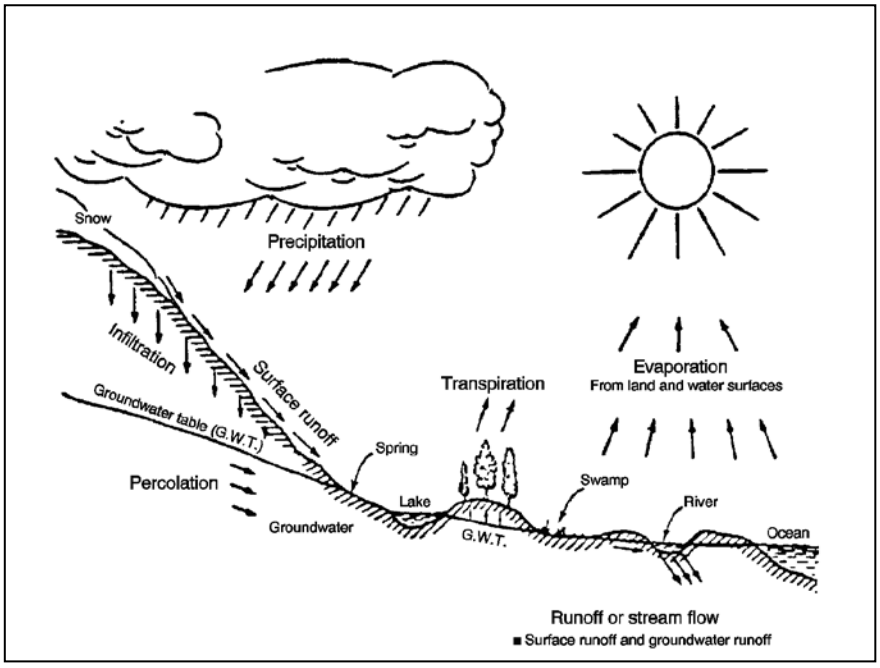


Figure 2.1.1
The Water Cycle (Fair et al., 1971)

2.1.2 - Precipitation Processes

While water vapor is a necessary factor in the formation of precipitation, it is not the sole requirement. Three basic steps are necessary for precipitation to form. First, a saturated condition in the atmosphere must exist. This condition is brought about mostly through the cooling which accompanies an ascending body of moist air. The saturated condition involves

the transformation of water in the air mass from vapor to liquid or solid state. This transformation, which is called condensation, occurs on small hygroscopic particles called condensation nuclei. As the air mass is lifted, expanded and cooled, the water vapor will condense on these particles to initiate the formation of precipitation. Snow will follow a similar process. Growth of the small water droplets to larger "precipitable" size is the third step. As the droplets form, there are influences which reduce their size through evaporation. Solar energy is one such influence. Heat generated from the physical transformation of the state of water molecules coupled with air movement, also contributes to evaporation. If evaporation is too great, then rainfall does not occur.

2.1.3 - Time Distribution of Rainfall

Time distribution of rainfall is important in the planning, sizing, and design of urban stormwater management systems. A complete approach to describing the time distribution of rainfall would include its probabilistic nature. One such study was performed in which data were collected over a 400 square mile area in east-central Illinois utilizing 40 rain gages. (Huff, 1970; Viessman, 1977).

The study by Huff (1970) found that the major portion of the total storm rainfall occurs in a small part of the total storm, regardless of storm duration, areal mean rainfall, and total number of showers or bursts in the storm period. The storms were classified into four groups (1st, 2nd, 3rd, and 4th quartiles) depending on the quartile, defined as a 25% time segment of the total storm duration, in which the greatest amount of total rainfall occurred (Huff, 1970; 1972). Using the Huff methodology, Tables 2.1.1 - 2.1.9 were derived from a study of rainfall at four stations (Indianapolis, Evansville, Fort Wayne, and South Bend) in the State of Indiana (Purdue et al., 1992). The information for Indianapolis is shown graphically in Figure 2.1.2. The axes of the curves are dimensionless cumulative rainfall and cumulative storm time. Each curve represents a different probability level. For example, a 10% probability curve may be interpreted as the distribution of rainfall that was exceeded in 10% of the storms. Example 2.1.1 illustrates the use of this data.

The Huff quartile groups represent typical rainfall distributions for 4 different storm duration ranges. Generally, in water resources modeling, the first quartile is taken to apply to storms less than or equal to 6 hours in duration. The second quartile is for storms greater than 6 hours and less than or equal to 12 hours while the third Huff quartile is for storms greater than 12 hours and less than or equal to 24 hours. Fourth quartile storms apply to storm durations greater than 24 hours (IDOT DWR, 1992).

Table 2.1.1
10% Huff Curve Ordinates (Purdue et al., 1992)

% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	42.00	16.36	14.36	19.35	45.44	15.56	13.64	17.14	42.68	15.00	13.24	17.50	39.65	16.67	15.38	17.50
20	64.35	32.73	25.26	30.00	69.63	32.50	23.62	28.75	63.64	31.43	24.57	28.75	63.67	30.92	25.00	28.00
30	76.36	58.10	33.33	36.36	79.14	60.00	30.83	37.50	78.18	55.31	33.45	37.60	75.58	54.91	32.76	38.69
40	83.84	76.36	41.82	43.53	86.09	77.27	40.00	43.40	85.26	75.00	41.67	42.86	82.52	73.33	41.20	45.00
50	89.63	87.50	54.72	50.00	89.09	88.72	53.85	50.00	89.05	86.70	54.55	50.00	87.68	85.71	53.45	50.00
60	92.50	92.54	78.18	54.55	92.85	93.94	77.62	55.17	92.00	91.30	77.10	55.56	90.91	91.30	77.30	57.14
70	95.00	95.69	92.35	63.64	95.00	95.77	90.33	62.55	94.40	95.12	91.20	63.64	93.68	94.13	90.89	63.60
80	97.00	97.00	96.43	82.50	96.97	97.50	96.30	83.33	96.51	97.14	96.67	82.82	96.15	96.56	95.29	81.82
90	98.57	98.67	98.73	96.73	98.78	98.98	98.78	96.92	98.18	98.57	98.57	96.84	98.21	98.35	98.16	96.09
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Table 2.1.2
20% Huff Curve Ordinates (Purdue et al., 1992)

% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	31.71	12.17	10.71	14.29	36.67	12.28	10.00	13.23	32.73	11.67	10.59	14.29	31.93	12.73	12.35	14.29
20	52.92	27.27	21.09	25.00	60.00	27.27	18.67	23.28	57.58	25.71	20.00	25.00	53.33	26.67	20.86	25.00
30	68.00	50.00	28.57	31.79	71.15	49.40	26.86	30.41	70.00	47.06	28.46	31.92	68.00	47.27	28.44	33.33
40	76.67	67.78	35.83	38.00	78.57	69.38	35.19	37.61	77.78	66.25	36.60	38.00	76.00	65.19	36.88	40.00
50	82.93	80.77	50.00	44.44	85.19	82.31	48.08	44.12	82.73	79.41	49.06	43.90	80.77	79.41	48.00	46.15
60	88.57	88.89	72.50	50.00	88.57	89.38	70.38	51.43	87.44	87.50	69.55	50.00	85.71	86.38	69.23	51.28
70	92.20	92.31	88.89	60.32	92.35	93.29	87.82	60.00	90.63	92.45	85.71	59.00	90.00	91.38	85.25	59.81
80	95.12	95.43	94.57	77.33	95.23	96.12	94.49	79.05	94.29	95.51	93.77	77.78	93.75	94.44	92.71	77.14
90	97.94	97.85	98.14	94.60	97.96	98.40	98.09	95.00	97.27	97.78	97.86	94.49	97.00	97.49	97.10	93.55
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Table 2.1.3
30% Huff Curve Ordinates (Purdue et al., 1992)

% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	26.67	9.88	8.47	10.00	30.00	9.86	8.00	10.00	26.92	9.47	8.57	11.61	26.67	10.61	10.00	12.00
20	48.57	23.81	16.92	20.00	52.80	22.98	16.36	20.00	50.00	22.86	16.67	20.16	47.69	24.00	18.57	21.22
30	62.86	44.74	24.00	28.00	66.67	42.77	24.00	26.79	63.33	42.00	23.59	27.59	62.00	41.82	25.00	28.57
40	71.22	62.91	31.79	33.33	74.19	62.50	32.00	33.67	71.05	60.32	33.15	33.33	70.00	60.00	33.33	35.00
50	77.42	76.35	45.61	40.63	80.00	77.14	44.44	40.44	77.78	75.13	45.16	40.00	76.25	74.19	44.95	41.67
60	83.81	83.87	66.47	47.69	85.45	85.00	67.34	47.65	83.33	84.05	64.29	46.67	81.82	82.11	64.71	49.13
70	89.76	90.00	84.73	56.94	89.39	90.73	84.21	57.14	87.60	89.76	82.57	55.71	87.60	87.80	82.00	56.52
80	93.50	94.03	92.50	74.62	93.55	94.73	92.50	75.83	92.22	93.62	91.43	73.85	91.20	92.73	90.70	74.17
90	97.22	97.20	97.36	92.51	97.33	97.88	97.30	93.16	96.25	97.09	96.94	92.22	95.71	96.71	96.11	91.20
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Table 2.1.4
40% Huff Curve Ordinates (Purdue et al., 1992)

% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	23.29	7.74	6.42	8.42	27.00	8.00	6.45	8.00	23.33	7.79	7.00	9.19	22.92	8.85	8.33	10.00
20	44.62	20.87	14.00	16.96	48.48	20.00	13.08	16.64	45.40	20.00	14.29	17.30	42.86	21.05	15.76	18.73
30	58.95	40.00	20.62	24.79	60.74	37.20	20.29	23.75	58.33	37.97	21.05	23.81	56.67	37.06	22.00	25.00
40	66.67	57.78	28.57	29.84	69.83	56.93	28.81	30.00	66.67	55.71	29.69	30.00	65.36	55.65	30.00	32.31
50	73.53	72.22	41.94	37.21	75.71	73.68	40.91	37.66	72.41	71.43	42.11	35.71	71.43	70.25	41.82	38.89
60	80.24	80.28	61.97	44.53	81.22	81.82	61.12	43.61	78.57	80.00	60.83	42.55	77.89	78.95	62.04	44.44
70	86.15	87.14	81.60	54.00	87.12	88.13	80.17	54.12	85.00	87.44	79.05	52.50	84.00	85.60	78.67	53.33
80	91.30	92.08	90.77	71.69	92.17	93.18	90.63	72.73	90.00	91.72	89.82	70.94	89.23	90.96	88.67	70.77
90	96.15	96.55	96.50	90.00	96.58	97.14	96.58	91.43	95.00	96.25	96.00	90.00	94.55	95.88	95.33	89.22
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Table 2.1.5
50% Huff Curve Ordinates (Purdue et al., 1992)

% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	20.00	6.50	5.26	6.67	22.82	6.28	5.13	6.92	20.00	6.67	6.00	7.14	20.00	7.50	7.00	8.26
20	40.80	18.13	11.55	14.25	44.69	17.33	11.11	14.04	41.11	17.14	12.23	14.23	40.00	18.57	13.33	16.35
30	54.95	35.85	17.06	20.00	57.11	33.33	16.67	20.51	54.83	34.17	18.86	20.00	51.67	34.00	20.00	22.73
40	62.50	52.94	24.24	26.09	65.33	53.09	25.44	27.06	62.00	52.18	26.15	25.71	60.89	51.43	27.50	28.50
50	68.75	67.86	37.78	33.33	71.43	69.57	37.93	34.21	68.42	66.67	38.46	33.33	67.35	66.67	39.13	34.04
60	76.67	76.52	58.33	40.00	78.15	78.57	57.39	40.91	75.00	76.36	57.23	38.00	75.00	75.17	58.46	40.20
70	83.05	83.81	78.03	50.00	84.66	85.60	77.44	50.79	81.62	84.29	76.11	48.50	80.83	82.32	75.98	50.00
80	89.70	90.67	88.68	68.57	90.00	91.72	88.54	69.70	87.50	90.00	87.69	68.24	86.67	88.89	86.79	67.50
90	95.00	95.89	95.29	88.37	95.36	96.50	95.88	89.36	93.75	95.56	95.08	87.88	92.89	94.78	94.17	87.50
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Table 2.1.6
60% Huff Curve Ordinates (Purdue et al., 1992)

% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	17.50	5.22	4.18	5.45	19.15	5.07	4.00	5.22	17.78	5.71	4.67	5.45	17.37	6.09	5.65	6.67
20	37.86	15.56	9.23	11.03	40.51	15.12	9.00	11.11	38.00	14.69	10.42	11.00	36.00	15.94	11.25	13.33
30	50.00	31.25	14.29	17.39	53.13	30.00	13.69	16.94	50.00	30.00	15.89	16.04	48.57	30.53	17.14	20.00
40	58.33	48.39	21.58	22.28	60.86	48.81	22.22	23.00	57.50	48.42	22.44	22.98	56.67	48.00	24.00	25.00
50	65.12	64.13	32.45	28.57	66.95	65.79	33.96	30.77	65.00	63.64	35.00	27.42	65.00	62.50	35.66	29.41
60	71.97	73.33	54.17	36.67	74.36	75.56	53.33	37.23	71.43	73.33	53.08	34.17	71.43	72.44	54.07	37.02
70	80.00	81.13	73.89	47.14	80.91	82.46	74.43	47.27	77.78	81.43	72.86	44.17	77.78	79.29	72.67	47.02
80	86.67	88.82	86.32	65.00	87.50	89.53	86.67	66.00	85.00	88.00	85.71	64.41	84.00	86.67	84.44	64.44
90	93.33	94.74	94.12	86.67	94.00	95.50	94.67	86.00	92.12	94.44	93.85	85.42	90.77	93.55	92.89	85.71
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Table 2.1.7
70% Huff Curve Ordinates (Purdue et al., 1992)

% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	14.44	4.12	3.16	4.00	15.88	4.08	3.13	3.75	15.00	4.62	3.64	4.13	14.29	5.00	4.38	4.62
20	35.00	13.19	7.14	8.26	36.67	12.33	7.10	8.72	35.00	12.73	8.42	8.33	33.33	13.68	10.00	10.53
30	47.01	26.36	11.76	14.21	49.88	25.88	11.33	14.00	46.67	26.00	13.04	12.86	45.00	26.79	14.89	14.59
40	53.65	44.21	18.10	18.46	56.79	44.40	17.55	18.18	53.87	44.08	18.82	17.24	52.86	44.24	20.41	20.55
50	60.71	60.81	28.91	24.88	63.37	61.11	29.30	25.00	61.29	60.00	30.77	24.34	60.71	59.46	31.37	25.00
60	68.00	70.59	48.89	31.43	70.00	71.58	49.41	32.86	67.50	70.00	49.52	30.26	67.65	68.40	50.48	31.58
70	75.00	77.84	70.65	42.50	76.54	79.76	70.34	43.33	75.00	78.00	70.00	40.00	75.00	76.47	70.00	41.74
80	83.38	86.00	84.44	61.05	85.00	87.32	84.08	61.33	82.35	85.71	83.28	60.00	80.00	83.89	82.19	60.00
90	91.25	93.44	93.00	84.54	92.50	94.11	93.62	82.04	90.50	93.16	92.73	82.50	88.75	91.82	91.25	82.50
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

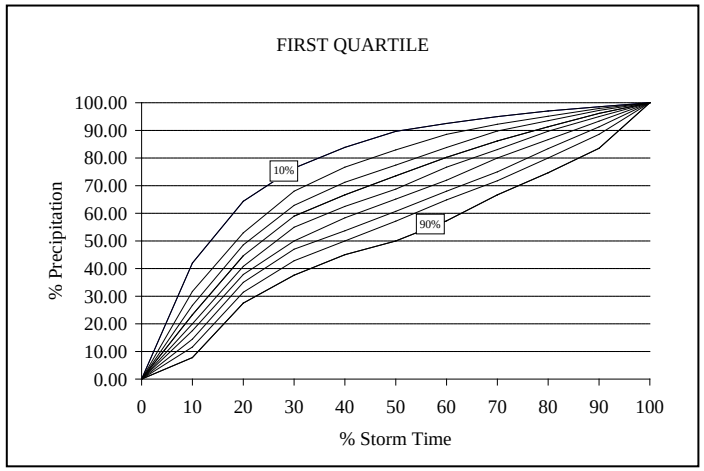
Table 2.1.8
80% Huff Curve Ordinates (Purdue et al., 1992)

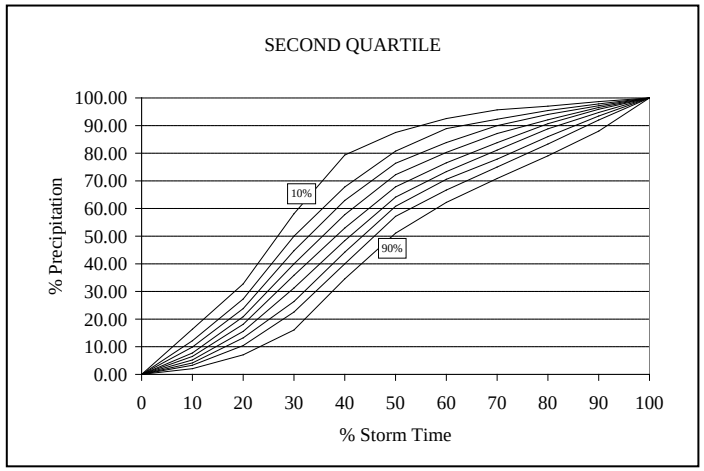
% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	11.59	3.33	2.12	2.93	12.40	2.92	2.22	2.78	11.01	3.43	2.82	2.94	11.43	3.83	3.24	3.57
20	31.43	10.43	5.26	6.25	33.33	9.80	5.17	6.36	32.00	10.61	6.06	5.89	30.67	10.73	7.45	7.69
30	42.82	22.56	8.45	9.09	45.83	22.02	8.70	10.22	42.86	22.22	9.16	9.12	41.00	22.05	10.97	11.11
40	50.00	40.00	14.29	13.85	51.06	40.00	13.60	13.50	50.00	39.19	14.29	12.92	49.23	40.22	16.67	16.00
50	57.14	57.14	23.33	18.52	59.14	56.73	24.00	17.89	56.34	56.00	25.00	16.67	56.67	55.56	26.74	21.05
60	64.84	66.67	44.00	26.00	65.37	66.67	43.48	27.29	64.29	66.41	45.09	24.85	65.22	65.00	46.11	26.32
70	71.76	75.00	66.67	37.04	72.00	75.56	66.33	38.33	71.43	74.32	67.11	35.00	70.20	73.08	66.67	36.47
80	80.00	83.33	81.94	56.64	80.00	84.72	80.70	55.56	80.00	82.92	80.95	54.29	77.14	80.43	80.00	54.78
90	88.57	92.00	91.60	80.00	89.71	92.50	91.67	77.73	88.18	91.43	90.77	78.71	86.67	90.00	89.33	79.00
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

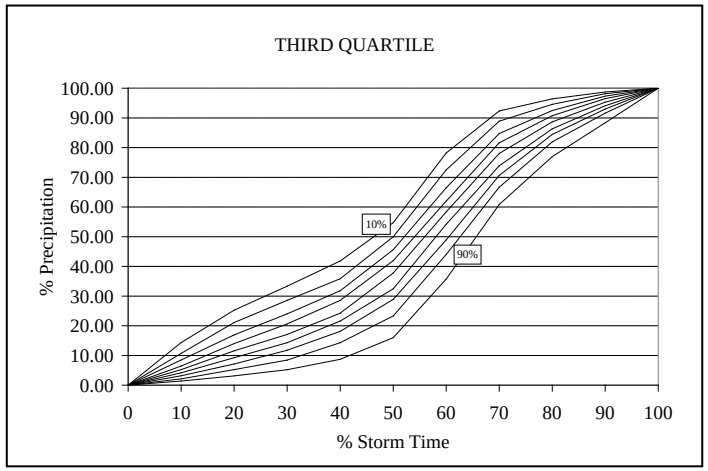
Table 2.1.9
90% Huff Curve Ordinates (Purdue et al., 1992)

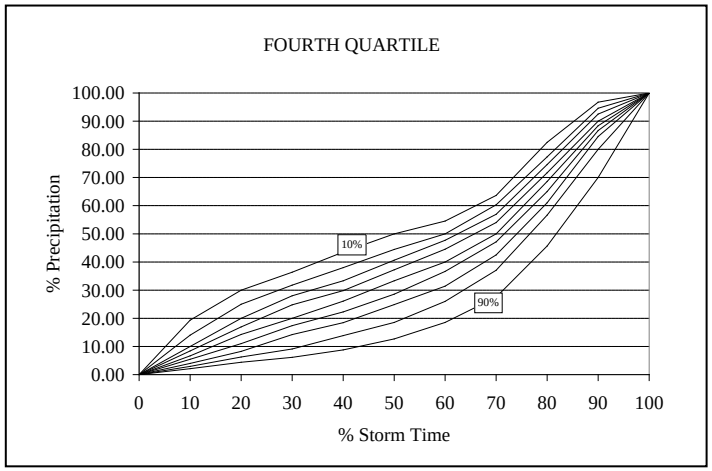
% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	7.78	2.07	1.37	2.13	7.69	1.80	1.27	1.61	7.50	2.16	1.89	1.82	8.00	2.48	2.12	2.08
20	27.50	7.07	3.13	4.40	28.89	6.48	3.39	3.87	27.27	7.14	3.77	3.89	27.27	7.54	4.65	4.55
30	37.50	16.07	5.21	6.12	38.79	16.67	5.65	6.25	38.46	15.11	6.00	5.45	35.17	16.32	7.50	7.25
40	45.00	34.58	8.70	8.78	46.15	34.29	9.52	8.92	45.57	33.00	10.14	7.69	42.86	34.69	12.43	10.53
50	50.00	51.09	16.00	12.67	51.22	51.90	13.71	11.65	50.00	50.00	19.23	10.00	50.00	50.00	20.51	13.89
60	57.27	62.22	35.71	18.52	60.00	62.00	34.00	20.00	58.57	62.00	36.11	15.56	59.11	60.00	40.00	20.00
70	66.67	70.85	60.83	27.50	66.67	71.11	61.75	30.60	66.67	69.81	61.67	25.60	66.67	67.50	62.22	28.57
80	74.67	79.07	76.96	45.67	74.29	80.00	76.67	46.32	75.00	78.67	77.00	45.00	72.14	76.67	76.10	45.71
90	83.53	88.00	88.35	70.00	84.00	90.00	88.64	69.27	84.00	88.21	88.13	70.83	82.00	86.00	85.71	70.91
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Figure 2.1.2
Huff Curves for Indianapolis (Purdue et al., 1992)









Example 2.1.1

This example problem illustrates the application of the Huff curves to generate a time distribution of rainfall.

If a storm in South Bend has a rainfall depth of 3.28 inches over a period of two hours, find the hyetograph (the time distribution of rainfall) and cumulative rainfall depths using a 50% Huff first quartile (Purdue et al., 1992) storm distribution.

Referring to Table 2.1.5, the 50% Huff I-quartile Curve for South Bend has the characteristics shown in the first and second columns below: Since the dimensionless time increment of the storm is 0.1 times the total storm times, the % storm times correspond to 0.1*120 minutes, or 12 minute durations as shown in column 3. The cumulative rainfall depth is found by multiplying the total rainfall depth by the cumulative percentage for that time. For example, at 70% storm time, the cumulative rainfall depth is 3.28*80.83/100 = 2.65 inches. These values are shown in column 4. The incremental rainfall values are obtained from the differences between the cumulative rainfall values. For example the incremental rainfall between 60 minutes and 72 minutes is 2.46 in. – 2.21 in. = 0.25 in. and is shown in column 5. The cumulative and incremental results are shown graphically in Figures 2.1.3 and 2.1.4 respectively.

% Dimensionless Storm Time	South Bend 50% I - Quartile Dimensionless Storm Depth	Time (minutes)	Cumulative Rainfall (inches)	Incremental Rainfall (inches)
0	0	0	0.00	0.00
10	20	12	0.66	0.66
20	40	24	1.31	0.66
30	51.67	36	1.69	0.38
40	60.89	48	2.00	0.30
50	67.35	60	2.21	0.21
60	75	72	2.46	0.25
70	80.83	84	2.65	0.19
80	86.67	96	2.84	0.19
90	92.89	108	3.05	0.20
100	100	120	3.28	0.23

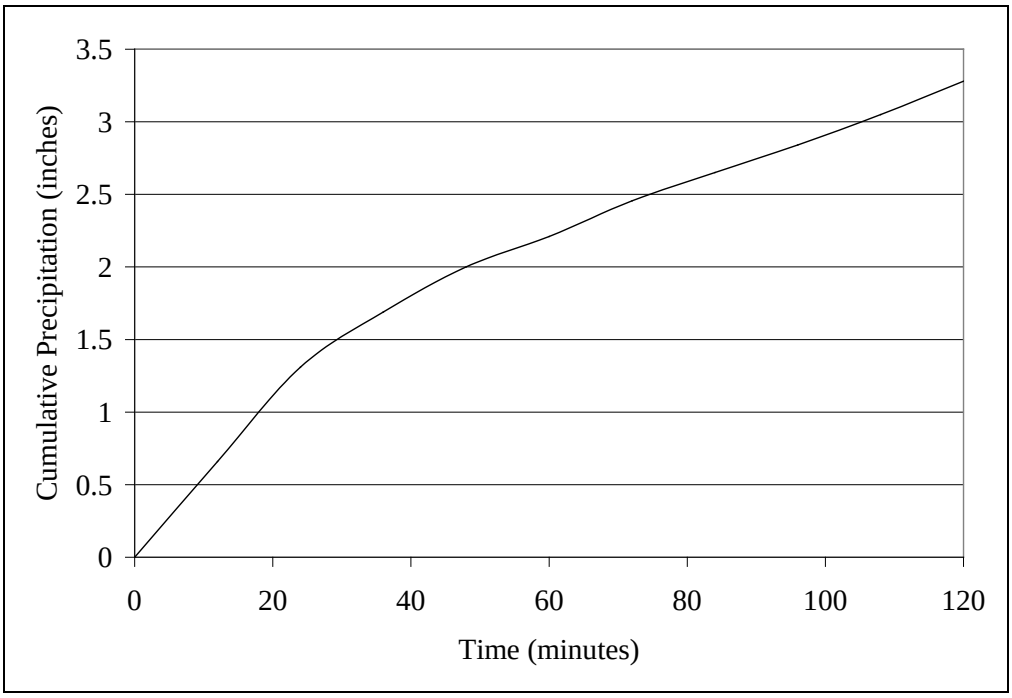


Figure 2.1.3
Cumulative Rainfall as a Function of Time, Example 2.1.1

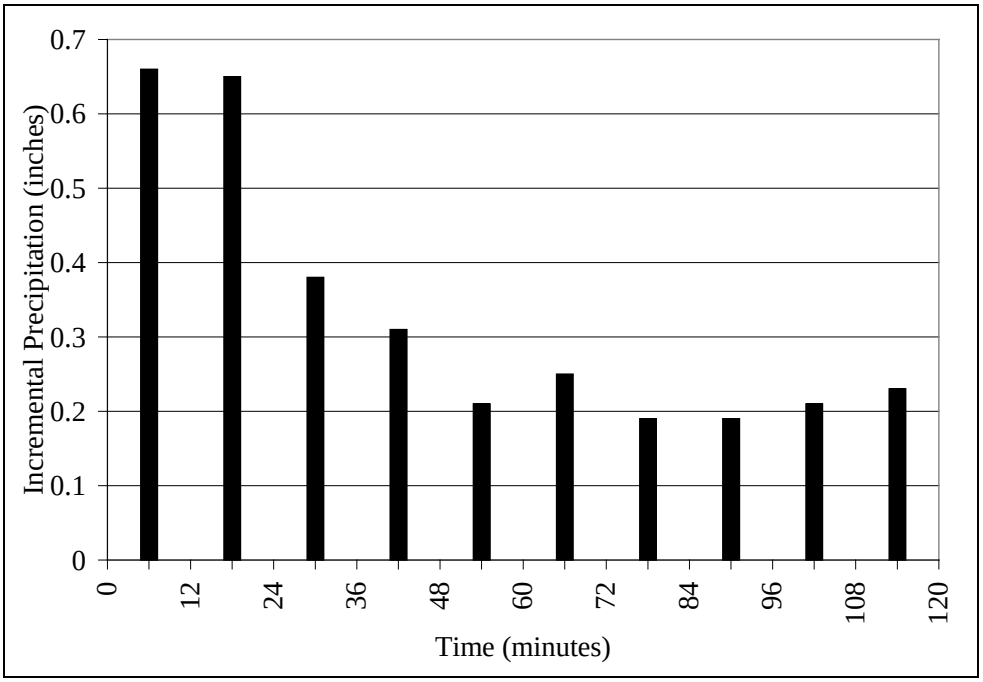


Figure 2.1.4
Incremental Rainfall as a Function of Time, Figure 2.1.1

Table 2.1.10
Percentage Distribution of Quartiles for West Lafayette, Indiana
(Rao and Chenchayya, 1974)

Quartile	Ten Minute Data		Hourly Data	
	Number	Frequency	Number	Frequency
I	57	33	64	48
II	56	32	31	23
III	27	16	9	7
IV	34	19	29	22
Total	174	100	133	100

The time distributions of 174 rainfall events for the Lafayette region were analyzed by Rao and Chenchayya (1974). In Table 2.1.10 the frequency of occurrence for each quartile is shown. From the analysis it can be seen that the first and second quartile storms occur most frequently for both the ten-minute and hourly data.

The Natural Resources Conservation Service (NRCS), formerly the Soil Conservation Service (SCS), has also developed rainfall distributions. (USDA, 1972) The SCS 24-Hour Rainfall Distributions are shown in Figure 2.1.5.Type II is the distribution commonly used by the NRCS in planning and design. The application of this distribution is shown in Chapters 3 and 8.

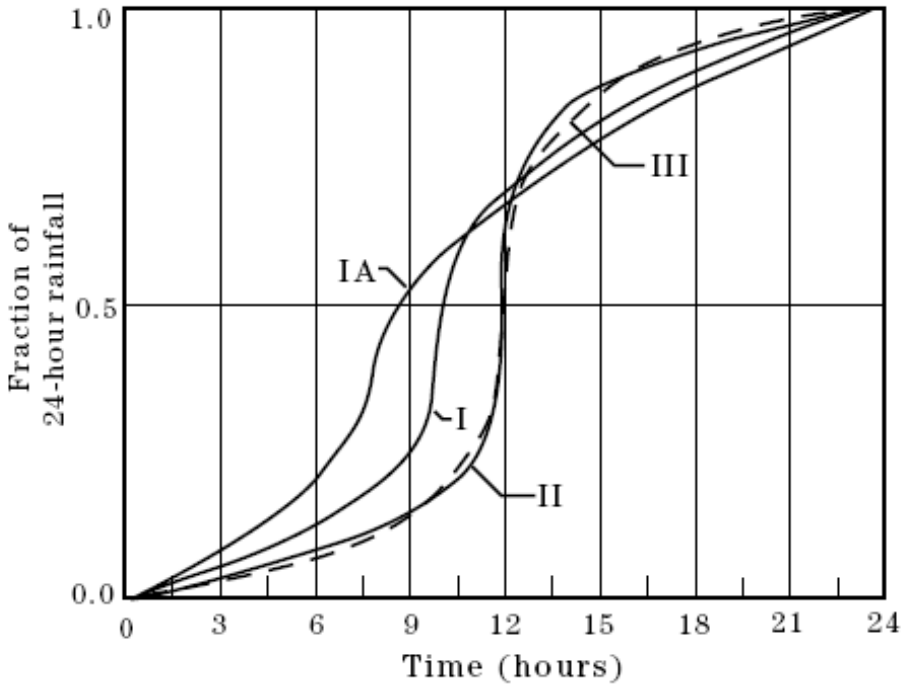


Figure 2.1.5
SCS 24-hour Rainfall Distribution (NEH-4)

In 2004, updated precipitation frequency estimates and temporal distributions of rainfall were provided for select areas of the United States in the National Oceanic and Atmospheric Administration's (NOAA) Atlas 14. As of February 2008, NOAA Atlas 14 is made up of three volumes, each representing a specific geographic region of the United States. For the purposes of this section, the focus will be on the information provided in Volume 2 of NOAA Atlas 14. The volumes and their corresponding geographic region are described by the following:

- Volume 1 – Semi-arid Southwest (includes Arizona, Southeast California, Nevada, New Mexico and Utah)

- Volume 2 – Ohio River Valley Basin and Surrounding States (includes Delaware, District of Columbia, Illinois, Indiana, Kentucky, Maryland, New Jersey, North Carolina, Ohio, Pennsylvania, South Carolina, Tennessee, Virginia, and West Virginia)

- Volume 3 - Puerto Rico and the Virgin Islands

The data utilized in the study was taken from 2846 daily precipitation stations, 994 hourly precipitation stations, and 96 N-min precipitation stations located throughout the Ohio River Valley Basin and states that border the study area. The data included station readings that went as far back as 126 years (since the time of the study, December 2000). Unlike the Huff distributions, NOAA Atlas 14 developed temporal distributions for specific time periods and

storm type (e.g., 1st Quartile, 2nd Quartile, etc.) rather than specific storm type. The distributions were developed for the 6-, 12-, 24-, and 96-hour time periods (Bonnin et al., 2006).

The study found that the temporal distributions varied very little throughout the entire study area. For example, data from the southeastern coastal states was compared with data from the northwest region of the Ohio River Valley Basin, and the distributions were nearly identical. Therefore, temporal distributions were developed for the entire study area as opposed to developing distributions for specific regions within the study area (Bonnin et al., 2006).

The temporal distributions for the study area are expressed as probabilistic relationships between the percentages of cumulative precipitation and storm duration. Plots of these relationships for the 6-, 12-, 24-, and 96-hour durations are included as Figure 2.1.6 below. The plots shown on Figures 2.1.7 – 2.1.10 categorize each storm duration by the quartile which recorded the greatest percentage of the total precipitation. The numerical data used to plot the temporal distribution graphs can be downloaded from the following website: http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_temporal.html (Bonnin et al., 2006).

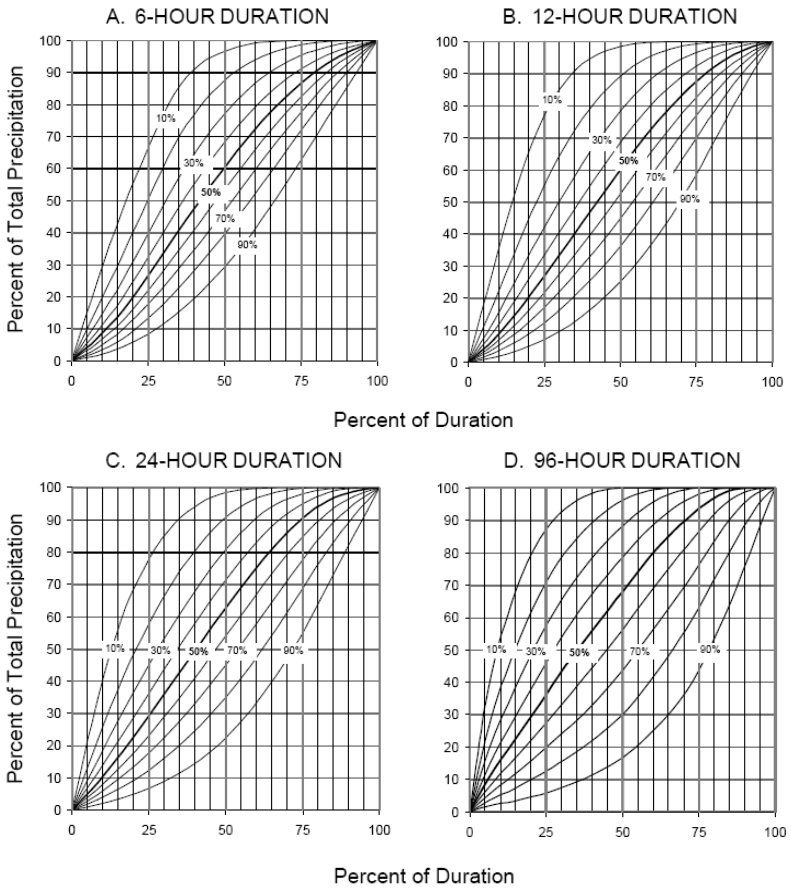


Figure 2.1.6
Temporal Distribution for 6- through 96-Hour Storm Duration (Bonnin et al., 2006)

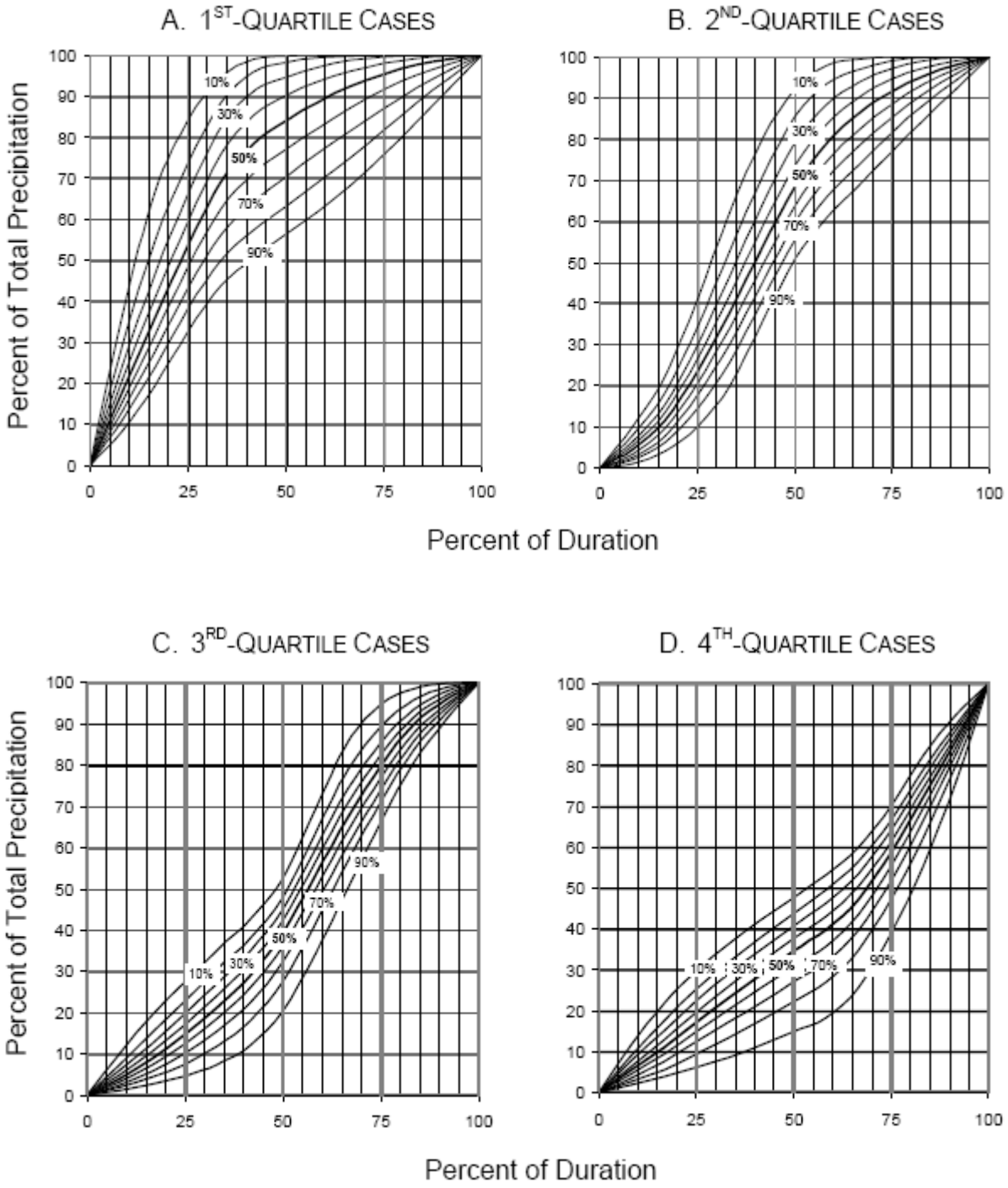


Figure 2.1.7
Temporal Distribution for the 6-Hour Storm Duration (Bonnin et al., 2006)

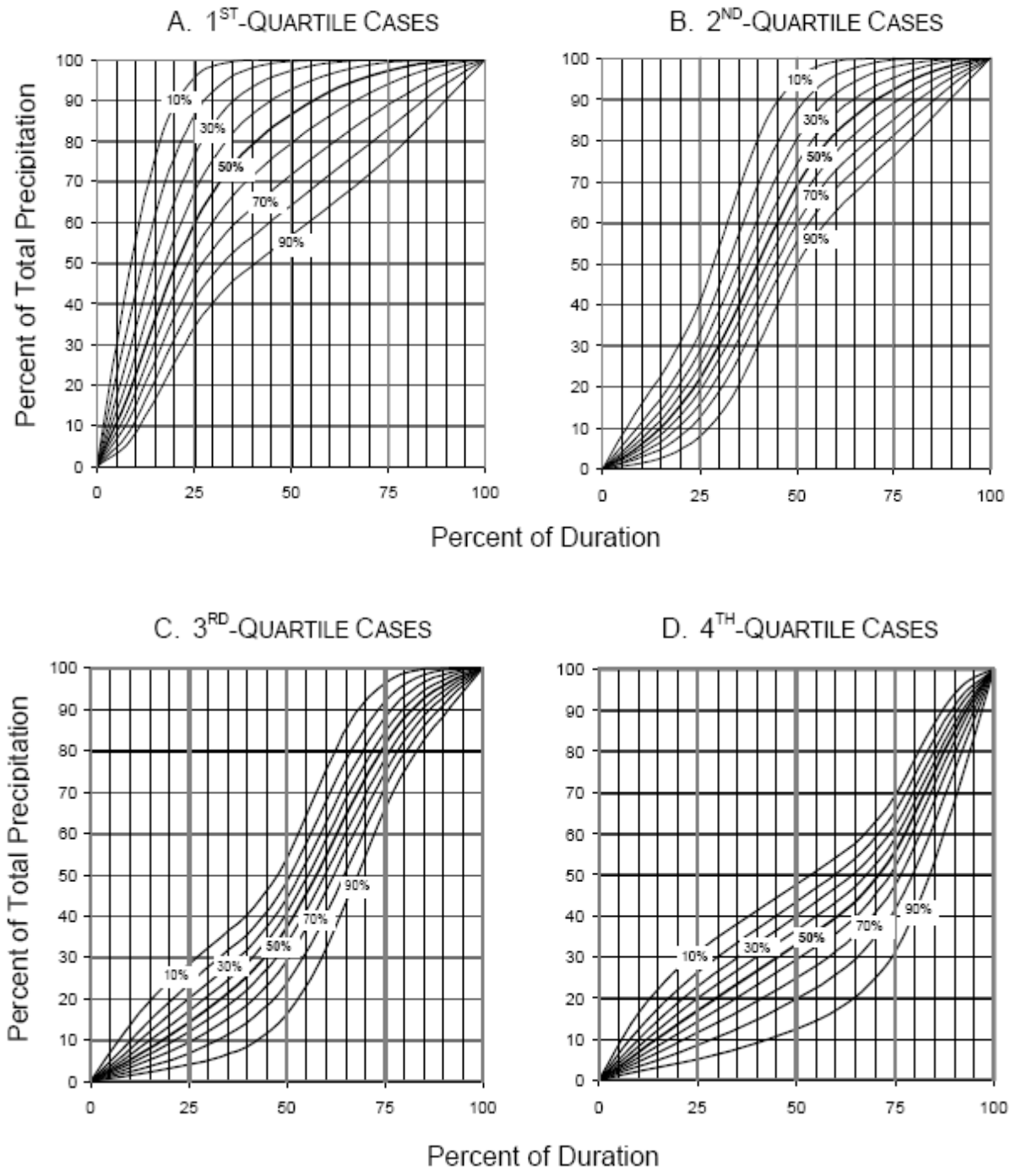


Figure 2.1.8
Temporal Distribution for the 12-Hour Storm Duration (Bonnin et al., 2006)

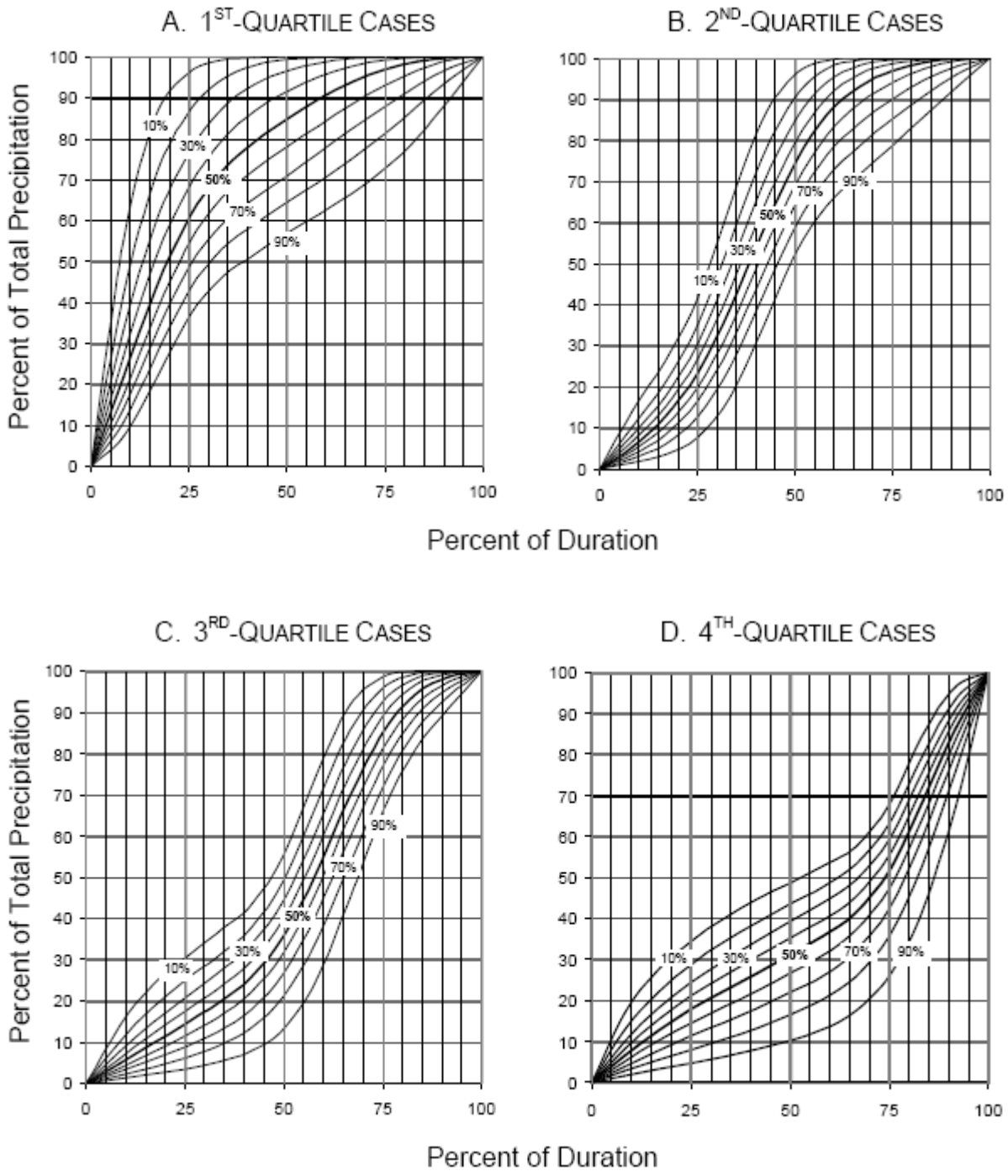


Figure 2.1.9
Temporal Distribution for the 24-Hour Storm Duration (Bonnin et al., 2006)

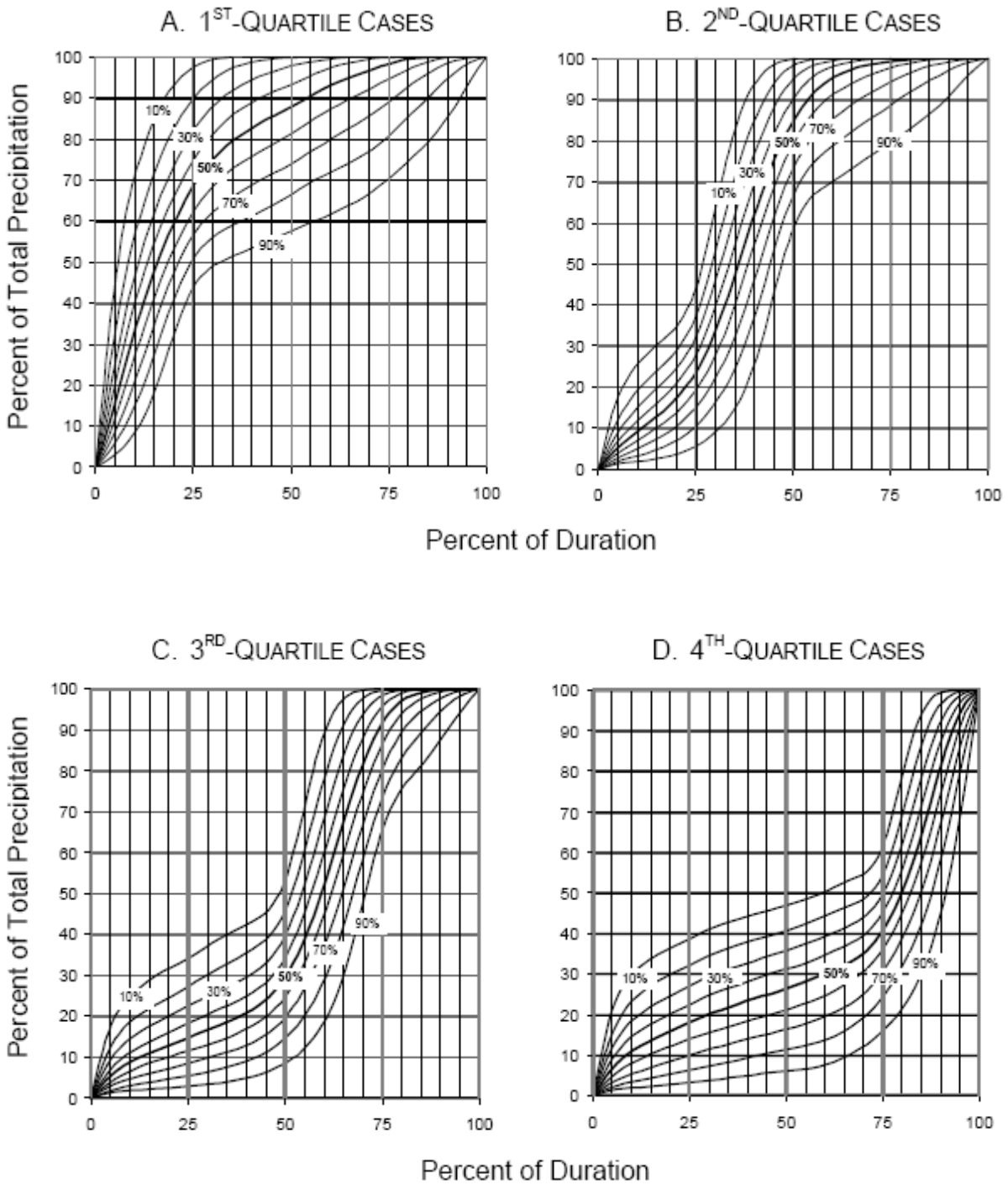


Figure 2.1.10
Temporal Distribution for the 96-Hour Storm Duration (Bonnin et al., 2006)

Example 2.1.2

This example problem illustrates the application of NOAA Atlas 14 to generate a time distribution of rainfall.

If Lafayette, Indiana experiences a 100-year, 6-hour storm event, find the hyetograph (the time distribution of rainfall) and cumulative rainfall depths using NOAA Atlas 14.

Precipitation frequency estimates for various durations and return intervals are available at user-specified locations from the NOAA Precipitation Frequency Data Server (PFDS) website at the following address: http://hdsc.nws.noaa.gov/hdsc/pfds/index.html. By selecting the precipitation data for Lafayette, it can be seen that the 100-year, 6-hour storm event has a rainfall depth of 4.96 inches.

Referring to Figure 2.1.6, the 50% temporal distribution for the 6-hour duration has the characteristics shown in the first and second columns below. These values were interpolated from the curve to give storm duration vs. depth at even time intervals. The methodology for calculating the time, cumulative rainfall, and incremental rainfall is identical to the methodology used in Example 2.1.1.

% Dimensionless Storm Time	50% 6-Hour Duration Dimensionless Storm Depth	Time (minutes)	Cumulative Rainfall (inches)	Incremental Rainfall (inches)
0	0	0	0.00	0
10	8.72	36	0.43	0.43
20	19.95	72	0.99	0.56
30	33.75	108	1.67	0.68
40	47.25	144	2.34	0.67
50	60.2	180	2.99	0.64
60	72.02	216	3.57	0.59
70	82.18	252	4.08	0.50
80	90.31	288	4.48	0.40
90	96.13	324	4.77	0.29
100	100	360	4.96	0.19

2.1.4 - Areal Distribution of Rainfall

Rainfall measurements are made at specific locations in a watershed. The structure of a storm and its internal variation are not represented by a single point measurement or even by many point measurements. (Hershfield, 1961; Eagleson, 1970).

As the area represented by a point measurement increases, the reliability of the data as a representation of an average over the entire region decreases. As drainage areas become larger than a few square miles, point data must be adjusted to estimate areal rainfall. Figure 2.1.7 was developed by Hershfield (1961) and demonstrates the relationship between average rainfall and the point rainfall over a watershed as a function of area and storm duration. The use of Figure 2.1.11 is illustrated in Example 2.1.3.

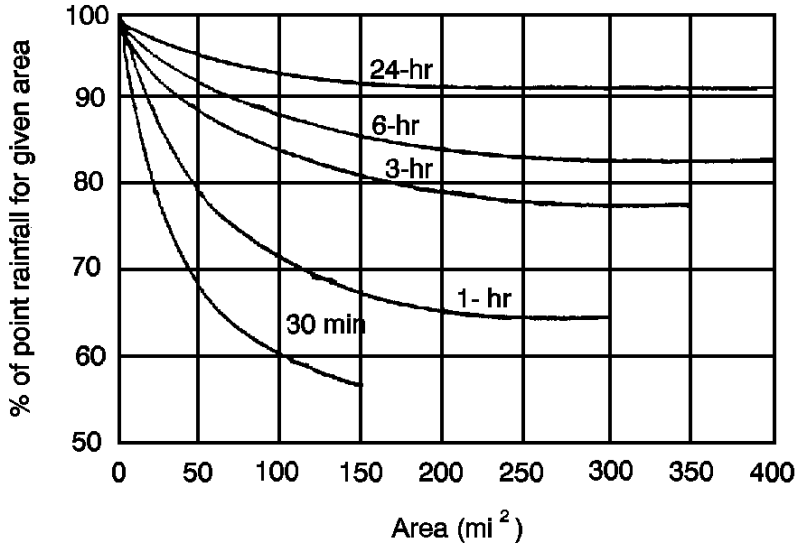


Figure 2.1.11
Area-Depth Curves (after Hershfield, 1961)

Example 2.1.3

A single gage is used to measure the rainfall over a 25 square mile watershed. If the gage collected 2.00 inches over a period of 3-hours, what is the estimated average rainfall over the entire watershed?

Referring to Figure 2.1.7, the 3-hour curve intersects a watershed area of 25 square miles at the ordinate 93%. Therefore the average rainfall over the entire watershed is 2.00*0.93= 1.86 in.

2.2 - COLLECTION AND ANALYSIS OF PRECIPITATION DATA

Drainage design engineers usually do not collect and analyze precipitation data. Rather, they use data that is already published. Information developed in this section will aid the designer to interpret the data published in sources such as the United States Weather Bureau (now knows as the National Weather Service) Technical Paper Number 40 and NOAA Technical Memo NWS HYDRO-35. This information is also beneficial in understanding concepts presented in other chapters of this manual.

2.2.1 - Sources of Hydrologic Data

Most of the precipitation data is archived along with temperature, solar radiation, dew point, relative humidity, wind speed, Palmer drought index and several other hydrologic quantities by the National Climatic Data Center (NCDC), a part of the National Oceanic and Atmospheric Administration (NOAA) of the U.S. Department of Commerce (more information available at their web site: http://www.ncdc.noaa.gov). In an effort to foster global cooperation, NCDC also maintains cooperative links with similar data centers throughout the world, and with other agencies like the World Meteorological Organization. In particular, precipitation data provided recorded by NCDC includes (a) rainfall measurements at ground stations; (b) estimates obtained from remote sensing operations such as radar and satellites; (c) snow accumulation from ground measurements or remotes sensors; and (d) snow covered area.

The Water Resources Division of the U.S. Geological Survey (USGS) has the primary responsibility of collecting and maintaining streamflow, stage, reservoir storage, groundwater levels, spring discharges and some water quality related data all over the country (see http://www.usgs.gov). The USGS collects real-time streamflow data and makes it available online for over 3000 stations. The Corps of Engineers, along with USGS and the National Weather Service (NWS) uses automated data acquisition systems for operating several multipurpose reservoir systems. The Geostationary Operational Environmental Satellites (GOES) have been used to transmit streamflow and precipitation data.

Furthermore, the USGS has developed and published regression equations for every state to estimate peak flood discharges. These regression equations were compiled into a micro-computer program titled the National Flood Frequency (NFF) Program. These equations are updated and reflect the increased availability of flood-frequency data and advances in flood-regionalization methods. These regression equations serve several purposes such as:

- Obtain estimates of flood frequencies for sites in ungaged basins.
- Obtain estimates of flood frequencies for sites in urbanized basins.
- Create hydrographs of estimated floods for sites in rural or urban basins.
- Create flood-frequency curves for sites in rural or urban basins.

The NFF program, an accompanying data base (NFFv3.mdb), and documentation can be

downloaded from the Web at http://water.usgs.gov/software/nff.html, or by anonymous file transfer protocol from ftp://water.usgs.gov/ (directory: /pub/software/surface_water/nff). Much of the documentation of the equations, maps, and other information pertaining to the regression equations for individual States is provided on line through links from the NFF web page (http://water.usgs.gov/software/nff.html). Much of Indiana data can be had from the website http://shadow.agry.purdue.edu, maintained by the Purdue Applied Meteorology Group.

2.2.2 - Precipitation Measurement and Interpretation

Variation in rainfall depths over an area is determined from rainfall depths observed at selected points in the watershed. Unless the rain gage density is high, accurate estimation of rainfall pattern and average values of rainfall depths usually cannot be obtained.

Rainfall is recorded with different levels of accuracy. First, there are the first-order Weather Service stations. These gages produce a continuous time-depth sequence which is usually transferred to an hourly sequence. Second, there are the recording-gage data of the hydrologic network which are published for clock-hour intervals. These data are processed to get hourly data. Thirdly, there are a very large number of nonrecording-gages, to obtain daily rainfall depths. (Hershfield, 1961).

After the data from particular gaging stations over an area have been collected, it may be necessary to average the depth of precipitation over an area. There are three methods of computing this average.

The first and the simplest method of obtaining the average depth is by using the arithmetic average. This method yields good estimates if the terrain is flat, and the gages are uniformly distributed and the individual gage catches do not vary widely from the mean.

In the second method, known as the Thiessen method, each gage is given a weight. The station locations are drawn on a map, and lines connecting the stations are drawn. Perpendicular bisectors of these connecting lines form polygons around each station. The area of each polygon is determined by planimetery and is expressed as a percentage of the total area. Weighted average rainfall for the total area is computed by multiplying the precipitation at each station by its assigned percentage of area and summing them up. The results from this method are regarded as more reliable than those obtained by simple arithmetic averaging.

The third and the most accurate method of estimating the average precipitation over a watershed is the isohyetal method. Station locations and amounts are plotted on a suitable map, and contours of equal precipitation (isohyets) are drawn. The average precipitation over an area is computed by multiplying the average precipitation between successive isohyets by the area of the watershed located between these isohyets, totaling these products, and then dividing the sum by the total area. The isohyetal method permits the use and interpretation of all available data and reflects orographic effects and storm distribution. Sample calculations of the three methods are shown in Figure 2.2.1

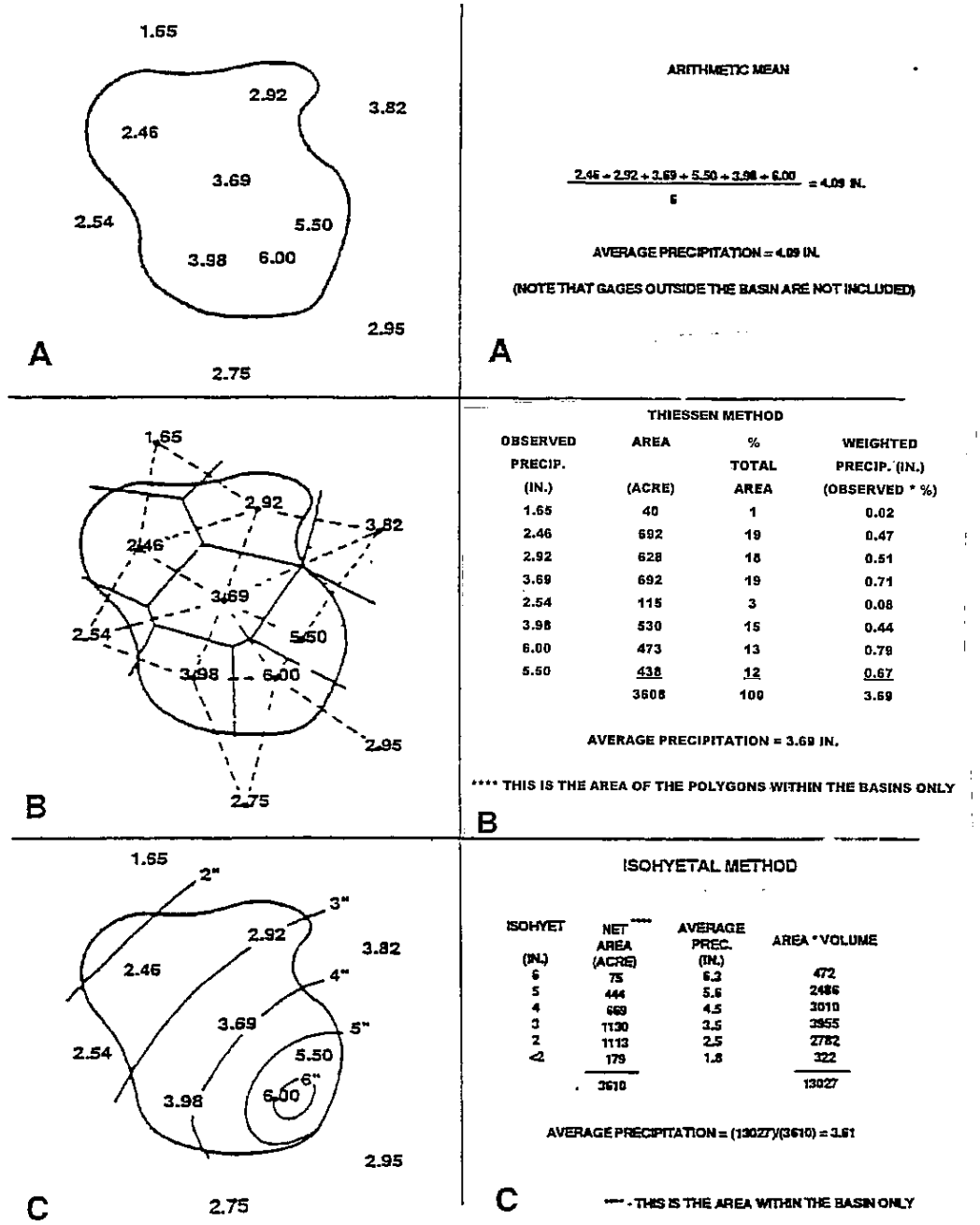


Figure 2.2.1
Areal Averaging of Precipitation by (a) Arithmetic Method (b) Thiessen Method (c) Isohyetal Method (Linsey et al., 1975)

2.2.3 - Statistical Analysis of Rainfall Data

Analysis of rainfall records to obtain design data is sometimes necessary. When more detailed information for a specific location is required, extreme value analysis is required. For this type of study, no less than 20 years of record is required if the approach is to have any statistical reliability. A brief discussion of statistical analysis of extreme rainfall data is presented in Appendix A.

The rainfall depth occurring over a specified duration is a basic unit of information used in drainage design. This information is used to estimate runoff from watersheds by using the techniques presented in Chapter 3. The selection of a design frequency is based on economic analysis and policy decisions.

Since theoretical aspects of frequency analysis for extreme values require that all data for the period of study be comparable, it is important that the basic data be thoroughly scrutinized. Data are comparable when all of it represents accurate, reliable observations. Data from individual storms are assumed to be independent. The following analysis must be applied only to extreme values. Changes in the location of a gage or other extraneous effects should be corrected.

The annual maximum series consists of only the largest value in any given year. The data are arranged in descending order of magnitude and assigned a rank (m) starting with one and increasing by one until the rank number equals the number of observations (n). For locations with limited data, the partial duration series may be more appropriate. The partial duration series consist of the n values larger than a threshold value regardless of the year of the storm event.

The return periods for the particular set of data are calculated by using Equation 2.2.1:

$$T_r = \frac{n+1}{m} \qquad (2.2.1)$$

where T_r is the return period of n-year event, n is the number of events or years of record, and m is the order or ranking number.

The return period may be transposed to frequency by using Eq. 2.2.2:

$$P = \frac{1}{T_r} = \frac{m}{1+n} \qquad (2.2.2)$$

where P = the frequency (average probability of occurrence in a year) of the event being equalled or exceeded.

For example, if it was found that three inches of rain in a given duration fell once in a nineteen year period of record, one might state that on the average, three inches of rainfall would occur

once every twenty years or with a frequency of P = 1 / (19+1) = 0.05.

Due to the small number of observations which are usually available, methods are needed to estimate rainfall magnitudes corresponding to larger return periods. Many well-defined theoretical probability distributions have been used to estimate rainfall magnitude at large return periods. It should be emphasized, however, that any theoretical distribution is not an exact representation of the natural process, but is only a probability description of the probabilistic structure of the process. Two of the commonly used distributions for rainfall analysis are the Gumbel's extreme value distribution and the Log Pearson Type III distribution. Either may be used as a formula or as a graphical approach. The general equation for each is Equation 2.2.3. (Chow et al., 1988),

$$p = \bar{p} + Ks \tag{2.2.3}$$

where p is the desired peak value for a specific frequency, $\bar{p}$ = arithmetic average of the given rainfall data, K is the frequency factor (use K for Gumbel distribution from Table 2.2.1 or K_t for Log Pearson Type III distribution from Table 3.5.2), and s is the standard deviation of the given rainfall data.

In utilizing Gumbel's distribution, the arithmetic average in Eq. 2.2.4 is used:

$$\bar{p} = \frac{1}{n}\sum_{i=1}^{n} p_i \tag{2.2.4}$$

where p_i is the individual extreme value of rainfall and n is the number of events or years of record.

The standard deviation is calculated by Eq. 2.2.5:

$$s = \left[\frac{1}{n-1}\sum_{i=1}^{n} (p_i - \bar{p})^2\right]^{\frac{1}{2}} \tag{2.2.5}$$

The frequency factor (K) (given in Table 2.2.1), which is a function of the return period and sample size, when multiplied by the standard deviation gives the departure of a desired return period rainfall from the average.

The Log Pearson Type III (LP (III)) distribution involves logarithms of the measured values. The mean and the standard deviation are determined using the logarithmically transformed data. The simplified expression for this distribution is given as: (see also Equation A-12 in Appendix A) (Chow et al., 1988).

$$\log p = \bar{y} + K_t s_y \tag{2.2.6}$$

$$\text{where} \qquad \bar{y} = \frac{1}{n}\sum_{i=1}^{n} y_i \tag{2.2.7}$$

$$\text{where} \qquad y_i = \log(p_i) \tag{2.2.8}$$

$$s_y = \left(\frac{1}{n-1}\sum_{i=1}^{n}(y_i - \bar{y})^2\right)^{\frac{1}{2}} \tag{2.2.9}$$

The skewness coefficient, C_s, is required to compute the frequency factor for this distribution. The skewness coefficient is computed by Eq. 2.2.10 (Chow et al., 1988).

$$C_s = \frac{n\sum_{i=1}^{n}(y_i - \bar{y})^3}{(n-1)(n-2)s_y^3} \tag{2.2.10}$$

By knowing the skewness coefficient and the recurrence interval, the frequency factor, K_t for the LP(III) distribution, is read off from Table 3.5.2. The antilog of the solution in Equation 2.2.6 will provide the estimated extreme value for the given return period.

Table 2.2.1
Frequency Factors (K) for the Gumbel Distribution

Sample Size	Recurrence Interval						
	10	20	25	50	75	100	1000
15	1.703	2.410	2.632	3.321	3.721	4.005	6.265
20	1.625	2.302	2.517	3.179	3.563	3.836	6.006
25	1.575	2.235	2.444	3.088	3.463	3.729	5.842
30	1.541	2.188	2.393	3.026	3.393	3.653	5.727
40	1.495	2.126	2.326	2.943	3.301	3.554	5.476
50	1.466	2.086	2.283	2.889	3.241	3.491	5.478
60	1.446	2.059	2.253	2.852	3.200	3.446	
70	1.430	2.038	2.230	2.824	3.169	3.413	5.359
75	1.423	2.029	2.220	2.812	3.155	3.400	
100	1.401	1.998	2.187	2.770	3.109	3.349	5.261

Example 2.2.1

Using the twenty-five years of data tabulated below, determine the 10-year and 50-year precipitation depths for a 1-hour duration storm in Coshocton, OH. Assume that the Gumbel distribution is applicable (adapted from Chow et al., 1988).

Rank	Max. Depth (in.) for 60-min Duration		Rank	Max. Depth (in.) for 60-min Duration
1	3.220		13	1.204
2	1.830		14	1.203
3	1.756		15	1.200
4	1.510		16	1.194
5	1.431		17	1.192
6	1.375		18	1.174
7	1.313		19	1.143
8	1.306		20	1.130
9	1.290		21	1.130
10	1.269		22	1.109
11	1.225		23	1.095
12	1.213		24	1.094
			25	1.063

Using Equations 2.2.4 and 2.2.5, the sample statistics are:

$$\bar{p} = \frac{1}{n}\sum_{i=1}^{n} p_i = \frac{1}{25}(3.220 + 1.830 + \ldots + 1.063) = 1.347$$

$$s = \left(\frac{1}{n-1}\sum_{i=1}^{n}(p_i - \bar{p})^2\right)^{\frac{1}{2}}$$

$$= \left(\frac{1}{24}((3.220 - 1.347)^2 + (1.830 - 1.347)^2 + \ldots + (1.063 - 1.347)^2\right)^{\frac{1}{2}}$$

$$s = 0.434$$

The rainfall depth is obtained by:

$$p_t^{T_r} = \bar{p} + Ks$$

where T_r is the return period (years) and t is the time (hours).

From Table 2.2.1, for a 10-year return period with sample size equal to 25, K is 1.575. From Table 2.2.1, for a 50-year return period with sample size equal to 25, K is 3.088.

p_1^{10} = 1.347+1.575*(0.434) = 2.031 inches
p_1^{50} = 1.347+3.088*(0.434) = 2.687 inches

2.2.4 - Hydraulic Risk

A return period of one hundred years implies that on the average that event will occur or be exceeded once every one hundred years, but does not guarantee that the event will occur every one hundred years. The concept of risk takes this into account by considering the chance of a particular event occurring within a given period. This risk is often something that the engineer must use in determining the economic feasibility of a design. Risk is determined by Eq. 2.2.11:

$$J = 1 - (1 - P)^n = 1 - \left(1 - \frac{1}{T_r}\right)^n \qquad (2.2.11)$$

where J is the risk of a certain event during a time interval, T_r is the return period, n is the number of years in the time interval.

Example 2.2.2

What is the risk of exceeding a 10-year return period storm in the next 5 years?

$T_r = 10$ and $n = 5$

$$J = 1-(1-\frac{1}{T_r})^n = 1-(1-\frac{1}{10})^5 = 0.41$$

So, the risk of exceeding this storm in the next 5 years is 41%.

2.2.5 - Depth and Intensity-Duration-Frequency Equations

Curves of depth or intensity-duration-frequency have been developed for various stations of Indiana (Purdue et al., 1992). These are very useful to the design engineer as input to the deterministic runoff models. To ease computational effort and in order to incorporate these curves in computer models, equations have been developed. An understanding of these equations or curves will aid the designer in their use for specific locations.

Depth and intensity-duration-frequency curve distributions are developed by using distributions as discussed in the last section. Depth and intensity are related, since intensity, i, is nothing more than depth, ΔD, divided by an increment of time, Δt, as shown in Equation 2.2.12.

$$i = \frac{\Delta D}{\Delta t} \tag{2.2.12}$$

It is important to realize that the intensity-duration-frequency values obtained do not represent any particular storm pattern or storm. It is the maximum amount of rain that has fallen for a particular time interval over the n-years of record.

The rainfall intensities, i, corresponding to a storm duration, t (hours), and a recurrence interval, T_r, can be represented in the form:

$$i = \frac{c\,T_r^{\alpha}}{(t+d)^{\beta}} \tag{2.2.13}$$

where c, d, α, and β are regional coefficients determined by evaluation of rainfall intensity-duration-frequency curves. The coefficients and exponents for several major cities in Indiana have been calculated by Purdue et al., 1992 and are shown in Table 2.2.2. The curves generated by this equation, for Indianapolis, are shown in Figure 2.2.2. In order to produce a smooth curve, curve fitting was used between the 0.6 hour and 2.0 hour values. Equation 2.2.13 is referred to as the intensity-duration-frequency (IDF) equation.

Example 2.2.3

Using the IDF equation, determine the 10-year, 15-minute rainfall intensity for the City of Indianapolis.

From Table 2.2.2, c=2.1048 α=0.1733 d=0.470 β=1.1289

Referring to the IDF equation,

$$i = \frac{c\,T_r^{\alpha}}{(t+d)^{\beta}} = \frac{2.1048\,(10)^{0.1733}}{(\frac{15}{60}+0.470)^{1.1289}} = 4.545 \text{ inches/hour}$$

Table 2.2.2
Regional Coefficients for the IDF Equation (Eq. 2.2.13)
(Purdue et al., 1992)

Station	c	α	d	β
0.083 hour < t ≤ 1 hour				
Indianapolis	2.1048	0.1733	0.470	1.1289
South Bend	1.7204	0.1753	0.485	1.6806
Evansville	1.9533	0.1747	0.522	1.6408
Fort Wayne	2.0030	0.1655	0.516	1.4643
1 hour < t < 36 hour				
Indianapolis	1.5899	0.2271	0.725	0.8797
South Bend	1.2799	0.1872	0.258	0.8252
Evansville	1.3411	0.2166	0.300	0.8154
Fort Wayne	1.4381	0.1878	0.525	0.8616

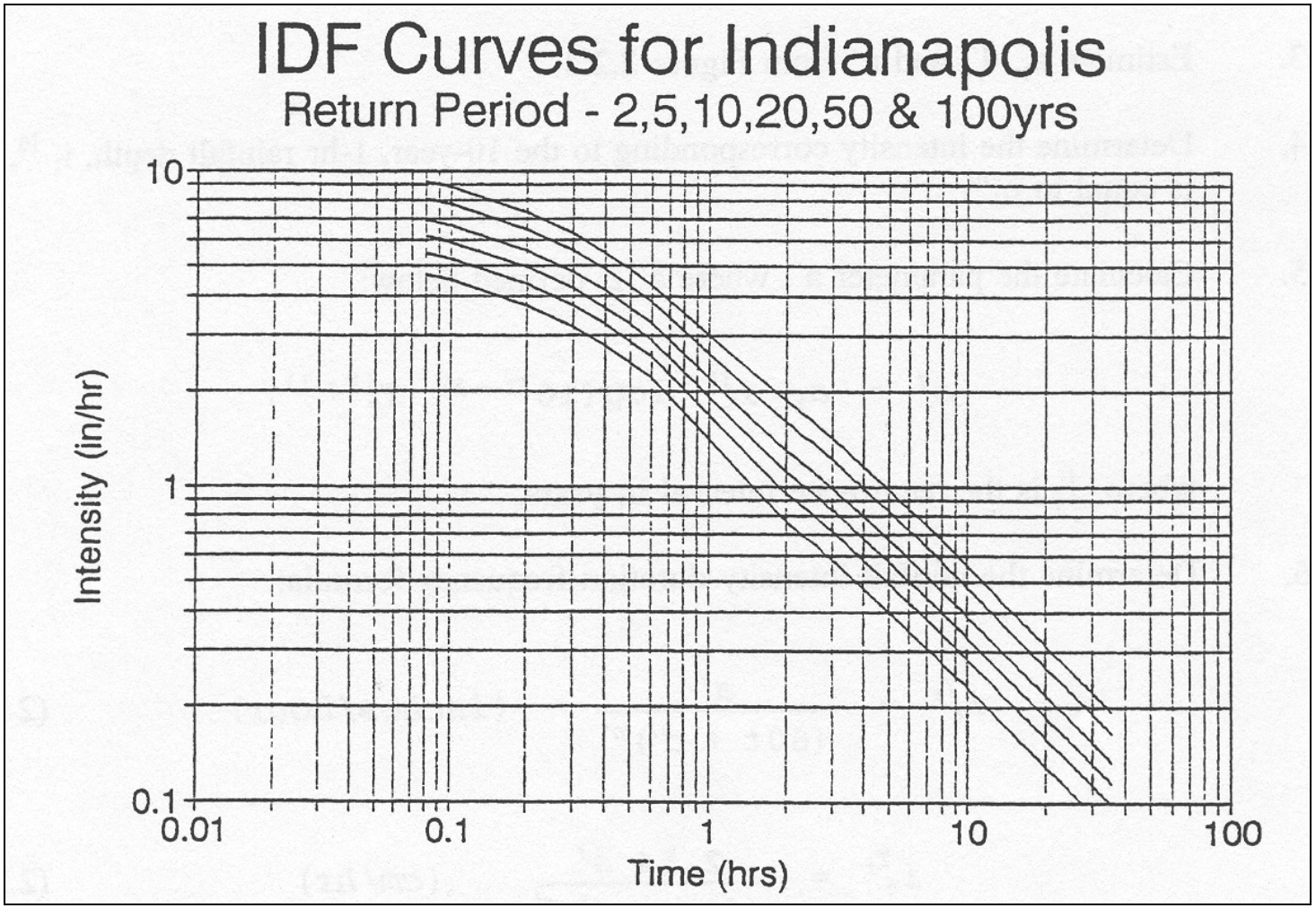


Figure 2.2.2
Intensity-Duration-Frequency Relationship for Indianapolis by IDF Equation (adapted from Purdue et al., 1992)

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LIST OF PARAMETERS

A	Area (acres)
A´	Area (ft^2 or m^2)
A_m	Area (mi^2)
B	Coefficient for Izzard's t_c formula
c,d,α,β	Regional coefficients for determining rainfall intensity-duration-frequency curves
C	Rational method runoff coefficient
c´	Retardance coefficient for Izzard's t_c formula
CN	Curve number
C_m	Map skew
C_s	Coefficient of skewness
C_w	Weighted skew
C_{comp}	Composite runoff coefficient
CN_{comp}	Composite curve number
DA	Drainage area (mi^2)
ΔD	Rainfall duration
F(t)	Total infiltration
G	WRC method parameter
H	WRC method parameter
i	Rainfall intensity (in/hr or cm/hr)
I_a	Initial abstraction
K	Coefficient for t_c equations
K_n	Coefficient used in outlier detection equation
K_T	Coefficient used in WRC method
L	Length (ft or m)
L´	Watershed lag
L_m	Length (miles)
μ	max [1, t+1 - NU]
n	Number of data values
n	Manning's roughness coefficient
N	Kerby's retardance coefficient
NP	Number of elements in the rainfall hydrograph up to last non-zero entry
NU	Number of coordinates in the unit hydrograph up to last non-zero entry
ν	min [t, NP]
P(t)	Accumulated rainfall (in)
PREC	Mean annual precipitation (inches)
q	Excess rainfall (in)
q_p	Peak flow (cfs)
q_t	Discharge at time t
Q	Peak runoff (cfs)
Q´	Total volume of triangular unit hydrograph (in)

LIST OF PARAMETERS (cont'd)

Q_t	Runoff at time t (cfs)
Q_t'	Accumulated Volume at time t
Q_{Tr}	Flow rate for corresponding return period
RC	Soil runoff coefficient
R(t)	Accumulated runoff (in)
s	Slope (ft/ft or m/m)
s´	Average surface slope (%)
s_y	Standard deviation
S	Ultimate abstraction
SL	Slope (feet per mile)
STOR	Storage (%)
t	Time (hrs)
t_b	Time (duration) of base
t_c	Time of concentration (min)
t_p	Time to peak
t_r	Recession duration
t_t	Travel time (min)
T_r	Return period
u	Ordinate of the unit hydrograph (cfs/in)
v	Velocity
V	Volume
$V(C_s)$	Variance of the station skew
$V(C_m)$	Variance of the map skew
V_{UH}	Volume of the unit hydrograph
V_t	Total volume of the triangular unit hydrograph
W	Weight
x_i	Annual maximum flow values
$\bar{y}$	Mean
y_i	Logarithmic annual maximum flow values
y_H	Logarithm of a high outlier
y_L	Logarithm of a low outlier

Chapter 3 - RUNOFF AND ITS ESTIMATION

3.1 - FACTORS AFFECTING RUNOFF

In the discussion throughout Chapter 2, it was noted that *only a part of* the rainfall is converted to surface runoff. Figure 3.1.1 presents a comprehensive view of one of the many possible interactions between rainfall and the earth as time increases from the beginning of rainfall.

The horizontal axis in Figure 3.1.1 represents the time from the start of the rainfall and the vertical axis represents the fraction of the rainfall rate (depth per unit time) absorbed by each of the components shown. This particular diagram represents an extensive storm of uniform intensity on a dry basin.

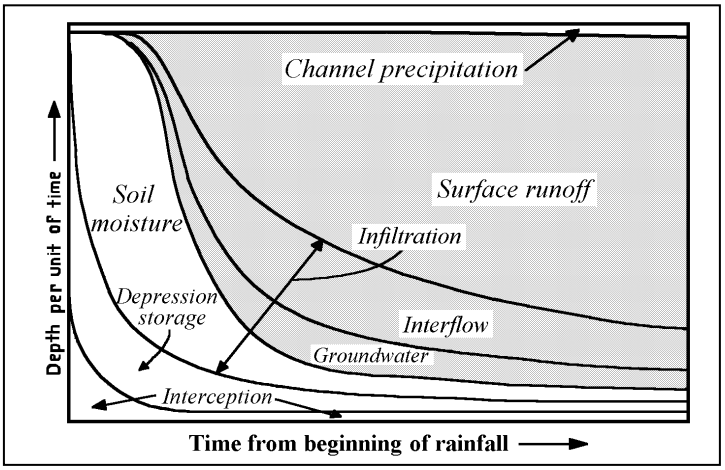


Figure 3.1.1
Schematic Diagram of the Disposition of Storm Rainfall
(after Linsley et al., 1975)

The shaded portion of the diagram represents the portion of the rainfall which will become flow measured at the point under consideration. During the early period of the storm, *channel precipitation*, rainfall that falls directly on the channel, is the only input to flow. As the storm progresses, other factors dominate: depression storage; interception; groundwater flow; interflow; infiltration; and surface runoff.

Depression storage is the volume of water which collects in natural depressions or ponds on impermeable surfaces. Once the rainfall intensity exceeds the local infiltration capacity of the soil, surface depressions, natural and man-made, begin to fill. After smaller depressions are filled, overland flow begins which in turn fills larger depressions or flows directly to the channel.

Interception is rainfall which is held in storage by vegetation and other wetted surfaces.

Infiltration is the seepage of rainfall into the subsurface. The infiltration capacity of the soil depends upon the soil type, moisture content of the soil, amount of organic matter present, vegetal cover, season of the year and rainfall intensity. As a storm progresses, infiltration rate decreases because the capillary spaces in the soil are filled. Infiltration is affected by urbanization as permeable soils are replaced with impermeable structures. Obviously, infiltration opportunity decreases and runoff increases.

The contribution of *groundwater* to channel flow does not fluctuate rapidly because of long flowpaths and low velocities through the soil. The groundwater contributes to the channel if the water table intersects the channel.

Interflow is the portion of water which infiltrates the soil surface and moves laterally through the upper layers of the soil until it re-emerges or enters the channel. It is dependent upon the soil type and the geology of the watershed under consideration. For some watersheds, subsurface flow may be the dominant contribution to stormwater runoff.

The last portion of Figure 3.1.1 is the *surface runoff* which starts at zero and increases as the storm progresses. As the storm progresses, the runoff level becomes a relatively constant percentage of rainfall. In urban areas with a high percentage of impervious area and low levels of detention storage, the major contributor to flow in the channel is surface runoff.

3.2 - THE RATIONAL METHOD

The rational method is one of the oldest, simplest, and most widely used and often criticized methods employed in the determination of peak discharges from a given watershed. It was first introduced into this country by Kuichling in 1889, and a survey indicated that it is used in 90 percent of the engineering offices in the United States (Ardis, et al., 1969). This popularity can probably be attributed to its simplicity, "ease" of application and tradition.

The fundamental idea behind the rational method is that the peak rate of surface outflow from a given watershed is proportional to the watershed area and average rainfall intensity over a period of time just sufficient for all parts of the watershed to contribute to the outflow. The constant of proportionality reflects the characteristics of the watershed, such as imperviousness, which affect runoff. The rational formula is written as:

$$Q = C\,i\,A \qquad (3.2.1)$$

where Q is the peak runoff (cubic feet per second - cfs), C is the ratio of peak runoff rate to average rainfall rate over the watershed during the time of concentration (runoff coefficient), *i* is the rainfall intensity (inches/hour), and A is the contributing area of watershed under consideration (acres).

It should be noted that the conversion from acres-inches/hour to cfs is 1.008. This value is rounded to 1.0 and it is for these units that the formula was termed “rational".

For metric units,

$$Q = 0.02778\, C\, i\, A \qquad (3.2.2)$$

where Q is the peak discharge (cubic meters per second - m^3/s), C is the ratio of peak runoff rate to average rainfall rate over the watershed during the time of concentration (runoff coefficient), *i* is the rainfall intensity (centimeters/hour - cm/hr), and A is the contributing area of watershed under consideration (hectares - ha).

The coefficient 0.02778 arises from the conversion from hectare-centimeters/hour to m^3/s.

In general, the rational method should be applied to drainage basins less than 200 acres (81 ha) in area and is best suited for well-defined drainage basins. Some local ordinances limit the use of the rational method to basins to areas much smaller than the 200 acres (81 ha). Application of the rational method is illustrated in Example 3.2.1.

The basic assumptions used in the application of the rational formula are as follows.

1. The return period of the peak discharge is the same as that of the rainfall intensity.
2. The rainfall is uniform in space over the watershed under consideration.
3. The storm duration associated with the peak discharge is equal to the time of concentration for the drainage area (the time for the most hydraulically-distant point to contribute to the peak outflow at the point under consideration).
4. The runoff coefficient C is not influenced by the return period.
5. The runoff coefficient C is independent of the storm duration for a given watershed and reflects any changes in infiltration rates, soil types and antecedent moisture conditions.

3.2.1 - Determination of Runoff Coefficient, C

Values of the runoff coefficient are given in Table 3.2.1 for rural areas and Table 3.2.2 for urban areas. Table 3.2.2 presents runoff coefficients for particular types of urban areas and Table 3.2.3 gives coefficients which are used to compute a weighted C based on the actual percentage of lawns, streets, roofs, etc. The determination of the runoff coefficient is illustrated in Example 3.2.1.

Table 3.2.1
Rural Runoff Coefficients (Schwab et al., 1966)

Vegetation and Topography	Soil Texture: Open Sandy Loam	Clay and Silt Loam	Tight Clay
Woodland			
Flat 0-5% slope	0.10	0.30	0.40
Rolling 5-10% slope	0.25	0.35	0.50
Hilly 10-30% slope	0.30	0.50	0.60
Pasture			
Flat	0.10	0.30	0.40
Rolling	0.16	0.36	0.55
Hilly	0.22	0.42	0.60
Cultivated			
Flat	0.30	0.50	0.60
Rolling	0.40	0.60	0.70
Hilly	0.52	0.72	0.82

As mentioned before, this coefficient represents the runoff-rainfall ratio and includes many factors such as type of cover, soil types, infiltration, evaporation, evapo-transpiration, and any antecedent moisture condition. For many years it has been known that C actually does not remain constant during a storm (Horner, 1910). The strong dependence on "engineering judgment" in selecting a runoff coefficient is one of the main weaknesses of the rational method.

Table 3.2.2
Urban Runoff Coefficients for the Rational Method (ASCE, 1992)

Description of Area	Runoff Coefficients
Business	
Downtown	0.70 to 0.95
Neighborhood	0.50 to 0.70
Residential	
Single-family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard	0.20 to 0.35
Unimproved	0.10 to 0.30

Table 3.2.3
Values Used to Determine a Composite Runoff Coefficient for an Urban Area
(ASCE, 1992)

Character of Surface	Runoff Coefficients
Pavement	
Asphalt and Concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, sandy soil	
Flat, 2 percent slope	0.05 to 0.10
Average, 2 to 7 percent slope	0.10 to 0.15
Steep, 7 percent slope	0.15 to 0.20
Lawns, heavy soil	
Flat, 2 percent slope	0.13 to 0.17
Average, 2 to 7 percent slope	0.18 to 0.22
Steep, 7 percent slope	0.25 to 0.35
Water Impoundment	1.00

3.2.2 - Determination of a Time of Concentration

The rational method assumes that the storm duration is at least as long as the time of concentration, which is the time for the most hydraulically distant point to contribute to the flowrate at the point under consideration. The time of concentration (t_c) is calculated in order to estimate the intensity of the design storm.

The time of concentration is often taken to be the travel time to a given location, which is the sum of the overland flow time, the gutter flow time, and the sewer flow time. The time of concentration at this location is the longest travel time when all paths are considered. For sewer design, this means that the inlet time for each sub-basin must be compared to the travel times from all upstream sub-basins and the longest time selected as the local time of concentration.

Table 3.2.4 presents some commonly used formulae to determine the overland flow time. Most of these equations relate this time to the basin length, slope, and surface roughness. Two equations, by Izzard and by Ragan, include rainfall intensity and hence, an iterative solution is necessary to compute t_c. The equation used for the computation of t_c should be chosen according to the specific site conditions, following the guidelines applicable to those equations. Gutter flow times can be estimated using the Manning formula which is discussed in Chapter 5. Often the inlet time is merely assumed. Commonly used values for the inlet time vary from 5 to 20 minutes (ASCE, 1986). The SCS (1986) has adopted the simple curves presented in Figure 3.4.5 for computing the time of concentration in urban areas.

These inlet times are added to the flow time in the pipe or channel to determine the travel time at the next point of interest. However, if the inlet time for this point is greater than the upstream travel time, the inlet time is used as t_c in the subsequent calculation.

The sewer flow time is usually calculated by choosing a pipe or channel configuration and calculating the velocity. The travel time is then found by:

$$t_t = \frac{L}{v}\frac{1\ minute}{60\ seconds} = \frac{L\,A'}{Q}\frac{1 minute}{60\ seconds} \qquad (3.2.3)$$

where t_t is the travel time in pipe (min), L is the reach length (ft, m), v is the velocity in reach (ft/sec, m/sec) = Q/A´, and A´ is the cross-section area (ft^2, m^2).

Table 3.2.4
Equations for Determining Overland Flow Time

GENERAL NOTE ON t_c FORMULAE: t_c will have units of minutes for all following equations.

Federal Aviation Administration (1970)

$$t_c = \frac{K\,(1.1 - C)\,L^{0.5}}{s'^{\frac{1}{3}}}$$

where K is equal to 1.8 for U.S customary units (3.26 for metric units), C is the rational method runoff coefficient, L is the length of overland flow in ft (m), and s´ is the average surface slope, %.

This method was developed from airfield drainage data assembled by the Corps of Engineers. The method is intended for use on airfield drainage areas, but has been used for overland flow in urban areas (Chow et al., 1988).

Kinematic Wave (Ragan, 1972)

$$t_c = \frac{K\,(L^{0.6}\,n^{0.6})}{i^{0.4}\,s^{0.3}}$$

where K is equal to 0.94 for U.S. customary units (2.78 for metric units), L is the length of overland flow in ft (m), *n* is Manning's roughness coefficient for sheet flow (tabulated in Table 3.2.6), *i* is the rainfall intensity, in/hr (cm/hr), and s is the average slope of overland flow path, ft/ft (m/m).

The length of the overland flow segment generally should be limited to 100 feet (30.5 m). This method requires iteration since both *i* and t_c are unknown. The steps for iteration are:

1. Assume a trial value of rainfall intensity (*i*)
2. Find the overland travel time (t_c)
3. Find the actual rainfall intensity for storm duration of t_c from the appropriate intensity-duration-frequency (IDF) curve for the area under consideration.
4. Compare rainfall intensities, if they are not the same, select a new trial rainfall and repeat step 1 (Wanielista, 1990).

Table 3.2.4 (cont'd)

Kerby (1959)

$$t_c = K\,(L\,N\,s^{-0.5}\,)^{0.467}$$

where K is equal to 0.83 (US Customary units) or 1.44 (Metric units), L is the length of flow in ft (m), s is the average slope of overland flow, ft/ft (m/m), and N is the retardance roughness coefficient given in Table 3.2.5.

The length used in the equation is the straight-line distance from the most distant point of the watershed to the outlet, measured parallel to the slope of the land until a well-defined channel is reached. Watersheds of less than 10 acres were used to calibrate the model; slopes were less than 1%; N values were 0.8 and less and surface flow dominated (McCuen, 1989).

Izzard (1946)

$$t_c = \frac{K(Bi + c')\,L^{\frac{1}{3}}}{s^{\frac{1}{3}}\,i^{\frac{2}{3}}}$$

where K is equal to 41.025 for U.S. customary units (113.391 for metric), B is equal to 0.0007 for U.S customary units (0.00027 for metric), c´ is the retardance coefficient given in Table 3.2.7, *i* is the rainfall intensity, in/hr (cm/hr), L is the length of flow path in ft (m), and s is the slope of overland flow path, ft/ft (m/m).

The product of *i* and L must be less than 500 in-ft/hr (390 cm-m/hr) to consider using this formula. In addition, well defined channels should **not** be present. This method was developed in laboratory experiments for the overland flow on roadway and turf surfaces.

Table 3.2.5
Values of N for Kerby's Formula (Kerby, 1959)

Type of Surface	N
Smooth impervious surface	0.02
Smooth bare packed soil	0.10
Poor grass, cultivated row crops or moderately rough bare surface	0.20
Deciduous timberland	0.60
Pasture or Overage grass	0.40
Conifer timberland, deciduous timberland with deep forest litter or dense grass	0.80

Table 3.2.6
Manning's *n* Roughness Coefficients for Sheet Flow (Engman, 1983)

Type of Surface	*n*
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (with no residue)	0.05
Cultivated soils	
Cover ≤ 20%	0.06
Cover ≥ 20%	0.17
Grass	
Short grass, prairie	0.15
Dense grass	0.24
Bermudagrass	0.41
Range	0.13
Woods	
Light underbrush	0.40
Dense underbrush	0.80

Table 3.2.7
Values of c for Izzard's Formula

Surface	c´
Smooth asphalt surface	0.007
Concrete pavement	0.012
Tar and gravel pavement	0.017
Closely clipped sod	0.046
Dense bluegrass turf	0.060

3.2.3 - Application of the Rational Method

The following procedure is used to apply the Rational Method.

Step 1: Determine the contributing basin area A (acres or hectares) by using USGS topographical maps, Indiana county drainage maps, maps developed from a survey of the area, or plans made specifically for the basin. This area is found by using a planimeter or digitizer.

Step 2: By the use of Table 3.2.1 for rural areas or Table 3.2.2 for urban areas, estimate the appropriate value of C. If the land use is mixed, a composite C value is estimated from Table 3.2.3 or is determined by:

$$C_{comp} = \frac{(C_1 A_1 + C_2 A_2 + ... + C_n A_n)}{A_t} \quad (3.2.4)$$

where C_1, C_2.....C_n are the runoff coefficients (taken from Tables 3.2.1 and 3.2.2) associated with component areas A_1, A_2....A_n and $A_t = A_1 + A_2 + ... + A_n$.

Step 3: Determine the time of concentration. The overland flow time of concentration for the basin can be determined by using one of the equations listed in Table 3.2.4 or the curves shown on Figure 3.4.5. The travel time in rills and channels are added to the overland flow travel times.

Step 4: Determine the intensity and peak discharge. Enter an intensity-duration-frequency curve with the storm duration equal to the time of concentration and, by choosing a curve corresponding to the appropriate return period, read off the intensity value. This is then multiplied by the area and runoff coefficient to determine the peak discharge rate.

If there is another basin downstream, the first time of concentration is added to the travel time in the channel or pipe using Equation 3.3. This is then compared to the inlet time of the second basin and the larger of two is used as the new time of concentration for the combined areas.

Example 3.2.1

This example problem illustrates the use of the rational method.

A watershed in South Bend is composed of three separate subbasins, Basin 1, Basin 2, and Basin 3, as shown in the Figure 3.2.1. Runoff from the uppermost basin, Basin 1, flows to point "a", where all flow from the subbasin is collected at an inlet. A pipe runs from point "a" to point "b", where another inlet collects the additional runoff from Basin 2. A second pipe runs from point "b" to point "c", where the additional runoff from Basin 3 is collected. If the basins have the following characteristics, calculate the runoff at points "a", "b", and "c" by the rational method for a 10-year return period. Assume the pipes flow with a velocity of 3 ft/sec. Pipe "a-b" has a length of 600 ft. Pipe "b-c" has a length of 90 ft.

Basin	*Area (acres)*	*Basin Travel Time (min)*	*Land Use*
Basin 1	2.4	10	Apartment Complex
Basin 2	13.7	37	Park
Basin 3	3.8	31	Light Industry

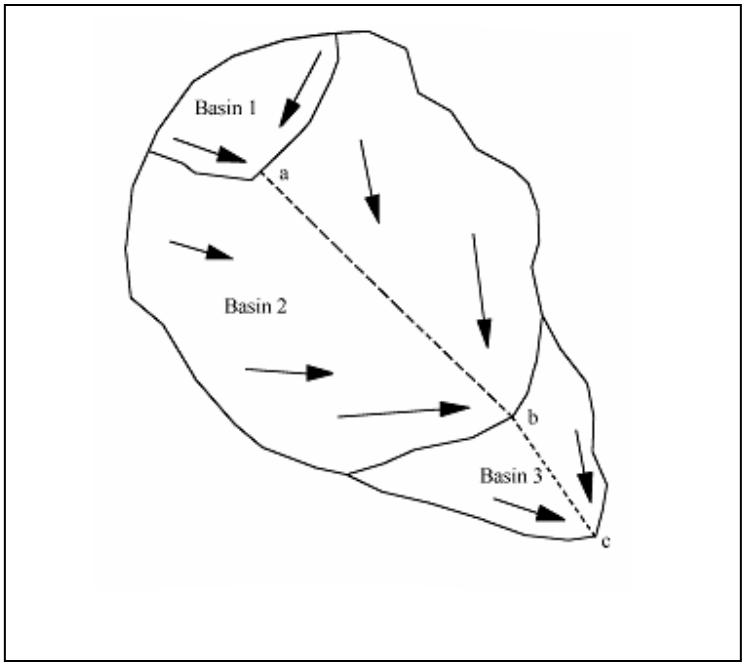


Figure 3.2.1
Hypothetical Watershed for Example 3.2.1

Step 1: The areas of each subbasin are given in the above table.

Step 2: Calculate the runoff coefficients. Obtain the runoff coefficients from Tables 3.2.1 - 3.2.2. Basin 1 (Apartment): $C_1 = 0.60$, Basin 2 (Parks, cemeteries): $C_2 = 0.15$, Basin 3 (Industrial, light): $C_3 = 0.65$.

Using Eq. 3.2.4, compute composite C values at points "a", "b" and "c":

Point "a": $C_{comp} = (0.60*2.4)/2.4 = 0.60$
Point "b": $C_{comp} = (0.60*2.4+0.15*13.7)/(2.4+13.7) = 0.22$
Point "c": $C_{comp} = (0.60*2.4+0.15*13.7+0.65*3.8)/(2.4+13.7+3.8) = 0.30$

Step 3: Calculate the time of concentration for points "a", "b", and "c".

Point "a": $t_c = 10$ *min*

Point "b": Compare (Basin 1 travel time + pipe flow time from "a" to "b") to Basin 2 travel time.

Using Eq. 3.2.3, pipe flow time, t, from "a" to "b" is 600/3 * 1/60= 3.33 minutes.

Basin 1 travel time + pipe flow time$_{a-b}$ = 10 + 3.33 = 13.33 minutes
Basin 2 travel time = 37 minutes $t_c = 37$ *minutes*

Point "c": Compare (the time of concentration at point "b" + pipe flow time from "b" to "c") to Basin 3 travel time.

Using Eq. 3.2.3, pipe flow time from "b" to "c" is 90/3 * 1/60 = 0.5 minutes.
t_c at "b" + pipe flow time$_{b-c}$ = 37 + 0.5 = 37.5 minutes
Basin 3 travel time = 31 minutes $t_c = 37.5$ *minutes*

Step 4: Calculate the rainfall intensities for points "a", "b",and "c". Recall that the time of the storm duration will be set equal to the t_c for each subbasin. Since the t_c is measured in minutes, it must be converted to hours.

Using Equation 2.2.13

$$i = \frac{c\,T_r^{\alpha}}{(t+d)^{\beta}}$$

Where : c=1.7204 α=0.1753 d=0.485 β=1.6806,
From Table 2.2.2 (South Bend for 5 min < t ≤ 60 min)

Point "a":

$$i = \frac{c\,T_r^{\alpha}}{(t+d)^{\beta}} = \frac{1.7204 \bullet (10)^{0.1753}}{(\frac{10}{60}+0.485)^{1.6806}} = 5.29\ inches/hr$$

Point "b":

$$i = \frac{c\,T_r^{\alpha}}{(t+d)^{\beta}} = \frac{1.7204 * 10^{0.1753}}{(\frac{37}{60}+0.485)^{1.6806}} = 2.19\ inches/hr$$

Point "c":

$$i = \frac{c\,T_r^{\alpha}}{(t+d)^{\beta}} = \frac{1.7204 * 10^{0.1753}}{(\frac{37.5}{60}+0.485)^{1.6806}} = 2.16\ inches/hr$$

Step 5: Calculate the flowrates at points "a", "b", and "c":

Point "a": Q= C *i* A = 0.60 * 5.29 * 2.4 = *7.62 cfs*
Point "b": Q= C *i* A = 0.22 * 2.19 * (2.4 + 13.7) = *7.76 cfs*
Point "c": Q= C *i* A = 0.30 * 2.16 * (2.4 + 13.7 + 3.8) = *12.90 cfs*

3.3 - NATIONAL RESOURCES CONSERVATION SERVICE CURVE NUMBER METHOD

The National Resources Conservation Service (NRCS) Curve Number (CN) Method was developed to determine the quantity of runoff from a given amount of precipitation (NEH-4, 2004). The CN method uses basin soil and cover types, rainfall depth, and the antecedent moisture condition to predict the runoff volume. The method is relatively easy to use and has wide applicability. It has been recommended for both rural and urban watersheds. Although there are no limits on the watershed area to which it can be applied, the area must be small enough so that the rainfall intensity is uniform. If the soil type varies within the basin, the basin should be divided into the sub-basins of similar character or a composite CN should be calculated.

3.3.1 - Theory of the Curve Number Method

The basic principle of the curve number method can be described in conjunction with Figure 3.3.1, which is a graph of quantity versus time. The upper line represents the total rainfall P(t) as a function of time. This is assumed to be a linear function for convenience. The dashed line concave to the horizontal axis represents the total abstraction or loss, L(t), as a function of time. The total abstraction is the sum of the initial abstraction, I_a, due mainly to surface wetting, depression storage, and the total infiltration, F(t).

Figure 3.3.1 is a diagram of accumulated rainfall, runoff and infiltration occurring during the storm being studied. It is assumed that no runoff occurs until I_a has been satisfied and that after a sufficiently long period of time, F(t) reaches a constant saturation value denoted by S-I_a. The symbol, S, stands for the ultimate total abstraction and depends on soil type, cover, and antecedent moisture condition.

The cumulative runoff, R(t), is the difference between the total rainfall and the total abstraction. The accumulated runoff is determined by Equation 3.3.1:

$$R(t) = \frac{(P(t) - 0.2\,S)^2}{P(t) + 0.8\,S} \qquad (3.3.1)$$

Where P(t) is the cumulative rainfall (inches).

In Equation 3.3.1, if $P(t) \leq 0.2S$; then R(t) is equal to zero.

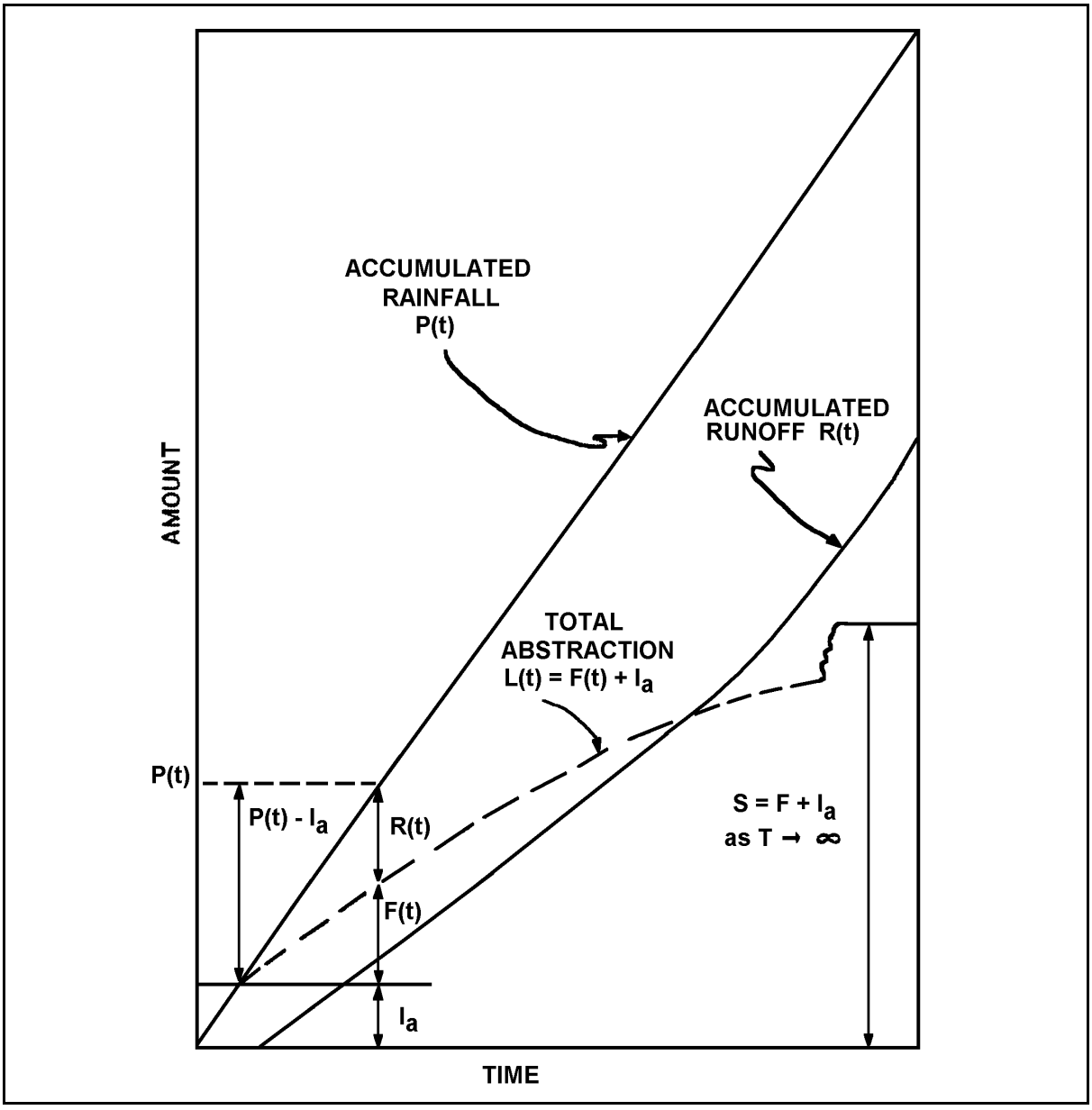


Figure 3.3.1
Diagram of Accumulated Rainfall, Runoff and Infiltration (SCS; NEH-4)

3.3.2 - Determination of the Parameter S

In the determination of the ultimate abstraction S, the soil characteristics are considered. Soils have been classified by the SCS into four groups based on infiltration capacity. The criteria for each soil group are described in Table 3.3.1. Soils common in the United States have been analyzed and placed in the appropriate category based on Table 3.3.1. Soils common to Indiana are given in Table 3.3.2. Soils at a particular site can be determined by using county soil maps distributed by the regional Soil Conservation Service office.

The antecedent moisture condition, AMC, which is an indication of the wetness of the basin under consideration, used to be a factor in the design. However, the latest SCS publications now refer to an average runoff condition, ARC, which is AMC II.

Table 3.3.1
Criteria Used by the Soil Conservation Service in the
Classification of Soils - National Engineering Handbook
Hydrology: Section 4 (SCS; NEH-4)

Soil Group	Characteristics
A	Soils in this category have a high infiltration rate even when thoroughly wetted and consist mainly of deep, well-to excessively-drained sands or gravels. (Lowest runoff potential).
B	Soils in this category have moderate infiltration rates when thoroughly wetted and consist of moderately deep to deep, moderately well to well-drained soils with moderately fine to moderately coarse textures.
C	Soils in this category have slow infiltration rates when thoroughly wetted and consist mainly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine textures.
D	Soils in this category have a very slow infiltration rate then thoroughly wetted and consist mainly of clay soils with high swelling potential, soils with a permanently high water table, soils with a clay or clay layer at or near the surface, and shallow water over nearly impervious material. (Highest runoff potential)

Once the soil classification is known, a curve number CN can be found given the type of cover in the watershed. The relationship between S and the curve number is:

$$S = \left(\frac{1000}{CN}\right) - 10 \qquad (3.3.2)$$

Table 3.3.2 and Table 3.3.3 may be used to determine the appropriate CN for the particular cover and soil type in the watershed. Higher curve numbers occur with higher impervious covers and with soils with slower infiltration rates. These numbers, when used with Equation 3.3.2 are consistent with the physical process. If a CN of 100 is used for an ideal impervious cover, the runoff predicated by Equation 3.3.1 is equal to the rainfall. The CN values presented in Tables 3.3.2 and 3.3.3 are for the ARC (AMC II).

3.3.3 - Application of the CN Method

Once a CN has been determined, a value of S may be calculated by using Equation 3.3.2. By selecting the rainfall depth for a particular storm, Equation 3.3.1 may be solved for runoff R(t). An alterative to using Equation 3.3.1 is the graphical solution presented in Figure 3.3.2. The units of R(t) are inches. By multiplying R by the basin area and converting units, volume of runoff may be determined for the watershed under consideration.

For a basin which has different soil types and/or land uses, a composite or weighted CN may be determined by Equation 3.3.3.

$$CN_{comp} = \frac{(CN_1 A_1 + CN_2 A_2 + \ldots CN_n A_n)}{A_t} \qquad (3.3.3)$$

Where CN_1, CN_2...CN_n are the curve numbers associated with component areas A_1, A_2,... A_n and $A_t = A_1 + A_2 + \ldots + A_n$.

Table 3.3.2
Hydrologic Soil Groups For Indiana Soils
(Shown in parentheses after the soil classification)

Abscota (A)
Ackerman (A/D)
Ade (A)
Adrian (A/D)
Alford (B)
Algansee (B)
Algiers (C/D)
Alida (B)
Allison (B)
Alvin (B)
Ambraw (B/D)
Andres (B)
Aptakisic (B)
Armiesburg (B)
Ashkum (B/D)
Atkins (D)
Aubbeenaubbee (B)
Ava (C)
Avonburg (D)
Ayr (B)
Ayrmount (B)
Ayrshire (C)

Banlic (C)
Barce (B)
Barry (B/D)
Bartle (D)
Battleground (B)
Baxter (B)
Beanblossom (B)
Beasley (C)
Beaucoup (B/D)
Beckville (B)
Bedford (C)
Beecher (C)
Belknap (C)
Belleville (B/D)
Belmore (B)
Berks (C)
Bethesda (C)
Bewleyville (B)
Billett (B)
Birds (C/D)
Birkbeck (B)

Bloomfield (A)
Blount (C)
Bobtown (C)
Bonnell (C)
Bonnie (C/D)
Bono (D)
Booker (D)
Boonesboro (B)
Boots (A/D)
Bourbon (B)
Bowes (B)
Boyer (B)
Brady (B)
Branch (B)
Brems (A)
Brenton (B)
Bromer (C)
Bronson (B)
Brookston (B/D)
Bruno (A)
Bryce (D)
Burside (B)

Cadiz(B)
Camden (B)
Cana (C)
Caneyville (C)
Carlisle (A/D)
Carmel (C)
Carmi (B)
Casco (B)
Catlin (B)
Celina (C)
Ceresco (B)
Chagrin (B)
Chalmers (B/D)
Chatterton (A)
Cheektowaga (D)
Chelsea (A)
Chetwynd (B)
Cincinnati (C)
Clarence (D)
Cleremont (D)
Clyde (B/D)
Cobbsford (D)

Coesse (C/D)
Cohoctah (B/D)
Coloma (A)
Colyer (D)
Comfrey (B/D)
Conotton (B)
Conover (C)
Conrad (A/D)
Coolville (C)
Corwin (B)
Cory (C)
Corydon (D)
Coupee (B)
Craigmile (B/D)
Crane (B)
Crawleyville (B)
Crider (B)
Crosby (C)
Crosier (C)
Cuba (B)
Cyclone (B/D)

Dana (B)
Darroch (B)
Darroch, bedrock subsratum (C)
Dearborn (B)
Del Rey (C)
Deputy (C)
Derinda (C)
Desker (A)
Dickinson (B)
Digby (B)
Door (B)
Dowagiac (B)
Driftwood (C/D)
Drummer (B/D)
Du Page (B)
Dubois (C)
Dunning (D)

Ebal (B)
Eden (C)

Edenton C)
Edwards (B/D)
Eel (B)
Eldean (B)
Elkinsville (B)
Elliott (C)
Elston (B)
Evansville (B/D)

Fabius (B)
Fairmount (D)
Fairpoint (C)
Farmington (C)
Faxon (B/D)
Fincastle (C)
Flanagan (B)
Foresman (B)
Fox (B)
Frederick (B)
Free (B/D)
Fulton (D)

Genesee (B)
Gessie (B)
Gilboa (B)
Gilford (B/D)

Gilford, stratified substratum (D)
Gilpin (C)
Ginat (D)
Glenhall (B)
Glynwood (C)
Gosport (C)
Granby (A/D)
Gravelton (B/D)
Grayford (B)
Griswold (B)
Grovecity (B)
Gudgel (C)
Guthrie (D)

Hagerstown (C)
Haney (B)
Hanna (B)
Harpster (B/D)
Haskins (C)
Haubstadt (C)
Haymond (B)
Hennepin (B)
Henshaw (C)
Hickory (C)
High Gap (C)
Hillsdale (B)
Holton (C)
Homer (B)
Hononegah (A)
Hoopeston (B)
Hoosierville (C)
Hosmer (C)
Houghton (A/D)
Hoytville (C/D)
Huntington (B)
Huntsville (B)

Iona (B)
Ipava (B)
Iroquois (B/D)
Iva (C)

Jasper (B)
Jennings (C)
Johnsburg (D)
Jules (B)
Junius (C)

Kalamazoo (B)
Kendallville (B)
Kentland (A/D)
Kerston (A/D)
Kibbie (B)
Kings (D)
Kokomo (B/D)
Kosciusko (B)

Kurtz (C)

Lafayette (B)
La Hogue (B)
Landes (B)
Lash (B)
Laramie (B)
Lawrence (C)
Lenawee (B/D)
Lindside (C)
Linkville (C)
Linwood (A/D)
Lisbon (B)
Lobdell (B)
Lomax (B)
Longlois (B)
Losantville (C)
Lucas (D)
Lydick (B)
Lyles (B/D)

Mahalasville (B/D)
Maplehille (C)
Marker (B)
Markham (C)
Markland (C)
Markton (C)
Martinsville (B)
Martisco (B/D)
Maumee (A/D)
McGary (C)
Medora (B)
Medway (B)
Mellott (B)
Mermill (B/D)
Metamora (B)
Metea (B)
Miami (B)
Miamian (C)
Middlebury (B)
Milford (B/D)
Millbrook (B)
Millgrove (B/D)

Notes: 1. Two hydrologic soil groups shown together, such as B/C, indicate the drained/undrained conditions.
2. Any modifiers shown, e.g., bedrock substratum, refer to a specific soil series phase found in the soil map legend.

Table 3.3.2
Hydrologic Soil Groups For Indiana Soils
(Shown in parentheses after the soil classification)

Millsdale (B/D)
Milton (C)
Monitor (C)
Montgomery (D)
Montmorenci (B)
Morley (C)
Morocco (B)
Moundhaven (A)
Muldavia (B)
Mulvey (B)
Mundelein (B)
Muren (B)
Muskego (A/D)
Muskingum (C)
Mussey (B/D)

Napoleon (A/D)
Nappanee (D)
Negley (B)
Nesius (A)
Newark (C)
Newglarus (B)
Newton (A/D)
Nicholson (C)
Nineveh (B)
Nolin (B)

Oakville (A)
Ockley (B)
Octagon (B)
Odell (B)
Oldenburg (B)
Onarga (B)
Ormas (B)
Orrville (C)
Oshtemo (B)
Otwell (C)
Ouiateno (A)
Owosso (B)

Palms (A/D)
Papineau (C)
Parke (B)
Parr (B)
Pate (C)
Patton (B/D)
Pekin (C)
Pella (B/D)
Peoga (C)
Peotone (B/D)
Petrolia (B/D)
Pewamo (C/D)
Philo (B)
Piankeshaw (B)
Pike (B)
Pinevillage (B)
Pinhook (B/D)
Piopolis (C/D)
Pipestone (B)
Plainfield (A)
Plano (B)
Pope (B)
Princeton (A)
Prochaska (A/D)
Proctor (B)

Quinn (B/D)

Ragsdale (B/D)
Ragsdale, overwash (B)
Rahm (C)
Rainsville (B)
Randolph (C)
Rarden (C)
Raub (C)
Rawson (B)
Reddick (B/D)
Reesville (C)
Rensselaer (B/D)
Rensselaer, nonstratified substratum (C)
Richardsville (B)
Riddles (B)
Ridgeville (B)
Rimer (C)
Riverdale (A)
Roby (C)
Rockcastle (D)
Rockfield (B)
Rockton (B)
Rodman (A)
Ross (B)
Rossmoyne (C)
Ruark (B/D)
Rush (B)
Russell (B)
Ryker (B)

Sable (B/D)
Saranac (C/D)

Saranac, gravelly substratum (C)
Saugatuck (C)
Sawabash (B)
Sciotoville (C)
Seafield (B)
Sebewa (B/D)
Selfridge (B)
Selma (B/D)
Seward (B)
Shadeland (C)
Shakamak (C)
Shipshe (B)
Shoals (C)
Sidell (B)
Simonin (B)
Sisson (B)
Skelton (B)
Sleeth (C)
Sloan (B/D)
Sparta (A)
Spinks (A)
St. Charles (B)
St. Clair (D)
Starks (C)
Steff (C)
Stendal (C)
Stockland (B)
Stonelick (B)
Stoy (C)
Strawn (B)
Strole (C)
Suman (B/D)
Sumava (B)
Sunbury (B)
Swanwick (D)
Switzerland (B)
Swygert (C)
Sylvan (B)
Symerton (B)

Taftown (B)
Taggart (C)
Tama (B)
Tawas (A/D)
Tecumseh (B)
Tedrow (B)
Thackery (B)
Throckmorton (B)
Tice (B)
Tilsit (C)
Tippecanoe (B)
Toledo (D)
Toronto (C)
Toto (B/D)
Tracy (B)
Trappist (C)
Treaty (B/D)
Trevlac (B)
Troxel (B)
Tuscola (B)
Tyner (A)

Uniontown (B)

Varna (C)
Vigo (D)
Vincennes (C/D)
Volinia (B)

Wabash (D)
Wakeland (C)
Wallkill (C/D)
Warners (C/D)
Warsaw (B)
Wasepi (B)
Washtenaw (C/D)
Watseka (B)
Waupecan (B)
Wauseon (B/D)
Wawasee (B)
Waynetown (C)
Wea (B)
Weikert (C/D)
Weinbach (C)
Weisburg (C)
Wellston (B)
Westland (B/D)
Wheeling (B)
Whitaker (C)
Whitson (D)
Wilbur (B)
Wilhite (C/D)
Willette (A/D)
Williamsport (C)
Williamstown (C)
Wingate (B)
Wirt (B)
Wolcott (B/D)
Woodmere (B)
Woolper (C)
Wynn (B)

Xenia (B)

Zaborosky (B)
Zadog (A/D)
Zanesville (C)
Zipp (D)

Notes: 1. Two hydrologic soil groups shown together, such as B/C, indicate the drained/undrained conditions.
2. Any modifiers shown, e.g., bedrock substratum, refer to a specific soil series phase found in the soil map legend.

Table 3.3.3
Runoff Curve Numbers for Urban Areas (USDA, 1986)

Cover Type and Hydrologic Condition		Curve Numbers for Hydrologic Soil Group			
		A	B	C	D
Undeveloped Areas					
Cultivated Land					
Without conservation treatment		72	81	88	91
With conservation treatment		62	71	78	81
Pasture or range land					
Poor condition		68	79	86	89
Good condition		39	61	74	80
Meadow					
Good condition		30	58	71	78
Wood or forest land					
Thin stand, poor cover, no mulch		45	66	77	83
Good cover		25	55	70	77
Fully developed urban areas (with established vegetation)					
Open space (lawns, parks, golf courses, cemeteries)					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved : curb and storm sewers (excluding right-of-way)		98	98	98	98
Gravel		76	85	89	91
Dirt		72	82	87	89
Urban Districts					
Commercial and business	(85% impervious)	89	92	94	95
Industrial (72% impervious)		81	88	91	93
Residential					
1/8 acre or less, townhouses	(65% impervious)	77	85	90	92
1/4 acre	(38% impervious)	61	75	83	87
1/3 acre	(30% impervious)	57	72	81	86
1/2 acre	(25% impervious)	54	70	80	85
1 acre	(20% impervious)	51	68	79	84
2 acre		46	65	77	82
Developing Urban Areas					
Newly graded areas (no vegetation)		77	86	91	94

Table 3.3.4
Runoff Curve Numbers for Agricultural Lands (USDA, 1986)

Cover Type and Hydrologic Condition	Curve Numbers for Hydrologic Soil Group			
	A	B	C	D
Pasture, grassland, or range with continuous forage for grazing				
Poor	68	79	86	89
Fair	49	69	79	84
Good	39	61	74	80
Meadow with continuous grass, protected from grazing and generally mowed for hay	30	58	71	78
Brush/brush-weed-grass mixture with brush being the major element				
Poor	48	67	77	83
Fair	35	56	70	77
Good	30	48	65	73
Woods and grass combination (orchard or tree farm)				
Poor	57	73	82	86
Fair	43	65	76	82
Good	32	58	72	79
Woods				
Poor	45	66	77	83
Fair	36	60	73	79
Good	30	55	70	77
Farmsteads	59	74	82	86

The curve number method may also be used in determining the time distribution of the runoff. In this manual, the CN method is used in conjunction with the synthetic dimensionless and triangular unit hydrograph methods to determine the storm hydrograph. The procedure used in this operation is outlined below.

1. Determine the basin curve number.

2. Given the rainfall depth and storm duration, determine the time distribution of the rainfall. This distribution can be the SCS Type II or Huff Distributions discussed in Chapter 2.

3. Divide the storm duration into convenient intervals and determine the rainfall depth at the start and end of each interval.

4. Determine the ultimate abstraction S by using Equation 3.3.2.

5. Using Equation 3.3.1 or Figure 3.3.2, determine the runoff for the beginning and end of the time intervals.

6. The difference between successive runoff values is the incremental runoff. The incremental runoff may then be used as the excess rainfall needed as input in hydrograph calculations.

The curve number method was developed for agricultural watersheds. When applying the method to urban watersheds, discrepancies are observed for low AMC's when compared to other methods. (Burke and Gray, 1979).

These discrepancies arise because runoff begins too slowly in urbanized basins if a composite CN is used. To overcome this discrepancy, *the runoff should be calculated separately for the pervious and impervious areas and the resultant hydrographs added together.*

The National Resources Conservation Service (NRCS) has developed two computer programs and the associated documentation to apply the CN method. The first model is the TR-20 program (SCS, 1982). This program was developed to generate runoff and simulate the movement of water through channels or reservoirs. TR-20 has been widely used for planning and design of water resource projects. It can be used to investigate the effects of varying storm durations, rainfall distributions, unit hydrograph, and AMC values. The TR-20 program is discussed in detail in Chapter 8.

The program TR-55 (SCS, 1986) provides users with a simplified procedure to evaluate the impacts of urbanization. In 1986, a computer version of TR-55 was published by the NRCS which updated the CN methodologies and incorporated the latest research in the field. However, the program does not allow for the routing of flows and uses only the SCS Types I, II and III rainfall distributions.

The U.S. Army Corps of Engineers (COE) has developed HEC-1, a program similar to TR-20 (COE, 1987). Like TR-20, many of the inputs can be varied and channel and reservoir routings can be performed. HEC-1 also allows for the variation of the loss rate which is useful in model calibration and verification. This program is also discussed in detail in Chapter 8.

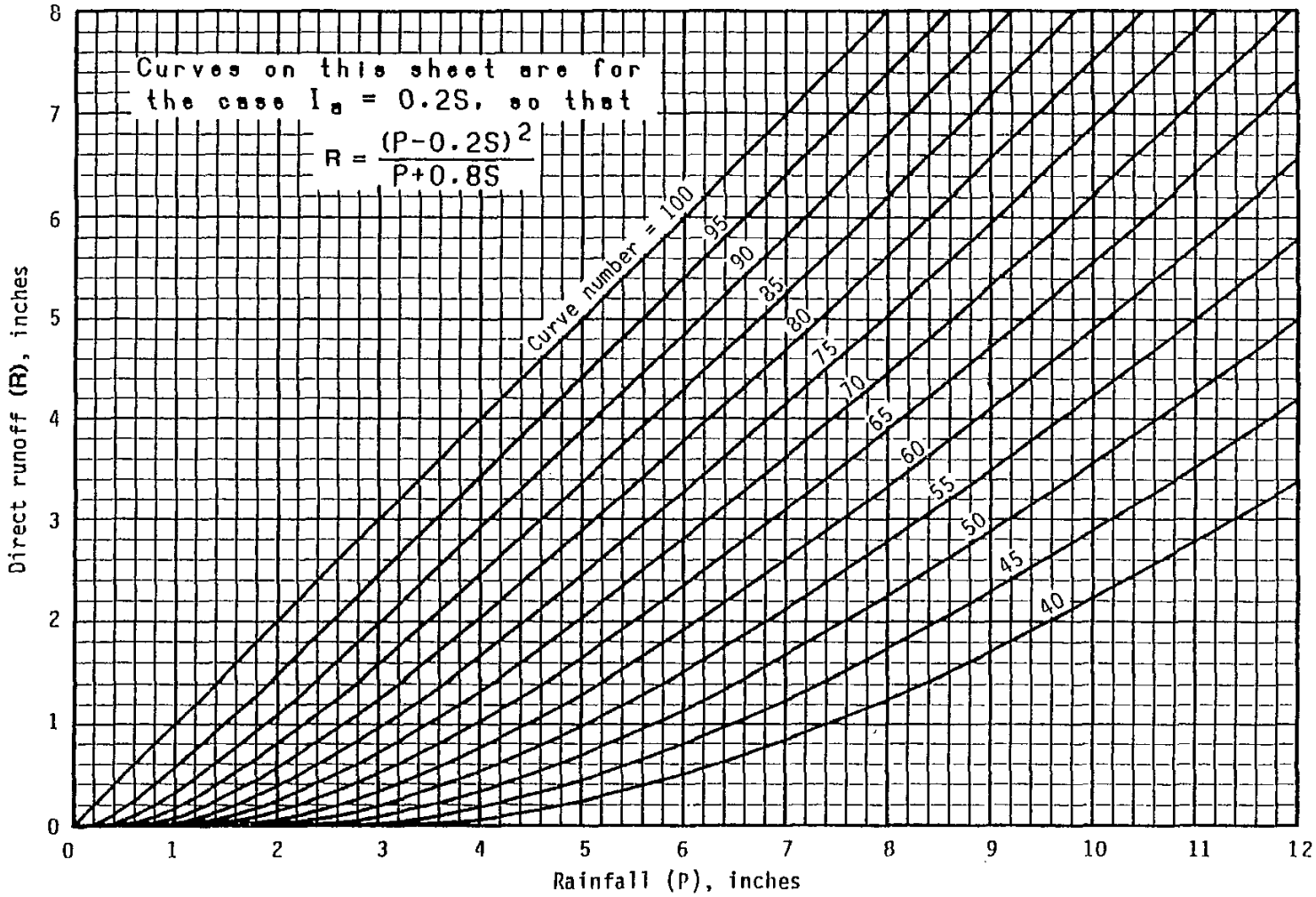


Figure 3.3.2
Graphical Solution of Equation 3.3.1 (SCS, 1986)

Example 3.3.1

This example illustrates the use of the NRCS Curve Number Method.

Determine the volume of runoff for a 50-year 2-hour storm in Indianapolis, Indiana with the following characteristics and antecedent moisture condition II.

Basin characteristics:

Subbasin	Soil Type	Practice	Condition	Area (acres)
1	Carmel	Cultivated Land w/ conservation treatment	Poor	55
2	Ockley	Meadow	Good	40
3	Hosmer	Residential; 1/8 acre	---------	25
4	Muskingum	Forest; good cover	Good	90

Step 1: Determine the soil type using Table 3.3.2 and the curve numbers using Table 3.3.3 to develop a composite curve number for the basin using Equation 3.3.3.

Subbasin	Soil Classification	CN
1	C	88
2	B	58
3	C	90
4	C	70

$$CN_{COMP} = \frac{55(88) + 40(58) + 25(90) + 90(70)}{210} = 74.81$$

Step 2: Determine the ultimate abstraction, S, using Equation 3.3.2.

$$S = \left(\frac{1000}{CN}\right) - 10 = \left(\frac{1000}{74.81}\right) - 10 = 3.37 \text{ inches}$$

Step 3: Determine the total rainfall for a 50-year return period and a 2-hour duration for Indianapolis, Indiana. From Equation 2.2.16 on page 2-30, the corresponding rainfall depth can be found.

$$i_2^{50} = \frac{56.34 \log(4.11(50)^{0.386})}{(60(2) + 9.2)^{0.78}} = 1.61 \text{ inches/hr}$$

For a duration of 2-hours the depth is 3.22 inches.

Step 4: Determine the cumulative runoff R(t) using Equation 3.3.1 or Figure 3.3.2.

$$R(t) = \frac{(P(t) - 0.2S)^2}{P(t) + 0.8S} = \frac{(3.22 - 0.2(3.37))^2}{3.22 + 0.8(3.37)} = 1.096 \text{ inches}$$

Step 5: Find the total runoff volume in cubic feet (ft^3).

$$1.096 \text{ inches} \bullet \frac{1 \text{ foot}}{12 \text{ inches}} \bullet 210 \text{ acres} \bullet \frac{43{,}560 \text{ } ft^2}{1 \text{ acre}} = 835{,}246 \text{ } ft^3$$

3.4 - HYDROGRAPHS

The physical processes of runoff and the factors which reduce or delay the amount of rainfall that becomes runoff, were discussed in Section 3.1.1. In the design of hydraulic structures, it is often necessary to know the time distribution of runoff as well as the peak magnitude. A hydrograph, which is simply a graph of flow versus time, enables both of these values to be determined.

A hydrograph may have four components: direct surface runoff, interflow, groundwater or base flow and channel precipitation. The initial rising portion of the hydrograph is termed as the rising limb, the portion in the vicinity of the peak is called the crest segment, and the falling

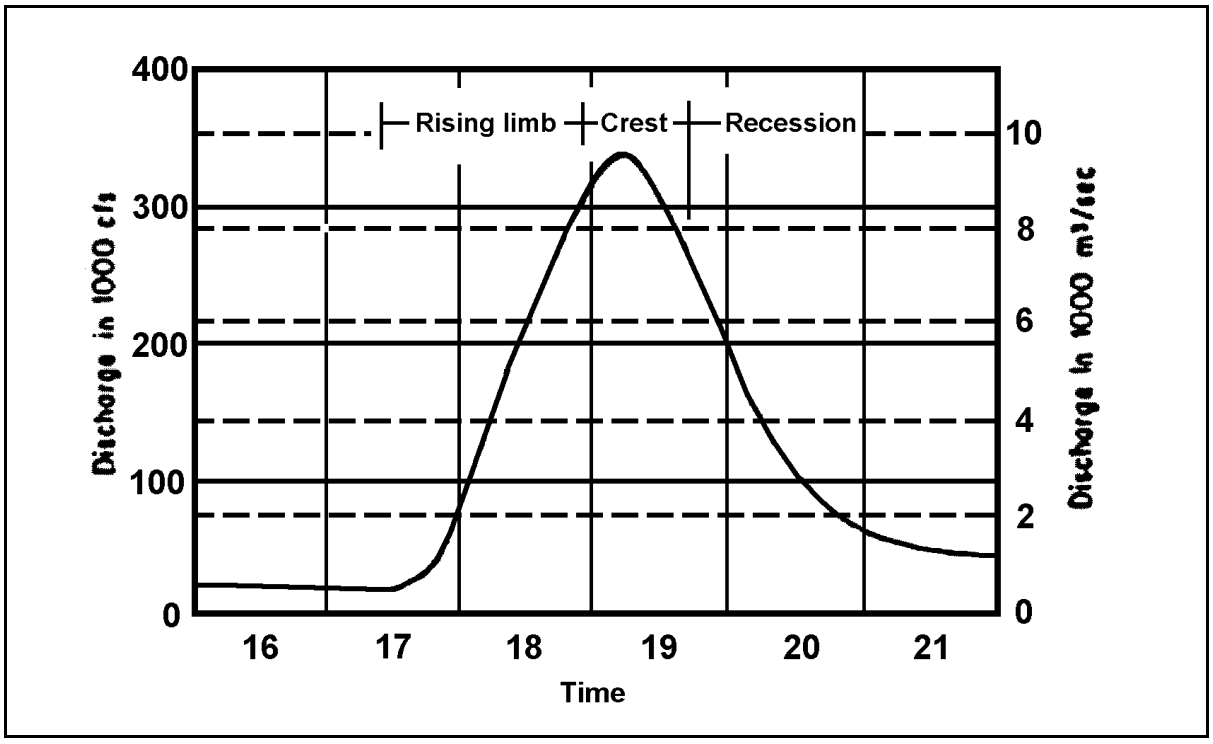


Figure 3.4.1
Definition of Hydrograph Terms (Linsley et al., 1975)

limb is termed the recession curve (Figure 3.4.1). The shape of a hydrograph depends on the precipitation and the basin characteristics. (Linsley et al., 1975)

If rainfall remains at a constant intensity for a sufficiently long period of time, a state of equilibrium is reached. This point represents the time when the rate of rainfall equals the rate of runoff. In Figure 3.4.2, two curves are shown. The inflection point on curve A represents the time when the entire drainage basin is contributing to the flow. At this inflection point, the maximum storage capability of the basin is only partially filled. As the storm progresses, the

state of equilibrium is reached and the curve levels off. In nature, the state of equilibrium is seldom attained. This is due to different factors associated with the runoff process and the fact that rarely will the rain fall with a constant intensity for a long period of time. Curve B represents a single-peaked hydrograph resulting from a finite rain of duration t_i.

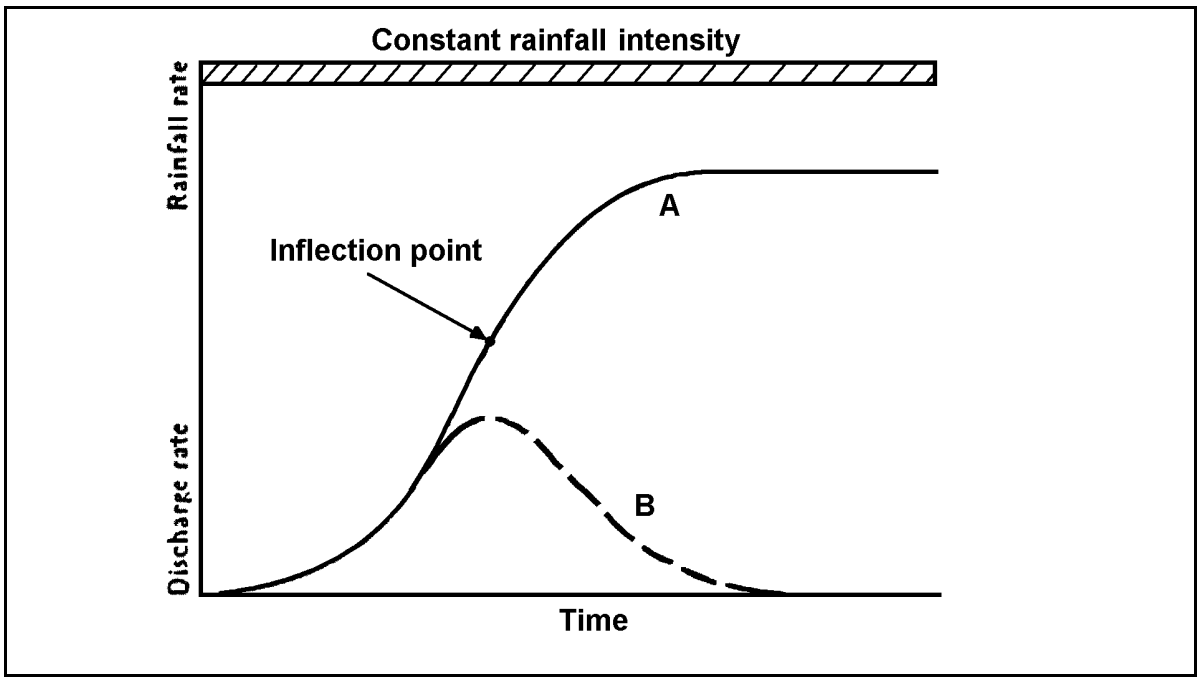


Figure 3.4.2
Equilibrium Discharge Hydrograph (Viessman et al., 1989)

There are four basic types of hydrographs (SCS; NEH-4):

a. Natural Hydrograph: Obtained directly from the flow records of a gauged stream. These hydrographs can be used to calibrate computer models for both the peak flowrate and volume.
b. Unit Hydrograph: A natural or synthetic hydrograph for one inch of direct runoff which is the amount of rain which becomes runoff after all losses and infiltration have been satisfied. The rainfall is assumed to occur uniformly over a watershed in a specified time.
c. Dimensionless Unit Hydrograph: Represents many unit hydrographs by using the time to peak and the peak rates as basic units and plotting the hydrographs in ratios of these units. This is also called the Index Hydrograph.
d. Synthetic Unit Hydrograph: A generalized unit hydrograph which contains adjustable parameters enabling it to be used for many watersheds.

The theory of the unit hydrograph and two synthetic hydrographs, the SCS Dimensionless Unit Hydrograph and the SCS Triangular Unit Hydrograph, are discussed in the following sections.

3.4.1 - The Unit Hydrograph

The techniques developed in this chapter provide a relationship between the rainfall input and the runoff so that hydrographs can be estimated for a particular basin with various temporal rainfall distributions. The unit hydrograph is defined as the hydrograph of direct runoff resulting from one inch of effective rainfall uniformly distributed in space and time over the watershed for a particular excess rainfall duration. The result is a hydrograph which is associated with a particular basin of constant physical conditions. *Direct Runoff* is the portion of the rainfall which finds its way to a channel and eventually reaches the point under consideration not long after the storm. This would include overland flow, interflow and groundwater. *Excess Rainfall* is the rainfall that actually contributes to direct runoff. Consequently, losses such as infiltration, evaporation and temporary or permanent storage are subtracted from the total rainfall.

The important concept behind the unit hydrograph theory is that the volume of excess rainfall equals the volume of runoff. The volume of runoff is the area under the hydrograph while the volume of excess rainfall is the depth times the area on which it falls. The depth in the unit hydrograph is one inch. In other words, the area under a hydrograph is equivalent to one inch of excess rainfall.

Once a unit hydrograph is computed, a rainfall depth corresponding to a storm of the same duration as the unit hydrograph may be developed. For example, a ten-minute unit hydrograph would be calculated on the depth of rainfall occurring in ten minutes. That depth is then multiplied by the unit hydrograph coordinates to develop the storm hydrograph associated with that rainfall depth. Two or more hydrographs, representing successive rainfall increments for a given time interval during the same storm, may be superimposed to determine the resulting storm hydrograph.

The unit hydrograph concept assumes that the volume of runoff is constant regardless of the temporal distribution. In nature, for the same rainfall depth, an increase in the precipitation duration lengthens the base and lowers the peak of the hydrograph. Normally this phenomenon changes the unit hydrograph only slightly. These changes are assumed to be negligible.

The unit hydrograph may be computed in several ways. The historical record of discharges may be analyzed to determine the unit hydrograph if a long streamflow record is available. Another method is to use synthetic unit hydrographs which have been developed by analyzing hydrographs from many basins.

3.4.2 - SCS Synthetic Unit Hydrographs

Two synthetic unit hydrographs are discussed in this section: the dimensionless unit hydrograph and the triangular unit hydrograph. Both of these incorporate empirical equations which relate the peak flow rate and the time to peak to basin parameters. These unit hydrographs were developed by the Soil Conservation Service and are based on the Curve Number Method in the

calculation of excess precipitation.

The dimensionless unit hydrograph shown in Figure 3.4.3 was developed by analyzing data from a large number of basins with varying geographic locations. Dimensionless ratios of q/q_p on the ordinate and t/t_p on the abscissa where q_p and t_p are the peak flow and time to peak, respectively, are used to specify the hydrograph. Table 3.4.1 lists the coordinates of the hydrograph as well as the cumulative mass curve which is also shown in Figure 3.4.3. 37.5% of the total runoff volume accumulates in the rising portion of the hydrograph.

The unit hydrograph may be represented as a triangle. Both the triangular unit hydrograph and curvilinear unit hydrograph are shown in Figure 3.4.4. If we say that t_P (the time to peak) is "one time unit" we can determine t_b, the base time of the triangular hydrograph, to be 2.67 t_p. It follows that the recession duration t_r is equal to 1.67 t_p.

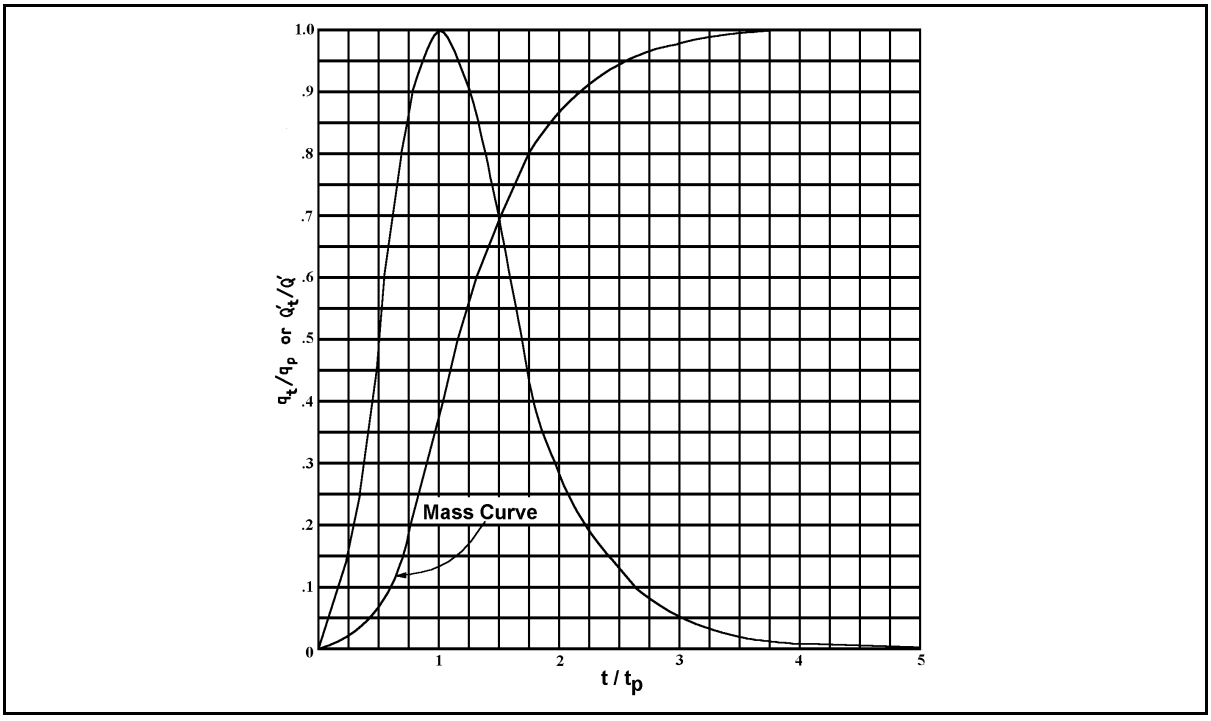


Figure 3.4.3
Dimensionless Unit Hydrograph and Mass Curve (SCS; NEH-4)

Table 3.4.1
Ratios for Dimensionless Unit Hydrograph
and Mass Curve (SCS; NEH-4)

Time Ratios	Discharge Ratios	Mass Curve Ratios
0.0	.000	.000
0.1	.030	.001
0.2	.100	.006
0.3	.190	.012
0.4	.310	.035
0.5	.470	.065
0.6	.660	.107
0.7	.820	.163
0.8	.930	.228
0.9	.990	.300
1.0	1.000	.375
1.1	.990	.450
1.2	.930	.522
1.3	.860	.589
1.4	.780	.650
1.5	.680	.700
1.6	.560	.751
1.7	.460	.790
1.8	.390	.822
1.9	.330	.849
2.0	.280	.871
2.2	.207	.908
2.4	.147	.934
2.6	.107	.953
2.8	.077	.967
3.0	.055	.977
3.2	.040	.984
3.4	.029	.989
3.6	.021	.993
3.8	.015	.995
4.0	.011	.997
4.5	.005	.999
5.0	.000	1.000

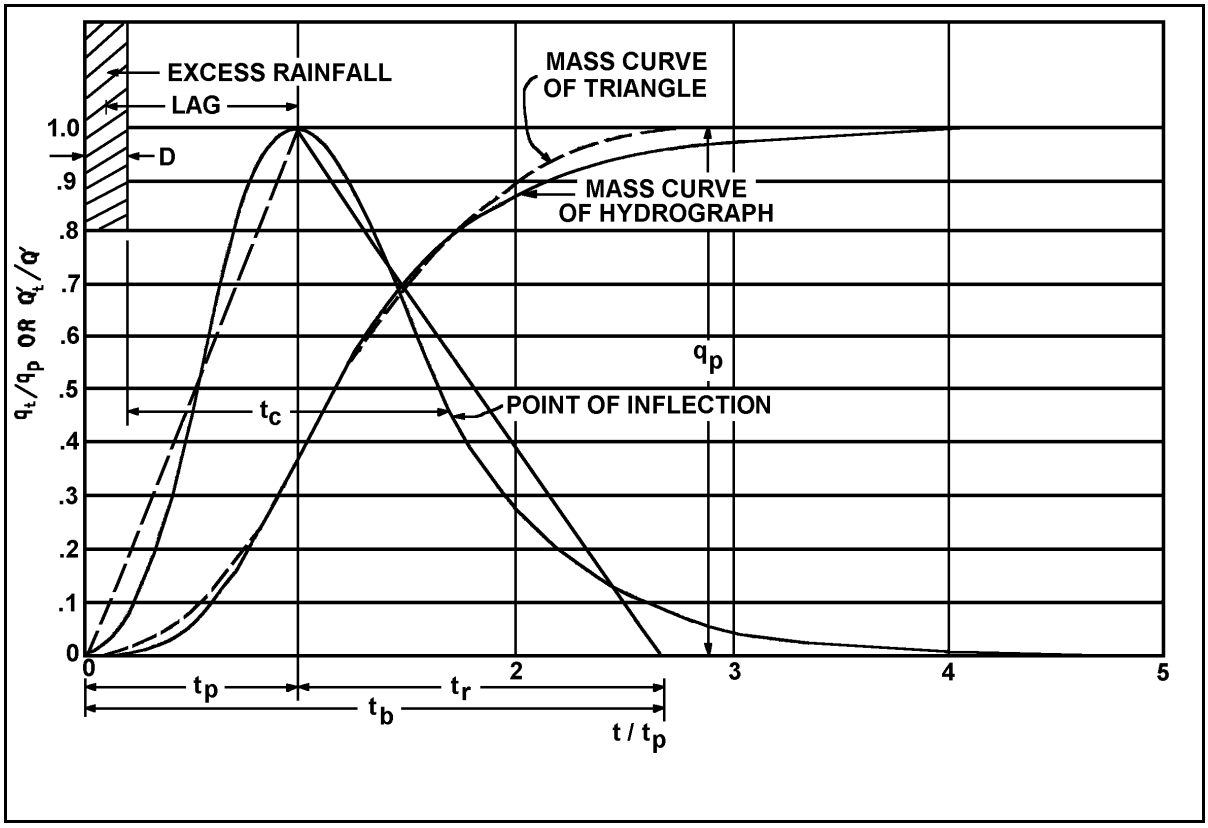


Figure 3.4.4
Dimensionless Curvlinear Unit Hydrograph and
Equivalent Triangular Hydrograph (SCS; NEH-4)

Other relationships associated with the unit hydrograph are as follows. The total volume of the triangular unit hydrograph Q´ in Figure 3.4.4 may be expressed as:

$$V_t = Q' = \frac{t_p\, q_p}{2} + \frac{t_r\, q_p}{2} \qquad (3.4.1)$$

where Q´ is in inches, t is in hours, and q_p has units of inches/hour. Equation 3.4.1 can be manipulated to give Equation 3.4.2 where q_p is in cfs, A_m is in mi^2 and Q´ is in inches.

$$q_p = \frac{484\, A_m\, Q'}{t_p} \qquad (3.4.2)$$

It has been observed by the SCS that the watershed lag (L´) is related to the t_c of the basin as L´ = 0.6 t_c. Consequently, the time to peak (t_p) may be expressed in terms of the duration of unit excess rainfall (ΔD) and L´ as shown in Equation 3.4.3.

$$t_p = \frac{\Delta D}{2} + L' = \frac{\Delta D}{2} + 0.6\, t_c \qquad (3.4.3)$$

$$\therefore \quad q_p = \frac{484\, A_m Q'}{t_p} = \frac{484\, A_m Q'}{\frac{\Delta D}{2} + 0.6\, t_c} \qquad (3.4.4)$$

The dimensionless unit hydrograph, Figure 3.4.3, has a point of inflection at approximately $1.7t_p$ and can be expressed in terms of t_c and ΔD as; $t_c + \Delta D = 1.7t_p$. Substituting t_p found in Equation 3.4.3 into $t_c + \Delta D = 1.7t_p$ and simplifying gives $\Delta D = 0.133t_c$. A small variation in ΔD is permissible, but it should not exceed $0.17t_p$ (Viessman et al., 1989).

3.4.3 - Comparison of Unit Hydrographs

The preceding relationships allow the development of the dimensionless unit hydrograph as follows:

Step 1: Find the time of concentration, t_c. This value may be obtained using Figure 3.4.5, in which the ordinate is entered with the travel path slope, then the diagonal line which represents either a paved or unpaved surface is intercepted, and a velocity is found by reading the abscissa. The travel time, t_t, is then found by using Equation 3.4.5. The time of concentration, t_c, is the sum of t_t values for various consecutive flow segments.

$$t_t = \frac{L}{3600\, v} \qquad t_c = t_{t1} + t_{t2} + \ldots t_{tn} \qquad (3.4.5)$$

Step 2: Determine the values of t_p, and q_p from Equations 3.4.3 and 3.4.4, where Q is equal to one for a unit hydrograph and ΔD is the rainfall duration.

$$q_p = \frac{484\, A_m Q'}{t_p} \qquad t_p = \frac{\Delta D}{2} + L' = \frac{\Delta D}{2} + 0.6\, t_c$$

Step 3: Determine the coordinates of the unit hydrograph. By using Table 3.4.1, the values of discharge and time may be found by multiplying the ratios by q_p and t_p. These may be plotted to develop the unit hydrograph.

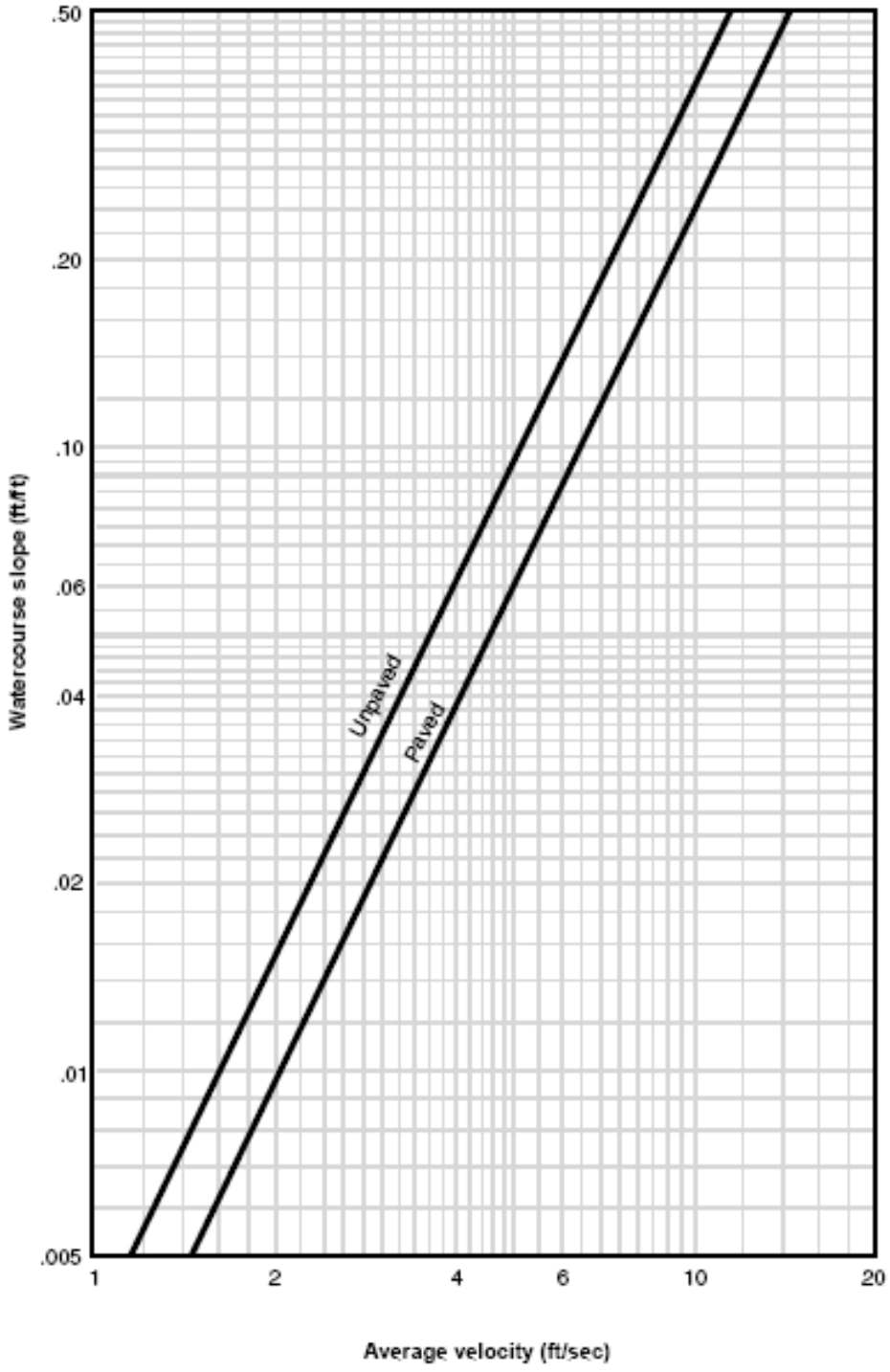


Figure 3.4.5
Average Velocities for Estimating Travel Time (USDA, 1986)

Step 4: Check that the volume of the unit hydrograph is equal to 1 inch of excess rainfall falling on the basin. This is accomplished by summing the coordinates of q_t and multiplying by ΔD. This may be expressed as:

$$V_{UH} = \Delta D \sum_{i=1}^{\left(\frac{5\,t_p}{\Delta D}\right)} q_i \qquad 3.4.6)$$

where V_{UH} has units of cfs-hr.

The value, V_{UH}, obtained in Equation 3.4.5 should be compared to the volume, V, obtained by Equation 3.4.6:

$$V = A_m * 645.33 \qquad (3.4.7)$$

Where V has units of cfs-hour and A_m has units of mi^2.

The results from Equations 3.4.5 and 3.4.6 should be very close. If there is a difference, the ordinates of the unit hydrograph should be adjusted.

The triangular unit hydrograph may be determined by using Step 2 outlined above and the relationship $t_b = 2.67\ t_p$.

3.4.4 - Storm Hydrographs

Once a unit hydrograph has been determined, the hydrograph for a given temporal distribution of a storm and basin characteristics can be determined as follows.

Before a hydrograph can be developed, the basin area (sq. mi.), basin curve number (see Section 3.3), antecedent moisture condition (see Section 3.3), storm duration, depth, and temporal distribution must be known. The following steps outline the procedure used in the application of dimensionless unit hydrograph to determine the storm hydrograph from a unit hydrograph.

Step 1: Tabulate the cumulative rainfall at ΔD intervals. Tabulate the unit hydrograph at ΔD time intervals.

Step 2: Using Equation 3.3.1 calculate the cumulative runoff using the curve number (see Section 3.3.3).

Step 3: Calculate the incremental runoff for each time interval.

Step 4: The storm hydrograph may be found using the results of Step 1, Step 3 and Equation 3.4.8.

$$Q_t = \sum_{i=\mu}^{\nu} (u_{t-i+1})(q_i) \qquad t = 1,...NP + N\,U - 1 \tag{3.4.8}$$

This is valid for NP < NU.

where Q_t is the runoff at time t (cfs), u is the ordinate of the unit hydrograph [cfs/inch], q is the excess rainfall [inches], NP is the number of elements in the rainfall hyetograph, NU is the number of coordinates in the unit hydrograph, μ is max [1,t+1-NU], and ν is min [t,NP].

These equations may be solved with a computer program such as TR-20 or HEC-1 or a small programmable calculator (Croley, 1977). They may also be solved manually as outlined in Example 3.4.1. The time interval for the hydrograph is assumed to be ΔD and it is assumed that NP < NU.

Step 5: Compute the volume of the new hydrograph by summing the ordinates and multiplying by the time interval. Compare this to the actual volume (area x accumulated runoff).

Example 3.4.1

A basin in South Bend, IN has an area of 300 acres (.47 mi^2) and a weighted curve number of 84. Determine the 2 hour - 50 year storm hydrograph by using a unit hydrograph. Assume that the water travels 9000 ft. on unpaved surface with a 1.6% slope.

Step 1: Find the time of concentration, t_c, using Figure 3.4.5. Enter the ordinate with a 1.6% slope and go across until the unpaved line is intersected and a velocity of 2.0 ft/sec is found. The time of concentration is equal to

$$t_c = \frac{L}{v} = \frac{9000\ ft}{2\ ft/\sec} = 4500\ seconds = 75\ minutes$$

Step 2: Determine the values of ΔD, t_p and q_p. From page 3-31:
$\Delta D = 0.133 t_c = 0.133(75) = 9.98$ minutes ≈ 10 minutes
$t_p = (\Delta D/2) + 0.6 t_c = (10/2) + 0.6(75) = 50$ min ≈ 0.83 hours

$$q_p = \frac{484\ A_m Q'}{t_p} = \frac{484(0.47\ mi^2)(1\ inch)}{0.83\ hours} = 274\ cfs$$

Step 3: Determine the coordinates of the dimensionless unit hydrograph using Table 3.4.1. ∆D was computed to be 10 minutes.

Example Calculation: $t = t/t_p * t_p = 0.8*50=40$ $q=q/q_p*q_p = 0.930*274 = 254.82$

Time Ratio t / t_p	Time t (min.)	Discharge Ratio (q / q_p)	Discharge q (cfs)
0.0	0	0.000	0
0.1	5	0.030	8.22
0.2	10	0.100	27.40
0.3	15	0.190	52.06
0.4	20	0.310	84.94
0.5	25	0.470	128.78
0.6	30	0.660	180.84
0.7	35	0.820	224.68
0.8	40	0.930	254.82
0.9	45	0.990	271.26
1.0	50	1.000	274.00
1.1	55	0.990	271.26
1.2	60	0.930	254.82
1.3	65	0.860	235.64
1.4	70	0.780	213.72
1.5	75	0.680	186.32
1.6	80	0.560	153.44
1.7	85	0.460	126.04
1.8	90	0.390	106.86
1.9	95	0.330	90.42
2.0	100	0.280	76.72
2.2	110	0.207	56.72
2.4	120	0.147	40.28
2.6	130	0.107	29.32
2.8	140	0.077	21.10
3.0	150	0.055	15.07
3.2	160	0.040	10.96
3.4	170	0.029	7.95
3.6	180	0.021	5.75
3.8	190	0.015	4.11
4.0	200	0.011	3.01
4.5	225	0.005	1.37
5.0	250	0.000	0.00

The values of t and q are plotted to get the unit hydrograph shown in Figure 3.4.6.

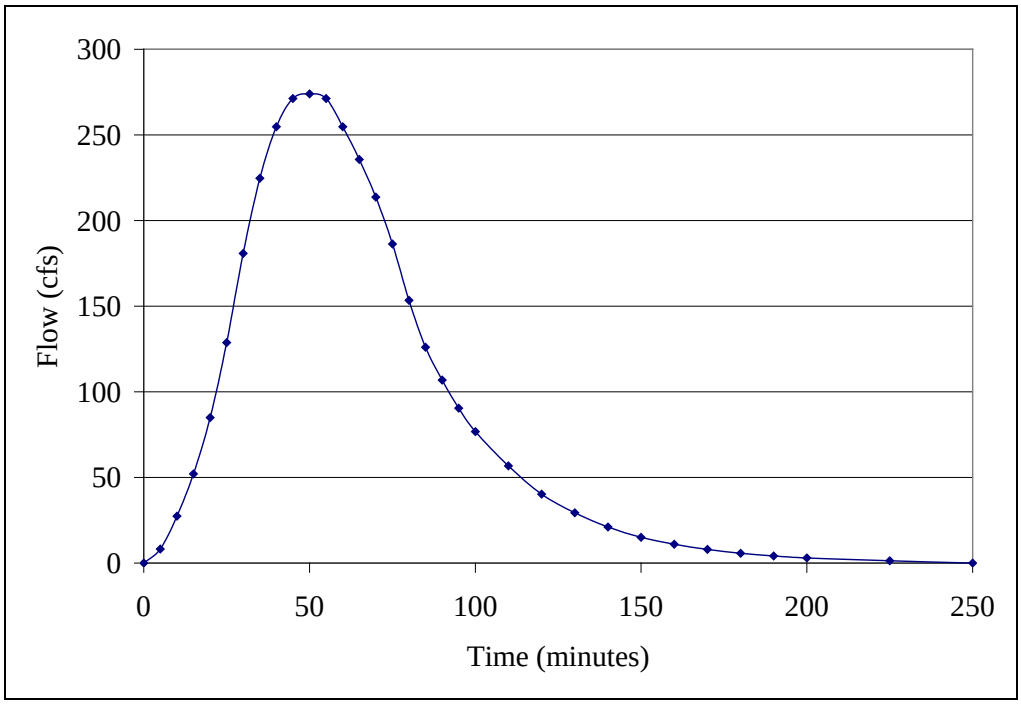


Figure 3.4.6
Dimensionless Unit Hydrograph for Example 3.4.1

Step 4: Check to see that the volume of the unit hydrograph is equal to 1 inch. Sum the unit hydrograph ordinates.

(0+8.22+27.40+...106.86+90.42+76.72)*5 + (56.72+40.28+…4.11+3.01)*10 + (1.37+0)*25

= 18,088 cfs-min. (*)

*Note: Discharges for Time Ratio greater than 4.5 are considered zero.

$$18{,}088\ \frac{ft^3\ \text{min}}{\text{sec}} * \frac{60\ \text{sec}}{\text{min}} * \frac{acre}{43{,}560\ ft^2} = 24.91\ acre\text{-}\ feet$$

$$1\ inch * \frac{1\ ft}{12\ inches} * 300\ acres = 25.00\ acre\text{-}\ feet$$

$$\frac{25.00 - 24.91}{24.91} * 100\% = 0.36\%\ difference$$

Once the unit hydrograph has been established, the storm hydrograph is computed by using the

temporal distribution of the rainfall.

Step 5: Find the rainfall depth. Using Equation 2.2.13 with the coefficients in Table 2.2.2, the 2-hour, 50-year rainfall intensity for South Bend is 1.36 in/hr. The 2-hour, 50-year rainfall depth is 2.72 in.

From Table 2.2.2, with a recurrence interval of 50-years and 2-hour duration, the coefficients for South Bend are:
c = 1.2799 α = 0.1872 d = 0.258 β = 0.8252

The corresponding rainfall intensity is:

$$i = \frac{c\,T_r^{\alpha}}{(t+d)^{\beta}} = \frac{1.2799\,(50)^{0.1872}}{(2+0.258)^{0.8252}} = 1.36 \text{ inches/hour}$$

For the 2-hour duration, the rainfall depth is 2*1.36 = 2.72 inches.

Tabulate the accumulated runoff at 10-minute intervals for the total rainfall depth of 2.72 inches using Huff's first quartile median distribution curve from Table 2.1.5.

Time (min)	Cum. % of Total Storm Time	Cum. % of Total Storm Rainfall	Cum. Rainfall Depth (inch)	Cum. Runoff R(t) (inch)*	Increm. Runoff R(t) (inch)
0	0	0	0.00	0.00	
10	8.33	16.66	0.45	0.00	0.00
20	16.67	33.34	0.91	0.11	0.11
30	25.00	45.84	1.25	0.27	0.16
40	33.33	54.74	1.49	0.41	0.14
50	41.67	61.97	1.69	0.53	0.12
60	50.00	67.35	1.83	0.63	0.10
70	58.33	73.72	2.01	0.75	0.12
80	66.67	78.89	2.15	0.85	0.10
90	75.00	83.75	2.28	0.95	0.10
100	83.33	88.74	2.41	1.05	0.10
110	91.67	94.08	2.56	1.16	0.11
120	100.00	100.00	2.72	1.29	0.13

$$* R(t) = \left(\frac{(P(t) - 0.2S)^2}{P(t) + 0.8\,S} \right) \text{ Where } S = \left(\frac{1000}{84} \right) - 10 = 1.90 \text{ inch}, R(t) = 0 \text{ if } P(t) < 0.2S$$

Step 6: Compute and construct the storm hydrograph. Multiplying the unit hydrograph in Figure 3.4.6 by the incremental runoff and summing values across as shown in Figure 3.4.7, the storm hydrograph presented in Figure 3.4.8 is developed. A blank form for the calculations is given in Figure 3.4.9.

Time (minutes)	Unit Hydrograph (cfs)	Excess Precipitation (inches)												Storm Hydrograph (cfs)
		0.00	0.11	0.16	0.14	0.12	0.10	0.12	0.10	0.10	0.10	0.11	0.13	
0	0.00	0.00												0.00
10	27.40	0.00	0.00											0.00
20	84.94	0.00	3.01	0.00										3.01
30	180.84	0.00	9.34	4.38	0.00									13.73
40	254.82	0.00	19.89	13.59	3.84	0.00								37.32
50	274.00	0.00	28.03	28.93	11.89	3.29	0.00							72.14
60	254.82	0.00	30.14	40.77	25.32	10.19	2.74	0.00						109.16
70	213.72	0.00	28.03	43.84	35.67	21.70	8.49	3.29	0.00					141.03
80	153.44	0.00	23.51	40.77	38.36	30.58	18.08	10.19	2.74	0.00				164.24
90	106.86	0.00	16.88	34.20	35.67	32.88	25.48	21.70	8.49	2.74	0.00			178.05
100	76.72	0.00	11.75	24.55	29.92	30.58	27.40	30.58	18.08	8.49	2.74	0.00		184.10
110	56.72	0.00	8.44	17.10	21.48	25.65	25.48	32.88	25.48	18.08	8.49	3.01	0.00	186.10
120	40.28	0.00	6.24	12.28	14.96	18.41	21.37	30.58	27.40	25.48	18.08	9.34	3.56	187.71
130	29.32	0.00	4.43	9.08	10.74	12.82	15.34	25.65	25.48	27.40	25.48	19.89	11.04	187.36
140	21.10	0.00	3.23	6.44	7.94	9.21	10.69	18.41	21.37	25.48	27.40	28.03	23.51	181.71
150	15.07	0.00	2.32	4.69	5.64	6.81	7.67	12.82	15.34	21.37	25.48	30.14	33.13	165.42
160	10.96	0.00	1.66	3.38	4.10	4.83	5.67	9.21	10.69	15.34	21.37	28.03	35.62	139.90
170	7.95	0.00	1.21	2.41	2.95	3.52	4.03	6.81	7.67	10.69	15.34	23.51	33.13	111.26
180	5.75	0.00	0.87	1.75	2.11	2.53	2.93	4.83	5.67	7.67	10.69	16.88	27.78	83.73
190	4.11	0.00	0.63	1.27	1.53	1.81	2.11	3.52	4.03	5.67	7.67	11.75	19.95	59.95
200	3.01	0.00	0.45	0.92	1.11	1.32	1.51	2.53	2.93	4.03	5.67	8.44	13.89	42.80
210	2.35	0.00	0.33	0.66	0.81	0.95	1.10	1.81	2.11	2.93	4.03	6.24	9.97	30.93
220	1.70	0.00	0.26	0.48	0.58	0.69	0.80	1.32	1.51	2.11	2.93	4.43	7.37	22.47
230	1.10	0.00	0.19	0.38	0.42	0.49	0.58	0.95	1.10	1.51	2.11	3.23	5.24	16.18
240	0.55	0.00	0.12	0.27	0.33	0.36	0.41	0.69	0.80	1.10	1.51	2.32	3.81	11.71
250	0.00	0.00	0.06	0.18	0.24	0.28	0.30	0.49	0.58	0.80	1.10	1.66	2.74	8.42
260			0.00	0.09	0.15	0.20	0.24	0.36	0.41	0.58	0.80	1.21	1.96	5.99
270				0.00	0.08	0.13	0.17	0.28	0.30	0.41	0.58	0.87	1.42	4.25
280					0.00	0.07	0.11	0.20	0.24	0.30	0.41	0.63	1.03	2.99
290						0.00	0.06	0.13	0.17	0.24	0.30	0.45	0.75	2.09
300							0.00	0.07	0.11	0.17	0.24	0.33	0.53	1.45
310								0.00	0.06	0.11	0.17	0.26	0.39	0.98

(Unit hydrograph values at 210 through 240 minutes were found through interpolation.)

Figure 3.4.7
Computation of the Storm Hydrograph for Example 3.4.1

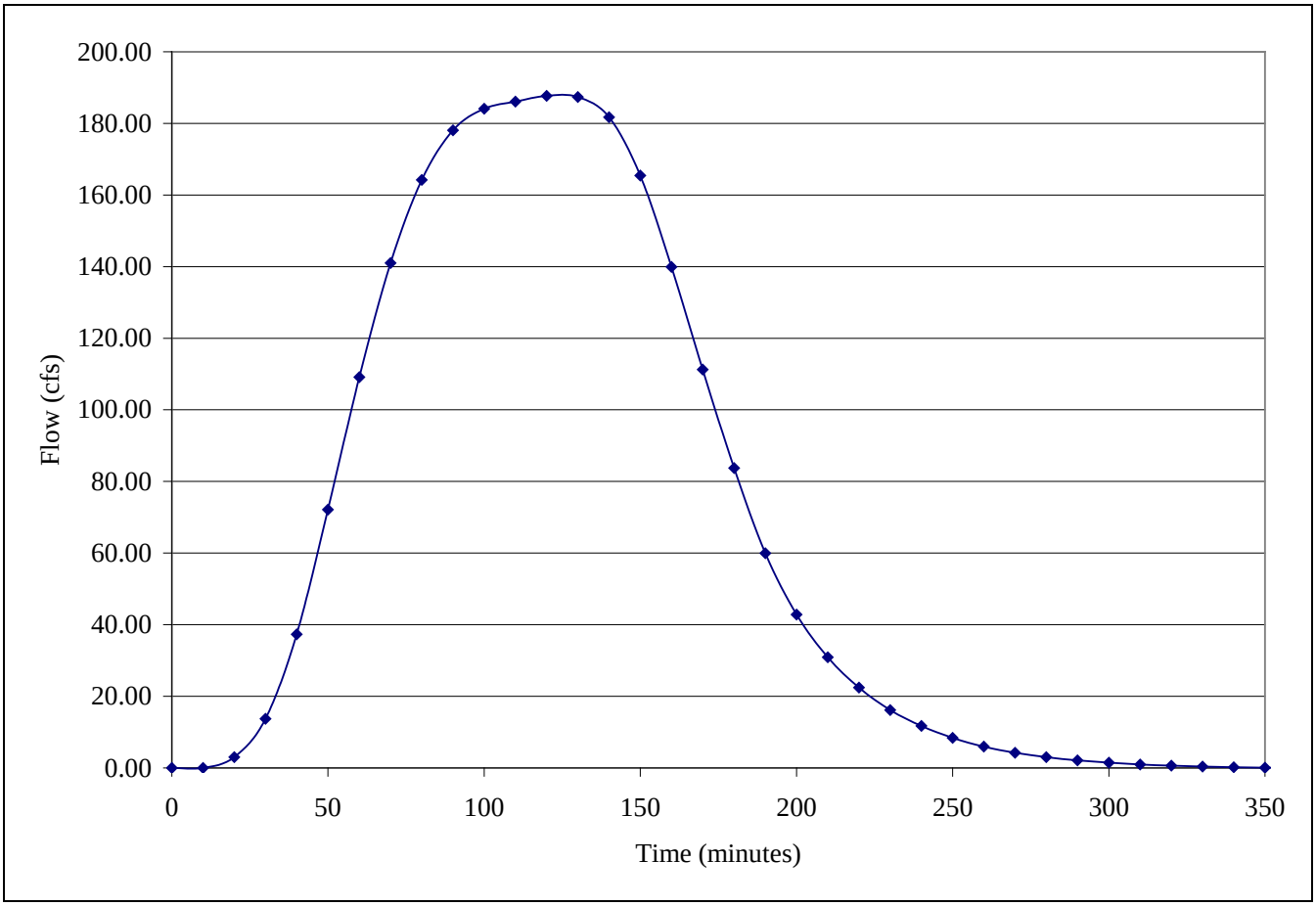


Figure 3.4.8
Storm Hydrograph for Example 3.4.1

Time (minutes)	Unit Hydrograph (cfs)	Excess Precipitation (inches)												Storm Hydrograph (cfs)

Figure 3.4.9
Calculation Sheet for Hydrograph Computation

3.5 - WATER RESOURCES COUNCIL METHOD

The use of the Water Resources Council Method (WRC) for flood flow frequency analysis has been mandated for projects involving Federal funds. The procedure for flood frequency analysis by using the WRC method and annual maximum flows x_i, i=1, 2,...n, is as follows (U.S. Water Resources Council, 1981).

Step 1: Convert x_i into logarithmic values, y_i.

$$y_i = \log(x_i)$$

Step 2: Compute the sample statistics

$$MEAN = \bar{y} = \frac{1}{n}\sum_{i=1}^{n} y_i$$

$$STANDARD\ \ DEVIATION = s_y = \left(\frac{1}{n-1}\sum_{i=1}^{n}(y_i - \bar{y})^2\right)^{\frac{1}{2}}$$

$$SKEWNESS\ \ COEFFICIENT = C_s = \frac{n\sum_{i=1}^{n}(y_i - \bar{y})^3}{(n-1)(n-2)s_y^3}$$

Step 3: Test for outliers. These are flowrates which are too high or low to be reconciled with the remaining data points. Inclusion of these data points will significantly affect the sample statistics, and thus the final results. The logarithms of high and low outlier thresholds are found by:

$$y_H = \bar{y} + K_n \bullet s_y \qquad y_L = \bar{y} - K_n \bullet s_y$$

where the values of K_n are shown in Table 3.5.1 and are used in *one-sided tests* that detect outliers at the 10-percent level of significance in normally distributed data. According to the Water Resources Council (1981), if information is available that indicates a high outlier is the maximum over an extended period of time, the outlier is treated as historic flood data and excluded from analysis. If useful historic information is not available to compare to high outliers, then the outliers should be retained as part of the systematic record. Flood peaks considered low outliers are deleted and the procedure is repeated (U.S. Water Resources Council, 1981).

Step 4: Consult Figure 3.5.1 to determine the map skew, C_m. Most of Indiana has a map skew of -0.4.

Table 3.5.1
Values of K_n for Various Sample Sizes

Sample Size, n	K_n	Sample Size, n	K_n
10	2.036	35	2.628
11	2.088	36	2.639
12	2.134	37	2.650
13	2.175	38	2.661
14	2.213	39	2.671
15	2.247	40	2.682
16	2.279	41	2.692
17	2.309	42	2.700
18	2.335	43	2.710
19	2.361	44	2.719
20	2.385	45	2.727
21	2.408	50	2.768
22	2.429	55	2.804
23	2.448	60	2.837
24	2.467	65	2.866
25	2.486	70	2.893
26	2.502	75	2.917
27	2.519	80	2.940
28	2.534	85	2.961
29	2.549	90	2.981
30	2.563	95	3.000
31	2.577	100	3.017
32	2.591	110	3.049
33	2.604	120	3.078
34	2.616	130	3.104
		140	3.129

Step 5: Determine the values of G and H, where:

$$G = -0.33 + 0.08|C_s| \quad if \quad |C_s| \le 0.9$$

$$G = -0.52 + 0.30|C_s| \quad if \quad |C_s| > 0.9$$

$$H = 0.94 - 0.26|C_s| \quad if \quad |C_s| \le 1.50$$

$$H = 0.55 \quad if \quad |C_s| > 1.50$$

Step 6: Determine the Variance of the station skew, $V(C_s)$.

$$V(C_s) = 10^{G - H \log(n/10)}$$

Step 7: Determine the weight, W, to place on the station skew.

$$W = \frac{V(C_m)}{V(C_s) + V(C_m)}$$

$V(C_m)$, the coefficient of map skew, has been estimated to be equal to 0.303.

Step 8: Determine the weighted skew of the station.

$$C_w = W \bullet C_s + (1 - W) \bullet 0.303$$

Step 9: Determine the frequency factors, K_t, corresponding to the various return periods using Table 3.12.

Step 10: Determine flow rates for the various return periods using the following relationships:

$$y_{T_r} = \bar{y} + K_T \; s_y$$

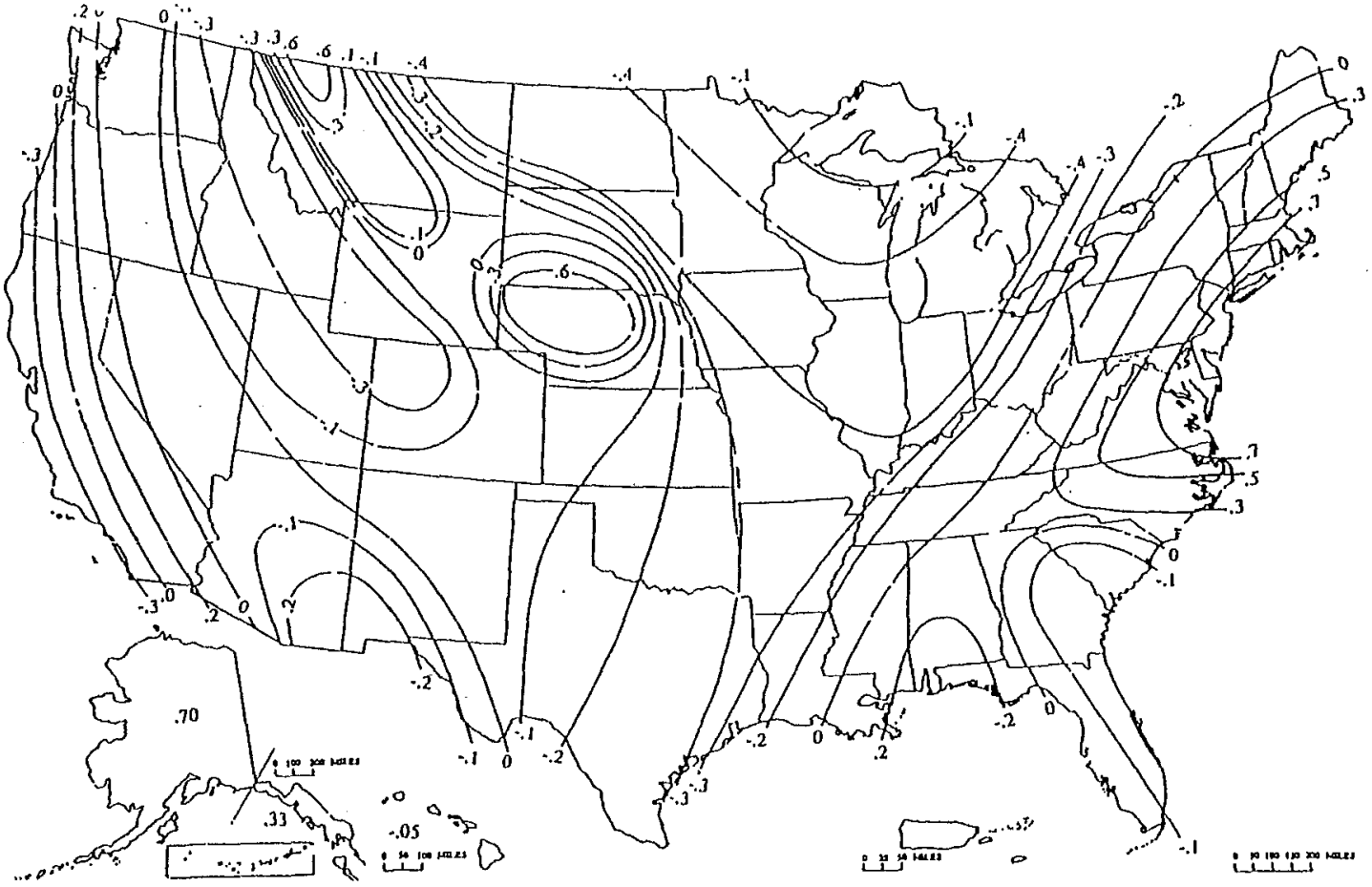
$$Q_{T_r} = 10^{y_{T_r}}$$

A computer program is available to perform the flood frequency analysis computations and plot the desired results with confidence intervals. The HEC-FFA program, formerly known as HECWRC, computes the flood frequencies in accordance with "Guidelines for Determining Flood Flow Frequencies," Bulletin 17B of the U.S. Water Resources Council (WRC), March 1982. The program and user's manual is available from the U.S. Army Corps of Engineers, (916) 756-1104 (HEC-FFA, 1992).

Table 3.5.2
K_T Values For Water Resource Council Method
(Log Pearson Type III Distribution)
Return Period in Years

Skew Coefficient, C_w	2	5	10	25	50	100
-3.0	0.396	0.636	0.666	0.666	0.666	0.667
-2.9	0.390	0.651	0.681	0.683	0.689	0.690
-2.8	0.384	0.666	0.702	0.712	0.714	0.714
-2.7	0.376	0.681	0.724	0.738	0.740	0.740
-2.6	0.368	0.696	0.747	0.764	0.768	0.769
-2.5	0.360	0.711	0.771	0.793	0.798	0.799
-2.4	0.351	0.725	0.795	0.823	0.830	0.832
-2.3	0.341	0.739	0.819	0.855	0.864	0.867
-2.2	0.330	0.752	0.844	0.888	0.900	0.905
-2.1	0.319	0.765	0.869	0.923	0.939	0.946
-2.0	0.307	0.777	0.895	0.959	0.980	0.990
-1.9	0.294	0.788	0.920	0.996	1.023	1.037
-1.8	0.282	0.799	0.945	1.035	1.069	1.087
-1.7	0.268	0.808	0.970	1.075	1.116	1.140
-1.6	0.254	0.817	0.994	1.116	1.166	1.197
-1.5	0.240	0.825	1.018	1.157	1.217	1.256
-1.4	0.225	0.832	1.041	1.198	1.270	1.318
-1.3	0.210	0.838	1.064	1.240	1.324	1.383
-1.2	0.195	0.844	1.086	1.282	1.379	1.449
-1.1	0.180	0.848	1.107	1.324	1.435	1.518
-1.0	0.164	0.852	1.128	1.366	1.492	1.588
-0.9	0.148	0.854	1.147	1.407	1.549	1.660
-0.8	0.132	0.856	1.166	1.448	1.606	1.733
-0.7	0.116	0.857	1.183	1.488	1.663	1.806
-0.6	0.099	0.857	1.200	1.528	1.720	1.880
-0.5	0.083	0.856	1.216	1.567	1.777	1.955
-0.4	0.066	0.855	1.231	1.606	1.834	2.029
-0.3	0.050	0.853	1.245	1.643	1.890	2.104
-0.2	0.033	0.850	1.258	1.680	1.945	2.178
-0.1	0.017	0.846	1.270	1.716	2.000	2.252
0.0	0.000	0.842	1.282	1.751	2.054	2.326
0.1	-0.017	0.836	1.292	1.785	2.107	2.400
0.2	-0.033	0.830	1.301	1.818	2.159	2.472
0.3	-0.050	0.824	1.309	1.849	2.211	2.544
0.4	-0.066	0.816	1.317	1.880	2.261	2.615
0.5	-0.083	0.808	1.323	1.910	2.311	2.686
0.6	-0.099	0.800	1.328	1.939	2.359	2.755
0.7	-0.116	0.790	1.333	1.967	2.407	2.824
0.8	-0.132	0.780	1.336	1.993	2.453	2.891
0.9	-0.148	0.769	1.339	2.018	2.498	2.957
1.0	-0.164	0.758	1.340	2.043	2.542	3.022

Skew Coefficient, C_w	2	5	10	25	50	100
1.1	-0.180	0.745	1.341	2.066	2.585	3.087
1.2	-0.195	0.732	1.340	2.087	2.626	3.149
1.3	-0.210	0.719	1.339	2.108	2.666	3.211
1.4	-0.225	0.705	1.337	2.128	2.706	3.271
1.5	-0.024	0.690	1.333	2.146	2.743	3.330
1.6	-0.254	0.675	1.329	2.163	2.780	3.388
1.7	-0.268	0.660	1.324	2.179	2.815	3.444
1.8	-0.282	0.643	1.318	2.193	2.848	3.499
1.9	-0.294	0.627	1.310	2.207	2.881	3.553
2.0	-0.307	0.609	1.302	2.219	2.912	3.605
2.1	-0.319	0.592	1.294	2.230	2.942	3.656
2.2	-0.330	0.574	1.284	2.240	2.970	3.705
2.3	-0.341	0.555	1.274	2.248	2.997	3.753
2.4	-0.351	0.537	1.262	2.256	3.023	3.800
2.5	-0.360	0.518	1.250	2.262	3.048	3.845
2.6	-0.368	0.499	1.238	2.267	3.071	3.889
2.7	-0.376	0.479	1.224	2.272	3.093	3.932
2.8	-0.384	0.460	1.210	2.275	3.114	3.973
2.9	-0.390	0.440	1.195	2.277	3.134	4.013
3.0	-0.396	0.420	1.180	2.278	3.152	4.000



Generalized skew coefficients of annual maximum streamflow. (*Source:* Guidelines for determining flood flow frequency, Bulletin 17B, Hydrology Subcommittee, Interagency Advisory Committee on Water Data, U. S. Geological Survey, Reston, Va. Revised with corrections March 1982.)

Figure 3.5.1

Example 3.5.1

Perform a flood frequency analysis of annual maximum flows from South Fork Wildcat Creek at Lafayette, Indiana.

Step 1: Transform x_i into logarithms, y_i. These are shown in the third column below.

$Year_i$	x_i (cfs)	y_i
1943	5770	3.5289
1944	3380	3.6739
1945	4720	3.5340
1946	3420	3.6107
1947	4080	3.7202
1948	5250	4.0792
1949	12000	3.6284
1950	4250	3.5658
1951	3680	3.7443
1952	5550	3.1847
1953	1530	3.5378
1954	3450	3.2095
1955	1620	3.6284
1956	4250	4.1004
1957	12600	3.9243
1958	8400	3.2601
1959	1820	3.5198
1960	3310	3.6990
1961	5000	3.8075
1962	6420	4.0334
1963	10800	3.5682
1964	3700	2.9750
1965	944	3.6990
1966	5000	3.9320
1967	8550	3.6964
1968	4970	3.5416
1969	3480	3.6064
1970	4040	3.5775
1971	3780	3.4378
1972	2740	3.7589
1973	5740	3.7738
1974	5940	3.6243
1975	4210	3.2405
1976	1740	3.9479
1977	8870	3.5999
1978	3980	3.8549
1979	7160	3.7497
1980	5620	3.6107
1981	4080	4.1790
1982	15100	3.4683
1983	2940	3.8261
1984	6700	3.6758
1985	4740	3.0043
1986	1010	3.5670
1987	3690	4.1584
1988	14400	3.9652
1989	9230	3.9484
1990	8880	3.6128
1991	4100	
1992		
	4.2529	
17900	3.7612	

Step 2: Compute the sample statistics of the logarithmic data. There are 50 data points (n=50).

$$\bar{y} = \frac{1}{n}\sum_{i=1}^{n} y_i = 3.6721$$

$$s_y = \left(\frac{1}{n-1}\sum_{i=1}^{n}(y_i - \bar{y})^2\right)^{\frac{1}{2}} = 0.2785$$

$$C_s = \frac{n\sum_{i=1}^{n}(y_i - \bar{y})^3}{(n-1)(n-2)s_y^3} = -0.2872$$

Step 3: Test for outliers. Referring to Table 3.5.1, for n=50, K_n=2.768

$$y_H = \bar{y} + K_n \bullet s_y = 3.6721 + 2.768 \bullet 0.2785 = 4.4430$$

$$Q_H = 10^{4.4430} = 27{,}732\ cfs$$

$$y_L = \bar{y} - K_n s_y = 3.6721 - 2.768 * 0.2785 = 2.9013$$

$$Q_L = 10^{2.9013} = 797\ cfs$$

All of the flows are within the range of 797 and 27,720 cfs. Therefore there are no outliers in this example.

Step 4: Determine the map skew, C_s from Figure 3.5.1.
Lafayette has a map skew of -0.4.

Step 5: Determine the values of G and H. C_s = -0.2872.

$$H = 0.94 - 0.26\,|C_s| = 0.94 - 0.26\,|-0.2872| = 0.865 \quad (for\ |C_s| _ 1.50)$$

$$G = -0.33 + 0.08\,|C_s| = -0.33 + 0.08\,|-0.2872| = -0.307 \quad (for\ |C_s| _ 0.9)$$

Step 6: Determine V(C_s), the variance of the station skew.

$$V(C_s) = 10^{G - H \log(n/10)} = 10^{-0.307 - 0.865 \log(50/10)} = 0.1226$$

Step 7: Determine the weight corresponding to station skew, W.

$$W = \frac{0.303}{V(C_s) + 0.303} = \frac{0.303}{0.1226 + 0.303} = 0.712$$

Step 8: Determine the weighted skew of the station.

$$C_w = W \bullet C_s + (1 - W) \bullet 0.303 = 0.712 \bullet (-0.2872) + (1 - 0.712) \bullet 0.303 = -0.117$$

Step 9: Determine the frequency factors, K_T corresponding to the different return periods. Interpolating from Table 3.5.2 between C_w values of -0.1 and -0.2 yields:

Return Period (T_r), years	K_T
2	0.020
5	0.847
10	1.268
25	1.710
50	1.991
100	2.239

Step 10: Determine the flow rates for the various return periods.

$$y_{T_r} = \bar{y} + K_T s_y \qquad Q_{T_r} = 10^{y_{T_r}}$$

T_r	K_T	y_{Tr}	Q_{Tr} (cfs)
2	0.020	3.677	4761
5	0.847	3.908	8091
10	1.268	4.025	10,598
25	1.710	4.148	14,071
50	1.991	4.227	16,850
100	2.239	4.296	19,754

3.6 - REGRESSION EQUATIONS

A great deal of research has been done to develop regression equations to generate peak discharges from ungaged rural and urban areas. Regional regression equations that were based on select parameters were developed for the State of Indiana by the USGS (1984). The parameters, which varied from region to region within the state, included such items as drainage area, slope, and soil runoff coefficients. The peak discharge could be calculated once all parameters were known.

Recently, regression equations that supersede the USGS equations were developed (Knipe and Rao, 2005). The USGS has converted this newer regression analysis into a GIS-based discharge determination program titled, "StreamStats." The program allows the user to specify the point of interest for a stream on the interactive map. Once the point of interest has been selected, StreamStats will:

- Delineate the drainage area at the user-specified location.
- Determine all region-specific parameters to be used in the regression analysis.
- Choose the appropriate regression equation for the selected area.
- Compute the peak discharges for the stream at different return intervals.

The StreamStats program has been developed by USGS for Indiana as well as several other states (2007). StreamStats for the State of Indiana is available at the following website: http://water.usgs.gov/osw/streamstats/indiana.html. A link to the regression analysis on which the Indiana StreamStats program is based is also available at this website.

A step-by-step guide to the StreamStats program is included as Example 3.6.1.

Example 3.6.1

This example illustrates the use of the USGS StreamStats computer software.

Use the USGS StreamStats program to estimate the 10- and 100-year flood flows for the 243 square mile watershed of the South Fork Wildcat Creek at Lafayette, Indiana.

Step 1: Go to the USGS StreamStats website for the state of Indiana, located at http://water.usgs.gov/osw/streamstats/indiana.html. The introductory screen is a zoomed out map of Indiana with its interstates, as shown in Figure 3.6.1.

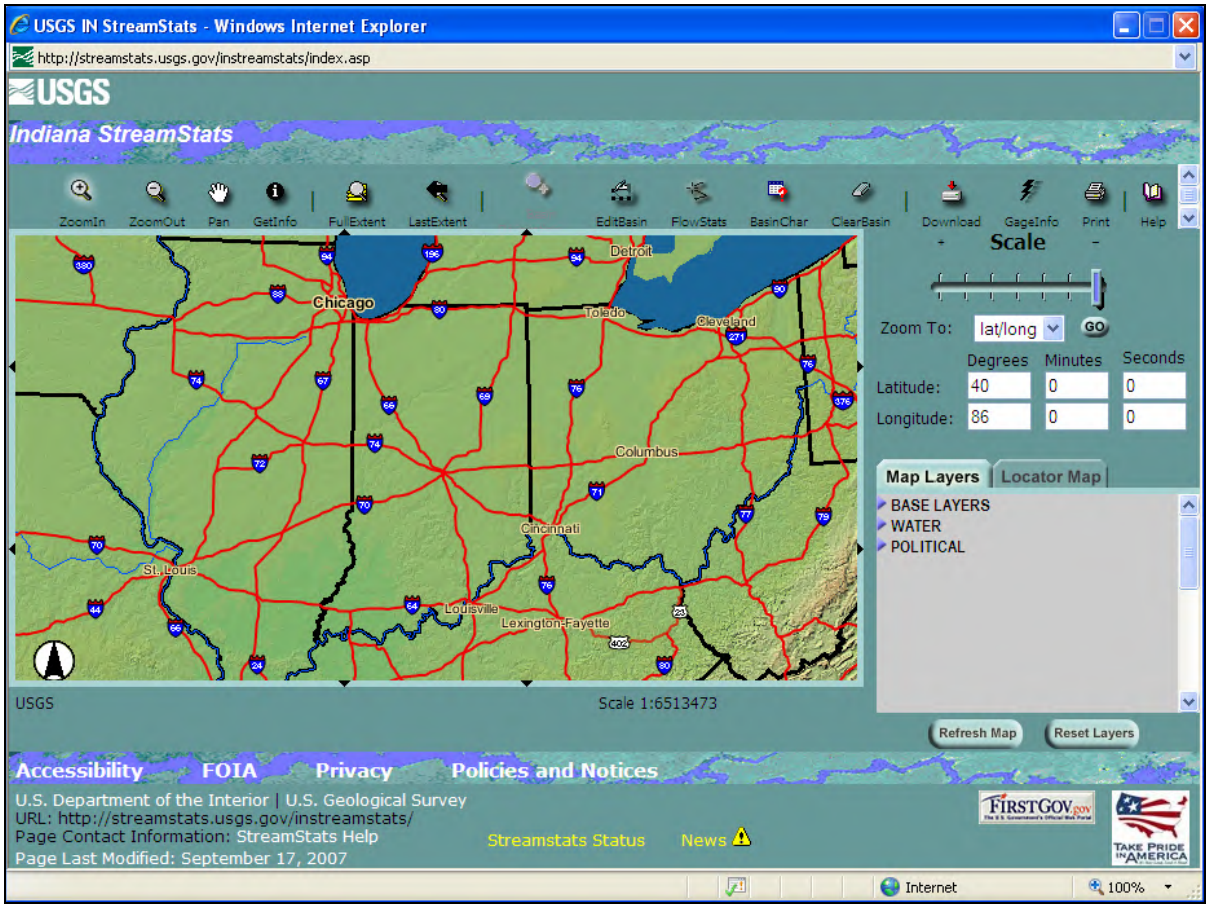


Figure 3.6.1
0Introductory Screen for the Indiana StreamStats Program

Step 2: Zoom in on the specified region until the "Basin Delineation" tool becomes available.

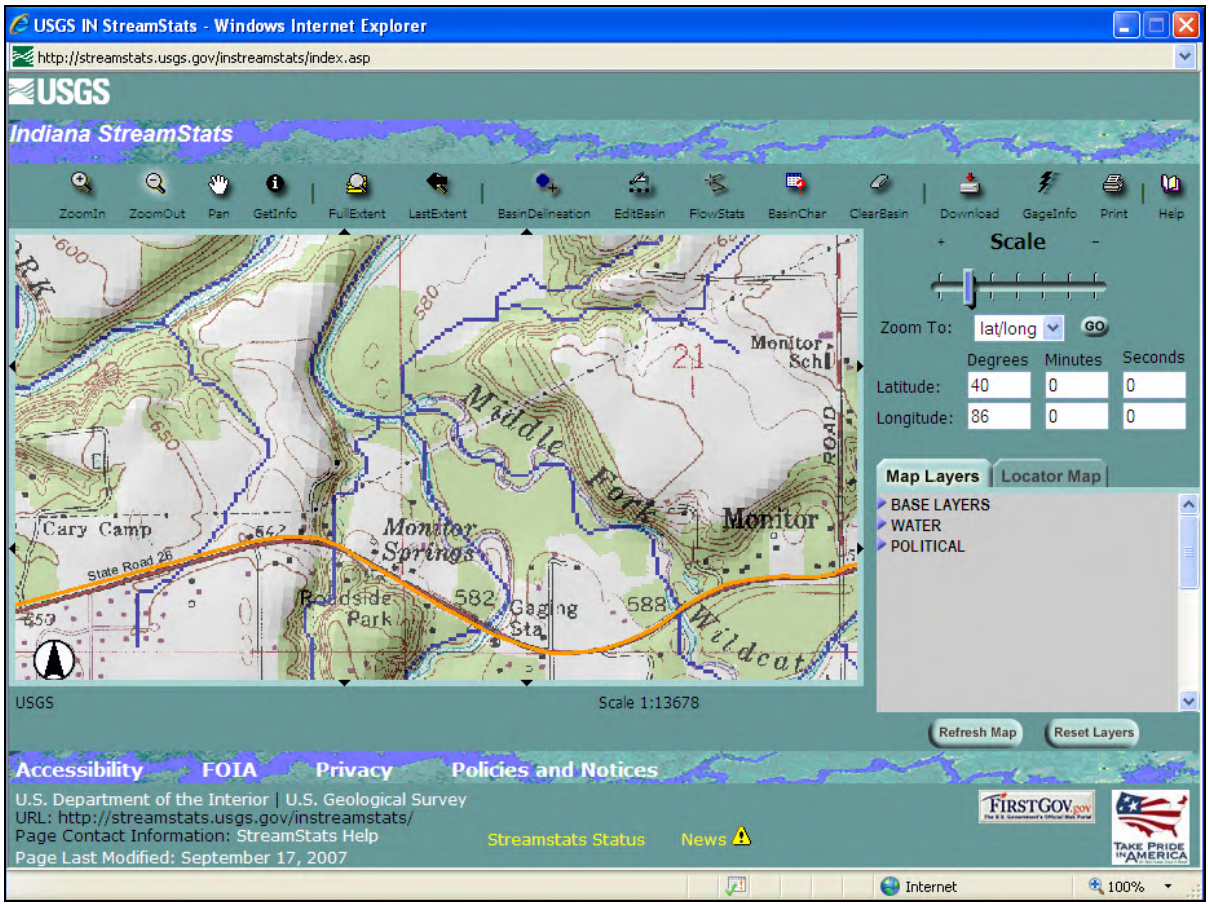


Figure 3.6.2
Zoomed In Area of the South Fork Wildcat Creek at Lafayette, IN

Step 3: Click on the "Basin Delineation" icon and click on the South Fork Wildcat Creek just upstream of its confluence with the Middle Fork Wildcat Creek. A new window will appear that shows the drainage area boundary outlined in pink. This is illustrated on Figure 3.6.3.

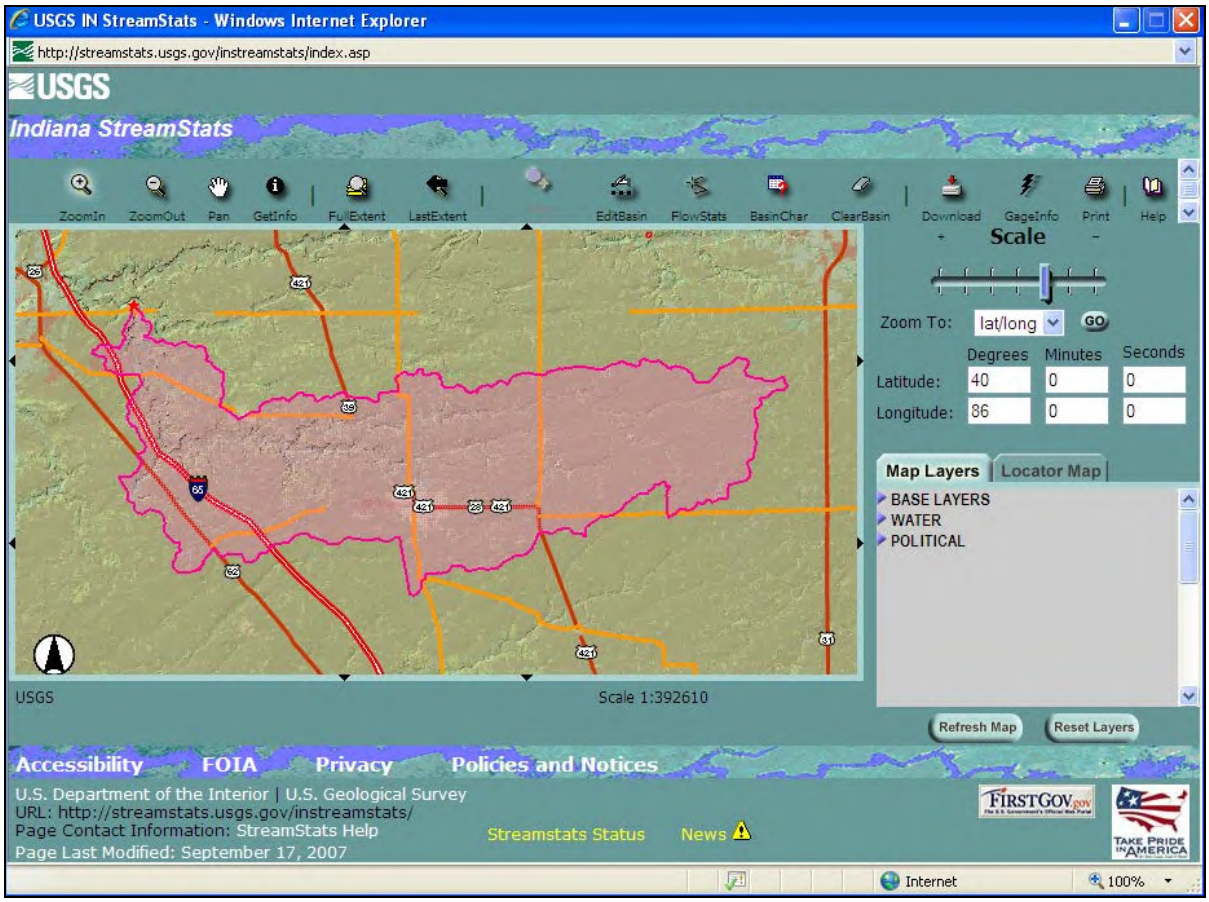


Figure 3.6.3
Basin Delineation for South Fork Wildcat Creek at Lafayette, IN

Step 4: Once the drainage area has been delineated, the flow statistics for the drainage area can be estimated. This is done by clicking on the "Flow Stats" Icon on the toolbar. A new window will appear that summarizes the basin and flow statistics for the delineated watershed, as shown in Figure 3.6.4.

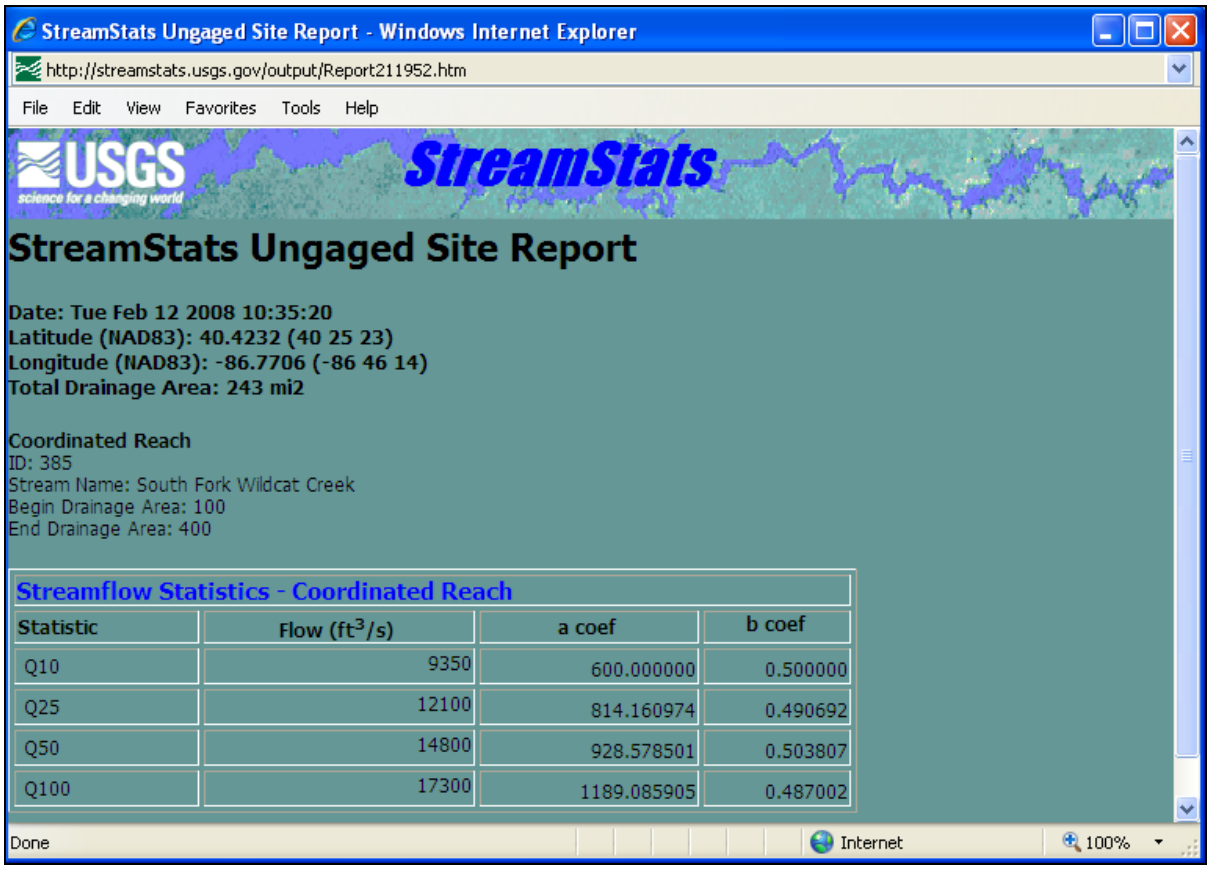


Figure 3.6.4
Peak Discharges for South Fork Wildcat Creek at Lafayette, IN

As shown in Figure 3.6.4, the 10- and 100-year flows for South Fork Wildcat Creek at Lafayette, Indiana are 9,350 and 17,300 cfs, respectively. Note that the total drainage area (243 square miles) is also listed in the output.

3.7 - COORDINATED DISCHARGES

The Corps of Engineers, USGS, SCS and Indiana Department of Natural Resources (IDNR) have coordinated the discharges of selected streams in Indiana (1990). This publication consists of discharge vs. drainage area curves which have been developed using available stream gage data. These data were statistically analyzed for the 10, 25, 50, and 100-year events and the peak flowrates at selected points (with a known drainage area) were plotted on log-log paper. A typical curve is shown on Figure 3.7.1.

This information should be used when performing Flood Insurance Studies or planning flood control projects and are useful for larger areas. For more information, contact the IDNR at the address at the end of the Reference section.

Example 3.7.1

Find the 10-year and 100-year flowrate for the South Fork Wildcat Creek at Lafayette using Figure 3.7.1 with a drainage area of 243 square miles.

From this exhibit the 10-year flowrate is approximately 9300 cfs and the 100-year flowrate is approximately 17,600 cfs.

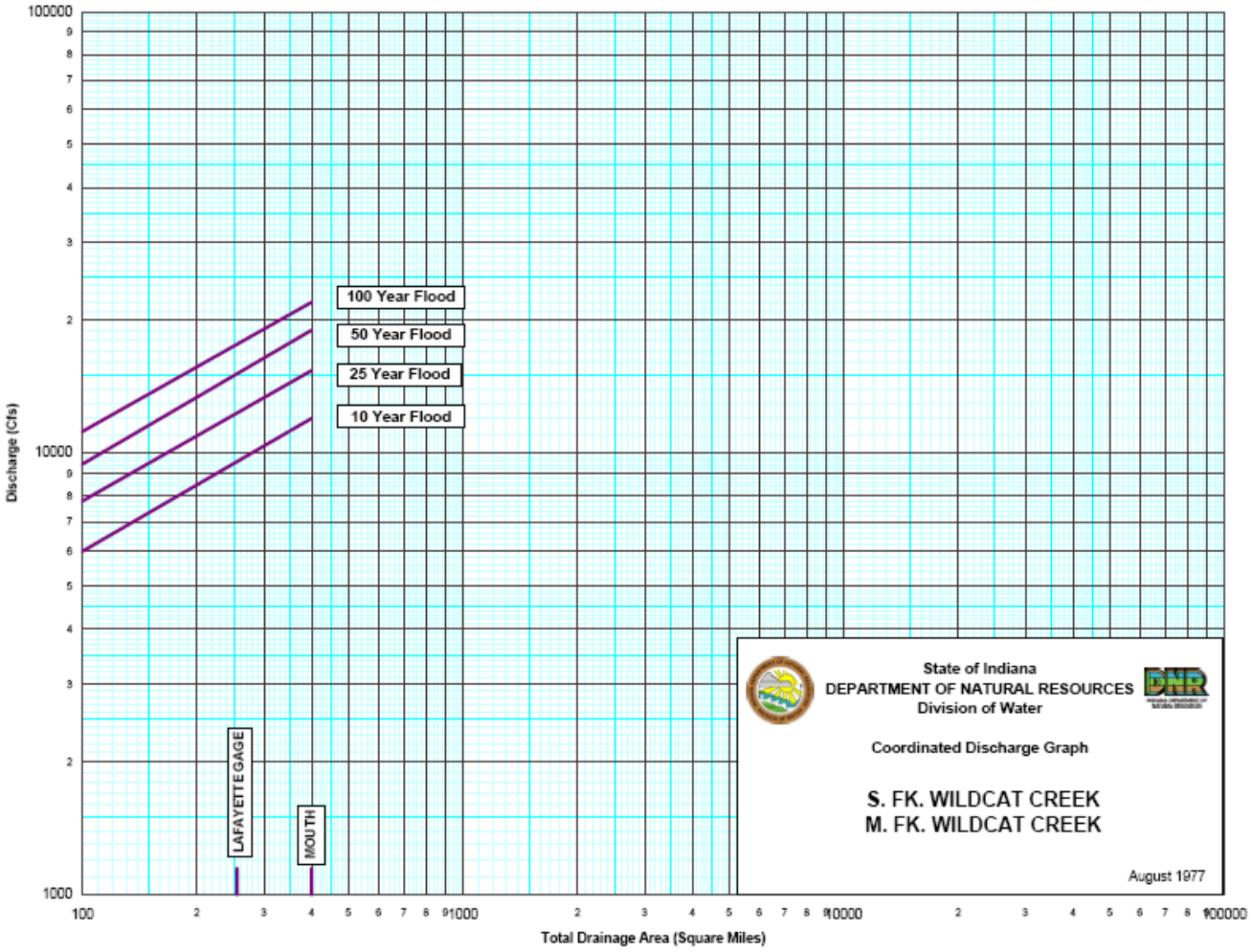


Figure 3.7.1
Coordinated Discharge Curve, Example 3.7.1 (IDOT DNR, 1990)

3.8 - FLOOD INSURANCE STUDIES

Flood Insurance Studies (FIS) have been performed by the Federal Emergency Management Agency (FEMA) for selected areas throughout Indiana to determine flood elevations for major waterways. As part of these studies discharges were calculated for various frequencies. These studies should be consulted when discharges are needed. For information on the areas where studies have been performed the FEMA maps can be obtained from the Indiana Department of Natural Resources, Division of Water. The current address is located at the end of the References.

Example 3.8.1

Find the 10-year and 100-year flowrate for the 243 mi^2 drainage area of the South Fork Wildcat Creek at Lafayette using the Flood Insurance Study by FEMA for Tippecanoe County, Unincorporated Areas. Compare these flowrates to those computed using the WRC method, regression equations and the coordinated discharges.

- A copy of the study is not included. From the tables at the beginning of the report the 10-year flowrate is 9,200 cfs and the 100-year flowrate is 17,500 cfs for the South Fork Wildcat Creek at Lafayette.

- The following table compares the results of Examples 3.5.1, 3.6.1, 3.7.1 and 3.8.1 which demonstrate four different approaches to finding the runoff from a drainage area. The values show fairly close results. The difference between the 100-year peak discharge computed by the Water Resource Council Method and the other methods is because the WRC Method calculations include the most recent yearly data that have some very high peak flows within the last 15 years. When the WRC Method is computed for the same number of years as the other methods the 10-year flowrate is 9,692 cfs and the 100-year flowrate is 18,652 cfs.

Table 3.8.1
Comparison of Results from Examples 3.5.1, 3.6.1, 3.7.1 and 3.8.1

Recurrence Interval	WRC Method	Regression Equations	Coordinated Discharges	Flood Insurance Study
10-Year	10,598 cfs	9,350 cfs	9,300 cfs	9,200 cfs
100-Year	19,754 cfs	17,300 cfs	17,600 cfs	17,500 cfs

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Coordinated Discharges of selected streams and Flood Insurance Rate maps can be obtained by writing to:

Department of Natural Resources, Division of Water
402 W. Washington St., Room W264
Indianapolis, Indiana 46204
(317) 232-4162

OR by visiting the Indiana Department of Natural Resources Webpage:
http://www.in.gov/dnr/water/surface_water/coordinated_discharges/index.html

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Chapter 4 - OPEN CHANNELS

LIST OF PARAMETERS

A	Cross-sectional area (ft^2 or m^2)
b	Bottom width of channel
B_S	Surface width of the channel
d	Depth of flow (normal to flow) (ft or m)
d_{max}	Maximum permissible depth of flow
D	Hydraulic depth (ft or m)
E	Specific energy (ft or m)
ΔE	Change in specific energy
F	Froude number
g	Acceleration constant due to gravity (ft/sec^2 or m/sec^2)
γ	Specific weight of water (lbf/ft^3 or N/m^3)
h_L	Head loss (ft or m)
K	Ratio of the tractive force
K´	Conveyance
K_b	Maximum shear on the bottom (lb/ft^2)
K_s	Maximum shear on the sides (lb/ft^2)
L	Length (ft or m)
n	Manning's roughness coefficient
n_E	Equivalent roughness coefficient
n_j	Roughness coefficient for a subsection
P	Pressure
P_w	Wetted perimeter
φ	Angle of the channel side-slope
Q	Volumetric flowrate (ft^3/sec or m^3/sec)
R	Hydraulic radius
R_O	Mean radius of the bend
S	Slope of the energy grade line (ft/ft or m/m)
S_o	Channel bed slope (ft/ft or m/m)
S_w	Water surface slope
T	Top width of channel
τ_b	Tractive force of channel bed (lb/ft^2 or
τ_o	Average bed shear force, tractive force (lb/ft^2 or
τ_s	Tractive force of channel side (lb/ft^2 or
θ	Angle of repose of soil
Δt	Time increment
V	Velocity (ft/sec or m/sec)
ΔV	Change in velocity (ft/sec or m/sec)
X	Composite length (ft or m)
ΔX	Increment of length (ft or m)
y	Depth of flow (vertically) (ft or m)

LIST OF PARAMETERS (cont'd)

y_c Critical depth
y_n Normal depth (ft or m)
z Elevation from datum (ft or m)
z Side-slope

Chapter 4 - OPEN CHANNELS

4.1 - INTRODUCTION

An open channel is a conveyance in which the liquid stream is not completely enclosed by solid boundaries. Thus the stream has a free surface subjected only to atmospheric pressure. Since the pressure is zero (gage pressure) at the surface, the energy available to cause fluid flow is due to changes in elevation from one section to another in the channel. The driving force for the flow is the component of the liquid weight along the slope of the channel. This driving force is resisted by a shear force transmitted from the boundaries to the liquid. The principle types of open channels are natural rivers and streams, artificial canals, drainage ditches, sewers, tunnels, and pipelines which are not completely filled.

The accurate solution of flow problems in open channels is difficult. Reliable experimental data are difficult to secure and there is a wide range of possible flow conditions. The cross-section of open channels may be of any shape, from circular to the irregular forms of natural streams. The channel surfaces vary greatly from concrete-lined channels to boulders, trees and other vegetation which occur in natural channels. In addition, the bed of the channel may be changing with time as erosion and deposition of bed materials occur during changing flow conditions.

The geometry and classification of open channels and the methods of flow computation, analysis and design are discussed in this chapter.

4.1.1 - Channel Geometry

A few definitions and relationships will be useful in the discussion of open channel flow. The term channel section refers to the cross-section of the channel normal to the direction of flow. Natural channel sections are irregular. Artificial channels (man-made) are usually designed with sections of a regular geometric shape. When these sections are regular, geometric properties of the section can be easily derived, based on the properties of the section and the depth of flow. If the section is irregular, these elements can be determined graphically.

The depth of flow, y, is the vertical distance from the liquid surface to the lowest point of the channel section. This depth of flow is often used interchangeably with the depth of the flow section, d. Strictly speaking, d is measured normal to the flow while y is measured vertically. It is only in channels with a steep longitudinal slope that there is a significant difference in the two values as seen in Figure 4.1.1.

The top width, T, is the width of the channel section at the free surface. The water area, A, is the cross-sectional area of the flow normal to the direction of flow. The wetted perimeter, P_w, is the length of the solid channel boundary in contact with the fluid in a given cross-section. The hydraulic radius, R, is the area divided by the wetter perimeter ($R = A/P_w$). The hydraulic depth, D, is the area divided by the top width ($D = A/T$). Table 4.1.1 presents all the geometric

elements for some commonly occurring sections.

4.1.2 - Classification of Flow in Open Channels

Since flow in open channels involves a free surface, it has many degrees of freedom. The classifications for these conditions include uniform and non-uniform (varied) flow; steady or unsteady flow; and supercritical (rapid) or subcritical flow (tranquil).

Uniform flow exists when the depth of flow does not change throughout a reach with a constant slope and cross section. Obviously, this condition rarely occurs in nature. For uniform flow to exist, the drop in potential energy due to the fall in elevation along the channel must be consumed, exactly, by the energy dissipation due to boundary friction and turbulence.

Non-uniform flow occurs when there is a change in depth due to a slope change, barrier or drop or a change in the cross-section so that the velocity increases or decreases in the direction of the flow. This type of flow is termed varied flow -- gradually varied flow if changes occur slowly, and rapidly varied if large changes occur in short distances in the channel.

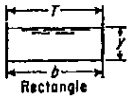
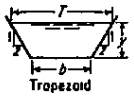
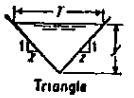
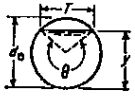
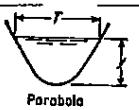
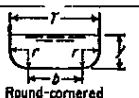
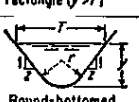
Steady flow occurs when the velocity at a point does not change with time -- that is $\Delta V/\Delta t = O$. When the flow is unsteady, $\Delta V/\Delta t \neq O$. Unsteady flows are difficult to analyze unless the change with time is sufficiently slow to permit a step type of analysis. Examples of unsteady flow are traveling surges and flood waves in an open channel.

The classification of flows as subcritical and supercritical flow is based on the Froude number ($F = V/\sqrt{(gD)}$), which is the ratio of the inertial force to that of gravity. If $F < 1.0$ the flow is subcritical (tranquil) flow, $F = 1.0$ it is critical flow and $F > 1.0$ it is supercritical (rapid) flow.

In summary, flow in open channels may be separated into three classifications:

1. uniform or non-uniform
2. steady or unsteady
3. subcritical or supercritical

Table 4.1.1
Geometric Elements of Channel Sections (Chow, 1988)

Section	Area A	Wetted perimeter P_w	Hydraulic radius R	Top width T	Hydraulic depth D	Section factor SF
 Rectangle	by	$b + 2y$	$\frac{by}{b+2y}$	b	y	$by^{1.5}$
 Trapezoid	$(b + zy)y$	$b + 2y\sqrt{1+z^2}$	$\frac{(b+zy)y}{b+2y\sqrt{1+z^2}}$	$b + 2zy$	$\frac{(b+zy)y}{b+2zy}$	$\frac{[(b+zy)y]^{1.5}}{\sqrt{b+2zy}}$
 Triangle	zy^2	$2y\sqrt{1+z^2}$	$\frac{zy}{2\sqrt{1+z^2}}$	$2zy$	$\frac{1}{2}y$	$\frac{\sqrt{2}}{2}zy^{2.5}$
 Circle	$\frac{1}{8}(\theta - \sin\theta)d_0^2$	$\frac{1}{2}\theta d_0$	$\frac{1}{4}\left(1 - \frac{\sin\theta}{\theta}\right)d_0$	$\frac{(\sin\frac{1}{2}\theta)d_0}{\text{or}}$ $2\sqrt{y(d_0-y)}$	$\frac{1}{8}\left(\frac{\theta - \sin\theta}{\sin\frac{1}{2}\theta}\right)d_0$	$\frac{\sqrt{2}}{32}\frac{(\theta-\sin\theta)^{1.5}}{(\sin\frac{1}{2}\theta)^{0.5}}d_0^{2.5}$
 Parabola	$\frac{2}{3}Ty$	$T + \frac{8}{3}\frac{y^2}{T}$ *	$\frac{2T^2y}{3T^2+8y^2}$ *	$\frac{3}{2}\frac{A}{y}$	$\frac{2}{3}y$	$\frac{2}{3}\sqrt{6}\,Ty^{1.5}$
 Round-cornered rectangle ($y > r$)	$\left(\frac{\pi}{2} - 2\right)r^2 + (b + 2r)y$	$(\pi - 2)r + b + 2y$	$\frac{(\pi/2-2)r^2+(b+2r)y}{(\pi-2)r+b+2y}$	$b + 2r$	$\frac{(\pi/2-2)r^2}{b+2r} + y$	$\frac{[(\pi/2-2)r^2+(b+2r)y]^{1.5}}{\sqrt{b+2r}}$
 Round-bottomed triangle	$\frac{T^2}{4z} - \frac{r^2}{z}(1 - z\cot^{-1}z)$	$\frac{T}{z}\sqrt{1+z^2} - \frac{2r}{z}(1 - z\cot^{-1}z)$	$\frac{A}{P}$	$2[z(y-r) + r\sqrt{1+z^2}]$	$\frac{A}{T}$	$A\sqrt{\frac{A}{T}}$

* Satisfactory approximation for the interval $0 < x \leq 1$, where $x = 4y/T$. When $x > 1$, use the exact expression $P = (T/2)[\sqrt{1+x^2} + 1/x \ln(x + \sqrt{1+x^2})]$.

4.1.3 - Energy Equation in Open Channels

In Figure 4.1.1 the geometric elements of open channel flow discussed in Section 4.1.1 are shown. If the energy equation is written between points 1 and 2 along the channel, the following relationship is found,

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma} + z_1 = \frac{V_2^2}{2g} + \frac{P_2}{\gamma} + z_2 + h_L \qquad (4.1.1)$$

where h_L = head loss through reach (ft) and γ = specific weight of water (lbs/ft^3).

Since atmospheric pressure acts on the free surface, the pressure terms becomes zero (gage pressure); and if the distance above the datum is written as (z + y), Equation 4.1.1 becomes:

$$\frac{V_1^2}{2g} + z_1 + y_1 = \frac{V_2^2}{2g} + z_2 + y_2 + h_L \qquad (4.1.2)$$

For small bed slopes (S_O) the values of z_1 and z_2 are essentially equal, this gives Equation 4.1.3.

$$\frac{V_1^2}{2g} + y_1 = \frac{V_2^2}{2g} + y_2 + h_L \qquad (4.1.3)$$

In Figure 4.1.1, S_w is defined as the water surface slope which for small slopes may be written as:

$$S_w = \frac{y_1 - y_2}{L} \qquad (4.1.4)$$

The slope of the energy grade line is observed to be:

$$S = h_L / L \qquad (4.1.5)$$

In uniform flow, the water surface slope (S_w), the energy slope (S) and the bed slope (S_O) are all equal to one another.

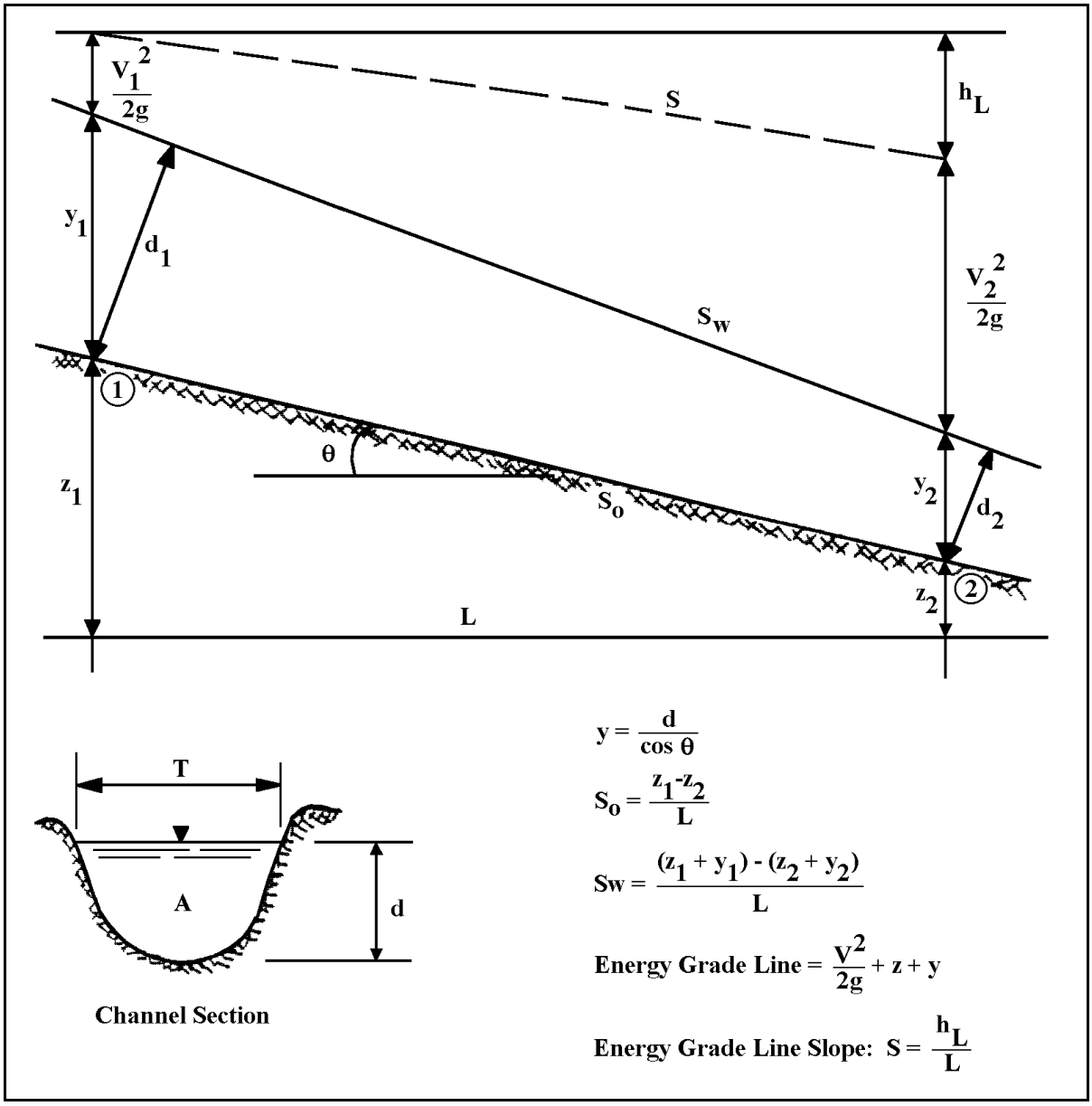


Figure 4.1.1
Properties of Open Channel Flow

4.2 - EQUATIONS FOR UNIFORM FLOW

One of the most widely used formulas for open channel flow is the Manning Equation. In English units, the formula is written as,

$$Q = \frac{1.49}{n}(R)^{2/3} A(S)^{1/2} \qquad (4.2.1)$$

where Q = volumetric flowrate (cfs), R = hydraulic radius (ft), A = cross-sectional area (ft^2), S = channel slope (ft/ft) or slope of the energy grade line (ft/ft), *n* = Manning's roughness coefficient.

(A nomograph solution of Equation 4.2.1 for circular pipe is given in Figure 4.4.3)

The same equation may be used with System International Units if the 1.49 factor is omitted. Table 4.2.1 includes various roughness factors for natural and man-made open channels.

It should be emphasized that strictly speaking, the Manning Equation applies only to uniform flow of water ($S = S_O = S_w$); although as will be shown later, it may also be applied to slowly varied non-uniform flow. Examples 4.2.1 and 4.2.2 help to illustrate the use of this equation.

When the Manning Equation is used for open channels, the computations involve the following six (6) variables:

1. The normal discharge, Q
2. The mean velocity of flow, V
3. The normal depth, y_n, associated with steady, uniform flow
4. The channel roughness coefficient, *n*
5. The channel slope, S
6. The geometric elements which depend upon the shape of the channel section, such as A, R, etc.

When any four of the above variables are given, the remaining two may be obtained using Equation 4.2.1 and the continuity equation, Q = AV. The following is a list of some of the possible problems in flow computation.

(a) The computation of the normal discharge, Q. In practical applications, this computation is required to determine the capacity of the channel section.

(b) To determine the velocity. This velocity is often required for the study of scouring and silting in channels.

(c) To compute the normal depth. This computation is necessary for the determination of the stage of flow in a given channel.

Table 4.2.1
Values of Manning's Roughness Coefficient, *n*
(Federal Highway Administration, 1961)

Table 1. Manning's Roughness Coefficients, n[1]

I. Closed Conduits	Manning's n Range[2]
A. Concrete pipes	0.011-0.013
B. Corrugated-metal pipe or pipe-arch.	
1. 2 2/3 by 1/2-in. corrugation (riveted pipe):[3]	
a. Plain or fully coated	0.024
b. Paved invert (range values are for 25 and 50 percent of circumference paved):	
(1) Flow full depth	0.018-0.021
(2) Flow 0.8 depth	0.016-0.021
(3) Flow 0.6 depth	0.013-0.019
2. 6 by 2-in. corrugation (field bolted)	0.03
C. Vitrified clay pipe	0.012-0.014
D. Cast-iron pipe, uncoated	0.013
E. Steel pipe	0.009-0.011
F. Brick	0.014-0.017
G. Monolithic concrete:	
1. Wood forms, rough	0.015-0.017
2. Wood forms, smooth	0.012-0.014
3. Steel forms	0.012-0.013
H. Cemented rubble masonry walls:	
1. Concrete floor and top	0.017-0.022
2. Natural floor	0.019-0.025
I. Laminated treated wood	0.015-0.017
J. Vitrified clay liner plates	0.015

II. Open Channels, Lined[4] (straight alignment):[5]	
A. Concrete with surfaces as indicated:	
1. Formed, no finish	0.013-0.017
2. Trowel finish	0.012-0.014
3. Float finish	0.013-0.015
4. Float finish, some gravel on bottom	0.015-0.017
5. Gunite, good section	0.016-0.019
6. Gunite, wavy section	0.018-0.022
B. Concrete, bottom float finished, sides as indicated:	
1. Dressed stone in mortar	0.015-0.017
2. Random stone in mortar	0.017-0.020

IV. Highway Channels and Swales with Maintained Vegetation[6] (values shown here are for velocities of 2 and 6 f.p.s.):	Manning's n Range[2]
A. Depth of flow up to 0.7 feet:	
1. Bermuda grass, Kentucky bluegrass, Buffalo grass:	
a. Mowed to 2-inches	0.045-0.07
b. Length 4-6 inches	0.05-0.09
2. Good stand, any grass:	
a. Length about 12-inches	0.09-0.18
b. Length about 24-inches	0.15-0.30
3. Fair stand, any grass:	
a. Length about 12-inches	0.08-0.14
b. Length about 24-inches	0.13-0.25
B. Depth of flow 0.7-1.5 feet:	
1. Bermuda grass, Kentucky bluegrass, Buffalo grass:	
a. Mowed to 2-inches	0.035-0.05
b. Length 4-6 inches	0.04-0.06
2. Good stand, any grass:	
a. Length about 12-inches	0.07-0.12
b. Length about 24-inches	0.10-0.20
3. Fair stand, any grass:	
a. Length about 12-inches	0.06-0.10
b. Length about 24-inches	0.09-0.17

V. Street and Expressway Gutters:	
A. Concrete gutter, troweled finish	0.012
B. Asphalt pavement:	
1. Smooth texture	0.013
2. Rough texture	0.016
C. Concrete Gutter with asphalt pavement:	
1. Smooth	0.013
2. Rough	0.015
D. Concrete pavement:	
1. Float finish	0.014
2. Broom finish	0.016
E. For gutters with small slope, where sediment may accumulate, increase above values of n by	0.002

Table 4.2.1 (cont.)
Values of Manning's Roughness Coefficient, *n*
(Federal Highway Administration, 1961)

B. Concrete, bottom float finished, sides as indicated:	
1. Dressed stone in mortar	0.015-0.017
2. Random stone in mortar	0.017-0.020
II. Open Channels, Lined[4] straight alignment):[5] (continued)	
3. Cement rubble masonry	0.020-0.025
4. Cement rubble masonry, plastered	0.016-0.020
C. Gravel bottom, sides as indicated:	
1. Formed concrete	0.017-0.020
2. Random stone in mortar	0.020-0.033
3. Dry rubble (riprap)	0.023-0.033
D. Brick	0.014-0.017
E. Asphalt:	
1. Smooth	0.013
2. Rough	0.016
F. Wood, planed, clean	0.011-0.013
G. Concrete-lined excavated rock:	
1. Good section	0.017-0.020
2. Irregular section	0.022-0.027
III. Open Channels, excavated[4] (straight alignment,[5] natural lining):	
A. Earth, uniform section:	
1. Clean, recently completed	0.016-0.018
2. Clean, after weathering	0.018-0.020
3. With short grass, few weeds	0.022-0.025
4. In gravelly soil, uniform section, clean	0.022-0.025
B. Earth, fairly uniform section:	
1. No vegetation	0.022-0.025
2. Grass, some weeds	0.025-0.030
3. Dense weeds or aquatic plants in deep channels	0.030-0.035
4. Sides clean, gravel bottom	0.025-0.030
5. Sides clean, cobble bottom	0.030-0.040
C. Dragline excavated or dredged:	
1. No vegetation	0.028-0.033
2. Light brush on banks	0.035-0.050
D. Rock:	
1. Based on design section	0.035
E. For gutters with small slope, where sediment may accumulate, increase above values of n by	0.002
VI. Natural stream channels:[8]	
A. Minor streams[9] (surface width at flood stage less than 100 ft.):	
1. Fairly regular section:	
a. Some grass and seeds, little or no brush	0.030-0.035
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035-0.05
c. Some weeds, light brush on banks	0.035-0.05
d. Some weeds, heavy brush on banks	0.05-0.07
e. Some weeds, dense willows on banks	0.06-0.08
f. For trees within channel, with branches submerged at high stage, increase all above values by	*0.01-0.02*
2. Irregular sections, with pools, slight channel meander; increase values given in 1 a-e about	*0.01-0.02*
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
a. Bottom of gravel, cobbles, and few boulders	0.04-0.05
b. Bottom of cobbles, with large boulders	0.05-0.07
B. Flood plains (adjacent to natural streams)	
1. Pasture, no brush:	
a. Short grass	0.030-0.035
b. High grass	0.035-0.05
2. Cultivated areas:	
a. No crop	0.03-0.04
b. Mature row crops	0.035-0.045
c. Mature field crops	0.04-0.05
3. Heavy weeds, scattered brush	0.05-0.07
4. Light brush and trees:[10]	
a. Winter	0.05-0.06
b. Summer	0.06-0.08
5. Medium to dense brush:[10]	
a. Winter	0.07-0.11
b. Summer	0.10-0.16
6. Dense willows, summer, not bent over by current	0.15-0.20

Table 4.2.1 (cont.)
Values of Manning's Roughness Coefficient, *n*
(Federal Highway Administration, 1961)

III. Open Channels, excavated[4] (straight alignment,[5] natural lining): (cont)	
E. Channels no maintained, weeds and brush uncut:	0.08-0.12
1. Dense weeds, high as flow depth----------	0.05-0.08
2. Clean bottom, brush on sides----------	0.07-0.11
3. Clean bottom, brush on sides, highest stage of flow----------	0.10-0.14
4. Dense brush, high stage----------	
VI. Natural stream channels:[8] (cont)	
8. Heavy stand of timber, a few down trees, little undergrowth:	
a. Flood depth below branches----------	0.10-0.12
b. Flood depth reaches branches----------	0.12-0.16
C. Major streams (surface width at flood stage more than 100 ft.): Roughness coefficient usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegatation on banks. Values of n may be somewhat reduced. Follow recommendation in publication cited[8] if possible. The value of n for larger streams of most regular section, with no boulders or brush, may be in the range of----------	0.028-0.033

Footnotes to Table 1

[1] *Estimates are by Bureau of Public roads unless otherwise noted.*

[2] *Ranges indicated for closed conduits and for open channels, lined or excavated, are for good to fair construction (unless otherwise stated). For poor quality construction, use larger values of n.*

[3] *Friction Factors in Corrugated Metal Pipe, by M.J. Webster and L.R. Metcalf, Corps of Engineers, Department of the Army; published in Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, vol. 85, No. HY9, Sept. 1959, Paper No. 2148, pp.35-67.*

[4] *For important work and where accurate determination of water profiles is necessary, the designer is urged to consult the following references and to select n by comparison of the specific conditions with the channel tested:*
Flow of Water in Irrigation and Similar Channels, by F.C. Scobey, Division of Irrigation, Soil Conservation Service, U.S. Department of Agriculture, Tech. Bull. No. 652, Feb. 1939; and
Flow of Water in Drainage Channels, by C.E. Ramser, Division of Agricultural Engineering, Bureau of Public Roads, U.S. Department of Agriculture, Tech. Bull. No. 129, Nov. 1929.

[5] *With channel of an alignment other than straight, loss of head by resistance forces will be increased. A small increase in the value of n may be made, to allow for the additional loss of energy.*

[6] *Handbook for Channel Design for Soil and Water Conservation, prepared by the Stillwater Outdoor Hydraulic Laboratory in cooperation with the Oklahoma Agricultural Experiment Station; published by the Soil Conservation Service, U.S. Department of Agriculture, Publ. No. SCS-TP-61, Mar. 1947, rev. June 1954*

[7] *Flow of Water in Channels Protected by Vegetative Linings, by W.O. Ree and V.J. Palmer, Division of Drainage and Water Control, Research, Soil Conservation Service, U.S. Department of Agriculture, Tech. Bull. No. 967, Feb. 1949.*

[8] *For calculation of stage or discharge in natural stream channels, it is recommended that the designer consult the local District Office of the Surface Water Branch of the U.S. Geological Survey, to obtain data regarding values of n applicable to streams of any specific locality. Where this procedure is not followed, the table may be used as a guide. The values of n tabulated have been derived from data reported by C.E. Ramser (see footnote 4) and from other incomplete data.*

[9] *The tentative values of n cited are principally derived from measurements made from fairly short, but straight, reaches of natural streams. Where slopes calculated from flood elevations along a considerable length of channel, involving meanders and bends, are to be used in velocity calculations by the Manning formula, the value of n must be increased to provide for the additional loss of energy caused by the bends. The increase may be in the range of perhaps 3 to 15 percent.*

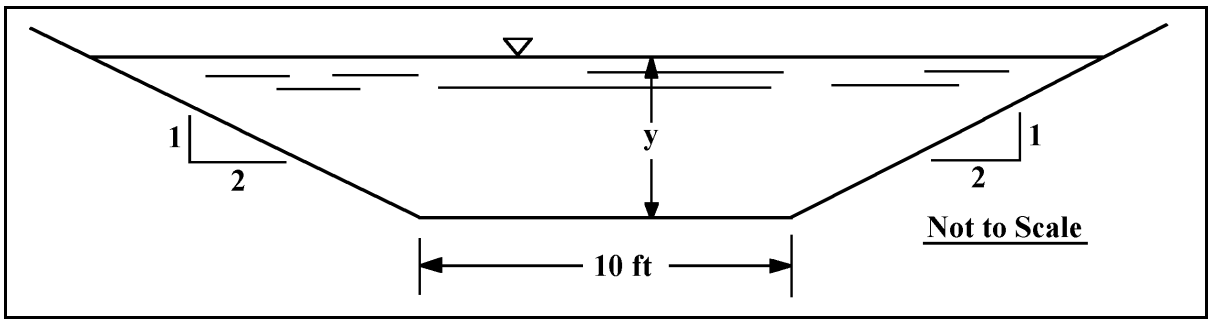
[10] *The presence of foliage on trees and brush under flood stage will materially increase the value of n. Therefore, roughness coefficients for vegetation in leaf will be larger than bare branches. For trees in channel or on banks, and for brush on banks where submergence of branches increases with depth of flow, n will increase with rising stage.*

(d) The determination of the channel roughness. This computation is used to ascertain the roughness coefficient in a given channel which can then be used in similar channels.

(e) The computation of the channel slope. This is needed for the adjustment of the slope in a given channel.

(f) The determination of the channel dimensions. This is needed in the design of open channels.

Example 4.2.1



Determine the normal depth for uniform flow in the trapezoidal channel shown below with a flowrate of 225 cfs, a slope of 0.0007 ft/ft and a Manning *n* of 0.015
From Table 4.1.1;

$$A = (b + zy)\, y$$

$$P_w = b + 2y\sqrt{1 + z^2}$$

$$R = A/P$$

$$A = (10 + 2y)y$$

$$P_w = 10 + 2y\sqrt{1 + 2^2} = 10 + 2y\sqrt{5}$$

$$R = \frac{(10 + 2y)y}{10 + 2y\sqrt{5}}$$

Hence,

$$Q = 225 = \frac{1.49}{0.015}(10 + 2y)y\left[\frac{10 + 2y)y}{10 + 2y\sqrt{5}}\right]^{2/3}(0.0007\ )^{1/2}$$

or

$$225 = 2.63(10 + 2y)y\left[\frac{(10 + 2y)y}{10 + 2y\sqrt{5}}\right]^{2/3}$$

solving by trial and error gives a normal depth $y_n = y = 3.16$ ft.

Example 4.2.2

Determine the normal discharge for a trapezoidal channel with a bottom width of 10 feet and side slopes of 2.5 on 1. The depth of flow is 5.4 ft. and the bottom slope, $S_O = 0.0008$. Assume that the channel has a concrete lining with a trowel finish.
From Table 4.2, $n = 0.012$, from Table 4.1.1 the area is

$$A = by + zy^2 = (10)(5.4) + 2.5(5.4)^2$$
$$A = 54 + 72.9 = 126.9 \text{ ft}^2$$

The wetted perimeter is:

$$P = b + 2y\sqrt{1 + z^2} = 10 + 2(5.4)\sqrt{1 + 2.5^2} \quad P = 39.08 \text{ ft.}$$

$$R = \frac{A}{P_w} = \frac{126.9}{39.08} = 3.25 \text{ ft.}$$

Substituting in Equation 4.2.1 gives:

$$Q = \frac{1.49}{(0.012)}(126.9)(3.25)^{2/3}(0.0008)^{1/2}$$
$$Q = 977.84 \text{ cfs}$$

4.2.1 - Specific Energy

Specific energy in a channel section is defined as the energy per weight of water flowing at any section of a channel, measured with respect to the channel bottom. Specific energy may be reduced to a dimension of feet. From Equation 4.1.2, it can be shown that the total energy at a section is equal to the sum of the potential and kinetic energy, if losses are neglected.

$$\frac{V^2}{2g} + y + z = CONSTANT$$

If the datum is taken at the channel bottom, the specific energy E is given by

$$E = y + \frac{V^2}{2g} \qquad (4.2.3)$$

which indicates that the specific energy is equal to the sum of the velocity head and the water depth. From the continuity equation (V = Q/A) we can rewrite Equation 4.2.3 as:

$$E = y + \frac{Q^2}{2g(A)^2} \qquad (4.2.4)$$

It is easily shown therefore, that the specific energy for a given channel and discharge is a function of the depth of flow only.

If the flow depth is plotted against the specific energy for a given channel section and discharge, a specific energy curve is obtained (Figure 4.2.2). This curve shows that for a given specific energy, there are two possible depths, called alternate depths, which correspond to that specific energy. It is also observed that at point C, a minimum specific energy occurs. It will be seen later that this point corresponds to a critical state of flow. At this point, the two alternate depths of flow apparently become one and the specific energy is at a minimum. This depth is termed the critical depth y_c.

When the depth of flow is larger than the critical depth, the velocity of flow is less than the critical velocity, and this flow is called subcritical. It should be noted that at or near the critical depth, the flow is unstable. This is because a minor change in the specific energy in this region will cause a major change in depth. Thus flow near the critical depth is often wavy. When the depth of flow is smaller than the critical depth, the velocity of flow is larger than the critical velocity, and this flow is called supercritical.

If the discharge changes, the specific energy will change accordingly as shown in Figure 4.2.2. It will be seen later that as the channel slope is increased, for a given discharge and cross-sectional shape, the flow will change from subcritical, to critical and finally to supercritical.

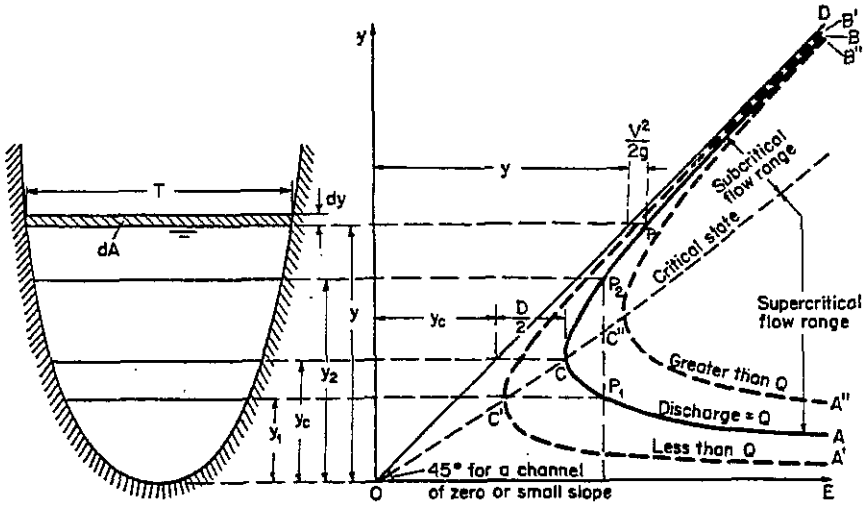


Figure 4.2.1
Specific-Energy Curve (Chow, 1988)

4.2.2 - Criterion for Critical State of Flow

The critical state of flow has been defined as the condition for which the Froude number is equal to 1 (Section 4.1.2). The Froude number is dimensionless. The Froude number equal to 1 occurs when the specific energy is a minimum for a given discharge. A theoretical criterion for critical flow may be developed as follows:

If Equation 4.2.4 is differentiated with respect to y, and noting that Q is a constant, Equation 4.2.5 is obtained

$$\frac{dE}{dy} = \frac{Q^2}{gA^3}\frac{dA}{dy} = 1 - \frac{V^2}{gA}\frac{dA}{dy} \tag{4.2.5}$$

The differential water area dA near the surface is equal to Tdy. We have seen that the hydraulic depth D = A/T and since dA/dy = T, Equation 4.2.5 becomes:

$$\frac{dE}{dy} = 1 - \frac{Q^2 T}{gA^3} = 1 - \frac{Q^2}{gA^2 D} = 1 - \frac{V^2}{gD} \tag{4.2.6}$$

At the critical state of flow, the specific energy is a minimum (dE/dy = 0).

$$\frac{V^2}{2g} = \frac{D}{2} \tag{4.2.7}$$

This is the criterion for critical flow, which simply states that at the critical flow state the velocity head is equal to one half the hydraulic depth. Equation 4.2.7 may also be written as $V^2/gD = 1$ (which as before states that the Froude number is equal to one).

Solving Equation 4.2.4 in a different manner for dE/dy = 0, the following expression is obtained.

$$\frac{Q^2}{gA^2 D} = 1 = \frac{Q^2 T}{gA^3} \qquad (4.2.8)$$

Solving for Q gives

$$Q = \left[\frac{g\,A^3}{T}\right]^{1/2} = \left[g\,A^2\,D\right]^{1/2} \qquad (4.2.9)$$

Equation 4.2.9 demonstrates that at critical conditions, the flow depends upon the flow depth only, and is not dependent upon the slope, roughness or water properties.

4.2.3 - Computation of Critical Flow Depth

The critical depth, y_c, is obtained by solving for the flow depth that satisfies Equation 4.2.9. For simple geometric shapes, the solution is relatively easy to determine. However, several iterations may be required to find the solution for an irregularly shaped channel such as a natural streambed. In fact, several valid critical depths may exist for irregular channels. The calculation of critical depth, y_c, is demonstrated in Examples 4.2.3 and 4.2.4.

Example 4.2.3

For the channel given in Example 4.2.1, calculate the critical depth for a flow of 225 cfs and *n* = 0.015 by trial and error, and by using Figure 4.2.3.

As before:

$A = (10 + 2y)y \qquad T = 10 + 4y$

$P_w = 10 + 2y\sqrt{5} \qquad D = \frac{A}{T} = \frac{(10 + 2y)y}{(10 + 4y)}$

$R = A/P_w$

From Equation 4.2.9

$$Q = 225 = \left[gA^2 D\right]^{1/2} = \frac{32.2^{1/2}\,[(10+2y)y\,]^{3/2}}{(10+4y\,)^{1/2}}$$

Solving by trial and error a critical depth, y_c, is found to be 2.16 ft.

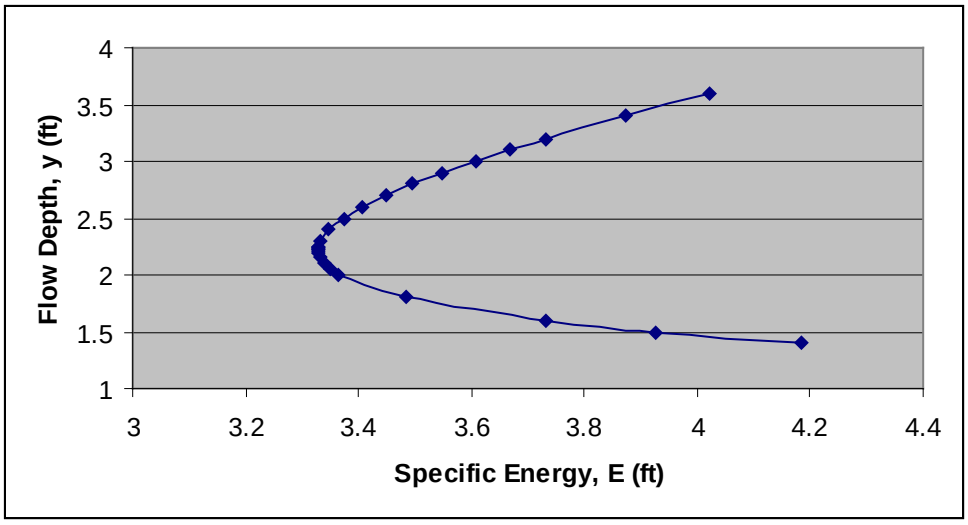
Example 4.2.4

An 8 ft. wide rectangular channel conveys a flow of 150 cfs. The flow depth at a section is 3.5 ft. Draw the specific energy diagram and find the critical depth and the minimum specific energy.

A simple way to construct the specific energy diagram is to use Equation 4.2.4. For a rectangular channel, area A = by = 4y, and Q is given as 150 cfs. Substituting these values into Equation 4.2.4 we obtain

$$E = y + \frac{Q^2}{2g(A\,)^2} = y + \frac{150^2}{2*32.2*(8y)^2} = y + \frac{5.459}{y^2}$$

For different values of y, we can find the corresponding values of E as shown.



The critical depth is obtained from Equation 4.2.9. For a rectangular channel, top width T is the same as the bottom width, b.

$$Q = \left[\frac{g\,A^3}{T}\right]^{1/2} \Rightarrow 150cfs = \left[\frac{32.2*(8y)^3}{8}\right]^{1/2} \Rightarrow 10.92 = y^3 \Rightarrow y = 2.22\,ft$$

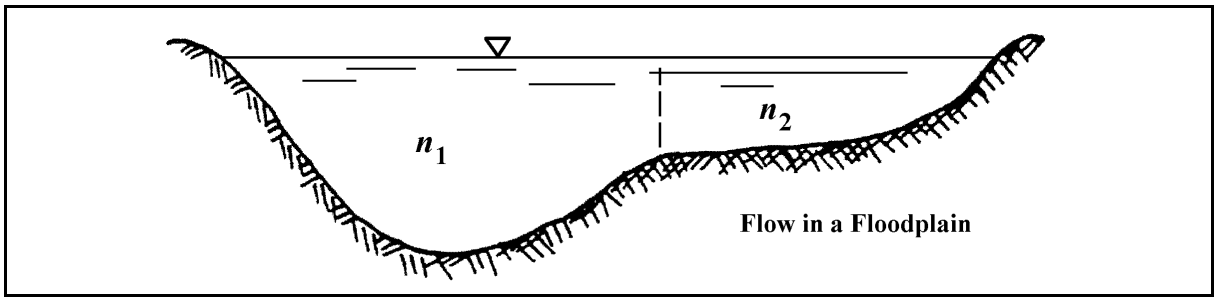
The minimum specific energy corresponding to this critical flow depth is

$$E_{\min} = y + \frac{Q^2}{2g(A)^2} = 2.22 + \frac{150^2}{2 * 32.2 * (8 * 2.22)^2} = 3.33\,ft$$

These values are also reflected in the graph, where the minimum specific energy of 3.33 ft corresponds to the flow depth of 2.22 ft.

4.2.4 - Steady Uniform Flow in a Floodplain

A practical open channel problem of importance is the computation of discharge through a natural floodplain. The figure below shows a hypothetical case.



In general, the floodplain is rougher than the river channel, and the depth is much less. The slopes of the energy grade-line for the two are the same and the discharge is determined for each section separately. Using the dashed line in the above figure, the discharges for each section are computed separately to determine the total discharge of the system.

Since both sections have the same slope, the total discharge may be expressed as:

$$\begin{aligned} Q &= Q_1 + Q_2 \\ &= K'_1\sqrt{S} + K'_2\sqrt{S} \\ &= (K'_1 + K'_2)\sqrt{S} \end{aligned} \qquad (4.2.11)$$

$$where\ K'_i = \frac{1.49}{n_i} A_i R_i^{2/3} \qquad (4.2.12)$$

For a particular section shape, K´ is a function of Mannings' *n* and the depth only. Example 4.2.5 illustrates the use of these equations.

The floodway is defined as the channel of a stream plus any adjacent floodplain areas, which must be kept free of encroachment so that the 100-year flood can be carried without substantial increases in flood heights. Minimum Federal standards limit such increases to 1.0 foot, provided that hazardous velocities are not produced (FEMA, 1984). In Indiana, however, state regulation limits such increases in flood heights to 0.14 feet. The portion of the floodway outside of the floodway but still subject to flooding is referred to as "floodway fringe."

Example 4.2.5

Determine the total discharge for the floodplain shown below. The channel slope is 0.0006 ft/ft. Compute the value of K´ for each section.

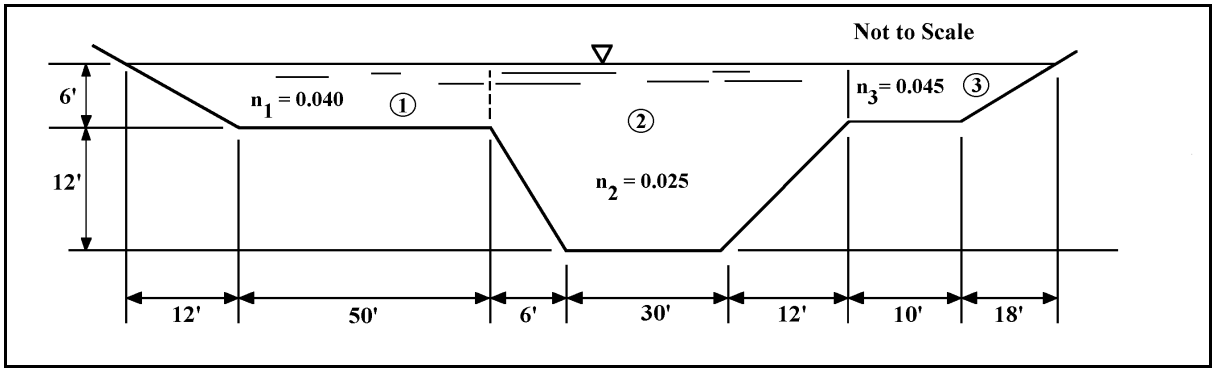
$$K'_i = \frac{1.49}{n_i} A_i R_i^{2/3}$$

* Section 1

$A_1 = 50 \times 6 + 1/2(12 \times 6) = 336 \text{ ft}^2$
$P_{w1} = 13.42 \text{ ft} + 50 \text{ ft} = 63.42 \text{ ft}$

$$R_1 = \frac{A_1}{P_w 1} = \frac{336\ ft^2}{63.42 ft} = 5.30\ ft$$

$$K'_1 = \frac{1.49}{0.040}(336)(5.30)^{2/3} = 38{,}046.62$$



* Section 2

$A_2 = 6(30 + 6 + 12) + 12(30) + 1/2(12)(12) + 1/2(6)(12) = 756 \text{ ft}^2$
$P_{w2} = 16.97 + 30 + 13.42 = 60.39 \text{ ft}$

$$R_2 = \frac{A_2}{P_w 2} = \frac{756\ ft^2}{60.39\ ft} = 12.52\ ft$$

$$K'_2 = \frac{1.49}{0.025}(756)(12.52)^{2/3} = 242{,}942.94$$

*Section 3

$A_3 = (10)(6) + 1/2(18)(6) = 114\ ft^2$
$P_{w3} = 10 + 18.97 = 28.97\ ft$

$$R_3 = \frac{A_3}{P_w 3} = \frac{114}{28.97} = 3.94\ ft$$

$$K'_3 = \frac{1.49}{0.045}(114)(3.94)^{2/3} = 9{,}416.21$$

The total flow therefore is equal to:

$$Q = (K'_1 K'_2 + K'_3)\sqrt{S} = (38046.58 + 242944.8 + 9416.21)\sqrt{0.0006} = 7{,}113.50\ cfs$$

4.3 - DESIGN CRITERIA FOR UNIFORM FLOW

4.3.1 - Site Selection

Drainage channels provide surface drainage and must be placed where they will adequately perform their drainage functions. In rural areas, as well as urban areas, topography, location of highways, structures, and other obstacles largely fix the location, alignment, and grade of the channel and determine the quantity of surface water entering it.

Design of the channel is dependent upon two principles. The first principle dictates that the design be in harmony with the physical laws and properties that control such items as topography, construction materials, and the hydraulics itself. Although the first two items are rather obvious, the last one has hidden implications. Although a cross-section can be determined to handle flows for normal conditions, more than often, these same cross-sections will be used for conditions that do not strictly apply. Designing a channel with a sharp curve using the exact same design section as the straight-run portion could lead to erosion of the curve to the point of failure. Assuming the flow as being uniform (in fact it is gradually varied) could in some instances result in the channel being overtopped somewhere upstream.

The second principle points out that the design layout of the drainage system requires knowledge and experience relating to both construction and maintenance. Construction involves the initial capital investment relating suitable channel sections, necessary safety precautions, types of channel lining for erosion protection, and aesthetics. Maintenance assures that the desired function of the channel survives. Consideration of both costs and benefits must be evaluated before any final decision may be concluded.

The actual layout of the drainage system should preferably be made on a topographic map which contains the location of all obstructions, and the accentuated ridge and drainage lines. In most cases it is sufficient to follow the natural grade lines of the drainage paths while in some instances these natural drainages must be intercepted and altered.

The approximate grade of the channel is computed from the topographic map. The grade affects both the size of the channel required to carry a given flow and the velocity at which the flow occurs. The flow should be subcritical whenever possible. Changes in channel alignment should be as gradual as the width of right-of-way and terrain permit. Whenever practicable, changes in alignment should be made in the reaches of the channel which have flatter slopes, particularly if the flow becomes supercritical on the steeper slopes.

Selection of the type of channel lining used; grass, earth, or artificial, can influence the location. An impermeable lined channel could be constructed in a sandy area whereas another type could not. The eventual selection must be based on the function or purpose of the channel itself. This choice must consider alternate proposals that achieve the same goals.

4.3.2 - Channel Cross-section Selection

For a channel conveying a given discharge for a given slope and Manning's roughness coefficient, some cross-section shapes are more efficient than others. When a channel is constructed, the costs are a function of the excavation and lining, both of which should be minimized. Based on the Manning equation, it can be shown that when the area of cross-section is minimum, the wetted perimeter is also minimum. This implies that both lining and excavation costs approach their minimum value for the same dimensional channel. Therefore, the best hydraulic section for a lined channel is one that has the least wetted perimeter, or its equivalent, and the least area for the type of section.

The geometric elements of six best hydraulic sections are listed below in Table 4.3.1. These sections may not always be practical due to the difficulties in construction and in the use of materials. In general, a channel selection by use of the best hydraulic efficiency gives a good starting point for design. Modifications for practicability, safety, or other imposed limits could be applied to these cross-sections. Because Table 4.3.1 is not in a directly usable form, and due to the fact that most channels are often designed in the shape of trapezoids, a nomograph method of design is presented in this section under "Design Procedures" (Section 4.3.5) that will increase the efficiency of the designer. The nomograph procedure is also helpful because the "best" section can be determined at various side-slopes that deviate from the optimum.

Table 4.3.1
Best Hydraulic Sections (Chow, 1988)

Cross section	Area A	Wetted perimeter P	Hydraulic radius R	Top width T	Hydraulic depth D	Section factor Z
Trapezoid, half of a hexagon	$\sqrt{3}\,y^2$	$2\sqrt{3}\,y$	$\frac{1}{2}y$	$\frac{1}{3}\sqrt{3}\,y$	$\frac{3}{4}y$	$\frac{3}{2}y^{2.5}$
Rectangle, half of a square	$2y^2$	$4y$	$\frac{1}{2}y$	$2y$	y	$2y^{2.5}$
Triangle, half of a square	y^2	$2\sqrt{2}\,y$	$\frac{1}{4}\sqrt{2}\,y$	$2y$	$\frac{1}{2}y$	$\frac{\sqrt{2}}{2}y^{2.5}$
Semicircle	$\frac{\pi}{2}y^2$	πy	$\frac{1}{2}y$	$2y$	$\frac{\pi}{4}y$	$\frac{\pi}{4}y^{2.5}$
Parabola, $T = 2\sqrt{2}\,y$	$\frac{4}{3}\sqrt{2}\,y^2$	$\frac{8}{3}\sqrt{2}\,y$	$\frac{1}{2}y$	$2\sqrt{2}\,y$	$\frac{2}{3}y$	$\frac{8}{9}\sqrt{3}\,y^{2.5}$
Hydrostatic catenary	$1.39586y^2$	$2.9836y$	$0.46784y$	$1.917532y$	$0.72795y$	$1.19093y^{2.5}$

One other factor might affect final cross-section selection. The best hydraulic section gives the minimum excavation only if the water surface is at the level of the bank top. Where the water surface is below the bank top, channels narrower than those of the best hydraulic section will result in minimum excavation.

Figure 4.3.1 gives freeboard heights that will take into account factors such as wave action due to wind that create the need for such additional depths of channels. Figure 4.3.1 also provides recommendations on the height of any artificial lining that may be used. Freeboard is recommended for all channel designs.

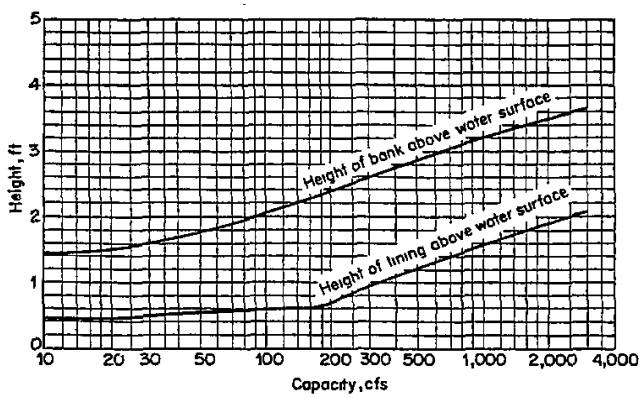


Figure 4.3.1
Recommended Freeboard and Height of Bank for Lined Channels
(U.S. Bureau of Reclamation, 1952)

Up to this point, the principle of best hydraulic section has been applied only to the design of non-erodible lined channels. In channels of this nature, high velocities may be tolerated. For erodible channels, the best hydraulic section must be modified to take into account the limits placed on excessive velocities.

In addition to the above criteria relating to channel cross-section selection, other factors must be considered. In urban areas, safety consideration may dictate the maximum side slopes and depths of flow which are desirable. If the channel is grassed and mowing of this side slope is necessary, the maximum slope is dictated by the ease with which it can be maintained.

4.3.3 - Roughness Coefficients

The greatest difficulty in using any of the various formulas for calculating the discharge capacity of channel, is the determination of a roughness coefficient. Since this manual makes extensive use of the Manning equation and because Manning's roughness coefficient *n* may be related to all other coefficients, the following discussion applies to selecting an appropriate value for Manning's *n* from Table 4.2.1.

A selection of a value of *n* implies that one is attempting to estimate the resistance to flow in a given channel. Before one can make any decision on that selection, the various factors that affect resistance must be understood. The first factor must be surface roughness. In one sense, surface roughness is represented by the size and shape of the grains of the material forming the wetted perimeter and producing a retarding effect on the flow. In a lined channel it is based on the artificial surface; whereas in a natural channel, it is based on the composition of grain sizes that make up the bed and sides.
Vegetation may also be regarded as a kind of surface roughness since it also produces a retarding effect on the flow. This effect is a function of the height, density, distribution, and type of vegetation. Any natural or artificial phenomena that affect the pattern or state of growth would also affect the roughness coefficients. The most prominent would be seasonal change. In this instance, *n* may increase in the growing season and diminish in the dormant season.

Obstructions in the channel are also regarded as a surface roughness, but not directly linked to the channel itself. Such debris may be removed, but in any case the effect is identical: retardation of the flow. The amount of increase in *n*, and therefore decrease in flow, depends on the nature of the obstructions, their size, shape, number, and distribution. A side effect of obstructions, other than surface roughness, is the creation of vortices and eddies that would cause an increase in turbulence and scour, and therefore, a decrease in available energy for transportation of flow. Such conditions would appear as an increase in the value of *n*.

Moving suspended material and bed load requires energy above that necessary to transport the water itself. The direct effect of the consumption of energy is to cause an additional head loss and, therefore, cause an apparent increase in channel roughness. Even when the bed load is not moving, the increase in obstructions causes increased turbulence. In either case, one would expect an increase in the value of *n*.

Up to this point, the channel alignment was considered straight. Smooth curvature with large radii will provide a relatively low increase of *n*, whereas sharp curvature with severe meandering will provide high increases of *n*. The degree of meandering of a channel course dictates the final correction along with the type of material used. For artificial channels it has been suggested that the value of *n* be increased 0.001 for each 20 degrees of curvature in 100 feet of channel. The meandering of natural streams, on the other hand, may cause extensive variation of *n*. For severe meandering the *n* value is usually increased by 30 percent, whereas for appreciable meandering the increase is 15 percent.

For natural channels, the coefficient of roughness is a function of stage and discharge. When the stage and discharge are low, the irregularities of the channel are exposed and their effects pronounced. As the stage increases, the sides or banks could relatively increase or decrease the roughness, depending on the composition and presence of obstructions. If water stage becomes too high, the water will flow out of its banks and thus change the relative roughness again. As described in an earlier section, it is best to break the channel into separate sections and simply find a mean for the value of *n*.

4.3.4 - Design Practices

Open channels may be broadly classified into rigid, lined, non-erodible channels and erodible channels which may be lined with grass, rip-rap, or even the bare soil itself. The design procedures for these two classes are distinctly different. Non-erodible, lined channels are the easiest to design. The major factors to consider are the kind of material forming the channel lining, the minimum velocity to avoid deposition if the water carries silt or debris, the channel bottom slope and side slopes, the freeboard, and the most efficient section.

The purpose of the lining is usually to prevent erosion, but in some instances, it may be employed to check seepage losses. Normally, excessive velocities can be ignored unless the water carries sand or gravel that may erode the strongest of channel linings, or unless the velocity is so high that there is a tendency of the water to pick up and move lining material. Such high velocities are extremely dangerous from the standpoint of safety also.

The minimum velocity is the lowest velocity that will prevent either the start of sedimentation or the growth of aquatic plants and moss. This velocity is very uncertain and exact values are dependent upon many factors. What has generally been accepted is to maintain a minimum mean velocity of three feet per second when the percentage of silt is small.

Channel bottoms and side slopes are usually restricted by the amount of space that is available. In populated areas, flat side slopes are safer on two counts. If someone should fall into the channel it is easier for that person to get out if the side slopes are flat. Also with flat side slopes, any vehicle that should stray into the channel may exit with minimal damage to the driver or the channel. The final selection of the side slopes and bottom width will dictate the depth of flow in the channel. This may be important as far as aesthetics or other intangible design inputs.

The freeboard of a channel is the additional vertical distance from the design water surface to the top of the channel. Besides providing additional capacity, its major role is to provide sufficient height to prevent waves or fluctuation in water surface from overflowing the sides. While no universally accepted rule exists for the determination of freeboard, the curves presented in Figure 4.3.1 represent a reasonable approach.

Figure 4.3.1 also provides a guide to the height of lining that should extend above the water surface. If the underlying soil is susceptible to erosion, then the channel lining or some other stabilizing material must continue to the top of the channel.

The design of erodible channels presents the engineer with the greatest challenge. Most channels, for cost reasons, must be unlined and it is necessary therefore that they be so proportioned as to prevent silting or scouring in objectionable quantities. The velocities of flow at all points of the cross-section must be sufficient to transport through the channel all sediment that enters it; however, the velocities must be low enough, at all points, to prevent scouring of erodible linings.

The present state of sedimentation theory is inadequate to permit a precise design for channels, but it is advanced far enough to give one a workable methodology. Channel design however, must still be based in large measure on engineering experience and judgement.

A method based on the concept of tractive force has been developed by engineers and consultants of the U.S. Bureau of Reclamation. The rationale of the solution is simple. When water flows in a channel, a force is exerted that acts in the direction of flow on the channel bed. This force, which is simply the pull of water on the wetted surface, is known as the tractive force.

Tractive force can be shown to equal

$$\tau_o = \gamma RS \qquad (4.3.2)$$

where τ_o = the average bed shear force, tractive force (lb/ft^2).

This is the average unit tractive force whereas the magnitude actually varies along the wetted perimeter as shown in Figure 4.3.2.

The coefficients K_S and $K_{b'}$ for maximum shear on the sides and bottom, respectively, depend on the width-depth ratio, b/d, and the side-slope *z*. However, for channels of ordinary size and shape (where b/d is greater than 3.0) the values of K_S and K_b may be taken as 0.75 and 1.0, respectively.

Thus, the maximum tractive force on the side-slope is only about 3/4 that on the bottom. This might imply that only the bottom is the critical point and one can ignore sides. However, motion of soil particles on the side-slope is also assisted by gravity, so that the resultant effect of tractive force and gravitational force is often such as to cause the critical point for incipient scour to be on the side rather than the bottom.
It can be shown that the ratio of the tractive force on the sides to that on the bottom is equal to:

$$K = \tau_s / \tau_b \tag{4.3.3}$$

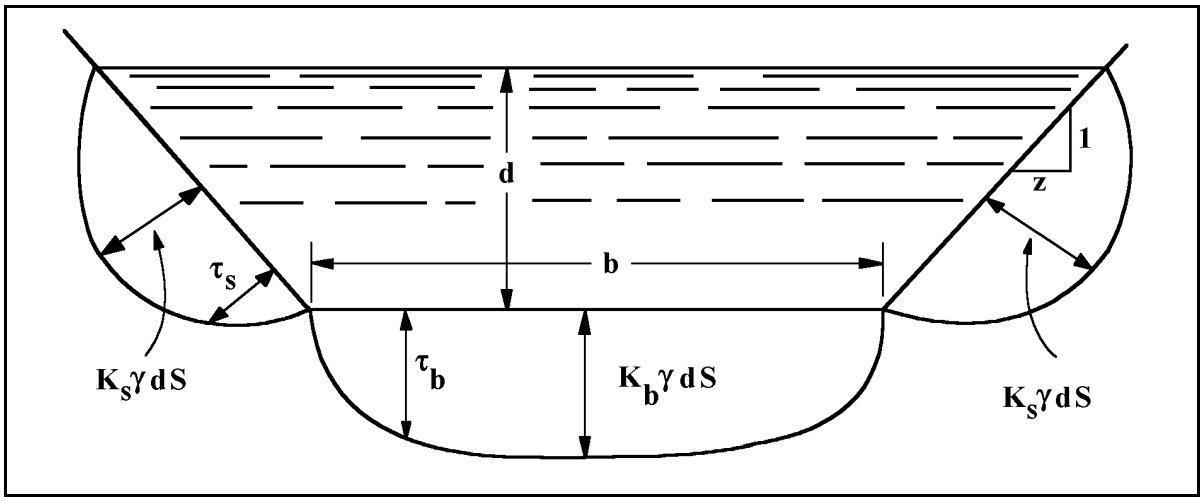


Figure 4.3.2
Tractive Stress Distribution on Channel Bed

$$K = \left[1 - \frac{\sin^2 \phi}{\sin^2 \theta} \right]^{1/2} \tag{4.3.4}$$

where φ = angle of the side-slope and θ = angle of repose for the soil.

Therefore, by calculating the permissible tractive force on the bottom, it is easy to determine the permissible tractive force on the sides. Equating this with the maximum tractive force on the channel section, a workable design can be developed.

A modification to the tractive force approach for erodible channel design is the concept of maximum permissible depth of flow. An examination of Equation 4.3.2 indicates that the

maximum tractive force that can exist without erosion occurring for a particular lining material depends directly on the hydraulic radius. Since the hydraulic radius and depth are related, it is possible to define a maximum permissible depth of flow which can occur before erosion takes place. This approach, which is a more convenient design procedure, will be used in this manual.

For wide channels of any shape (and for a given channel slope, depth, and lining), the vertical velocity distribution in the central and deepest section, where wall effects are negligible, should be identical. Also, the first scour occurs at the deepest portion of the channel since the wall or bottom shear stress is greatest in that portion.

If the depth of flow, channel slope, lining, and soil are the same in both channels shown in Figure 4.3.3, then the flow rate and the mean channel velocity for the two channels will be different. But in the central section of both channels, represented by ΔX, the velocity distribution and bottom shear stress will be nearly identical. Therefore, in both channels, there exists a limiting depth of flow above which scour will occur, and this depth, d_{max}, is the same for all wide channels of the same longitudinal slope, lining, and underlying soil. This concept has been verified by tests at Mississippi State University (1968), the Louisiana Department of Highways (1971) and by Anderson (1973). Thus, for unlined channels and rip-rapped channels, charts can be prepared which display the maximum depth of flow as a function of the channel slope and the degree of erodibility of the soil or the size of the rip-rap used. Such charts, which are taken from Normann (1975), are shown in Figures 4.3.4 and 4.3.5. The use of these charts is illustrated by Examples 4.3.1 and 4.3.2.

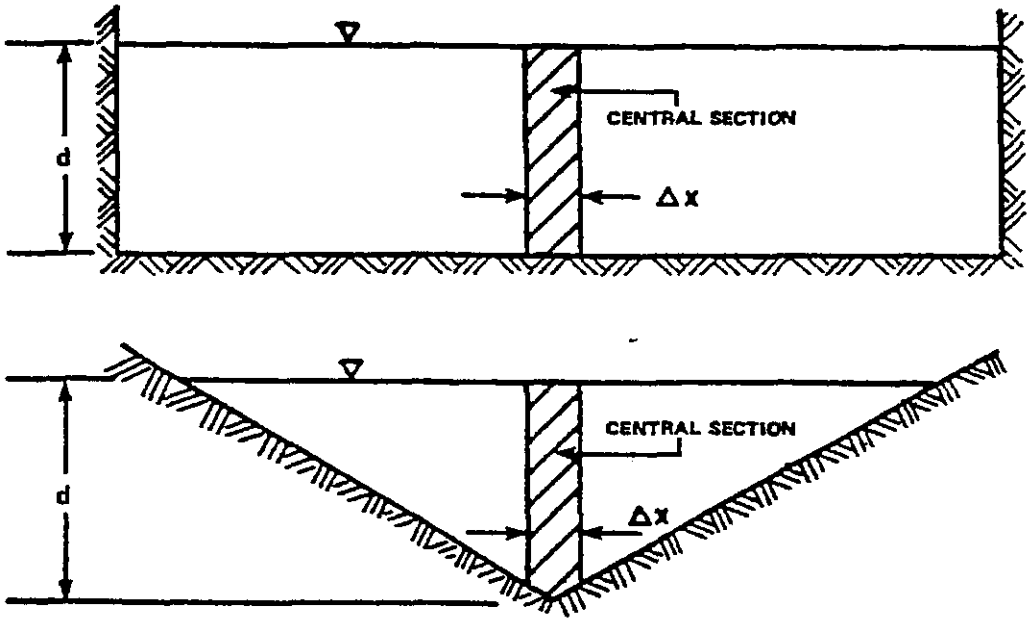


Figure 4.3.3
Schematic Diagram of Channels of Different Shapes (Norman, 1975)

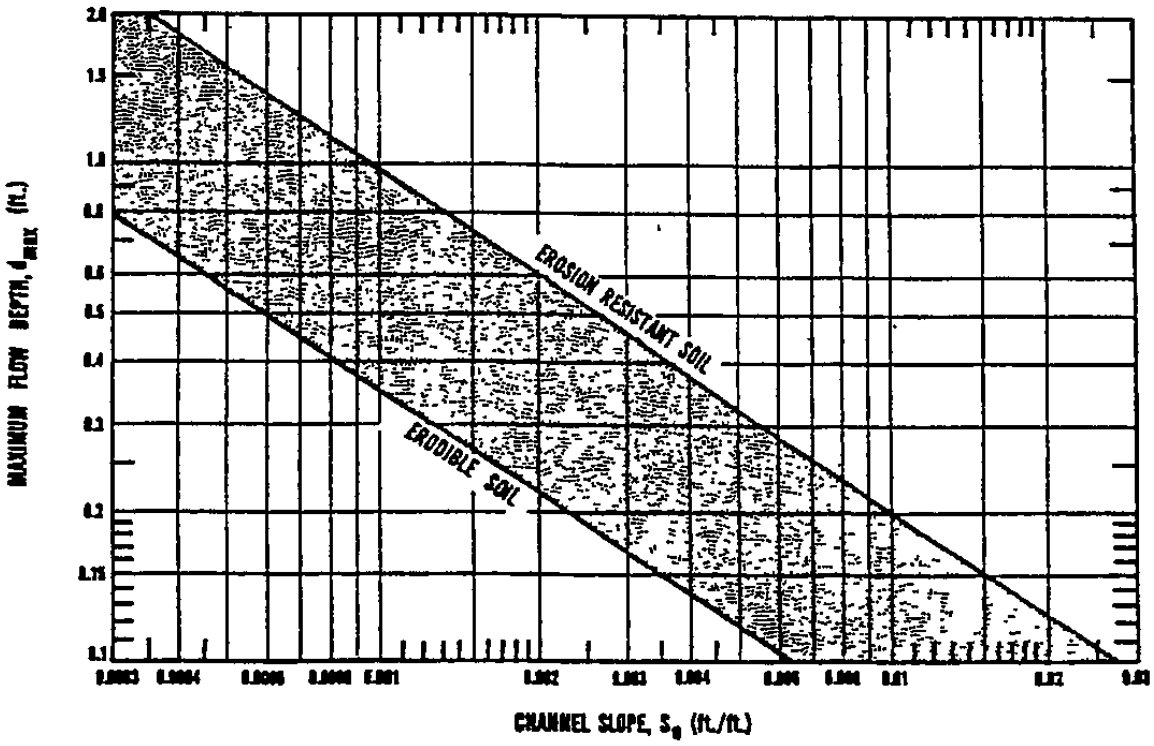


Figure 4.3.4
Maximum Permissible Depth of Flow (d_{max}) for Unlined Channels (Normann, 1975)

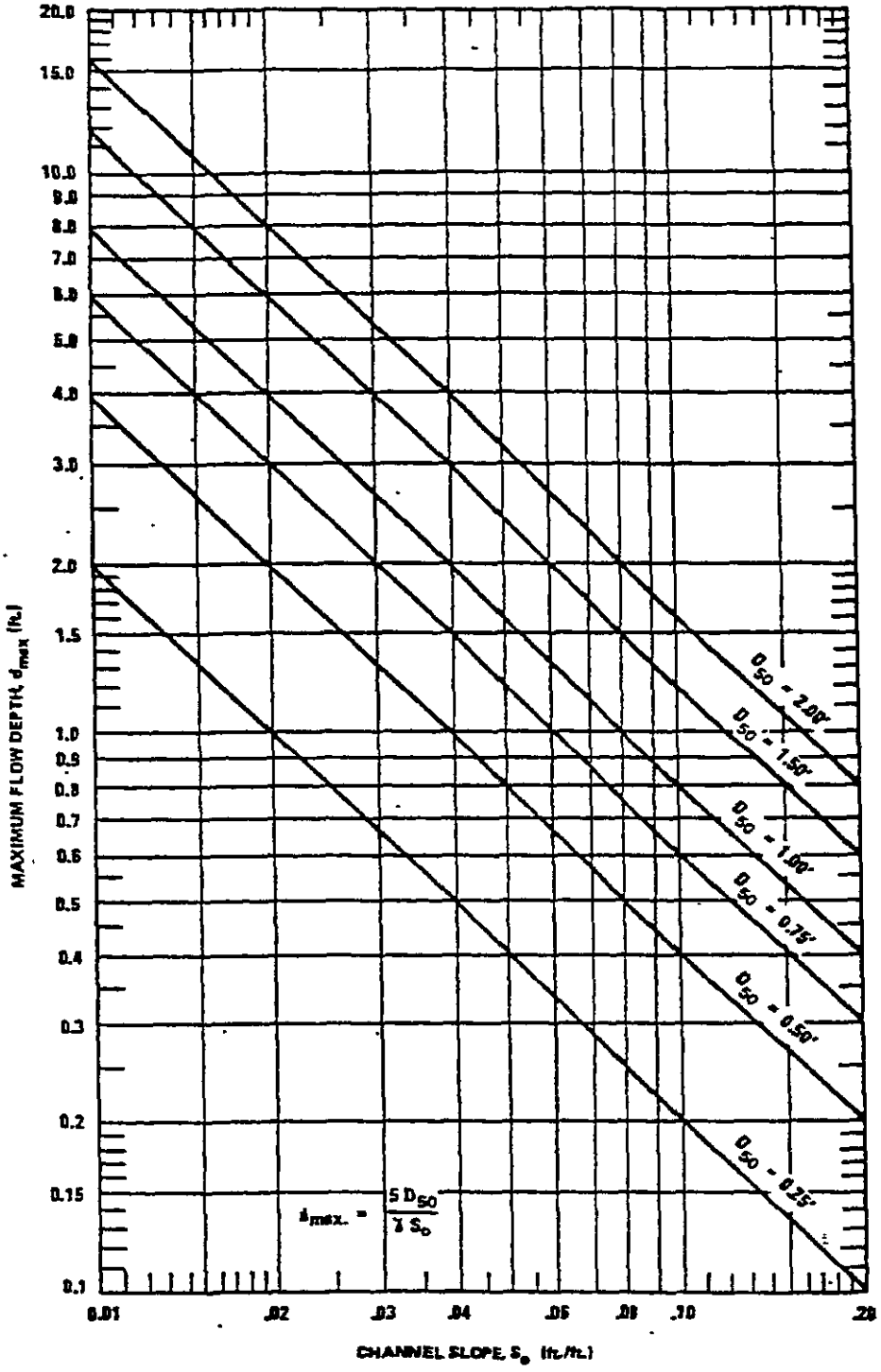


Figure 4.3.5
Maximum Permissible Depth of Flow (d_{max}) for
Channels lined with Rock Rip-rap (Normann, 1975)

Note: D_{50} is the particle size of gradation, of which 50% of the mixture is finer by weight.

The design of a grass-lined channel proceeds in somewhat the same manner. The resistance to

flow in a grass-lined channel depends both upon the type of grass and its length. Work by the Soil Conservation Service (1954) has resulted in a classification scheme for grassed channels known as the degree of retardance. These groupings for various grasses, ranging from type A with very high retardance through type E with very low retardance, are shown in Table 4.3.2.

Table 4.3.2
Classification of Degree of Retardance for Various Kinds of Grasses (Chow, 1988)

Retardance	Cover	Condition
A Very high	Weeping love grass	Excellent stand, tall (av 30 in)
	Yellow bluestem ischaemum	Excellent stand, tall (av 36 in.)
B High	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall (av 12 in.)
	Native grass mixture (little bluestem, blue grama, and other long and short Midwest grasses)	Good stand, unmowed
	Weeping love grass	Good stand, tall (av 24 in)
	Lespedeza sericea	Good stand, not woody, tall (av 19 in.)
	Alfalfa	Good stand, uncut (av 11 in.)
	Weeping love grass	Good stand, mowed (av 13 in.)
	Kudzu	Dense growth, uncut
	Blue grama	Good stand, uncut (av 13 in)
C *Moderate*	Crab grass	Fair stand, uncut (10 to 48 in.)
	Bermuda grass	Good stand, mowed (av 6 in.)
	Common lespedeza	Good stand, uncut (av 11 in.)
	Grass-legume mixture—summer (*orchard grass, redtop,* Italian rye grass, and common lespedeza)	Good stand, uncut (6 to 8 in.)
	Centipede grass	Very dense cover (av 6 in.)
	Kentucky bluegrass	Good stand, headed (6 to 12 in.)
D Low	Bermuda grass	Good stand, cut to 2.5 in height
	Common lespedeza	Excellent stand, uncut (av 4.5 in.)
	Buffalo grass	Good stand, uncut (3 to 6 in.)
	Grass-legume mixture—fall, spring (orchard grass, redtop, Italian rye grass, and common lespedeza)	Good stand, uncut (4 to 5 in)
	Lespedeza sericea	After cutting to 2 in height, very good stand before cutting
E Very low	Bermuda grass	Good stand, cut to 1.5 in. height
	Bermuda grass	Burned stubble

* U.S. Soil Conservation Service [41].

The ideal grass would be one that grows rapidly, but not over one inch in height, and develops a strong, deep-root system quickly. The selection of grass for a channel lining depends on the climate and the soil in which the plant is to survive. The development of the root system, which is dependent upon the individual plant, will add stability to the channel. In some cases it will be necessary to introduce two different types of grasses. One might be for fast establishment until a more permanent grass might grow out. It is most important to consider the condition of the grass after development. If the channel will be frequently cut, then a lower retardance or smaller roughness coefficient could be used in the Manning equation. This implies more capacity for the same cross-section.

Once the type of grass is selected, curves such as shown in Figure 4.3.7 (Normann, 1975) can be used to establish the maximum permissible depth of flow for some retardance classifications.

Again, according to the investigation by the Soil Conservation Service, it was found that the mean velocity of flow was related to the hydraulic radius R and the channel slope S_O. The curves which are presented in Figures 4.3.8 - 4.3.12 illustrate this relationship for the 5 types of vegetation retardance.

Example 4.3.1

Design a channel lining for a trapezoidal channel with a 4 foot bottom width and 4:1 side slopes. Design the permanent lining for a 10-year recurrence interval runoff. The soil has average erodibility. The channel top width is restricted to 12 feet. Channel slope is 1.5 percent. Prepare design analysis for several different permanent linings (bare soil, redtop grass, 3" rip-rap and concrete).

Detailed calculations are shown in Figure 4.3.6.

Note that the bare soil would convey very little on the 1.5 percent slope. Redtop grass 6 inches long is the best lining investigated.

DRAINAGE CHANNEL LINING DESIGN

DATE: 8/28/80

PROJECT: EXAMPLE 4.3.1 DESIGNER: CBB

STATION: _____ TO STATION: _____ CHECKED BY: _____

DRAINAGE AREA = 25 ACRES DATE: _____

HYDROLOGIC COMPUTATIONS:

Given: Q = 18 cfs (for a 10-year return period)

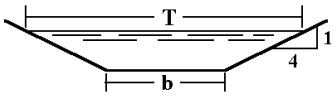
DESIGN FLOW: Q 10 = 18 cfs

DESIGN FLOW FOR TEMPORARY LINING: Q --- = --- cfs

SOIL ERODIBILITY: Average

CHANNEL DESCRIPTION: MAX. TOP WIDTH = 12 ft

$T = b + (z_1 + z_2) * d = 4 + 8d$



S_o = 0.015

AVAILABLE LININGS:
1. Bare Soil
2. Redtop Grass (4")
3. Riprap (3")
4. Concrete

	LINING	d_{max}	b	A	R	V	Q = AV	T	REMARKS
1.	Bare Soil	0.12	4	0.54	0.107	1.83	0.99	4.96	no good
2.	Redtop Grass	0.90	4	6.84	0.60	2.70	18.47	11.2	OK
3.	Riprap (3")	1.3	4	11.96	0.81	5.11	60.48	14.4	no good
4.	Concrete	1.0*	4	8	0.65	10.53	84.25	12	over designed
									high velocity
		* No d_{max}							

Figure 4.3.6
Sample Calculations for Example 4.3.1 (Normann, 1975)

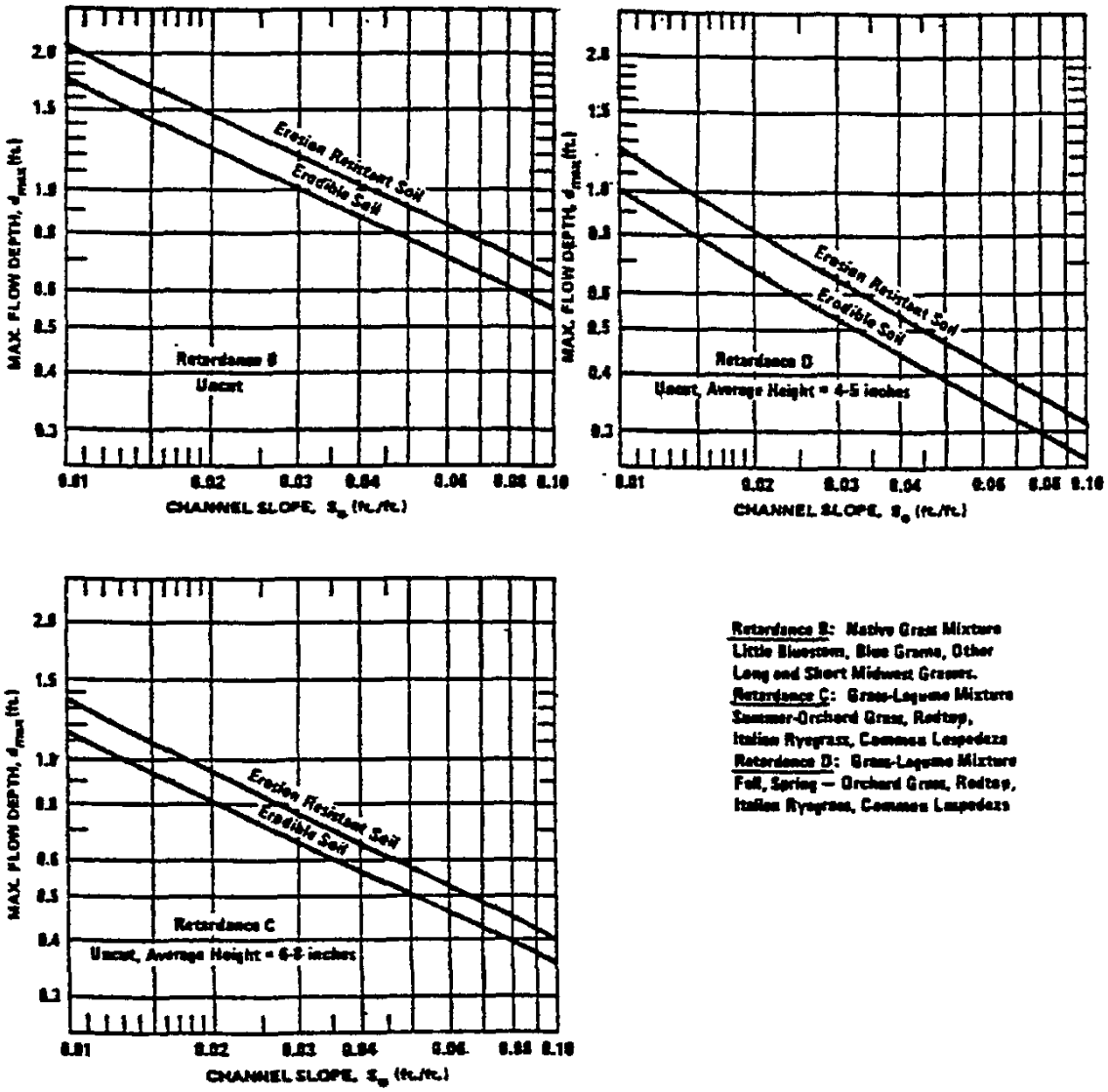


Figure 4.3.7
Maximum Permissible Depth of Flow (d_{max}) for Channels lined with Grass Mixtures, Good Standing, Uncut. (Normann, 1975)

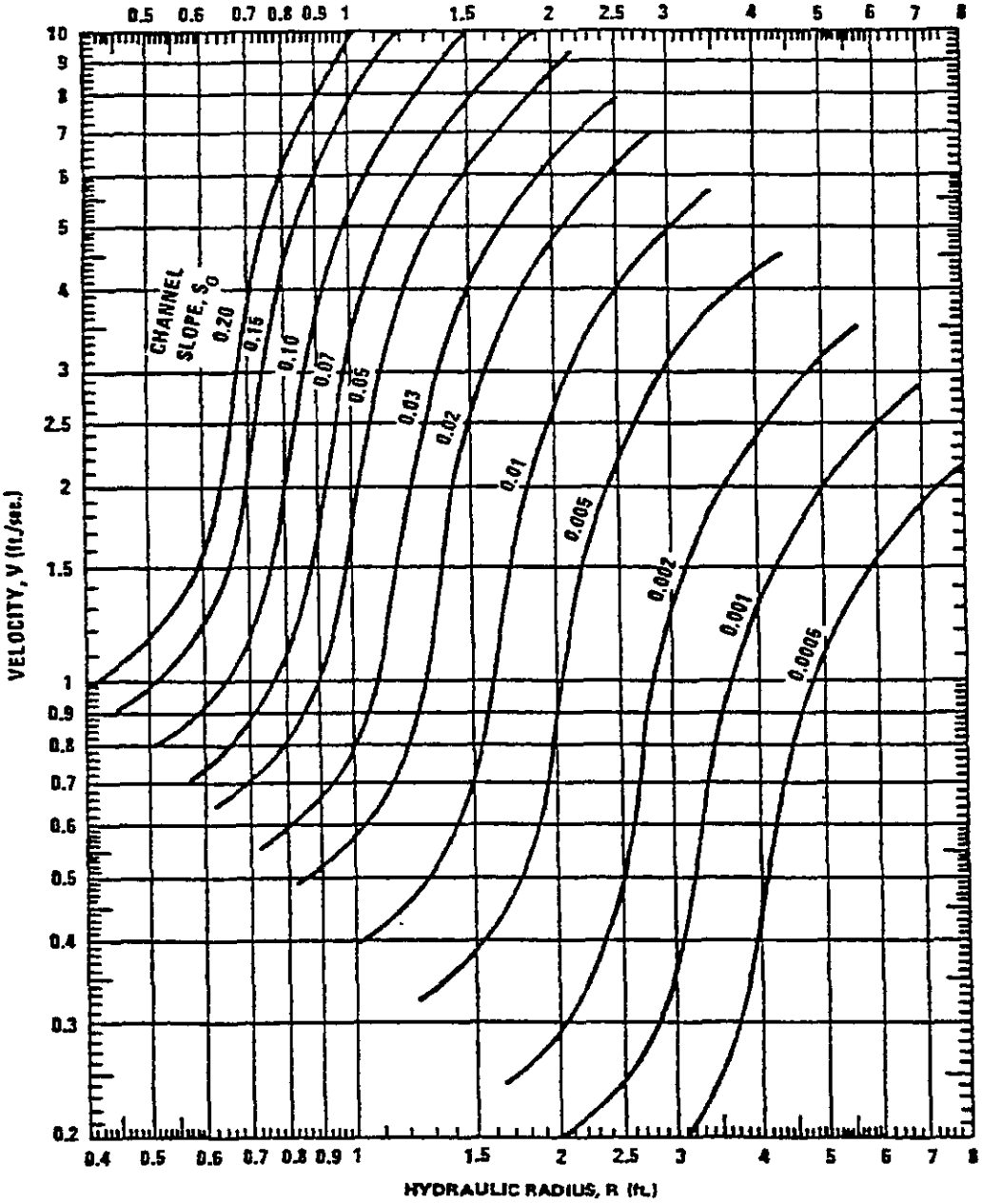


Figure 4.3.8
Flow Velocity for Channels Lined with Vegetation of Retardance A (Normann, 1975)

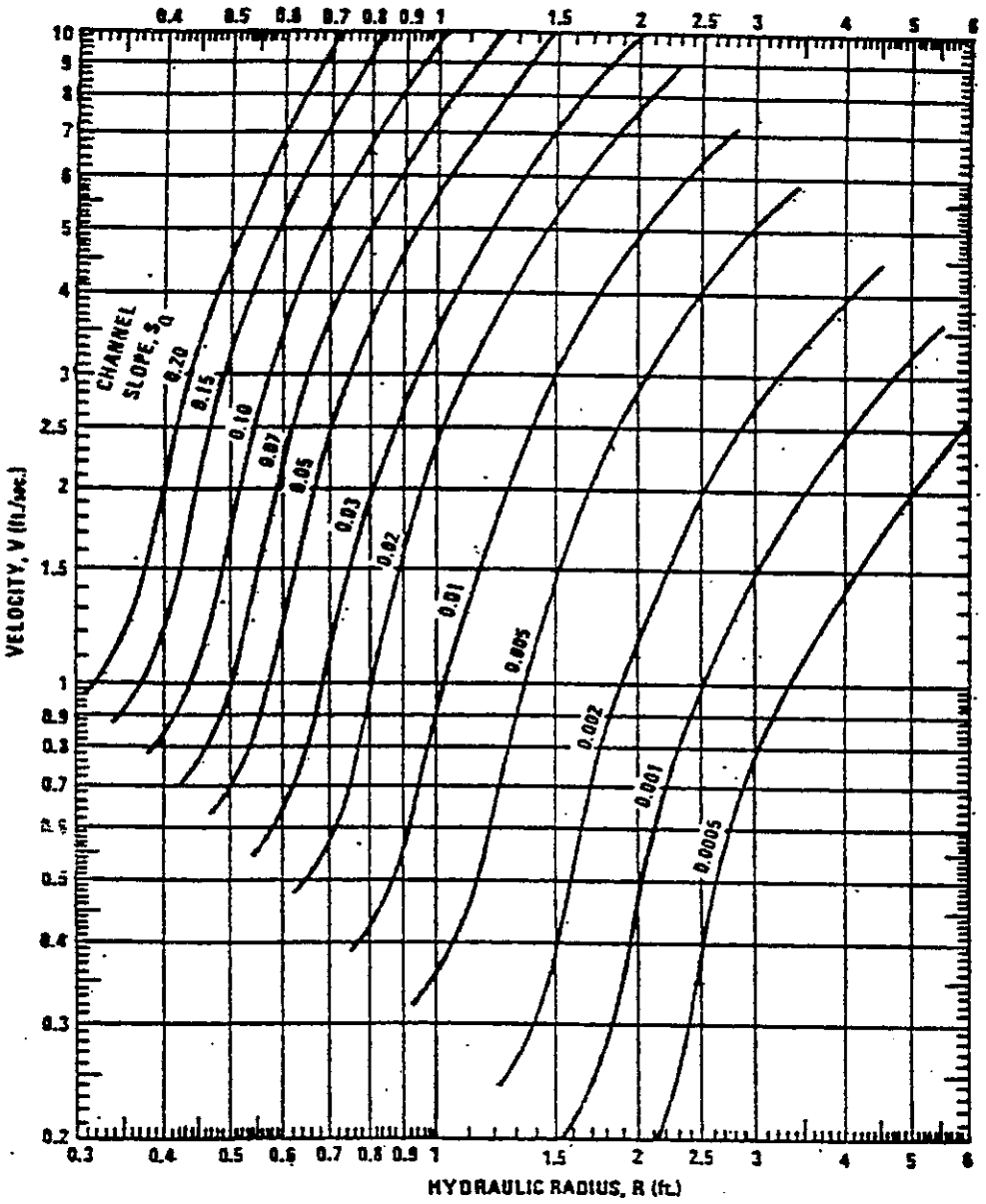


Figure 4.3.9
Flow Velocity for Channels Lined with Vegetation of Retardance B (Normann, 1975)

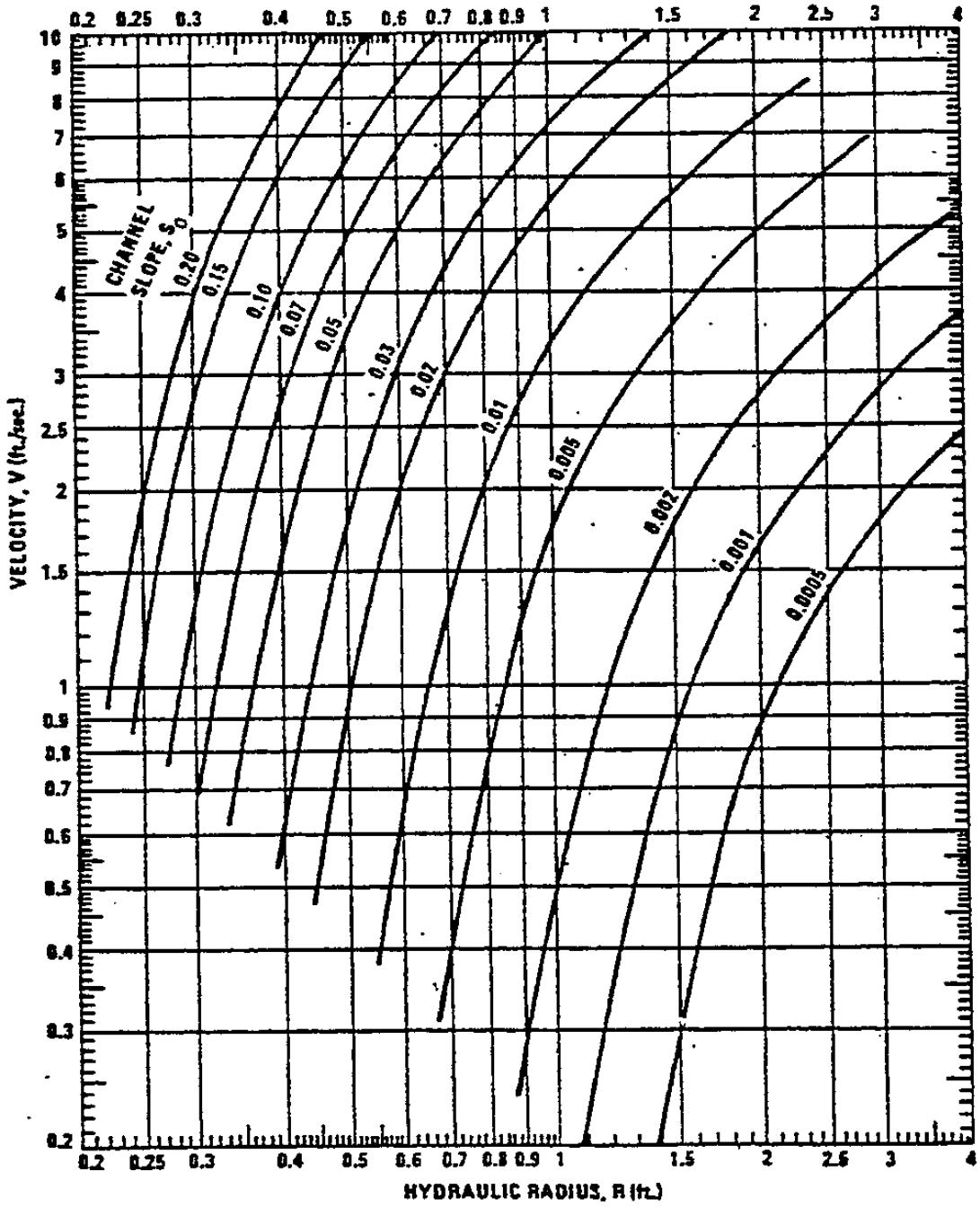


Figure 4.3.10
Flow Velocity for Channels Lined with Vegetation of Retardance C (Normann, 1975)

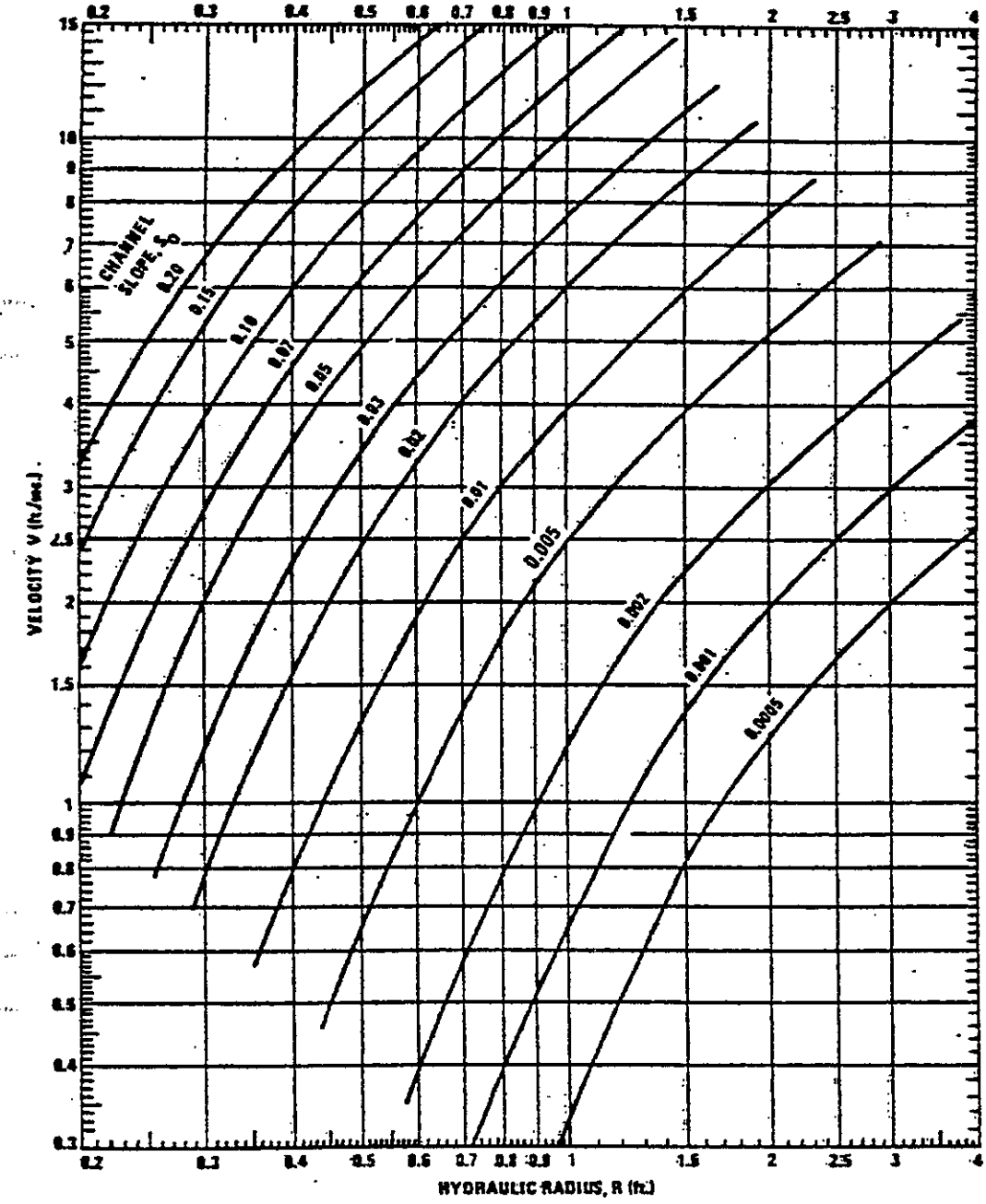


Figure 4.3.11
Flow Velocity for Channels Lined with Vegetation of Retardance D (Normann, 1975)

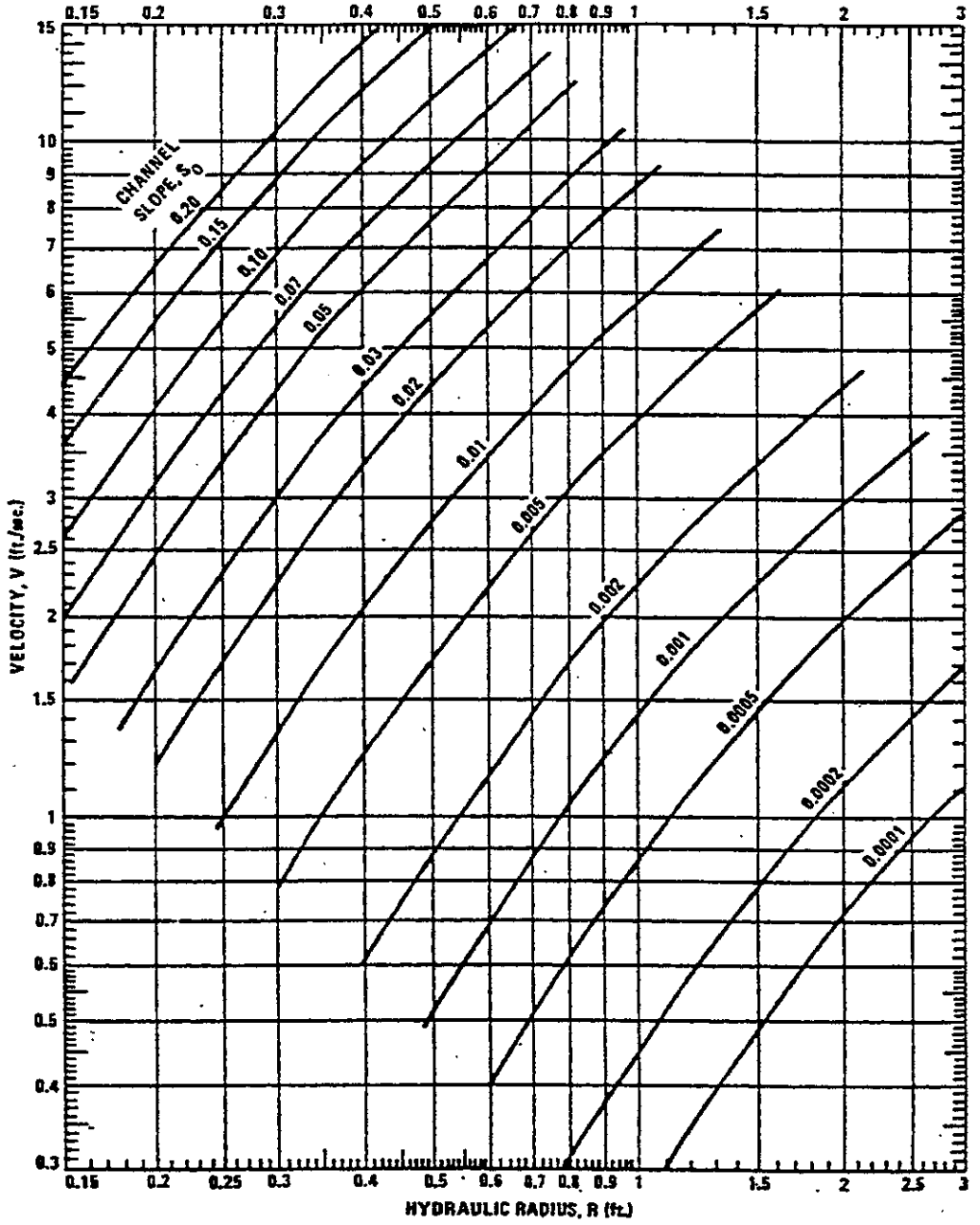
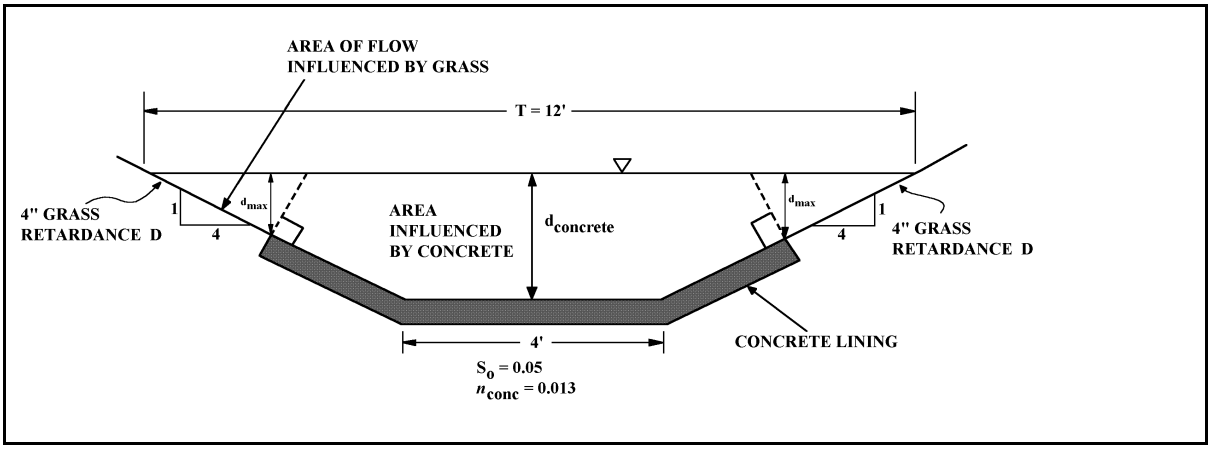


Figure 4.3.12
Flow Velocity for Channels Lined with Vegetation of Retardance E (Normann, 1975)

Example 4.3.2 (Normann, 1975)

Compute the capacity of the channel in Example 4.3.1, if the top width is limited to 12 feet, and the channel bottom and part of the sides is concrete lined. The channel slope is 0.05; the side slope is 4:1; and the bottom width is 4 feet. A grass mixture (retardance D) is to be an un-mowed, 4-inch length, on the average.



d_{max} (grass mixture) = 0.45 ft (Figure 4.3.7) (Take the average value)

For T_{max} = 12 ft.; d (concrete) = 1.0 ft

$A_{total} = (4.0)(1.0) + 4(1.0)^2 = 8.0 \text{ ft}^2$

$P_{w\ total} = 4.0 + 2(\sqrt{17})(1.0) = 12.25 \text{ ft}$

For shear boundary, take normal to side slopes at edge of concrete lining.

$A_{grass} = (4d_{max} + 0.25d_{max})_{grass} * d_{max\ grass} = (4.25)(0.45)^2 = 0.86 \text{ ft}^2$

$P_{w\ grass} = 2(\sqrt{17})\, d_{max} = 2(\sqrt{17})(.45) = 3.71 \text{ ft}$

$R_{grass} = (0.86 / 3.71) = 0.23 \text{ ft}$

$V_{grass} = 1.00 \text{ ft/sec}$ (Figure 4.14)

$Q_{grass} = AV = (1.00)(0.86) = 0.86 \text{ cfs}$

$A_{concrete} = A_{total} - A_{grass} = 8.0 \text{ ft}^2 - 0.86 \text{ ft}^2 = 7.14 \text{ ft}^2$

$P_{w\ concrete} = P_{w\ total} - P_{w\ grass} = 12.25 \text{ ft} - 3.71 = 8.54 \text{ ft}$

$R_{concrete} = 7.14 \text{ ft}^2 / 8.54 \text{ ft} = 0.84 \text{ ft}$

$Q_{concrete} = (1.49/n)\, AR^{2/3}\, s_o^{1/2} = (1.49/0.013)(7.14)(0.84)^{2/3}(0.05)^{1/2}$

$Q_{total} = 163 \text{ cfs}$

Outlet velocity $= (Q_{total} / A_{total}) = 163 / 8.0 = 20.4 \text{ ft/sec}$

4.3.5 - Design Procedures (after Normann, 1975)

A. Lined, Non-Erodible Channels (See Example 4.3.1)

Step 1: Perform the necessary hydrologic computations to establish the design discharge value. (See Chapter 3)

Step 2: Select a cross-sectional shape based on economics, aesthetics, safety, maintenance, etc. Select the appropriate *n* value from Table 4.2.1.

Step 3: Determine the depth of flow using the methodology from Example 4.2.1. If this depth seems excessive, widen the channel section and determine a new depth of flow using the methodology from Example 4.2.1.

Step 4: Check for the state of flow. Calculate the cross-sectional area of flow and hydraulic depth of flow (See Table 4.1.1). Calculate V= Q/A. Calculate the Froude number from $F = V/\sqrt{(gD)}$. If this Froude number is equal to or greater than one, then critical or supercritical flow exists and trouble could be experienced with standing waves, flow around bends, and general flow instability. Another cross-sectional shape or the use of check structures should be considered.

Step 5: Modify the final section dimensions for practicality and add a freeboard depth to the channel.

B. Erodible Channels

Step 1: Perform hydrologic computations.

Step 2: Select design flows for permanent lining material and for temporary linings based on subjective risk analyses.

Step 3: Define soil erodibility.

- The difficulties involved in defining the erodibility of cohesive soils is well described by Partheniades (1971), based on the MSU work, which covered ten (10) soils of different characteristics.
- Soils with a gravel, sand and clay mixture are erosion resistant; fine-grained sands or silts are erodible; while plastic and semi-plastic soils are in the intermediate range.
- The soil erodibility factor for the Universal Soil Loss Equation, developed by Wischmeier and Smith (1978), can also be used as a guide to soil erodibility. If the designer has no knowledge of the erodibility of the soil at a particular channel site, a reasonable estimate of d_{max} may be obtained by interpolating halfway between the "erosion resistant" and "erodible" lines of the maximum permissible depth charts (Figures 4.3.4 and 4.3.7).

Step 4: Define channel shape, slope and maximum top width and lining.

- For purposes of safety, construction, maintenance, and erosion resistance, it is suggested that the channel side slopes be kept as flat as possible. Ideally, side slopes should be 3:1 or flatter for erosion resistance. Flatter slopes may be necessary for safety or other reasons.

Step 5: Determine d_{max} for the selected slope, and soil erodibility (Figure 4.3.4 or 4.3.8).

Step 6: Determine hydraulic radius (R) and area (A) for the selected channel geometry and d_{max} (Table 4.1).

Step 7: Determine the velocity of flow using:

(a) For rock rip-rap: Use Manning Formula with Roughness Coefficient determined by

$$n = 0.0395\, D_{50}^{1/6} \tag{4.3.5}$$

where D_{50} = mean stone size (feet).

(b) For bare soil: Use

$$V = 22.81\, R^{0.591}\, S_0^{0.286} \tag{4.3.6}$$

(c) For grass lined channels: Use appropriate chart of Figures 4.3.8 through 4.3.12.

Step 8: Determine Q = VA

If Q does not satisfy the design Q, select another channel size and return to Step 4 or select another lining material and return to Step 5. Also, consider the feasibility of additional inlets to reduce Q.

A computation sheet, shown in Figure 4.3.13 has been developed to facilitate the foregoing design procedure (Normann, 1975). In addition a nomograph for determining the geometric properties of trapezoidal channels is presented in Figure 4.3.14.

DRAINAGE CHANNEL LINING DESIGN

DATE: _____

PROJECT: _____ **DESIGNER:** _____

STATION: _____ **TO STATION:** _____ **CHECKED BY:** _____

DRAINAGE AREA = _____ **ACRES** **DATE:** _____

HYDROLOGIC COMPUTATIONS:

DESIGN FLOW: Q _____ **=** _____ **cfs**

DESIGN FLOW FOR TEMPORARY LINING: Q _____ **=** _____ **cfs**

SOIL ERODIBILITY: _____

CHANNEL DESCRIPTION: MAX. TOP WIDTH = _____ **ft**

S_o = _____

AVAILABLE LININGS:

LINING	d_{max}	b	A	R	V	Q = AV	T	REMARKS

Figure 4.3.13
Computation Sheet for Drainage Control Lining Design (after Norman, 1975)

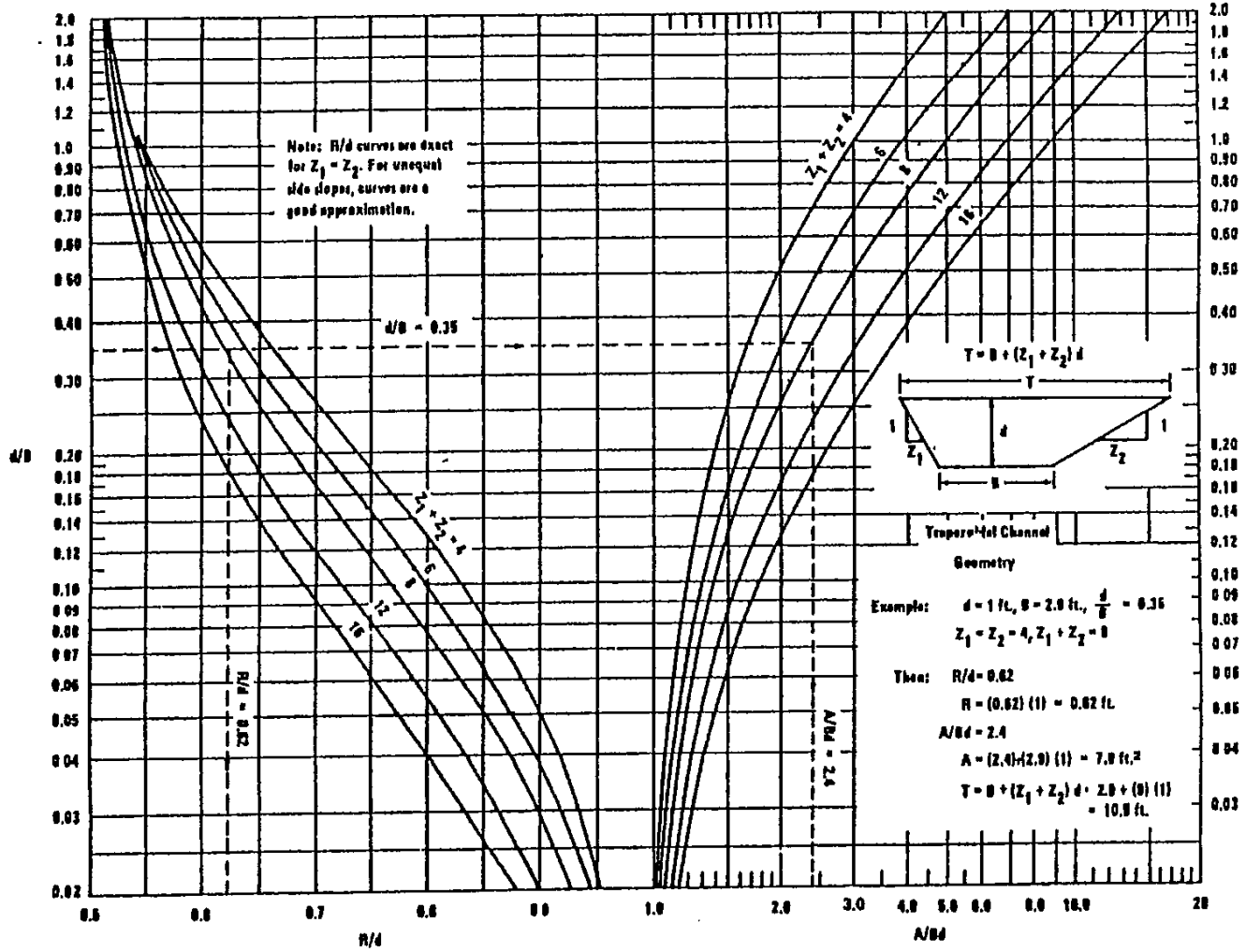


Figure 4.3.14
Nomograph for Design of Trapezoidal Channel Section (Normann, 1975)

4.3.6 - Flow in Bends

Flow around a bend in an open channel creates currents which impose higher shear stresses on the channel sides and bottom. According to Anderson (1973), the location of the maximum shear varies depending on the position in the bend.

Figure 4.3.15 has been prepared to adjust the D_{50} of rock rip-rap lining for the higher shear stresses in a bend, depending on the surface width of the channel (B_S = T) and the mean radius of the bend (R_O). To use the chart, determine K_3 based on the ratio B_S/R_O. Then, multiply the D_{50}, determined for the straight channel reaches, by K_3 to obtain a larger stone size for use in the bend. For instance, assume that a channel with a top width of 12 feet requires a stone size of D_{50} = 0.5 feet in the straight reaches. Assume that the channel has a bend with a 24 ft. radius. Then B_S/R_O = 12/24 = 0.5 from Figure 4.18, for B_S/R_O = 0.5, K_3 = 1.18.

$$(D_{50})_{Bend} = 1.18(D_{50})_{Straight} = (1.18)(0.5) = 0.59 \ ft$$

Therefore, the stone in the bend area should have a D_{50} of 0.59 ft. Since it is not possible to predict the location of the maximum shear, the entire channel cross-section must be protected with the same stone.

Figure 4.3.15 also indicates the B_S/R_O values requiring additional protection with other lining materials. It appears that as long as B_S/R_O is less than 0.4, K_3 is less than 1.1 and no additional protection is necessary. When B_S/R_O is greater than 0.4, the lining in the bend area should be reinforced with rip-rap

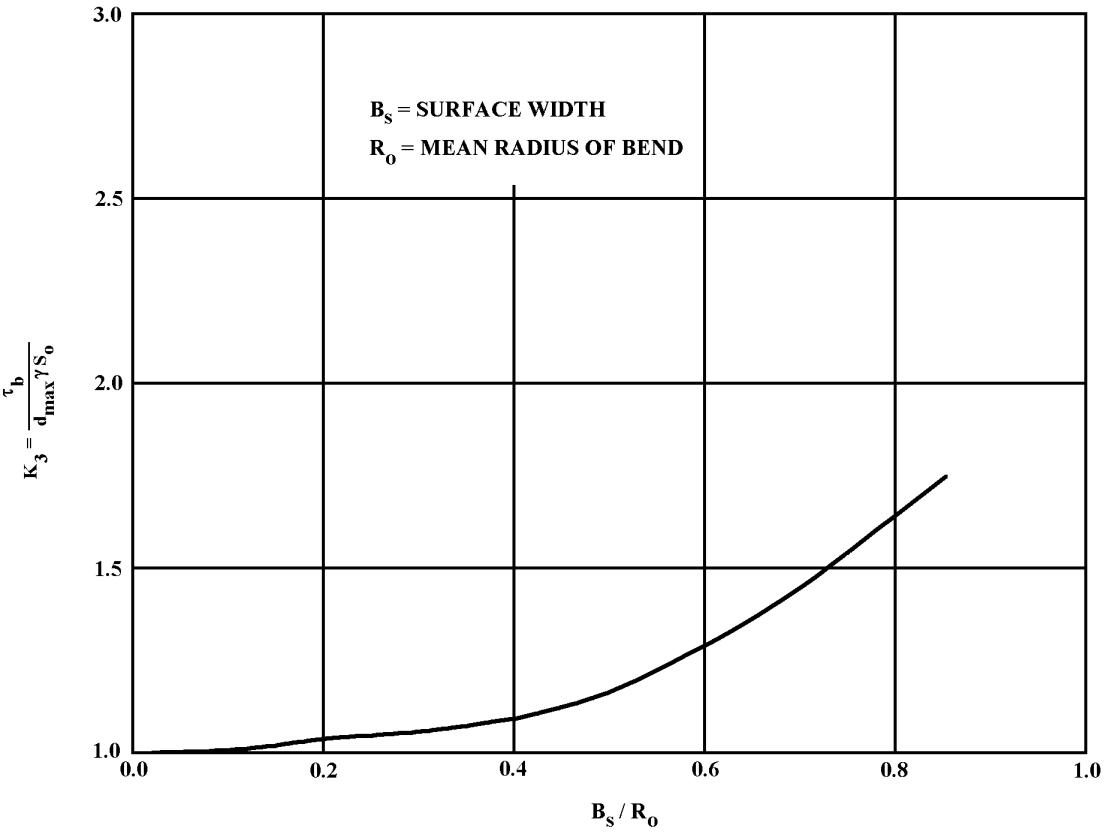


Figure 4.3.15
Ratio of Maximum Boundary Shear in Bends to Maximum
Bottom Shear in Straight Reaches (Anderson, 1973)
Note: $B_s = T$

4.4 - Gradually Varied Flow

Gradually varied flow is defined as steady flow which has a gradual change in depth along the channel length. This is typical of many natural streams or channels. The analysis and computation of this type of flow is quite complex. This section will develop the basic theory and give a simplified design procedure used in the analysis of this flow.

The basic assumptions for gradually-varied-flow theory presented here are:

1. The head loss for a given section is identical of that for uniform flow with the same velocity and section hydraulic radius.
2. The channel has a constant cross-section.
3. The Manning roughness coefficient *n* is constant for the length of the channel under consideration and does not vary with depth.
4. The depth of flow is the same as the distance normal to the free surface.

4.4.1 - Development of Analysis

Recalling Equation 4.1.1 and Figure 4.1.1, the total energy at a given section may be written

$$H = \frac{V^2}{2g} + Z + d\cos\theta \qquad (4.4.1)$$

as:

By differentiating the above equation with respect to x, the slope of the energy grade line dH/ dX is given as S_f. If the slope decreases, the values are negative; if the slope increases, the values are positive. Therefore, Equation 4.4.1 may be simplified by solving for the

$$\frac{dd}{dx} = \frac{S_o - S_f}{\cos\Theta + \frac{d(V^2/2g)}{dd}} \qquad (4.4.2)$$

water surface slope dd / dx.
For small slopes a cosϴ = 1 and dd / dx = dy / dx which gives

This is the general equation for gradually varied flow. It is easily seen that for uniform flow $S_O = S_f$ and dy / dx = 0. When $S_O > S_f$, dy / dx is positive; and when $S_O < S_f$, dy / dx is negative.

$$\frac{dy}{dx} = \frac{S_o - S_f}{1 + d(V^2/2g)dy} \qquad (4.4.3)$$

Table 4.4.1 lists the many possible types of flow profiles which may occur in open channels and their classification. Figure 4.4.1 presents these conditions graphically. In engineering design the most typical application of the gradually varied flow equation is backwater.

Backwater occurs as a result of a tributary stream which may have a lower depth than the larger channel to which it contributes. The result is a subsequent backing up of the tributary and a change in the water surface profile. The depth and extent of this backwater is needed to provide the planner and engineer with data to insure that no property or lives are endangered by the increased depths.

There are many detailed computer programs which have been developed to analyze this effect which have been shown to produce reliable results. HEC-RAS by the Army Corps of Engineers (COE, 1990), WSP-2 by the Soil Conservation Service (USDA, 1987) or WSPRO by the United States Geologic Survey and Federal Highway Administration (USGS and FHWA, 1988) are recommended, if available. However, if a small channel is being analyzed or a rough estimate is desired, the following section presents a method which may be used.

Table 4.4.1
Types of Flow Profiles in Prismatic Channels (Chow, 1988)

Channel slope	Designation			Relation of y to y_n and y_c			General type of curve	Type of flow
	Zone 1	Zone 2	Zone 3	Zone 1	Zone 2	Zone 3		
Horizontal $S_0 = 0$	None			$y > y_n > y_c$			None	None
		$H2$			$y_n > y > y_c$		Drawdown	Subcritical
			$H3$		$y_n > y_c > y$		Backwater	Supercritical
Mild $0 < S_0 < S_c$	$M1$			$y > y_n > y_c$			Backwater	Subcritical
		$M2$			$y_n > y > y_c$		Drawdown	Subcritical
			$M3$		$y_n > y_c > y$		Backwater	Supercritical
Critical $S_0 = S_c > 0$	$C1$			$y > y_c = y_n$			Backwater	Subcritical
		$C2$			$y_c = y = y_n$		Parallel to channel bottom	Uniform-critical
			$C3$		$y_c = y_n > y$		Backwater	Supercritical
Steep $S_0 > S_c > 0$	$S1$			$y > y_c > y_n$			Backwater	Subcritical
		$S2$			$y_c > y > y_n$		Drawdown	Supercritical
			$S3$		$y_c > y_n > y$		Backwater	Supercritical
Adverse $S_0 < 0$	None			$y > (y_n)^* > y_c$			None	None
		$A2$			$(y_n)^* > y > y_c$		Drawdown	Subcritical
			$A3$		$(y_n)^* > y_c > y$		Backwater	Supercritical

* y_n in parentheses is assumed a positive value.

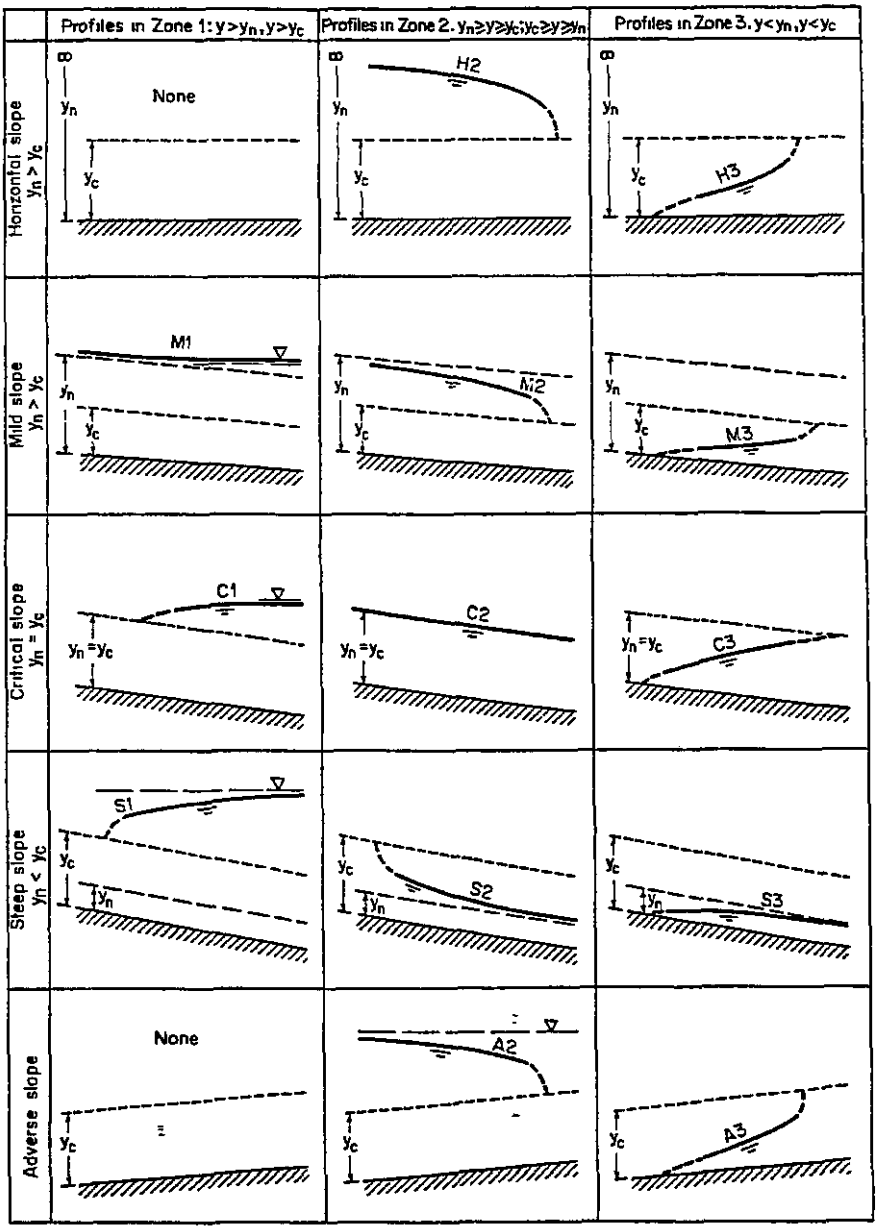


Figure 4.4.1
Classification of Flow Profiles of Gradually Varied Flow (Chow, 1988)

The method presented here is the direct step method. This simple method breaks the channel into short reaches and the computations are done from step to step, and the distance to specified depths determined. This method may be applied to prismatic channels. For more sophisticated manual methods the reader is referred to Chow (1988).

From Figure 4.4.2 the following expression which relates the energy head in a reach of length ΔX, can be developed.

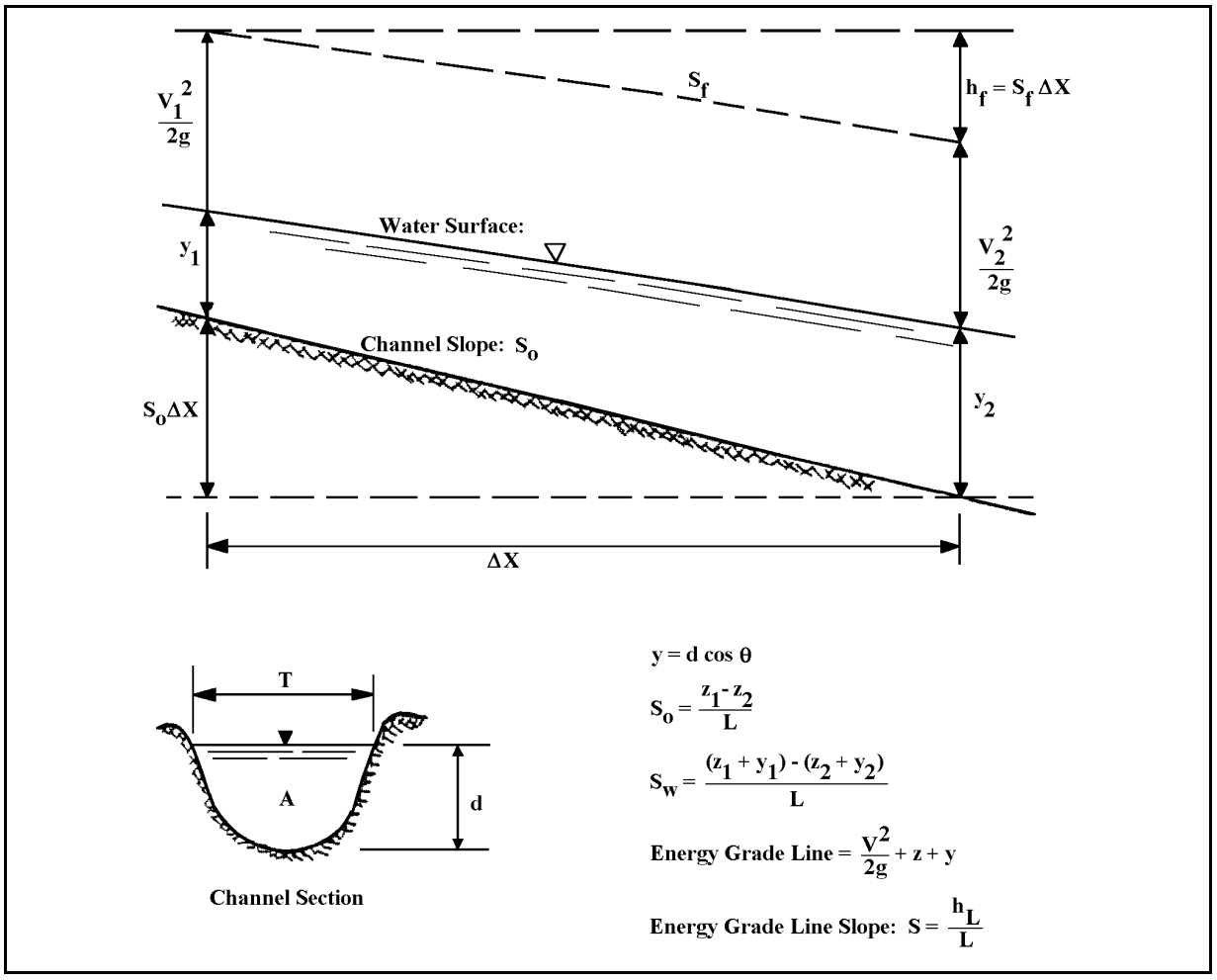


Figure 4.4.2
A Channel Length Used for the Development of the Direct Step Method

$$S_o \Delta X + y_1 + \frac{V_1^2}{2g} = y_2 + \frac{V_2^2}{2g} + S_f \Delta X \qquad (4.4.4)$$

Since the specific energy E has been defined as y + $\frac{V^2}{2g}$ we can solve Equation (4.4.2) for the increment of length ∆X.

$$\Delta X = \frac{E_2 - E_1}{S_o - S_f} = \frac{\Delta E}{S_o - S_f} \qquad (4.4.5)$$

S_O is found from channel characteristics and S_f is determined from the Mannings' Formula:

$$S_f = \frac{n^2 V^2}{2.22\, R^{4/3}} \qquad (4.4.6)$$

The procedure used in determining the water surface profile is listed below:

Step 1: With a starting depth y, which is usually a known condition, proceed upstream in equal depth increments.

Step 2: Compute the area and the hydraulic radius for the given depth y.

Step 3: Compute the velocity by dividing the flow rate (given) by the flow area and calculate the velocity head ($V^2/2g$).

Step 4: Determine the specific energy, E = (y + $V^2/2g$), and change in specific energy (∆E) from the previous step.

Step 5: Compute S_f from Equation 4.4.6.

Step 6: Compute the average friction slope between the steps using an arithmetic average.

Step 7: Compute the difference between the channel slope and the average friction slope.

Step 8: Compute the length between the sections by Equation 4.4.5.

Example 4.4.1 outlines the above procedure.

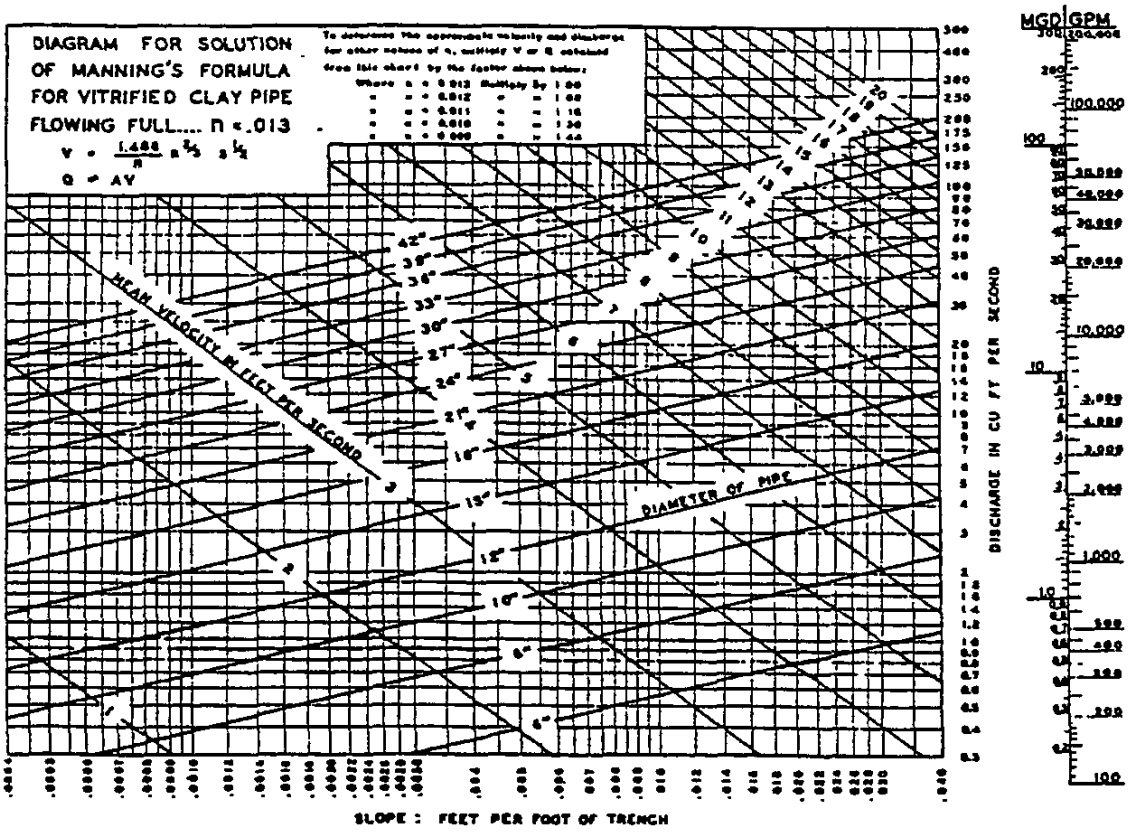


Figure 4.4.3
Graphical Solution of the Manning Equation for Circular Pipe Flowing Full

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Example 4.4.1

For a rectangular concrete channel with a ten foot width, flow rate of 360 cfs, Manning *n* of 0.015 and a slope of 0.0023 ft/ft, determine the backwater profile. The downstream depth is 5 feet. Work upstream in 0.1 ft increments until a depth of 4.4 ft is reached.

The computations are shown on the next page. For each depth, the area, hydraulic radius, velocity and velocity head may be determined. The energy, change in energy, friction slope and the difference between the bed slope and the friction slope also can be calculated. Finally, the incremental reach ΔX and the composite length X are found. For this example, the depth of 4.4 feet occurs 893 feet upstream.

y (ft)	A (ft^2)	R (ft)	$R^{4/3}$ ($ft^{4/3}$)	v (ft/sec)	$v^2/2g$ (ft)	E (ft)	ΔE (ft)	S_f (ft/ft)	$\overline{S_f}$ (ft/ft)	$S_o - S_f$ (ft/ft)	ΔX (ft)	X (ft)
5.0	50	2.50	3.39	7.20	0.80	5.80	-	0.00155	-	-	-	-
4.9	49	2.47	3.35	7.35	0.84	5.74	0.06	0.00163	0.00159	0.00071	84.50	84.50
4.8	48	2.45	3.30	7.50	0.87	5.67	0.07	0.00173	0.00168	0.00062	112.90	197.40
4.7	47	2.42	3.25	7.66	0.91	5.61	0.06	0.00183	0.00178	0.00052	115.36	312.76
4.6	46	2.40	3.20	7.83	0.95	5.55	0.06	0.00194	0.00189	0.00041	144.89	457.65
4.5	45	2.37	3.16	8.00	0.99	5.49	0.05	0.00205	0.00200	0.00030	164.66	622.31
4.4	44	2.34	3.11	8.18	1.04	5.44	0.05	0.00218	0.00212	0.00018	270.71	893.02

4.5 - HY-8 PROGRAM

The Federal Highway Administration (FHWA) has developed analytical and empirical techniques for the hydraulic analysis and design of culverts. The design engineer may utilize the FHWA publications to analyze culverts for a single design discharge and, with some additional effort, develop a culvert performance curve. In addition, these techniques allow the consideration of inflow and outflow hydrographs, storage and routing, and energy dissipation. Different scenarios require considerable effort to compute by hand. The HY-8 program was developed to quickly and accurately solve these culvert system techniques.

A culvert system is composed of the actual hydraulic structure, as well as hydrologic inputs, storage and routing considerations, and energy dissipation devices and strategies. HY-8 automates the methods presented in HDS 5, "Hydraulic Design of Highway Culverts", HEC-14, "Hydraulic Design of Energy Dissipators for Culverts and Channels", HEC-19, "Hydrology", and information published by pipe manufacturers pertaining to the culvert sizes and materials. HY-8 is composed of the four previously named programs: culvert analysis, hydrograph generation, routing, and energy dissipation. A full description of the program and its capabilities may be obtained by consulting the FHWA references at the end of this chapter.

The current version of HY-8 is Culvert Hydraulic Analysis Program, Version 7.0. The program is Windows based and can be downloaded from http://www.fhwa.dot.gov/engineering/hydraulics/software/hy8/. FHWA is currently incorporating Energy Dissipator, Hydraulic Jump, Broken Band and other features into HY-8. For those wishing to use an energy dissipator program, they should use the older DOS version that contains the energy dissipator module.

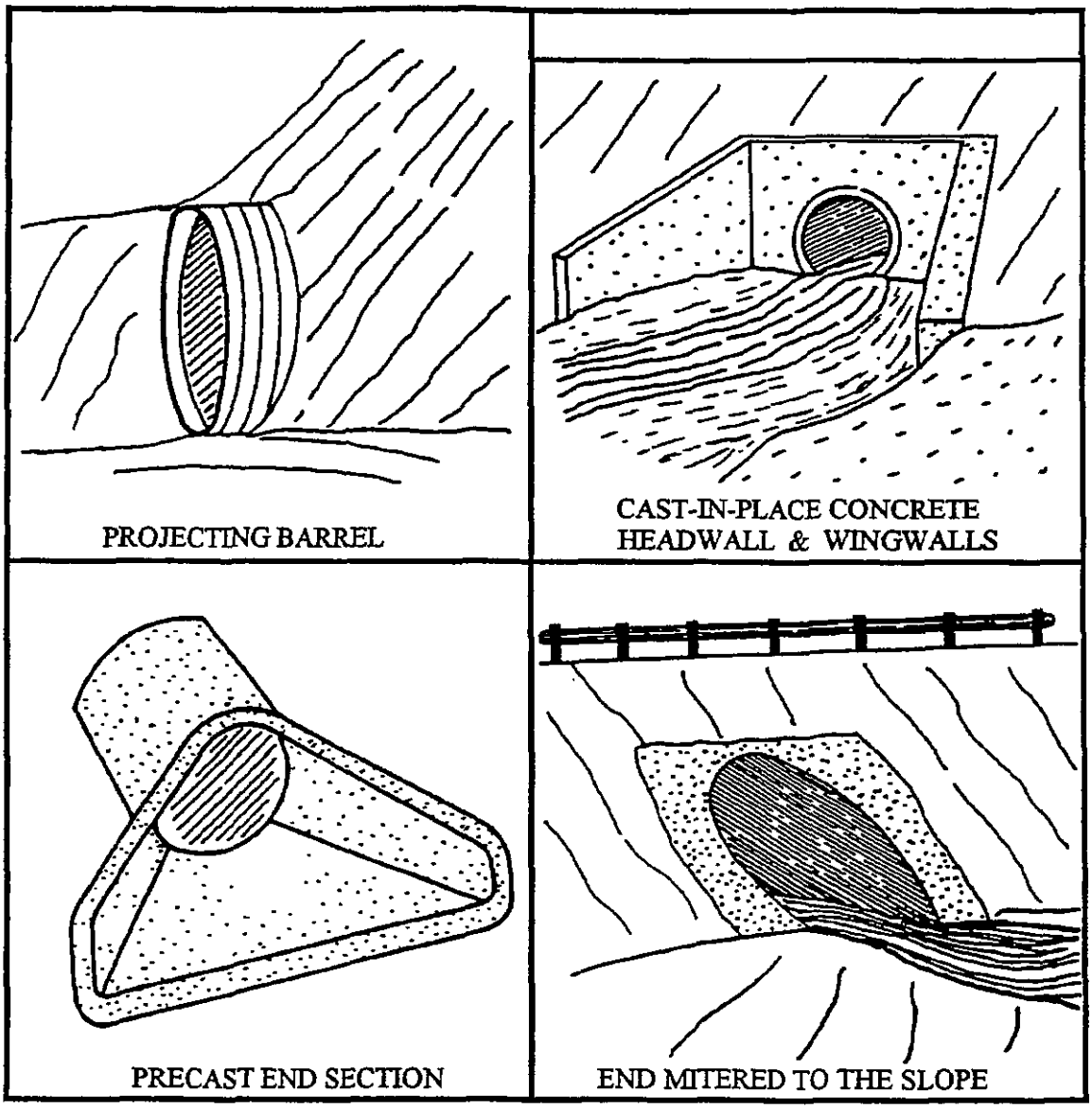


Figure 4.5.1
Four Standard Inlet Types (schematic) (FHWA, 1985)

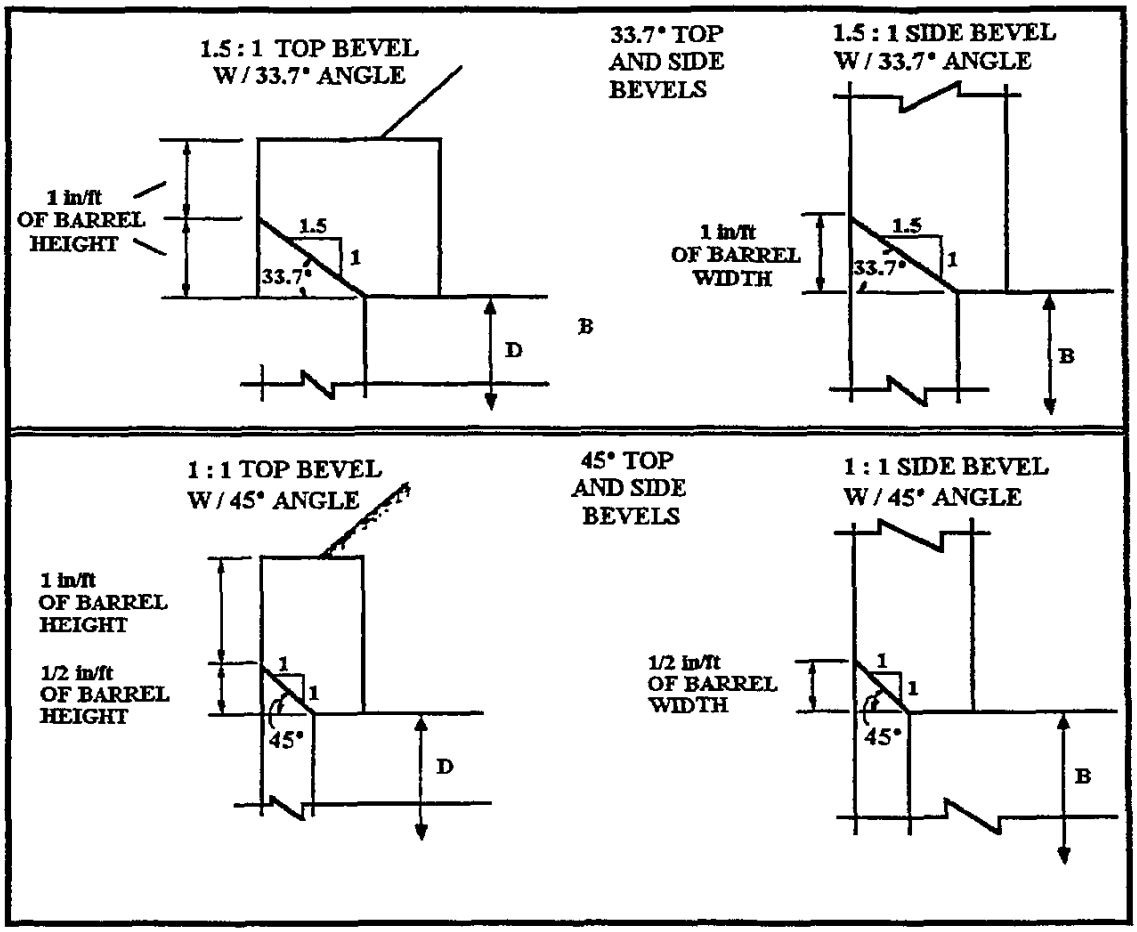


Figure 4.5.2
Beveled Edge Inlets (FHWA, 1985)

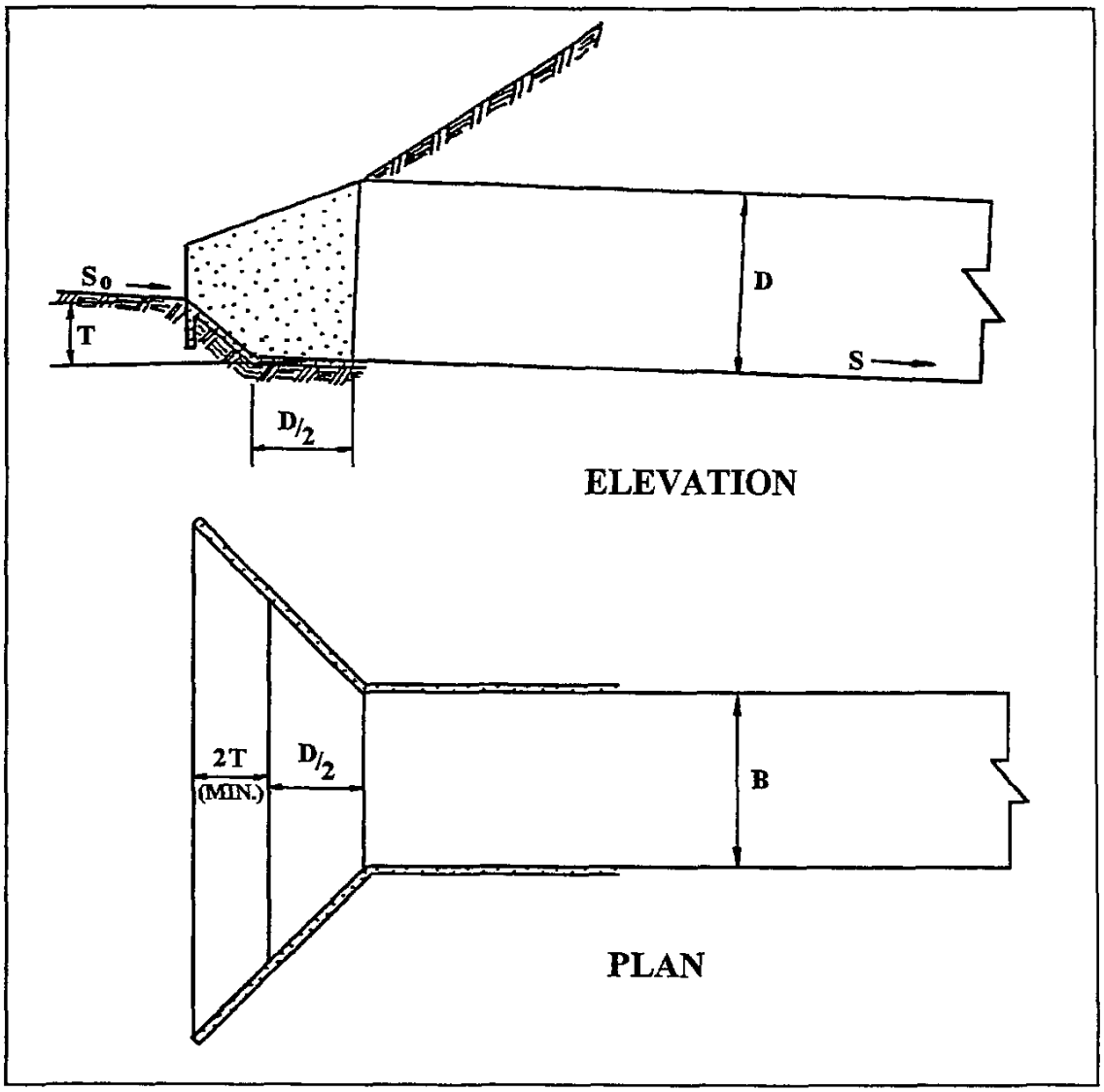


Figure 4.5.3
Culvert with Depressed Apron and Wingwalls (FHWA, 1985)

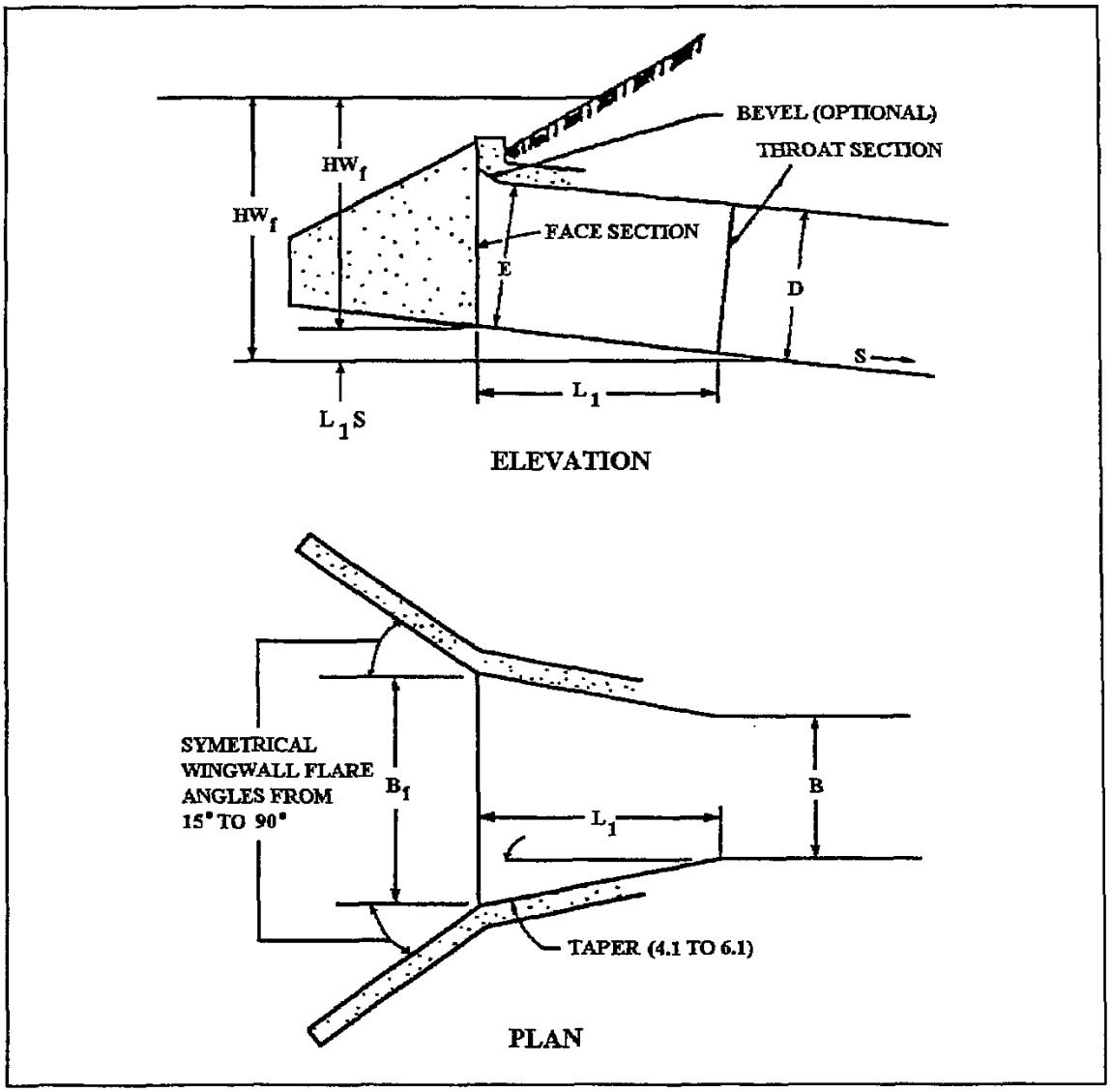


Figure 4.5.4
Side-tapered Inlet (FHWA, 1985)

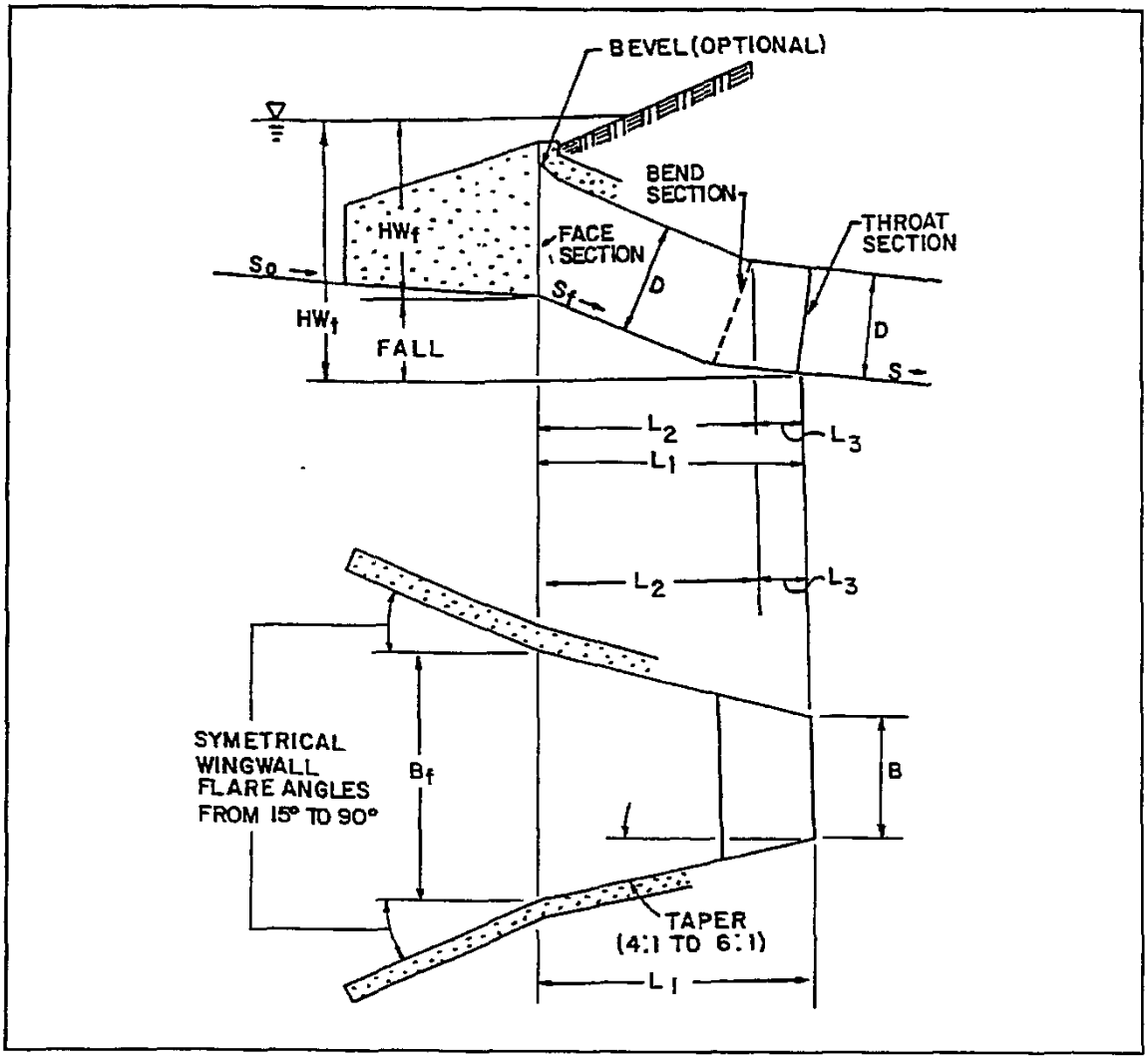


Figure 4.5.5
Slope-tapered Inlet with Vertical Face (FHWA, 1985)

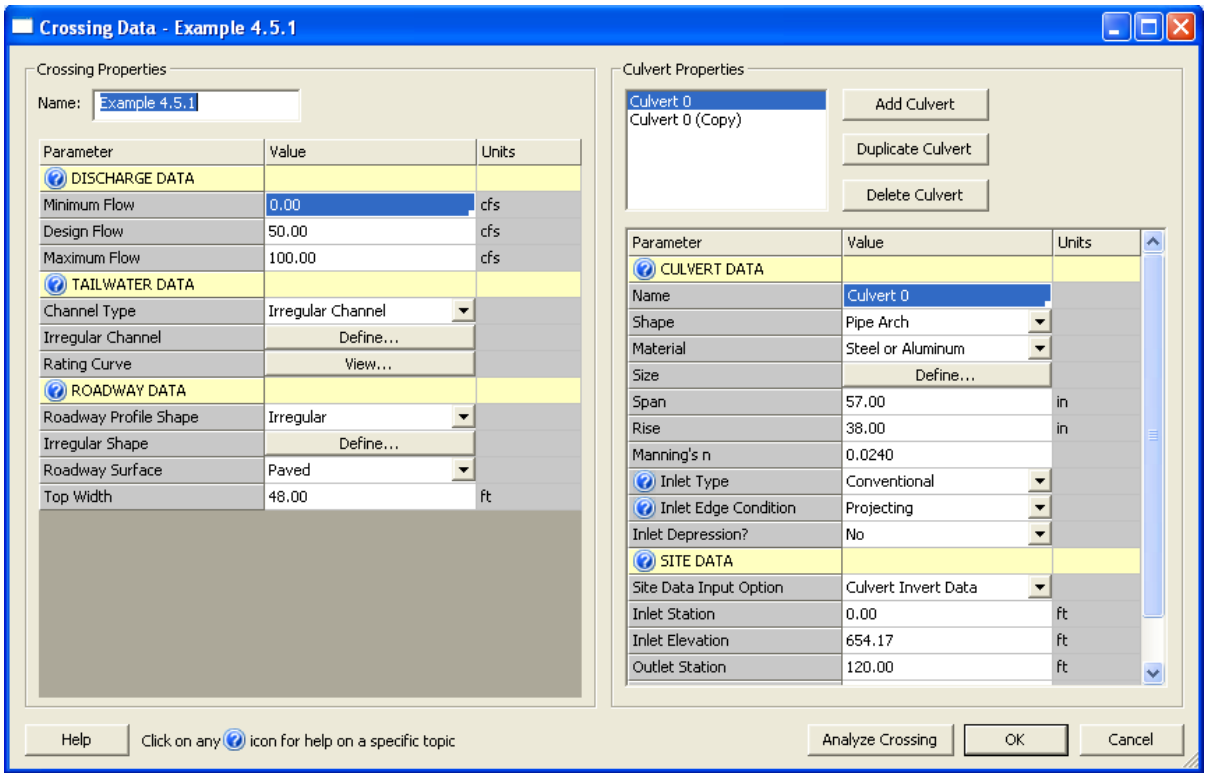
Example 4.5.1

This example illustrates the use of the HY-8 computer program for developing an elevation-discharge curve resulting from a twin culvert.

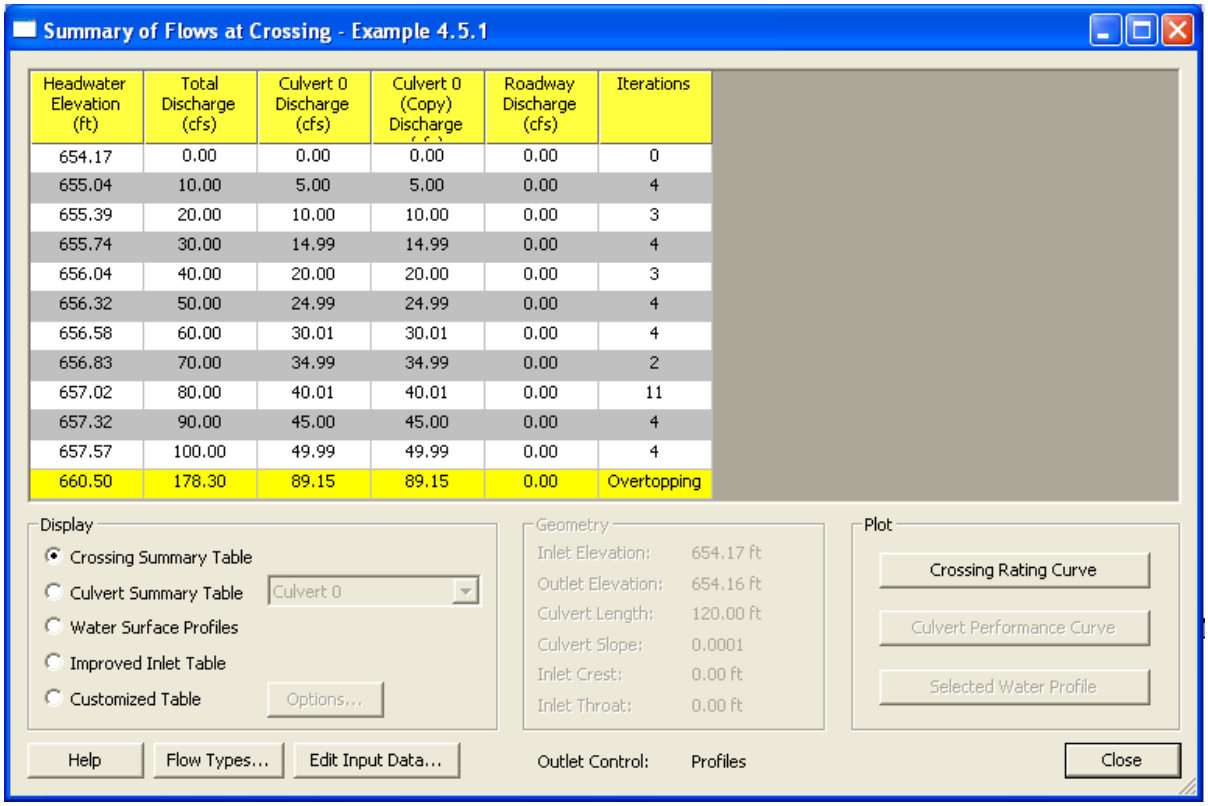
Using 120 feet long twin 38-inch by 57-inch CMP culverts, determine the elevation-discharge relationship at the culvert outlet given a downstream and roadway cross-section shown below.
The following additional data describes the twin culverts and is used for input for the HY-8 computations:

Invert Elevation	654.17 ft.	Outlet Elevation	654.16 ft.
Culvert Length	120 ft.	Inlet Type	Conventional
Inlet Edge and Wall	Projecting	Inlet Depression	None

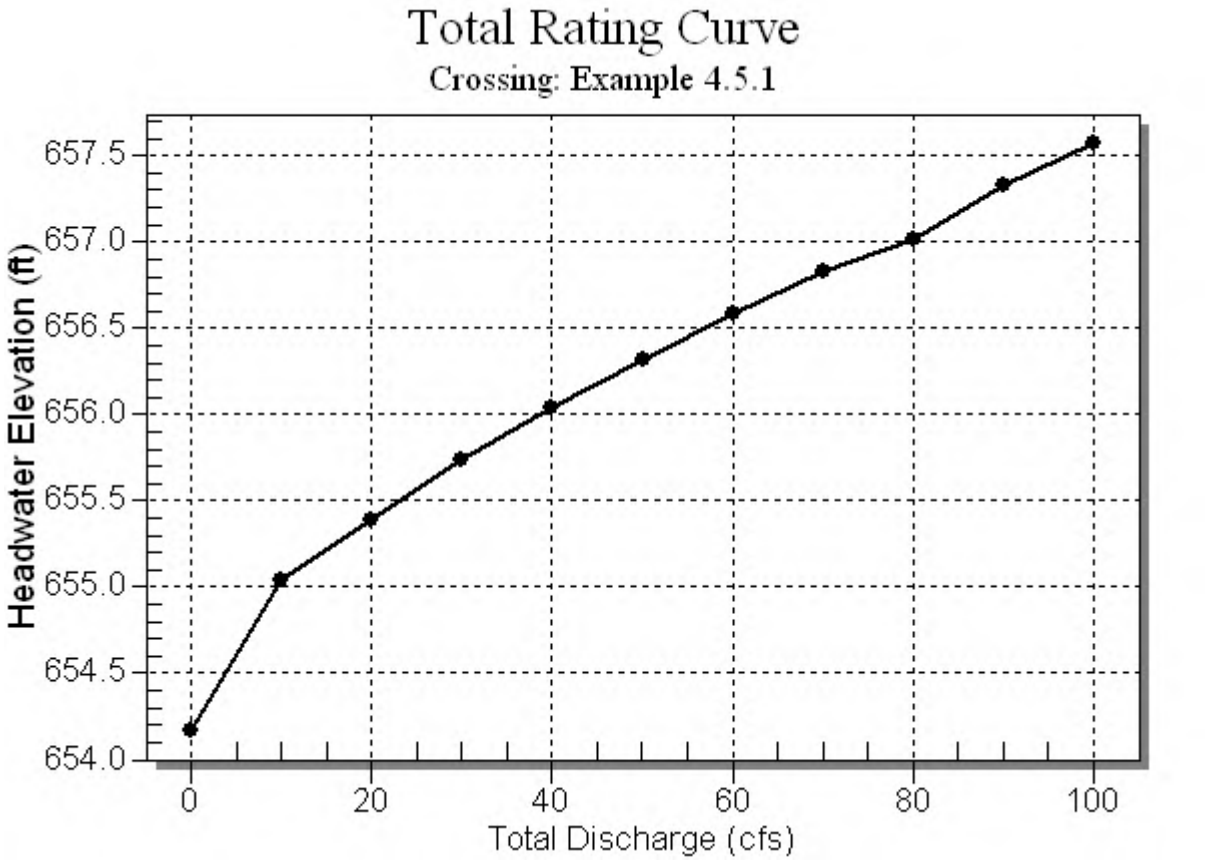
In addition, the roadway elevations in the vicinity of the culverts are entered into the program using x, y coordinates. The data is easily entered in the Windows based HY-8 program. There are several choices for tailwater data, roadway data and culvert data that can be selected from pull down menus. A summary of the input is shown below.



The following pages contain the HY-8 results that are presented in a report format as well as some graphs produced by the program. There is no overtopping for the culvert flows ranging between 0 and 50 cfs. The flow at which roadway overtopping occurs is at 178 cfs.



Rating Curve Plot for Crossing: Example 4.5.1



Culvert Summary Table - Culvert 0

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth(ft)	Outlet Control Depth(ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00	0.00	654.17	0.00	0.00	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
10.00	5.00	655.04	0.79	**0.87**	3-M2t	1.98	0.43	0.47	0.68	2.93	0.64
20.00	10.00	655.39	1.13	**1.22**	3-M2t	3.17	0.65	0.71	0.92	3.25	0.76
30.00	14.99	655.74	1.37	**1.57**	3-M2t	3.17	0.78	0.88	1.09	3.57	0.85
40.00	20.00	656.04	1.59	**1.87**	3-M2t	3.17	0.92	1.01	1.22	4.22	0.90
50.00	24.99	656.32	1.83	**2.15**	3-M2t	3.17	1.07	1.10	1.31	5.08	0.92
60.00	30.01	656.58	2.05	**2.41**	2-M2c	3.17	1.23	1.23	1.40	5.84	0.95
70.00	34.99	656.83	2.26	**2.66**	2-M2c	3.17	1.35	1.35	1.48	6.27	0.98
80.00	40.01	657.02	2.46	**2.85**	2-M2c	3.17	1.46	1.46	1.55	6.59	1.01
90.00	45.00	657.32	2.65	**3.16**	2-M2c	3.17	1.56	1.56	1.61	6.86	1.04
100.00	49.99	657.57	2.85	**3.40**	2-M2c	3.17	1.66	1.66	1.68	7.16	1.07

Display
- Crossing Summary Table
- Culvert Summary Table — Culvert 0
- Water Surface Profiles
- Improved Inlet Table
- Customized Table — Options...

Geometry
- Inlet Elevation: 654.17 ft
- Outlet Elevation: 654.16 ft
- Culvert Length: 120.00 ft
- Culvert Slope: 0.0001
- Inlet Crest: 0.00 ft
- Inlet Throat: 0.00 ft

Plot
- Crossing Rating Curve
- Culvert Performance Curve
- Selected Water Profile

Help | Flow Types... | Edit Input Data... | Outlet Control: Profiles | Close

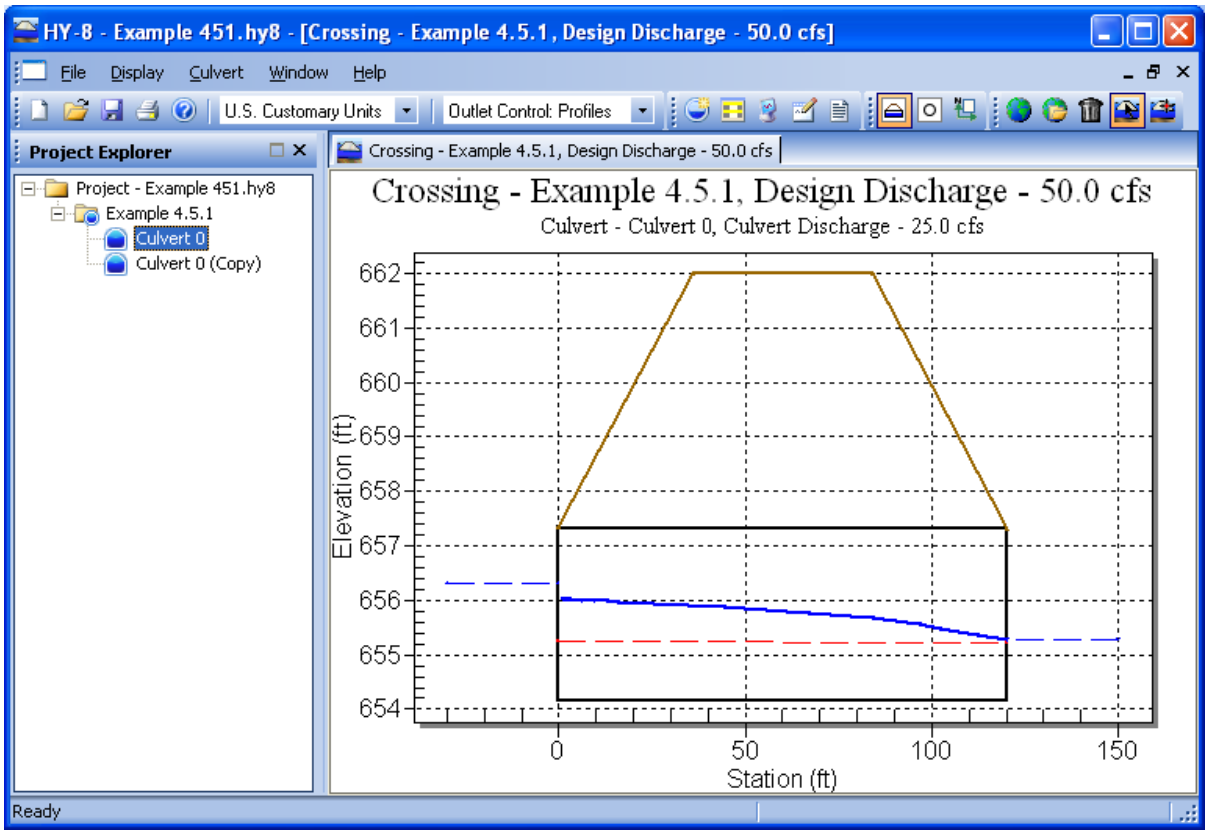


Table 1 - Summary of Culvert Flows at Crossing: Example 4.5.1

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 0 Discharge (cfs)	Culvert 0 (Copy) Discharge (cfs)	Roadway Discharge (cfs)	Iterations
654.17	0.00	0.00	0.00	0.00	0
655.04	10.00	5.00	5.00	0.00	4
655.39	20.00	10.00	10.00	0.00	3
655.74	30.00	14.99	14.99	0.00	4
656.04	40.00	20.00	20.00	0.00	3
656.32	50.00	24.99	24.99	0.00	4
656.58	60.00	30.01	30.01	0.00	4
656.83	70.00	34.99	34.99	0.00	2
657.02	80.00	40.01	40.01	0.00	11
657.32	90.00	45.00	45.00	0.00	4
657.57	100.00	49.99	49.99	0.00	4

Table 2 - Culvert Summary Table: Culvert 0

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00	0.00	654.17	0.000	0.000	0-NF	0.000	0.000	0.000	0.000	0.000	0.000
10.00	5.00	655.04	0.791	0.874	3-M2t	1.983	0.435	0.471	0.681	2.930	0.644
20.00	10.00	655.39	1.126	1.217	3-M2t	3.167	0.649	0.714	0.924	3.249	0.764
30.00	14.99	655.74	1.372	1.571	3-M2t	3.167	0.784	0.883	1.093	3.569	0.849
40.00	20.00	656.04	1.586	1.873	3-M2t	3.167	0.920	1.005	1.215	4.220	0.896
50.00	24.99	656.32	1.827	2.151	3-M2t	3.167	1.073	1.105	1.315	5.085	0.923
60.00	30.01	656.58	2.049	2.413	2-M2c	3.167	1.231	1.231	1.401	5.840	0.954
70.00	34.99	656.83	2.256	2.660	2-M2c	3.167	1.349	1.349	1.478	6.272	0.984
80.00	40.01	657.02	2.457	2.847	2-M2c	3.167	1.456	1.456	1.549	6.590	1.013
90.00	45.00	657.32	2.653	3.155	2-M2c	3.167	1.563	1.563	1.615	6.859	1.040
100.00	49.99	657.57	2.852	3.402	2-M2c	3.167	1.658	1.658	1.676	7.160	1.066

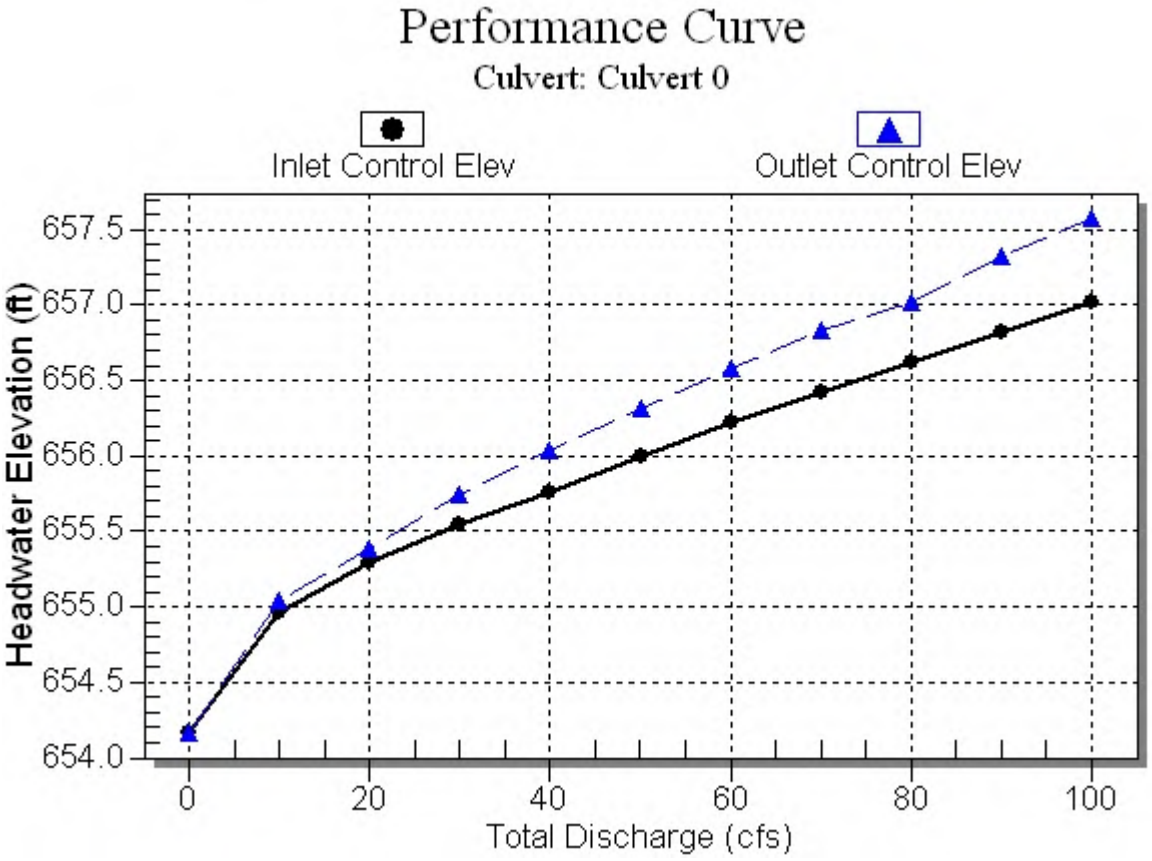
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Inlet Elevation (invert): 654.17 ft, Outlet Elevation (invert): 654.16 ft

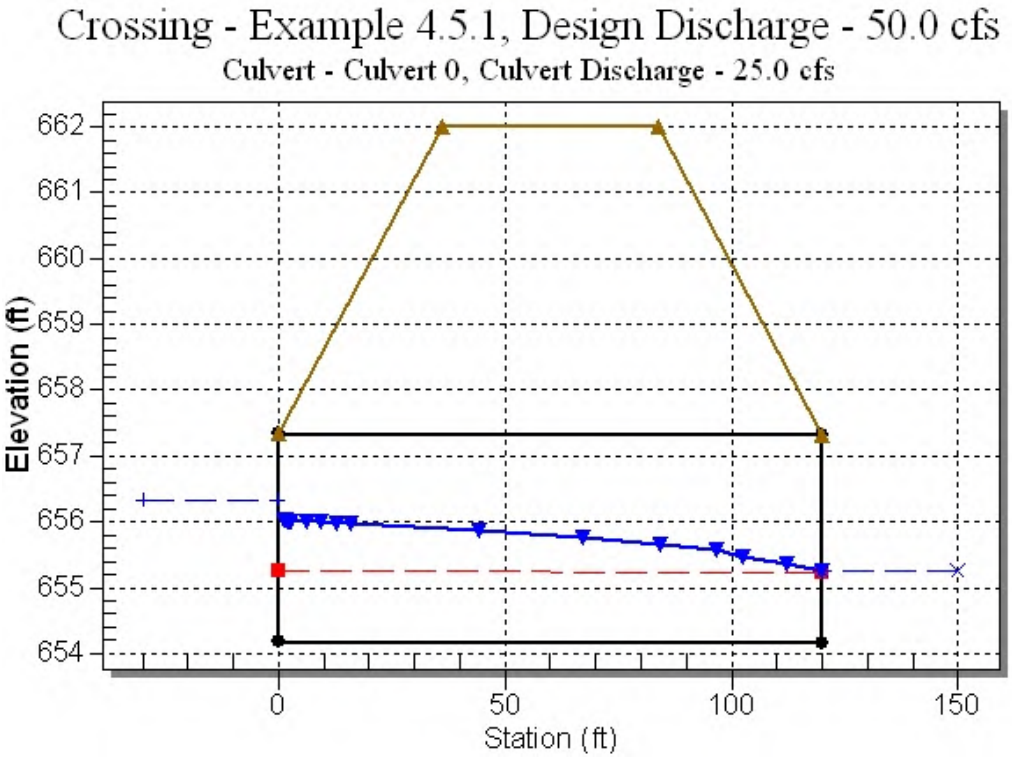
Culvert Length: 120.00 ft, Culvert Slope: 0.0001

****

Culvert Performance Curve Plot: Culvert 0



Water Surface Profile Plot for Culvert: Culvert 0



Site Data - Culvert 0

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 654.17 ft

Outlet Station: 120.00 ft

Outlet Elevation: 654.16 ft

Number of Barrels: 1

Culvert Data Summary - Culvert 0

Barrel Shape: Pipe Arch

Barrel Span: 57.00 in

Barrel Rise: 38.00 in

Barrel Material: Steel or Aluminum

Barrel Manning's n: 0.0240

Inlet Type: Conventional

Inlet Edge Condition: Projecting

Inlet Depression: None

Table 3 - Culvert Summary Table: Culvert 0 (Copy)

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00	0.00	654.17	0.000	0.000	0-NF	0.000	0.000	0.000	0.000	0.000	0.000
10.00	5.00	655.04	0.791	0.874	3-M2t	1.983	0.435	0.471	0.681	2.930	0.644
20.00	10.00	655.39	1.126	1.217	3-M2t	3.167	0.649	0.714	0.924	3.249	0.764
30.00	14.99	655.74	1.372	1.571	3-M2t	3.167	0.784	0.883	1.093	3.569	0.849
40.00	20.00	656.04	1.586	1.873	3-M2t	3.167	0.920	1.005	1.215	4.220	0.896
50.00	24.99	656.32	1.827	2.151	3-M2t	3.167	1.073	1.105	1.315	5.085	0.923
60.00	30.01	656.58	2.049	2.413	2-M2c	3.167	1.231	1.231	1.401	5.840	0.954
70.00	34.99	656.83	2.256	2.660	2-M2c	3.167	1.349	1.349	1.478	6.272	0.984
80.00	40.01	657.02	2.457	2.847	2-M2c	3.167	1.456	1.456	1.549	6.590	1.013
90.00	45.00	657.32	2.653	3.155	2-M2c	3.167	1.563	1.563	1.615	6.859	1.040
100.00	49.99	657.57	2.852	3.402	2-M2c	3.167	1.658	1.658	1.676	7.160	1.066

****

Inlet Elevation (invert): 654.17 ft, Outlet Elevation (invert): 654.16 ft

Culvert Length: 120.00 ft, Culvert Slope: 0.0001

****

Table 4 - Downstream Channel Rating Curve (Crossing: Example 4.5.1)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)	Velocity (ft/s)	Shear (psf)	Froude Number
0.00	653.95	0.00	0.00	0.00	0.00
10.00	654.63	0.68	0.64	0.15	0.18
20.00	654.87	0.92	0.76	0.21	0.18
30.00	655.04	1.09	0.85	0.25	0.20
40.00	655.17	1.22	0.90	0.27	0.22
50.00	655.26	1.31	0.92	0.30	0.22
60.00	655.35	1.40	0.95	0.31	0.22
70.00	655.43	1.48	0.98	0.33	0.21
80.00	655.50	1.55	1.01	0.35	0.21
90.00	655.56	1.61	1.04	0.36	0.21
100.00	655.63	1.68	1.07	0.38	0.21

Tailwater Channel Data - Example 4.5.1

Tailwater Channel Option: Irregular Channel

Channel Slope: 0.0036

User Defined Channel Cross-Section:

Coord No.	Station (ft)	Elevation (ft)	Manning's n
1	0.00	656.00	0.0900
2	0.00	655.20	0.0900
3	30.00	655.00	0.0900
4	43.00	654.00	0.0800
5	49.00	653.95	0.0800
6	53.00	654.00	0.0800
7	85.00	655.00	0.0900
8	140.00	656.00	0.0000

Roadway Data for Crossing: Example 4.5.1

Roadway Profile Shape: Irregular Roadway Shape (coordinates)

Irregular Roadway Cross-Section:

Coord No.	Station (ft)	Elevation (ft)
1	0.00	662.00
2	115.00	661.00
3	190.00	660.60
4	265.00	660.50
5	375.00	660.80
6	500.00	661.00
7	615.00	662.00

Roadway Surface: Paved

Roadway Top Width: 48.00 ft

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Chapter 5 - FLOW IN GUTTERS AND INLETS

LIST OF PARAMETERS

a	Depth of inlet depression (inches or cm)
A	Drainage area (acres or hectares)
A_g	Clear opening of gutter inlet (ft^2 or m^2)
A_i	Clear opening of grate inlet (ft^2 or m^2)
C	Rational method runoff coefficient
C_w	Weir coefficient for a curb inlet
d	Depth of flow at gutter (ft or m)
d´	Depth of flow in street at some distance from gutter (ft or m)
d_o	Depth of water over orifice (ft or m)
d_w	Depth of water over weir (ft or m)
D_H	Distance from high point to tangent point
D_L	Distance from low point to tangent point
E	Efficiency of curb inlets
E_o	Ratio of flow in depressed section to total gutter flow
g	Acceleration due to gravity (ft/sec^2 or m/sec^2)
G	Tangent grade
H	Curb opening height (height between top of curb opening to bottom of depression) (ft or m)
i	Rainfall intensity (inches/hr)
K	Neenah coefficient for grate capacity
L	Actual curb opening length (ft or m)
L_c	Length of curb opening to intercept 100% of gutter flow (ft or m)
n	Manning's roughness coefficient
P	Perimeter of grate opening (ft or m)
Q_b	Bypassed flow (cfs or m^3/sec)
Q_B	Remaining flow in street between distance x and T (cfs or m^3/sec)
Q_c	Carryover flow from upgrade inlet (cfs or m^3/sec)
Q_G	Gutter flow (cfs or m^3/sec)
Q_i	Capacity of inlet (cfs or m^3/sec)
Q_x	Flow in street between gutter and a distance x from the gutter (cfs or m^3/sec)
S_e	Equivalent cross slope for curb inlets
S_L	Longitudinal slope
S_T	Transverse slope
S_w	Slope of the depressed curb inlet area
T	Width of flow (ft or m)
v	Velocity of flow in gutter (ft/sec or m/sec)
W	Width of depression (ft or m)
W_s	Width of slotted drain inlets (ft or m)
X	Width (ft or m)
y	Depth of flow in street (ft or m)

Chapter 5 - FLOW IN GUTTERS AND INLETS

5.1 - INTRODUCTION

In many drainage systems, stormwater enters the conveyance system through an inlet. Often these openings are located along a gutter which is designed to convey overland flow to the inlets. These inlets may be located directly in the gutter, curb or in both the curb and gutter. It is important that the gutter and inlet be properly designed to adequately collect the stormwater in order to minimize the potential flooding of the roadway, danger to pedestrians, and disruption of traffic.

Figure 5.1.1 is a diagram illustrating the importance of properly designed inlets. In the left-hand side of the figure the encroachment of the stormwater into the street when the inlets are properly spaced is shown. The dashed line is the street encroachment with no carry-over from the upstream inlet and the solid line is encroachment with carry-over. In either case, these flows would not be very likely to disrupt traffic. However, the flow on the right-hand side shows pavement encroachment far into the street for the case with fewer inlets. This would obviously be a hazard to traffic and pedestrians.

In this chapter, the methods for determining the capacity of both gutters and inlets are discussed. Techniques used in the sizing and spacing of various types of inlets are presented. Example problems illustrate the application of these methods. Computer programs which reduce the amount of hand calculations required in designing inlets are provided.

5.2 - FLOW IN GUTTERS

Gutters are used to convey surface runoff into an inlet or other opening so that it may eventually enter the drainage system. The depth of water in the gutter and the top width of the water surface in the street are important design parameters. These values are dependent on the longitudinal slope, S_L, transverse slope, S_T, and roadway Manning's roughness coefficient, *n*. A relationship between these fixed values, and the unknown depth and the top width can be determined in the following manner.

Flow in a gutter with a curb can be approximated by using a triangular section as shown in Figure 5.2.1. The gutter is on a longitudinal slope of S_L and a transverse slope of S_T.

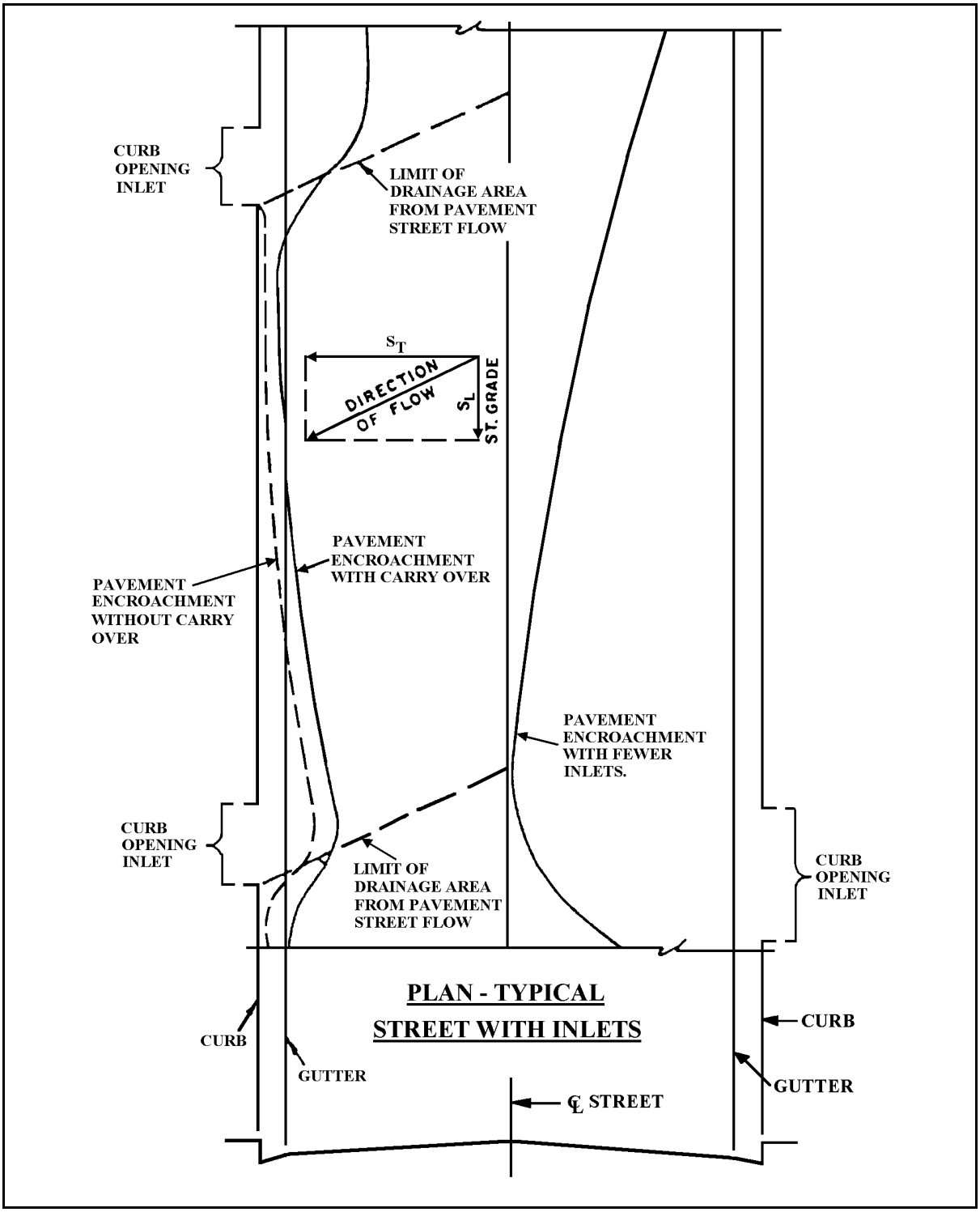


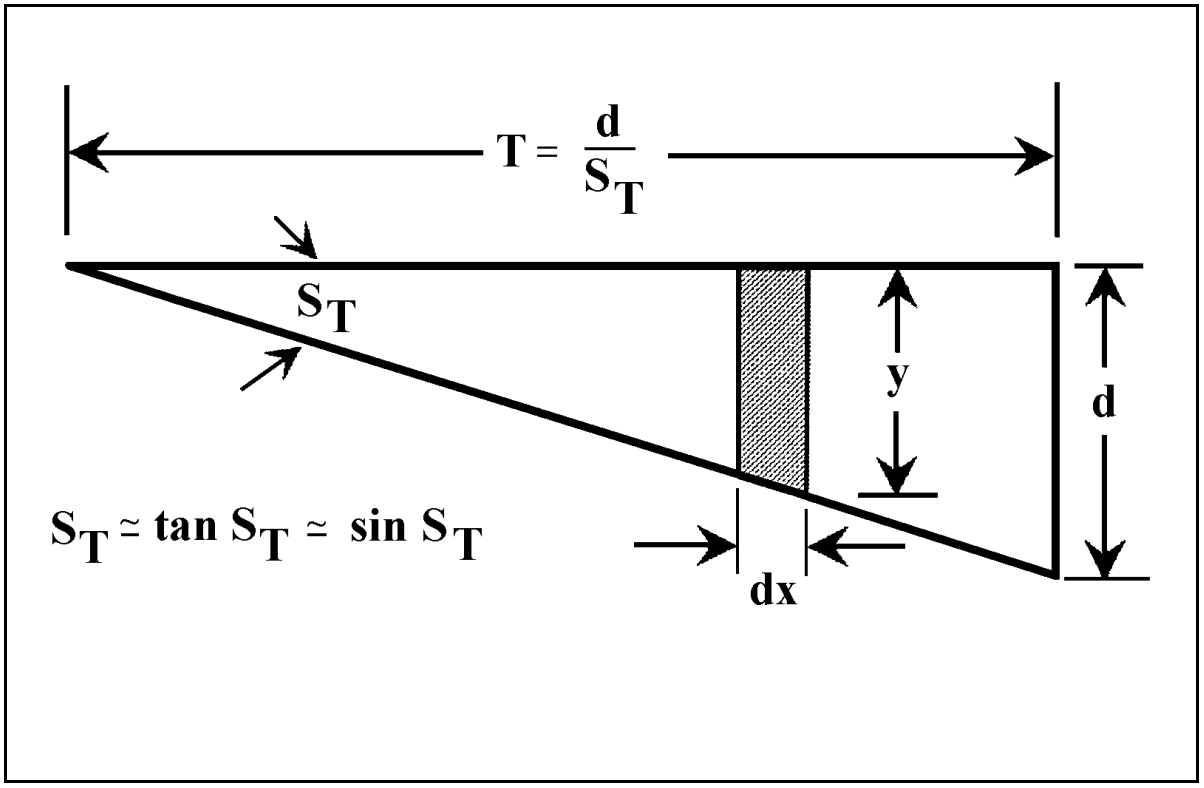
Figure 5.1.1
Diagram of Gutter and Pavement Flow Patterns (Wright, 1968)

The incremental flow rate, dQ_G, in the gutter is:

$$dQ_G = vy\,dx \tag{5.2.1}$$

where v is the velocity in the cross section of width dx (ft/sec), dx is the incremental width of cross section (ft), and y is the depth of flow at dx (ft).

Figure 5.2.1 Flow in a Gutter



The velocity, v, may be estimated using Manning's Equation (Equation 4.2.1) with slope, S_L, and the hydraulic radius equal to the depth, y,

$$v = \frac{1.49}{n}\, y^{2/3}\, S_L^{1/2} \tag{5.2.2}$$

where *n is* Manning's roughness coefficient (Table 4.2.1).

Substituting this relationship into Equation 5.2.1, the following equation is obtained.

$$dQ_G = \left[\frac{1.49}{n} y^{5/3} S_L^{1/2} \right] dx \qquad (5.2.3)$$

The incremental width, dx, may be expressed as dy/S_T, where S_T is the transverse slope of the cross-section (because $dy/dx = \tan S_T = S_T$).

Substituting this into Equation 5.2.3 and integrating y from 0 (zero) to d gives:

$$Q_G = \frac{1.49}{n} \frac{S_L^{1/2}}{S_T} \frac{3}{8} d^{8/3} = \frac{0.56}{n} \frac{S_L^{1/2}}{S_T} d^{8/3} \qquad (5.2.4)$$

The depth of flow against the gutter is given by Equation 5.2.5, while the width of the flow, T, is determined from:

$$T = \frac{d}{S_T}$$

$$d = \left[1.79\, Q_G\, n \frac{S_T}{S_L^{1/2}} \right]^{3/8} \qquad (5.2.5)$$

Figure 5.2.2 is a nomograph solution of Equation 5.2.4 (to find Q_G). It may also be used to calculate the depth of flow, d, (Equation 5.2.5).

After calculating the runoff from a sub basin and knowing the roadway slopes, S_T and S_L, and pavement roughness *n*, the depth and width of flow can be calculated. When the depth reaches a height greater than that of the curb or the width of flow extends too far into the street, an inlet must be placed to capture the flow.

Sometimes it is desired to know the discharge and depth in only a part of the channel. This information is often used in determining the capacity of grate inlets. Figure 5.2.3 shows a gutter cross-section. It is desired to find the discharge only in the portion of the channel having width x. The total discharge, Q_G, in the cross-section may be found using Equation 5.2.4 or Figure 5.2.2. The depth of flow at x can be calculated using Equation 5.2.6.

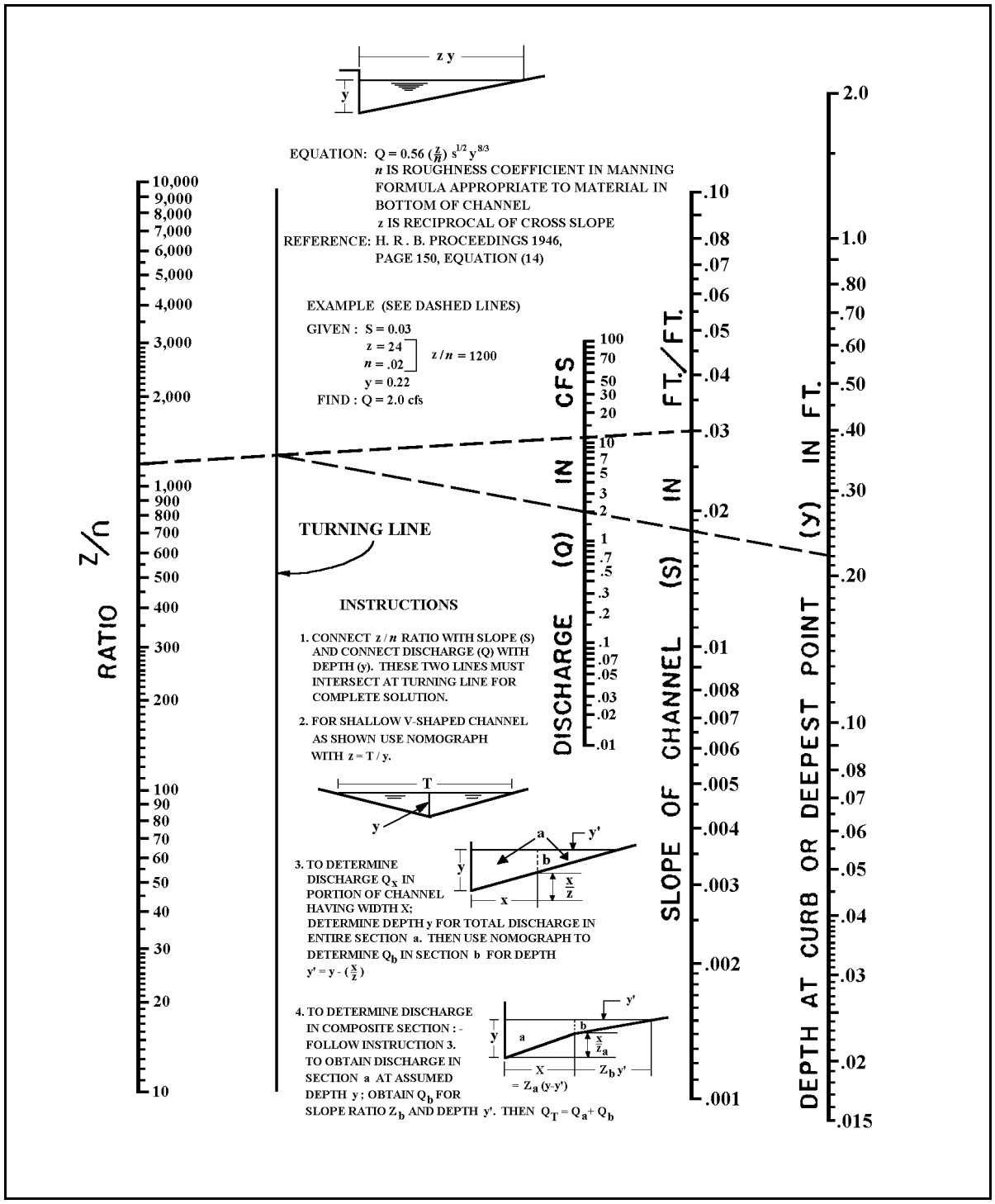


Figure 5.2.2
Nomograph Solution of Equations 5.2.4 and 5.2.5
(Bureau of Public Roads, 1968)

$$d' = d - x S_T \qquad (5.2.6)$$

The value of d' can then be used with Equation 5.2.4 or Figure 5.2.3 to calculate the flowrate, Q_B. Then the discharge in the portion of the channel within a distance x from the gutter, Q_X, can be calculated as:

$$Q_x = Q_G - Q_B \qquad (5.2.7)$$

The information presented here is required to design inlets properly. Applications of these relationships are illustrated in Example 5.2.1.

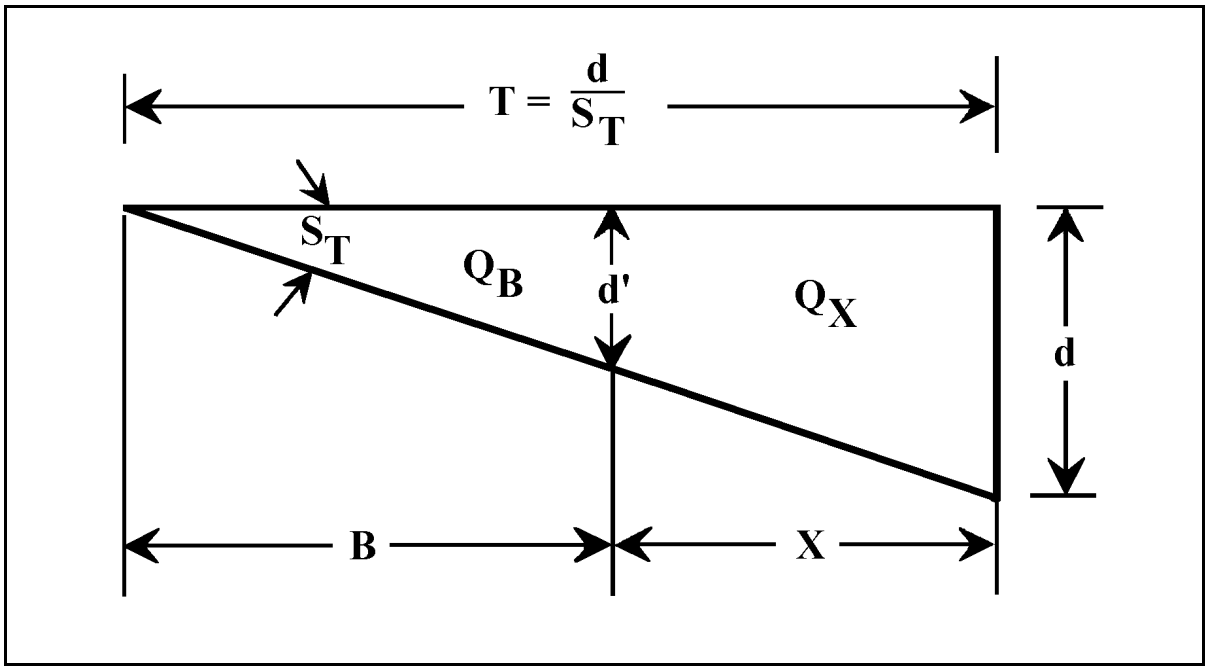


Figure 5.2.3
Gutter Cross-section

Example 5.2.1 (Flow in gutters)

Given: a concrete gutter with a rough asphalt pavement and a flowrate of 3.0 cfs, transverse slope of 1/4 inch per foot, and longitudinal slope of 3%.

Calculate: the depth of flow, the top width and the flowrate within 2 feet of the curb and depth at a distance of 2 feet from the curb.

From Table 4.2.1, the roughness is found to be 0.015. The transverse slope is calculated as:

$$S_T = \frac{1/4}{12} = \frac{1}{48} = 0.02$$

The depth of water at the gutter can be calculated using Equation 5.2.5

$$d = \left[1.79\, Q_G\, n \frac{S_T}{S_L^{1/2}}\right]^{3/8} = \left[(1.79)(3.0)(0.015)\frac{(0.02)}{(0.03)^{1/2}}\right]^{3/8} = 0.17 \ ft.$$

Therefore the top width is:

$$T = \frac{d}{S_T} = \frac{0.17}{0.02 \ ft/ft} = 8.44 \ ft.$$

The depth of flow two feet from the gutter is calculated using Equation 5.2.6.

$$d' = d - xS_T = 0.17 - 2(0.02) = 0.13 \ ft.$$

The flowrate in the street from 2 to 8.50 feet (Q_B) is computed using Equation 5.2.4.

$$Q_b = \frac{0.56}{n}\frac{S_L^{1/2}}{S_T} d^{8/3} = \frac{0.56}{0.015}\frac{(0.03)^{1/2}}{0.02}(0.13)^{8/3} = 1.40 cfs$$

The flowrate in the street between the gutter and two feet from the gutter (Q_x) is then computed as:

$$Q_x = Q_G - Q_B = 3 - 1.40 = 1.60 \ cfs$$

5.3 - FLOW IN INLETS

There are basically four types of inlets which are used in drainage design: gutter inlets; combination inlets; curb opening inlets; and slotted drain inlets. The capacity of each of these depends upon the inlet size, longitudinal and transverse slopes, street roughness, and the depth that the inlet is depressed (for gutter and combined inlets). The methods used in determining the capacity of the inlets as a function of the previously mentioned variables are discussed in this section.

Regardless of inlet type, there are some general design considerations. A few of these are listed below:

1. Is the inlet to be placed on a continuous grade or in a "sag"?
2. Is the inlet to be designed for 100% efficiency? (Efficiency is the ratio of the runoff passing over the inlet to the runoff captured)
3. Is the inlet to be depressed? (For gutter and combined inlets only)
4. Will the inlet be placed in an area where it will be susceptible to clogging?

These questions will help the designer to select the inlet(s) which will best suit the design criteria. It should be pointed out that regardless of the storm sewer design capacity, the inlet must be able to admit the stormwater into the storm system or flooding will occur with a frequency greater than the sewer system design would indicate.

5.3.1 - Gutter Inlets

A gutter inlet is an opening in the gutter through which water is admitted into the storm sewer system. The capacity of a gutter inlet decreases with an increase in longitudinal slope, and increases with an increase in transverse slope, grate length, width depression (described below) and grate efficiency. It has also been shown (Larson, 1948) that the capacity of a gutter inlet is increased by allowing a small percentage of the flow to bypass the grate (possible only on a continuous grade). This is achieved by the increased depth, and therefore an increase in the quantity of water captured. In order to intercept <u>all</u> the water, the grate length must be increased. As an example, an inlet grate with a constant transverse slope, S_T, may intercept 82% of the flow when the grate width is 50% of the total flow width. However, to capture the remaining 18% of the flow, the grate length must be doubled. This is shown qualitatively in Figure 5.3.1 where the grate length, L, would have to be extended to L_i to capture all of the flow Q_G.

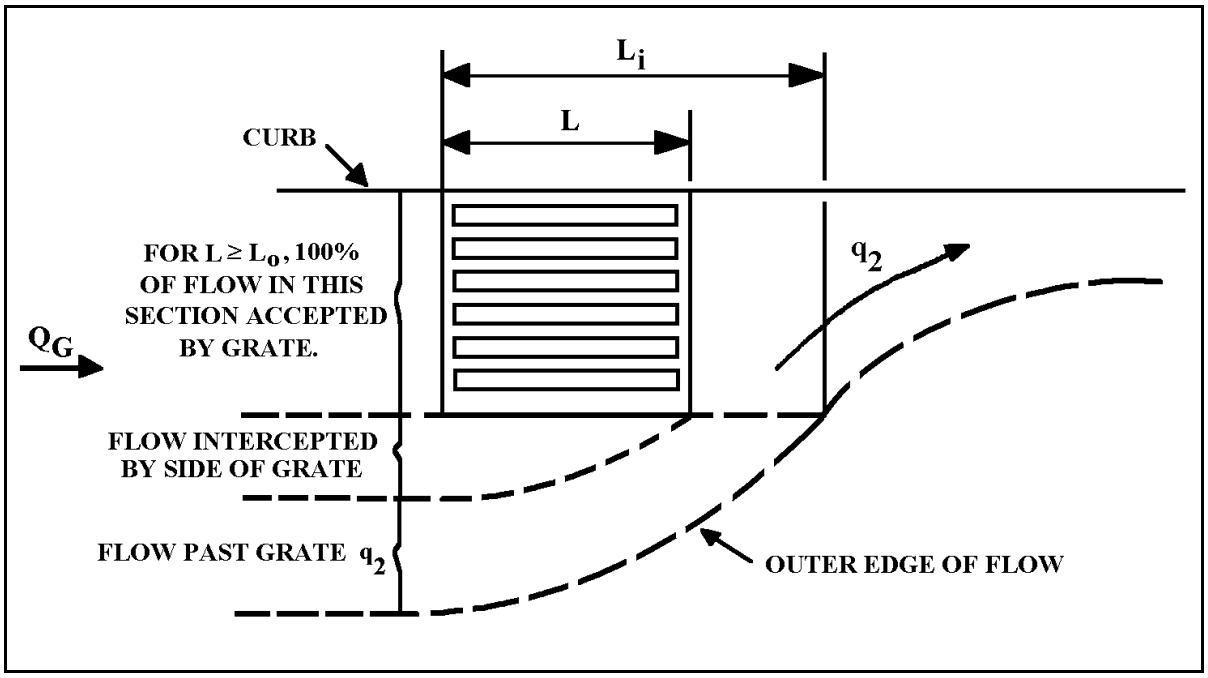


Figure 5.3.1
Plan of Grated Inlet showing Flow Lines (Wright, 1968)

The inlet grate must not be selected only on the basis of capacity. The potential for flooding caused by clogging of the grate with debris may dictate the use of curb inlets or combination inlets. Also, longitudinal parallel bars may cause a hazard to bicyclists and depressions may be dangerous in high speed traffic.

There are many possible bar configurations for the gutter grates. Various companies have published grate size data for an assortment of applications. (Neenah, East Jordan Iron Works, etc.) See References at the end of this Chapter.

For inlets which are on a continuous grade, the capacity of a grate inlet may be easily calculated with a method developed by Neenah (1987). The discharge-depth relationship for a grate inlet is written as

$$Q_i = K\, d^{5/3} \qquad (5.3.1)$$

where Q_i is the capacity of grate inlet (cfs), K is the coefficient dependent on the longitudinal slopes, S_L, transverse slopes, S_T, and grate configuration, and d is the depth of flow (ft).

Graphs published by Neenah (Neenah, 1987) for many of their manufactured grates present K as a function of S_T and S_L. Figure 5.3.2 is a typical chart. Each of these graphs published by Neenah is the result of 96 separate test points. The charts are applicable for Manning's roughness coefficient, *n*, between 0.013 - 0.014, and longitudinal slopes between 1 and 6 percent.

The maximum allowable depth, d, in Equation 5.3.1 may be obtained from a rating table or Equation 5.2.5. The spacing between the inlets will depend on the maximum allowable depth in the gutter or the maximum width of flow in the street. Example 5.3.1 illustrates the use of the charts. (NOTE: The use of Neenah Charts is not to be interpreted as an endorsement of their products by the author or INLTAP).

For gutter inlets located in a "valley", a sump condition will exist and all the water will be captured eventually. The rate at which the flow enters the inlet is determined by the depth of water above the grate. For depths less than 0.3 feet, the inlet acts as a weir and the discharge is calculated from Equation 5.3.2,

$$Q_i = 3.0\,P(d_w)^{1.5} \qquad (d_w < 0.3\ ft) \tag{5.3.2}$$

where Q_i is the discharge rate into the inlet (cfs), d_w is the depth of water above grate top for weir flow (ft) and P is the perimeter of grate opening (neglect bars and side against curb) (ft).

For depths exceeding 0.4 feet, the grate starts to act as an orifice. For orifice condition the discharge is calculated by Equation 5.3.3.

$$Q_i = 4.89\,A_i(d_o)^{0.5} \qquad (d_o > 0.4\ ft) \tag{5.3.3}$$

where A_i is the clear opening of grate (ft^2) (obtained from grate manufacturer catalogs) and d_o is the depth of water above the grate top for orifice flow (ft).

The U.S. Dept. of Transportation (1984) recommends that a factor of safety of two be used when the danger of the grate clogging exists. The use of these equations is illustrated by Example 5.3.2.

The FHWA Hydraulic Toolbox is available at:

http://www.fhwa.dot.gov/engineering/hydraulics/software/toolbox404.cfm

The Neenah K values can be found on their website at:

http://www.nfco.com/municipal/resources/manning-equation-calculator

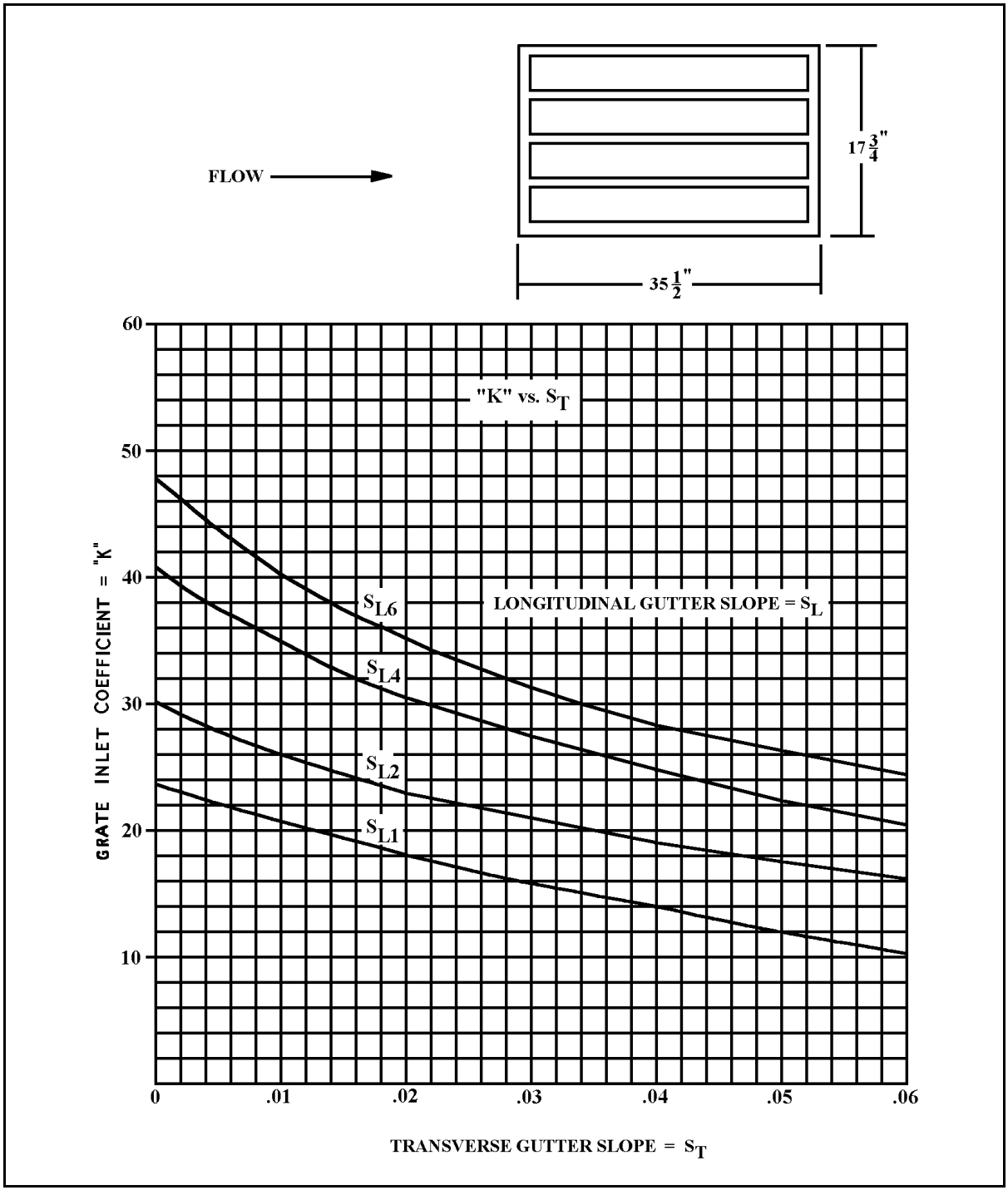


Figure 5.3.2
Typical Inlet Grate Capacity Chart (Neenah, 1987)
Note: S_{L6} = 6%, etc.

Example 5.3.1 (Gutter inlets - continuous grade)

Given: the grate shown in Figure 5.3.2 with a longitudinal slope of 4 %, transverse slope of 2 %, gutter flow rate of 3.0 cfs and roughness coefficient of 0.014.

Calculate: the grate capacity.

From Figure 5.3.2, K is found to be 30.3. The depth is found using Equations 5.2.5.

$$d = \left[1.79(3.0)(0.014)\frac{0.02}{(0.04)^{1/2}} \right]^{3/8} = 0.16 \ ft.$$

Therefore, by using Equation 5.3.1 the grate capacity is found as:

$$Q_i = K\, d^{5/3} = 30.3(0.16)^{5/3} = 1.43\, cfs$$

The inlet efficiency is therefore 1.43/3.0 = 48%. It would be necessary to consult the catalog and select a different grate or use two grates in series if a greater interception efficiency is needed.

Example 5.3.2 (Gutter inlets - sump condition)

Given: a gutter inlet, located in a sag, with a width of 2.5 feet, length of 2.0 feet, and a 50% clear opening.

Calculate: the depth of flow over the inlet when the discharge into the grate is 1.0 cfs and 10.0 cfs.

For a flow of 1 cfs, start by using Equation 5.3.2.

$$Q_i = 3.0\, P(d_w)^{1.5} \qquad \therefore \qquad d_w = \left[\frac{Q_i}{3.0\, P} \right]^{0.67}$$

$$P = 2.0 + 2.5 + 2.5 = 7.0 \ ft.$$

Using the recommended factor of safety of 2, P = 3.5. Therefore, for a flow of 1.0 cfs (using Equation 5.3.2 and rearranging to find d_w (the depth of flow for weir flow):

$$d_w = \left[\frac{1.0}{(3.0)(3.5)}\right]^{0.67} = 0.21 \text{ ft.} < 0.3 \text{ ft. (weir flow)}$$

Check the depth for orifice flow (d_o):

$$d_o = \left[\frac{1.0}{(4.89)(1.25)}\right]^{2} = 0.027 \text{ ft.}$$

Because the depth is less than 0.3 feet for both d_w and d_o, Equation 5.3.2 is applicable for Q_i = 1.0 cfs. For Q_i = 10.0 cfs, the depth is calculated using Equation 5.3.3.

$$Q_i = 4.89\, A\, d_o^{0.5} \quad \therefore \quad d_o = \left[\frac{Q_i}{4.89\, A}\right]^{2} \quad \text{(orifice flow)}$$

where A = (2.5)(2.0)(0.5) = 2.50 ft^2 or using the factor of safety, A = 1.25 ft^2.
Therefore,

$$d_o = \left[\frac{10.0}{(4.89)(1.25)}\right]^{2} = 2.68 \text{ ft.} \geq 0.4 \text{ ft. (orifice flow)}$$

$$d_w = \left[\frac{10.0}{(3.0)(3.5)}\right]^{0.67} = 0.96 \text{ ft.} \quad \text{(weir flow)}$$

Checking for both the weir and orifice flow, the results indicate that the inlet acts as an orifice because the depth is greater than 0.4 feet. The resulting depth for Q_i = 10.0 cfs is 2.68 feet.

A curb inlet, as the name implies, is located directly in the curb, causing little interference to traffic. This type of inlet is not as susceptible to clogging and may be used for streets with mild slopes. The capacity of a curb inlet depends on street slopes (longitudinal and transverse), curb opening length, the top width and depth of the flow at the curb, and the depth and width of the inlet depression. The inlet depression and geometry has a significant effect on its capacity. The symbols and notation for curb inlets are shown on Figure 5.3.3.

The U.S. Dept. of Transportation has revised the methodology for determining the curb inlet lengths required for a set of inputs. The total interception of the gutter flow on a section of pavement with a uniform transverse slope can be calculated by Equation 5.3.4,

$$L_c = 0.6\, Q_G^{0.42}\, S_L^{0.3} \left(\frac{1}{n\, S_T} \right)^{0.6} \qquad (5.3.4)$$

where Q_G is the total gutter flow (cfs), and L_c is the length of curb opening required to intercept 100% of the gutter flow (feet).

The efficiency of curb inlets shorter than L_c is determined by Equation 5.3.5,

$$E = 1 - \left(1 - \frac{L}{L_c} \right)^{1.8} \qquad (5.3.5)$$

where L is the actual curb opening length (feet).

The length of opening for depressed inlets is computed by Equation 5.3.4 but substituting S_e in lieu of S_T. This is the equivalent cross-slope which is calculated by Equation 5.3.6, and shown on Figure 5.3.3,

$$S_e = S_T + S_w\, E_o \qquad (5.3.6)$$

where S_w is the transverse slope of the gutter [S_w = (a/12W)], a is the depth of inlet depression (inches), W is the width of depression (feet), and E_o is the ratio of flow in depressed section to total gutter flow.

A graphical solution to E_o is provided by Figure 5.3.4. The use of these equations is illustrated by Example Problem 5.3.3.

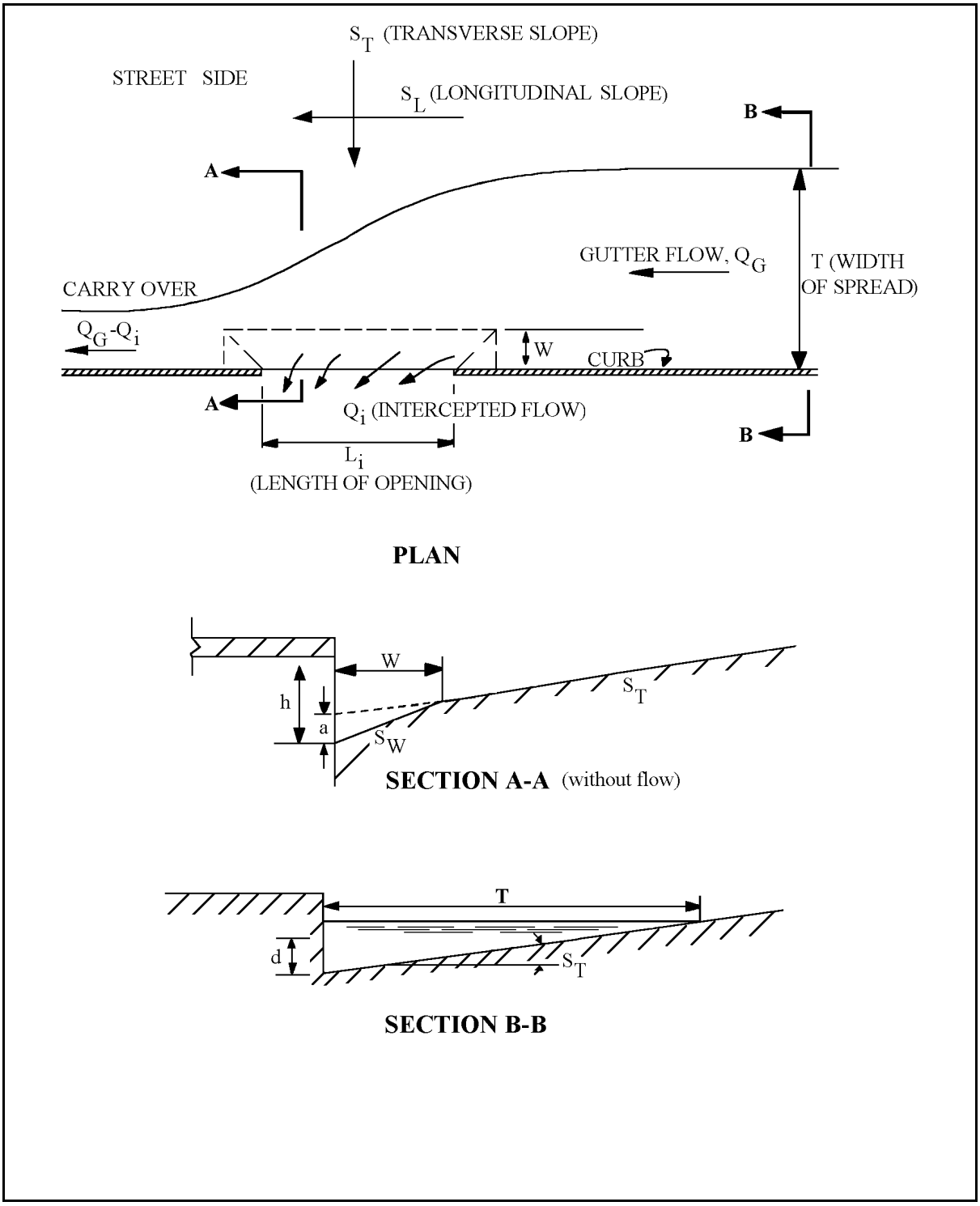


Figure 5.3.3
Plan View and Notations Used for Curb Inlets (Bauer and Woo, 1964)

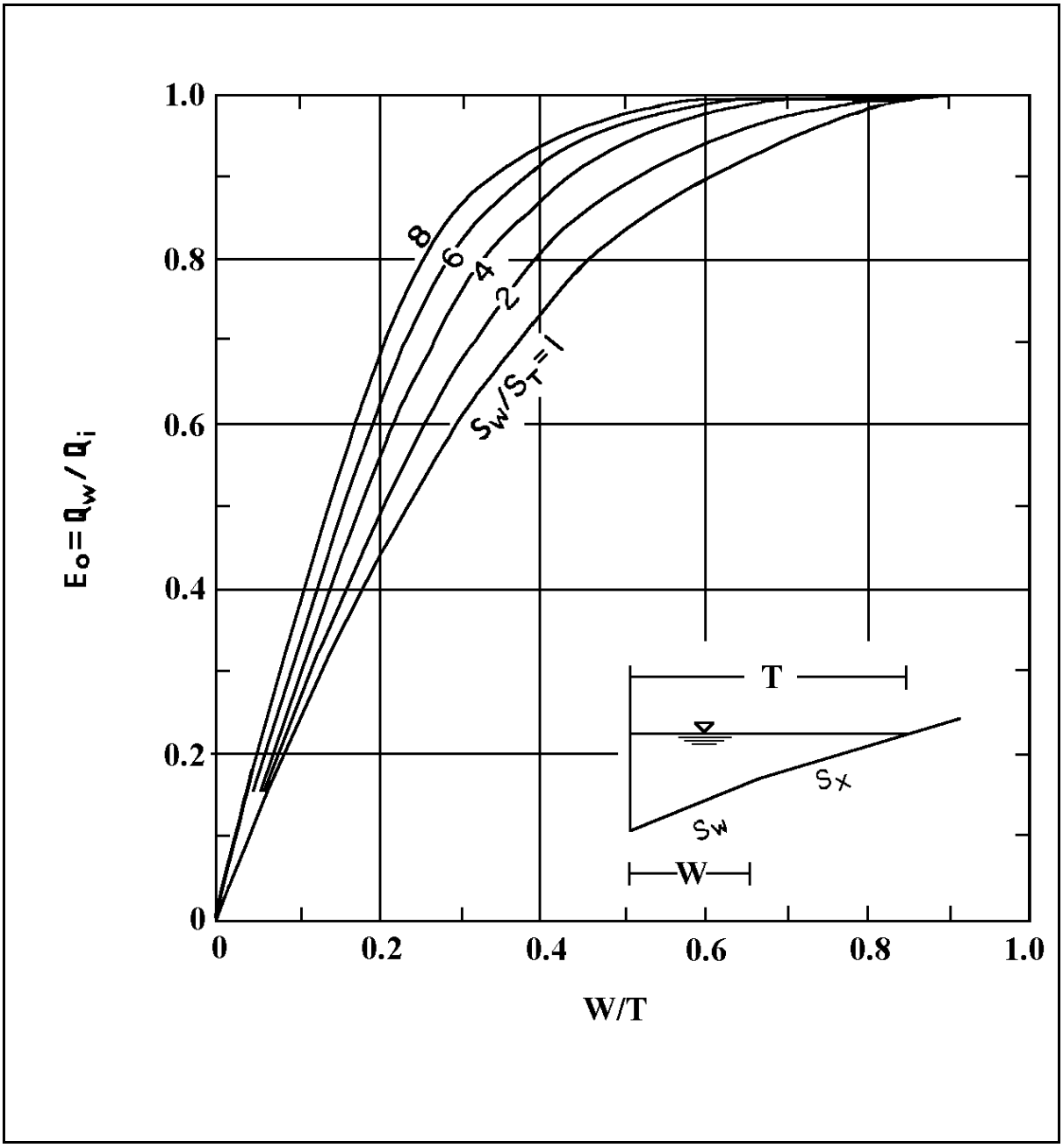


Figure 5.3.4
Ratio of Frontal Flow to Total Gutter Flow (U.S. DOT, 1969)

For a sump condition, the capacity of the inlet depends upon the depth of water at the curb, the opening length and the height of the curb opening. The inlet will act as a weir until the entrance is submerged. When the depth is greater than 1.4 times the curb opening height, the inlet will operate as an orifice. The flow is indeterminate for the intermediate region.

The weir coefficient for a curb inlet is less than the usual coefficient because experimental measurements for determining the coefficients were taken in the curb and not in the inlet, and because there is a drawdown in the water surface between the point where the measurements are made and the weir. The weir location for a depressed curb inlet is at the edge of the gutter while the weir length is dependent on the width of the depression and the curb opening length. For a non-depressed curb inlet, the weir location is at the lip of the opening and the corresponding length is equal to the inlet length. The weir coefficient for depressed curb inlets and those without depression are approximately equal. For a depressed curb inlet which acts as a weir, the equation for the interception capacity is given by Equation 5.3.7,

$$Q_i = C_w (L + 1.8W)\, d^{1.5} \qquad \left(d \le h + \frac{a}{12} \right) \tag{5.3.7}$$

where C_w is equal to 2.3, W is the width of depression (feet), a is the depth of depression (inches), and h is the curb opening height (see Figure 5.3.3)

For a curb inlet without depression which acts as a weir (W=0), the equation for the interception capacity is Equation 5.3.8.

$$Q_i = C_w L\, d^{1.5} \qquad (d \le h) \tag{5.3.8}$$

A depressed curb inlet acts as an orifice when the depth exceeds 1.4h. The interception equation for this condition is Equation 5.3.9,

$$Q_i = 0.67\, A_g \left[2g \left(d - \frac{h}{2} \right) \right]^{1/2} \qquad (d \ge 1.4\, h) \tag{5.3.9}$$

where A_g is the area of opening equal to hL.

Equation 5.3.9 is also applicable to depressed curb-opening inlets and the depth at the inlet includes any gutter depression.

Example 5.3.3 (Curb inlet - continuous grade)

Given: a gutter with a flowrate of 2.0 cfs, transverse slope of 2 percent, longitudinal slope of 3 percent, depression width of 2 feet, and inlet depression of 2 inches. The gutter has a concrete broom finish (n = 0.016).

Calculate: the intercepted flow for curb inlet lengths of 5, 10 and 15 feet.

From Equation 5.2.5 the depth of flow in the channel at the curb is found to be:

$$d = [1.79(2.0)(0.016)\frac{(0.02)}{\sqrt{0.03}}]^{3/8} = 0.15 \ ft.$$

The gutter flow spread is:

$$T = \frac{d}{S_T} = \frac{0.15}{0.02} = 7.6 \ ft.$$

The slope of the depressed area is:

$$S_w = \frac{a}{12W} = \frac{2}{24} = 0.083$$

$$The\ ratio\ of\ \frac{S_w}{S_T} = \frac{0.083}{0.02} = 4.17$$

$$The\ ratio\ of\ \frac{W}{T} = \frac{2}{7.6} = 0.26$$

From Figure 5.3.4, the ratio E_o = 0.73

Therefore, S_e is be calculated from Equation 5.3.6:

$$S_e = S_T + S_W E_o = 0.02 + 0.083(0.73) = 0.081$$

Example 5.3.3 (Continued)

The curb opening length for 100% efficiency can now be calculated from Equation 5.3.4 as:

$$L_C = 0.6\,Q^{0.42}\,S_L^{0.3}\left[\frac{1}{n\,S_e}\right]^{0.6} = (0.6)(2^{0.42})(0.03^{0.3})\left[\frac{1}{(0.016)(0.081)}\right]^{0.6}$$

$$L_C = 15.14 \text{ ft.}$$

The efficiency for a five foot curb opening can be calculated from Equation 5.3.5 as:

$$E = 1 - \left(1 - \frac{L}{L_C}\right)^{1.8} = 1 - \left(1 - \frac{5}{15.14}\right)^{1.8} = 0.51$$

This means that the total intercepted flow is (0.51)(2 cfs) or 1.02 cfs. The same procedure is used to calculate the efficiency and flows captured by a 10 and 15 foot long opening. The results are summarized below:

Length (ft)	E_o	Q_i *(cfs)*
5	0.51	1.02
10	0.86	1.72
15	1.00	2.00

Example 5.3.4 (Curb inlet - sump condition)

Given: a curb inlet located in a sump with the following information.

$S_T = 0.05$ L = 5 feet
h = 5" = 0.42' T = 8 feet

Calculate: the intercepted flow for an *undepressed* curb opening.

$$d = T\,S_T = (8)(0.05) = 0.4\ ft. = 4.8\ inches$$

Since d < h, the flow is weir type and Q_i is computed from Equation 5.3.8.

$$Q_i = C_W\,L\,d^{1.5} = (2.3)(5.0)(0.4)^{1.5} = 2.91\ cfs$$

For a *depressed* curb opening with a = 2", W = 2 feet (all other information remaining the same).

$$h + \frac{a}{12} = 0.42 + \frac{2}{12} = 0.59\ ft. > (1.4)d \quad \therefore use\ Equation\ 5.3.7$$

$$Q_i = 2.3(L + 1.8W)\,d^{1.5} = 2.3(5.0 + 1.8(2))(0.4)^{1.5} = 5.0\ cfs$$

The depressed curb opening inlet is about 72% more efficient then the undepressed opening.

5.3.3 - Combined Inlets

Combined inlets have both a gutter and curb inlet and are used when there is a possibility that the gutter grate may be clogged frequently. The capacity of a combined inlet on a continuous grade may be computed by ignoring the curb inlet and calculating the interception of the gutter inlet alone.

Combination inlets are strongly recommended when a sump condition exists. The curb inlet will provide relief if the gutter grate becomes clogged. The capacity of the grate may be computed using Equation 5.3.2 or Equation 5.3.3 depending upon the depth of the water. The safety factor of two, however, is usually ignored.

5.3.4 - Slotted Drain Inlets

A typical slotted inlet drain is shown on Figure 5.3.5. This type of inlet consists of a long narrow inlet acting as a slot into a pipe. These inlets are used on curbed or uncurbed sections and minimize traffic interference. The disadvantages of the slotted drain inlet are the deposition in the pipe and the fact that there is only a minimal amount of information available on their ability to handle debris. However it is easy and relatively inexpensive to add length to a slotted drain inlet. The U.S. Dept. of Transportation analysis of the slotted drain inlets suggests that when the width of the drain is greater than 1.75 inches, the interception capacity be computed by using the same methods as a curb inlet. In other words, Equations 5.3.4 and 5.3.5 are used to compute the capacity of a slotted inlet on grade.

Slotted drains located in a sag act as weirs until a depth of about 0.2 feet, depending on the slot width and length. When the depths exceed 0.4 feet slotted drains act as orifices. Depths between 0.2 feet and 0.4 feet are in a transition stage. When the slot acts as an orifice, the intercepted flow is calculated by Equation 5.3.10,

$$Q_i = 0.8\, L W_s (2gd)^{0.5} \qquad (d \geq 0.4\ ft) \tag{5.3.10}$$

where g is the acceleration due to gravity and W_s is the width of slot (feet); for

$$W_s = 1.75", \; Q_i = 0.94\, Ld^{0.5}.$$

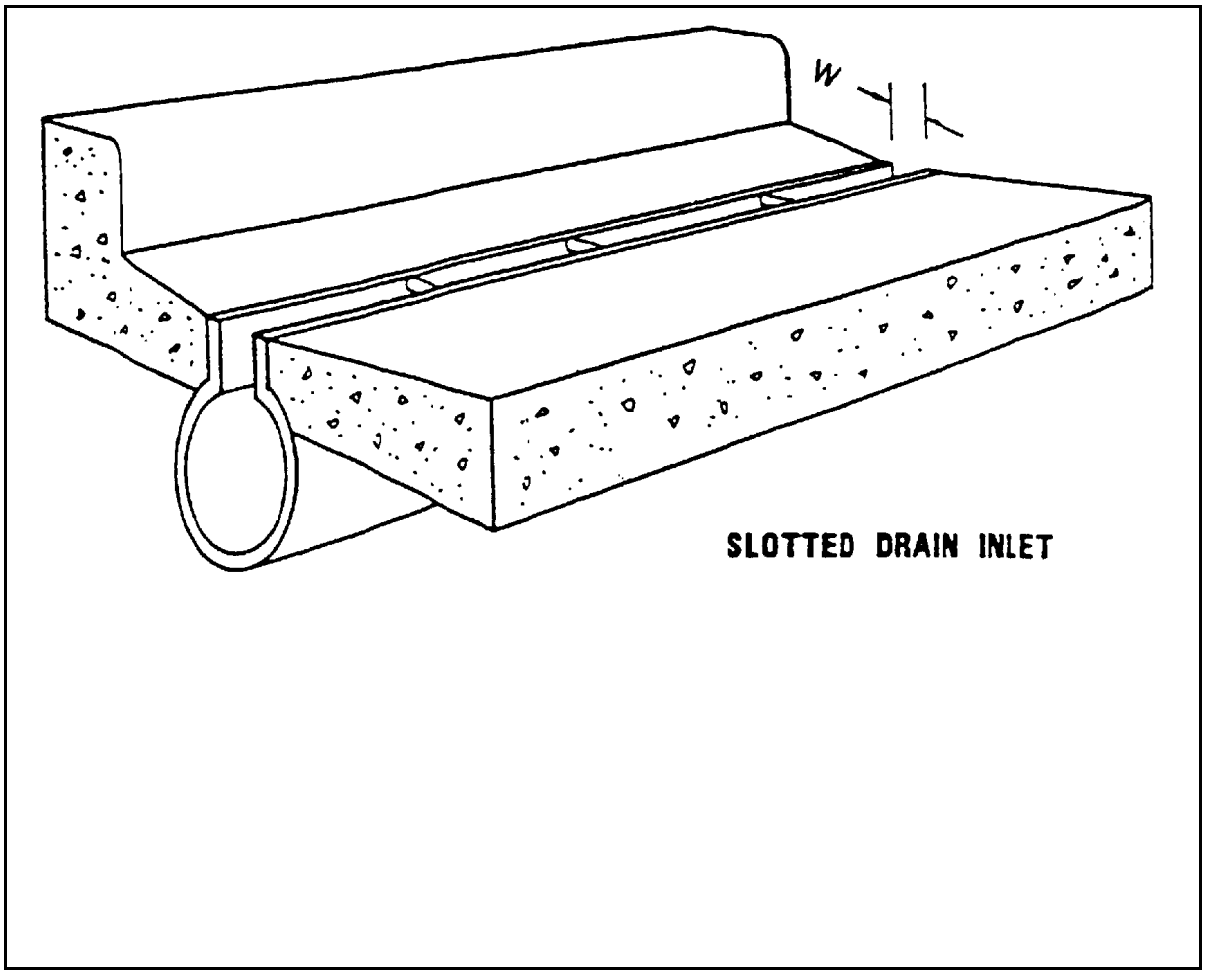


Figure 5.3.5
Typical Slotted Drain Inlet (U.S. DOT, 1984)

For depths between 0.2 and 0.4 feet the capacity can be calculated using the orifice equation with the coefficient dependent on depth, slot width, and length. A graphical solution of a slotted drain inlet in a sump condition is presented in Figure 5.3.6.

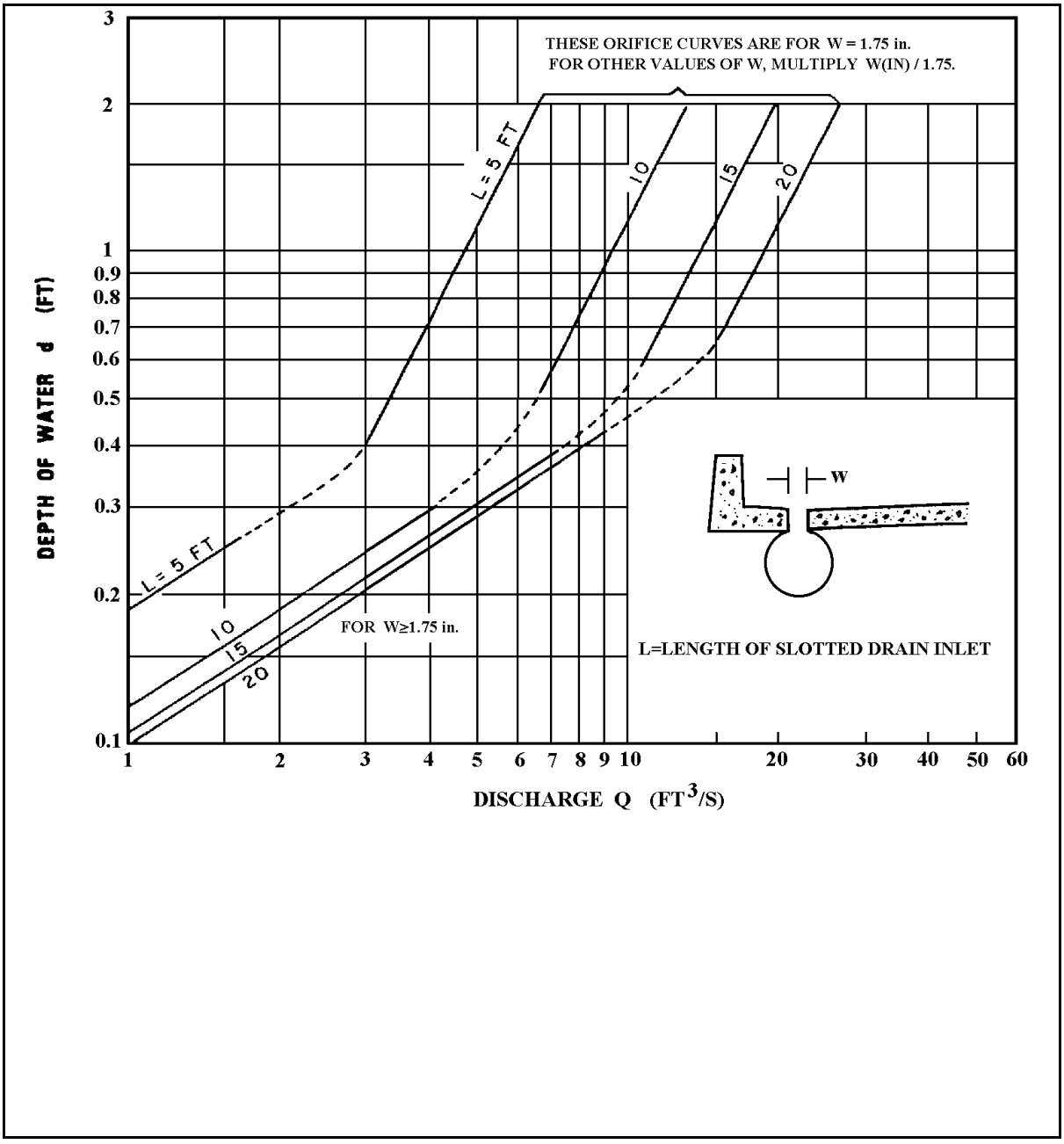


Figure 5.3.6
Slotted Drain Inlet Capacity in Sump Locations, (U.S. DOT, 1984)

Example 5.3.5 (Slotted Drain Inlet - Sump Condition)

Given: a slotted inlet with the following information:

W_s = 1.75 in.	S_T = 0.05
Q = 5 cfs	S_L = 0.02

Calculate: the length of slotted drain inlet to limit depth at curb to 0.3 feet and 0.6 feet.

a. For d = 0.30, we have transition flow, and from Figure 5.3.6 the length of inlet required to intercept 100% of the flow is determined to be 15 feet.

b. For d = 0.60 feet, we have orifice flow and we can either use Figure 5.3.6 or Equation 5.3.10.

$$Q_i = 0.94\,LD^{0.5}, \quad for\ W_s = 1.75 \qquad \therefore$$

$$L = \frac{Q_i}{(0.94)(d)^{0.5}} = \frac{5\,(cfs)}{(0.94)(0.6)^{0.5}} = 6.9 \cong 7\ \ ft.$$

5.4 - INLET DESIGN

The design criteria and methods for the design of stormwater inlets are presented in this section. This information will aid in inlet selection and spacing. Listed below are some general requirements which must be considered prior to inlet selection:

1. The inlet should have sufficient capacity to intercept stormwater from the gutter.

2. The inlet should be designed so as to control the ponding of flow.

3. The inlet should be able to pass small debris (e.g. leaves) while screening out larger, harmful debris (e.g. branches of trees).

4. The inlet should have sufficient strength to resist traffic loadings.

5. Inlet grates should not be hazardous to bicycles.

5.4.1 - Design Criteria

The following is a list of some general criteria for the design of stormwater inlets:

Return Period: The inlet should be designed for a ten (10) year return period. (This is a requirement of the Indiana Department of Transportation, Hydraulics Engineering Section).

Encroachment: The spread, top width or encroachment of stormwater into the street should be less than 4 feet for streets without parking; and less than 12 feet or a maximum depth of 0.35 feet for a street with parking.

Minimum Gutter Slope: The minimum longitudinal slope of the gutter, S_L, is 0.4%,

Minimum Transverse Slope: The minimum transverse slope of the street, S_T, is 1%.

Inlet Locations: Inlets should be placed at all low points in the grade of the gutter and at intersections to prevent stormwater from flowing across traffic lanes and crosswalks. In addition, inlets should be placed when the top width (see above) or the flow depth exceeds the maximum allowable limits.

Spacing of Inlets: Inlets should be spaced so as to capture the flow from the drainage area contributing to the inlet plus any stormwater not intercepted by the stream inlet (carryover) less any flow intentionally bypassed. Using the rational method to calculate the flow from the contributing drainage area (see Section 3.2) the flow in the gutter can be written as,

$$Q_G = CiA + Q_c \qquad (5.4.1)$$

where Q_G is the flowrate in gutter at the inlet location (cfs), Q_c is the carryover from upgrade

inlet (cfs), (Q_c is zero for the first inlet), C is the runoff coefficient, A is the drainage area (acres), and i is the rainfall intensity (inches/hr).

The capacity of the inlet Q_i which is a function of the longitudinal and transverse slopes and inlet geometry can be determined by using the appropriate equations or figures. The depth of flow used in these equations or figures is determined by using the criteria presented above under "Encroachment". If the spacing is to be the maximum distance apart, the inlet capacity is found by Equation 5.4.2,

$$Q_i = Q_G - Q_b \tag{5.4.2}$$

where Q_b is the flow bypassed by this inlet (cfs). (This could be zero for 100% efficiency)

The runoff coefficient C for the contributing area is calculated and it is recommended (Izzard, 1946) that a five-minute time of concentration be used to compute the rainfall intensity. This is usually a conservative assumption but it is also the maximum intensity which is usually found on an intensity-duration-frequency curve, for a given frequency.

The contributing area is assumed to be the product of the length and a constant width. If this assumption and Equation 5.4.2 are substituted into Equation 5.4.1, the length between inlets is given by Equation 5.4.3,

$$L = \frac{43{,}560\,(Q_i + Q_b - Q_C)}{Ciw} \tag{5.4.3}$$

where L is the distance between inlets (ft), w is the width of contributing area (ft) (assumed to be a constant), C is the composite runoff coefficient for contributing area (see Chapter 3).
NOTE: 43,560 converts acres to square feet and $Q_i + Q_b = Q_G$.

Figure 5.4.1 is included to aid in the computations for a stormwater collection system. This method is illustrated in Example 5.4.1.

COMPUTATION SHEET FOR INLET SPACING

Inlet Location			Roadway and Gutter Data							Inlet Data			Contributing Area Data			Inlet Spacing			
Inlet Number	Station	Road or Street	Inlet Type and Condition	Trans. Slope S_T (%)	Longit. Slope S_L (%)	Manning's Roughness Coefficient *n*	Depth of Flow d (ft)	Top Width T (ft)	Flow Rate Q (cfs)	Depression Width W (ft)	Depression Depth a (in)	Inlet Capac. Q_i (cfs)	Width w (ft)	Composite Runoff Coefficient C	Rainfall Intensity (in/hr)	Upgrade Carryover $Q_C=Q_G - Q_i$ (cfs)	Inlet Bypass Q_b (cfs)	$Q_i+Q_b-Q_C$ (cfs)	Inlet Spacing L (ft)

Figure 5.4.1
Computation Sheet for Inlet Spacing

Table 5.4.1 shows the suggested inlet spacing for the allowable water spread. Main trunk lines should be sized with pipe flowing full, but not placed under pressure for the 10-year peak discharge (Q_{10}). The hydraulic gradeline shall not exceed the top of the inlet or manhole for the 50-year peak discharge (Q_{50}). (INDOT, 1993)

Table 5.4.1
Allowable Water Spread (INDOT, 1993)

TYPE OF FACILITY	ALLOWABLE SPREAD "T"
Freeways	Edge of travel lane for Q_{50}
Multilane Non-Freeways	Spread acceptable across one entire travel lane for Q_{10}
All Two-Lane Freeways	4 feet onto travel lane for Q_{10}
Bridge Decks	4 feet onto travel lane for Q_{10}
Ramps	8 feet of roadway must remain clear of water during Q_{10}

It is often necessary to locate an inlet along a vertical curve or on a roadway with multiple slopes. With this situation, the equivalent longitudinal slope is a function of the vertical curve grade and the distance from the high or low point to the tangent point. The terms used in determining S_L for a vertical curve are presented in Figure 5.4.2.

For an inlet located on the vertical crest curve, the mean slope, $\overline{S_L}$, is calculated using Equation 5.4.4.

$$\overline{S_L} = \frac{LG}{2D_H} \tag{5.4.4}$$

where G is the tangent grade and D_H is the distance from high point to tangent point.

If Equation 5.4.4 is substituted into Equation 5.2.4 (to calculate Q_G) and the result is substituted into Equation 5.4.3 (with $Q_C = 0$), then Equation 5.4.5 gives the spacing for the first inlet on a crest vertical curve.

$$L = \left[\frac{24{,}394}{Ciwn}\frac{d^{8/3}}{S_T}\right]^2 \frac{G}{2D_H} \qquad (L < D_H) \tag{5.4.5}$$

After the initial spacing is calculated, the mean slope from Equation 5.3.9 is used with the depth of water, transverse slope and inlet geometry to determine the grate efficiency, carryover and bypass.

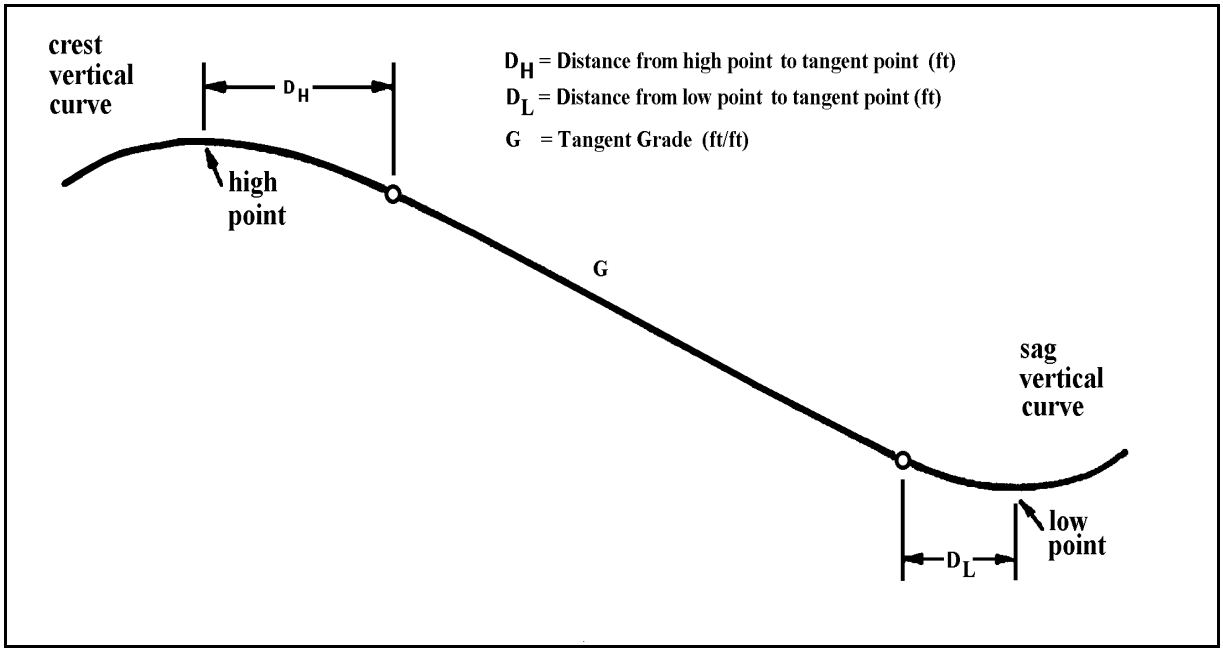


Figure 5.4.2
Grade Profile for a Hypothetical Roadway

The procedure for locating an inlet on a sag vertical curve is analogous to the crest vertical curve, except a safety factor of two is used to compensate for the high flooding potential of "valleys" and the likelihood of clogging. The mean slope is calculated by using Equation 5.4.6 and the spacing is found by using Equation 5.4.7.

$$\overline{S_L} = \frac{LG}{4D_L} \tag{5.4.6}$$

$$L = \left[\frac{24{,}394}{wCin}\frac{d^{8/3}}{S_T}\right]^2 \frac{G}{4D_L} \qquad (L < D_L) \tag{5.4.7}$$

It is assumed that L falls within the distance D_H or D_L in Equations 5.4.5 and 5.4.7. If it does not, the mean slope of the street is simply calculated by weighting individual slopes between the break points over a given length of gutter, as shown in Figure 5.4.2. This mean slope is used to compute spacing as though the inlet is located on a constant grade.

Example 5.4.1 (Gutter Inlet Spacing - Rational Method)

Given: A 24' wide road without parking, shown on Figure 5.4.3, is located in Indianapolis. It has a longitudinal slope of 1.0 percent and a transverse slope of 3.0 percent. Assume that the

street has an asphalt pavement with a smooth finish and that a 15 foot strip of the adjacent parkway contributes to the inlet.

Calculate: the spacing and location of the first two (2) gutter inlets, using the grate shown in Figure 5.3.2, design data presented in Section 5.3.4, and design calculation sheet in Figure 5.4.1.

Step 1: Determine the maximum flow in the street

Since the maximum top width can only be 4 feet, the depth of flow can be calculated as:

$$d = S_T T = (0.03)(4) = 0.12 \ \ feet$$

From Table 4.2, n = 0.013

The maximum gutter flowrate is determined using Equation 5.2.4.

$$Q_G = \frac{0.56\,(0.01)^{1/2}}{(0.013)(0.03)}(0.12)^{8/3} = 0.50 \ cfs$$

Step 2: Using the depth and street slopes, calculate the grate capacity.

From Figure 5.3.2, K = 16, and from Equation 5.3.1:

$$Q_i = K(d)^{5/3} = 16(0.12)^{5/3} = 0.47 \ cfs$$

The bypass Q_b is 0.5 cfs - 0.47 cfs or 0.03 cfs. This will be passed to the second inlet.

Step 3: Compute a composite runoff coefficient

Assume a value C of 0.25 for the parkway. The composite coefficient for a 1 foot length of pavement is:

$$C = \frac{12(0.95) + 15(0.25)}{27} = 0.56$$

Step 4: Calculate the rainfall intensity

For the recommended 10-year return period and a five-minute time of concentration, the rainfall intensity for Indianapolis is found using the equation on page 2-22 from Example 2.2.3.

$$i = \frac{c\,T_r^{\alpha}}{(t + d)^{\beta}} \ \ \therefore \ \ i_5^{10} = \frac{2.1048\,(10)^{0.1733}}{\left(\frac{5}{60} + 0.47\right)^{1.1289}} = 6.12 \ inches/hr$$

Step 5: Calculate the spacing to the first inlet by using Equation 5.4.3

This is the required distance from the start of a subbasin to the first inlet.

$$L = \frac{43{,}560\,(Q_i + Q_b - Q_C)}{Ciw} = \frac{(43{,}560)(0.50)}{(0.56)(6.12)(12+15)} = 235 \text{ feet}$$

This is the spacing for one side of the street only. If the same width of parkway contributed to both sides, then there would be an inlet at this location on the opposite side.

Step 6: Calculate the spacing to the next inlet

The total area contributing to the next inlet is reduced since this inlet must catch the carry-over of 0.03 cfs.

$$L = \frac{(43{,}560)(0.47 + 0.03 - 0.03)}{(0.56)(6.12)(12+15)} = 221 \text{ feet}$$

NOTE: This assumes that there will be a carry-over of 0.03 cfs to the next inlet on this grade.

These calculations are also shown in Figure 5.4.4.

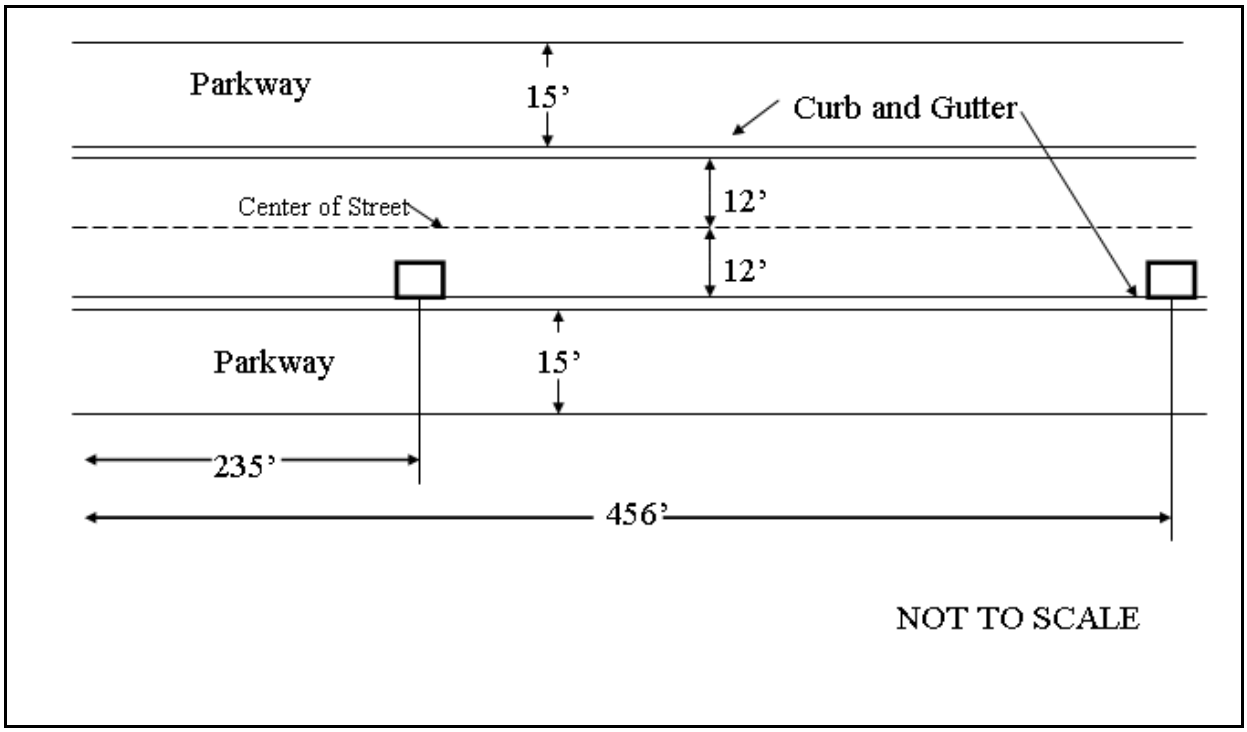


Figure 5.4.3
Depiction of Street Described in Example 5.4.1

COMPUTATION SHEET FOR INLET SPACING

Inlet Location			Roadway and Gutter Data							Inlet Data			Contributing Area Data			Inlet Spacing			
										Depression									
Inlet Number	Station	Road or Street	Inlet Type and Condition	Trans. Slope S_T (%)	Longit. Slope S_L (%)	Manning's Roughness Coefficient *n*	Depth of Flow d (ft)	Top Width T (ft)	Flow Rate Q (cfs)	Width W (ft)	Depth a (in)	Inlet Capac. Q_i (cfs)	Width w (ft)	Composite Runoff Coefficient C	Rainfall Intensity (in/hr)	Upgrade Carryover $Q_C=Q_G - Q_i$ (cfs)	Inlet Bypass Q_b (cfs)	$Q_i+Q_b-Q_C$ (cfs)	Inlet Spacing L (ft)
1	2 + 35	Sample	Grate Cont. Grade	3.0	1.0	0.013	0.12	4	0.5	- -	- -	0.47	27	0.56	6.12	- -	0.03	0.50	235
2	4 + 56	Sample	Grate Cont. Grade	3.0	1.0	0.013	0.12	4	0.5	- -	- -	0.47	27	0.56	6.12	0.03	0.03	0.47	221

Figure 5.4.4
Sample Calculation for Example 5.4.1

5.5 - COMPUTER PROGRAMS

It should be obvious that the rather tedious calculations required for the sizing and location of gutters and inlets are well suited to the computer. A couple of computer programs which have been developed for solving problems similar to the examples in this chapter are listed below.

5.5.1 - HEC-12

The HEC-12 Drop Inlet Design and Analysis Program automates calculations for finding the interception capacity of roadway and median inlets. It will calculate the pavement spread and required length of the drop inlet to intercept all storm flow. HEC-12 uses the procedure outlined by the Federal Highway Administration's Hydraulic Engineering Circular No. 12, "Drainage of Highway Pavements." The peak discharge can be directly entered or the program can calculate it using the Rational Method.

The program handles the following inlet types: curb, grate, combination grate and curb, 4-inch bridge scupper, slotted drain, and grate in trapezoidal ditch. It can handle inlets on continuous grades or in sumps, analyze existing inlets, and add the discharge from one inlet to another. (U.S. DOT, 1984)

5.5.2 - Neenah Foundry Company's Inlet Grate Capacities

The Neenah Foundry Company has a computer program available for Inlet Grate Capacities. The program allows the user to evaluate grate efficiencies under differing flow conditions. Two separate sub-programs are present. One evaluates inlet capacities, where the parameters of flow, longitudinal and transverse slopes, plugging factor and grate selection are input. The program then displays the depth of flow onto the pavement, and the amount of flow captured by the individual grate being evaluated. The program has the capability of evaluating a compound gutter section where the transverse slope of the gutter is greater than the transverse slope of the pavement.

The second sub-program evaluates an inlet at low points, i.e. sump conditions. The free open area of the grate and the wetted perimeter are input and a graph of head vs. flow is generated for both weir and orifice flow (Neenah, 1990). The program and supporting documentation can be obtained from the Neenah Foundry Company, Product Engineering Department, P.O. Box 729, Neenah, Wisconsin, 54957; (414) 725-7000.

Chapter 5 - REFERENCES

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Chapter 6 - STORMWATER STORAGE

LIST OF PARAMETERS

A	Area (acres or hectares)
A’	Cross-sectional area of soil (ft^2 or m^2)
A_D	Contributing developed area
A_o	Area of the orifice (ft^2 or m^2)
A_T	Total contributing area (acres or hectares)
A_U	Contributing undeveloped area
C_d	Coefficient of discharge
C_D	Developed runoff coefficient
C_U	Undeveloped runoff coefficient
d	Depth (ft or m)
D	Pipe diameter (ft or m)
ΔD	Rainfall duration
g	Acceleration due to gravity (32.2 ft/sec^2 or 9.81 m/sec^2)
h_L	Head loss or the gradient over a flow distance L (ft or m)
h_o	Height of water above the orifice (ft or m)
h_p	Height of water above the pipe opening (ft or m)
h_w	Height of water above the weir opening (ft or m)
i_d	Rainfall intensity at time t_d (in/hr or cm/hr)
i_U	Undeveloped rainfall intensity (in/hr or cm/hr)
I(t)	Inflow at time t (cfs or m^3/sec)
K	Coefficient of permeability
K_e	Entrance loss coefficient
K_o	Outlet loss coefficient
L	Length (ft or m)
n	Manning’s roughness coefficient
O(t)	Outflow or runoff (cfs or m^3/sec) or (cfs/acre or m^3/sec / hectare)
P(t)	Accumulated rainfall (inches)
Q	Infiltration flows (cfs or m^3/sec)
Q_U	Peak undeveloped flow (cfs or m^3/sec)
q_p	Peak flow (cfs or m^3/sec)
$Q_I(t)$	Magnitude of the inflow hydrograph at time t
$Q_O(t)$	Magnitude of the outflow hydrograph at time t
R(t)	Direct surface runoff (inches or cm)
S	Ultimate abstraction
$S(t_d)$	Required storage (acre-ft)
ΔS	Change in storage (ft^3 or m^3)
S_{max}	Required detention storage
t’	Time at which inflow is equal to outflow
t_b	Time of base
t_c	Time of concentration
t_d	Storm duration

LIST OF PARAMETERS (cont'd)

t_p	Time to peak
t_t	Time of travel (hrs)
Δt	Time interval (sec)
v	Velocity (ft/sec or m/sec)
V	Volume (ft^3 or m^3)
$V_I(t_d)$	Inflow volume
$V_o(t_d)$	Outflow volume

CHAPTER 6 - STORMWATER STORAGE

6.1 - INTRODUCTION

In Chapter 3, the response of an "urbanized" watershed to a storm was shown to be considerably different than that of rural, undeveloped watershed. Land which once allowed infiltration of rainwater is covered with impervious parking lots, streets, and building or homes, thereby causing a higher percentage of rainfall to become surface runoff. Improvements such as streets, curbs, and storm sewers collect and convey this runoff to its point of disposal more rapidly than in an undeveloped watershed, resulting in a higher runoff volume in a shorter period of time.

These increased runoff rates produced by the urbanization of a watershed may overtax succeeding portions of the larger drainage network causing flooding problems downstream. To circumvent this difficulty, storage facilities are provided to receive the runoff from the developed watershed while releasing water to the larger drainage network at a reduced rate. This reduced rate is determined by using parameters fixed by local ordinance or by calculating the available capacity of the downstream conveyance system.

Various types of storage facilities used and the methods employed in computing required storage volumes are discussed in this chapter. Design considerations are outlined and example problems are included to illustrate applications.

6.1.1 - Types of Storage Facilities

Storage facilities can be separated into two general categories: detention and retention.

Detention storage involves detaining or slowing the runoff and then releasing it. A *detention basin* has a positive outlet that completely empties all runoff between storms. In some situations, the excavation of a detention facility may extend below the water table or outlet level where the bottom is sealed by sedimentation. This case is referred to as a *detention pond* or *wet bottom detention basin*. The detention pond also has a positive outlet and releases all temporary storage.

Retention facilities retain runoff for an indefinite amount of time and have no positive outlet. Runoff is removed only by infiltration through a porous bottom or by evaporation. Retention ponds and lakes are examples of retention facilities that may be built in a development and in many cases, enhance the overall project.

Both types of storage are very common, although the designed retention becomes less practical as the size of the drainage area increases. Retention ponds may need 2 to 7 times more permanent pool volume than the temporary storage volume needed for a detention basin (Maidment, 1993). The use and design of these facilities are outlined in Section 6.3.

6.2 - COMPUTATION OF STORAGE VOLUMES

As previously stated, storage facilities are designed to receive the stormwater collected by a drainage system for a development and release it at a specified rate which may be zero for retention facilities. The primary goal is to provide the storage volume necessary to offset the impacts of urbanization. If infiltration and evaporation are neglected during the period of runoff, the continuity equation for a detention pond may be written as:

$$I(t) - O(t) = \frac{\Delta S}{\Delta t} \qquad (6.2.1)$$

where I(t) is the inflow to the pond from the sewer network at time (t) (cfs), O(t) is the outflow from the pond into the larger drainage network at time (t) (cfs), and ΔS is the change in storage (ft^3) in time interval (Δt)(sec).

Equation 6.2.1 may also be written as:

$$(I_1 + I_2)\frac{\Delta t}{2} - (O_1 + O_2)\frac{\Delta t}{2} = S_2 - S_1 \qquad (6.2.2)$$

where subscripts 1 and 2 denote the flows, volumes, and storage at times t_1 and t_2 and $\Delta t = t_1 - t_2$.

When the inflow and outflow hydrographs are known, the largest value of $S_2 - S_1$ found in Equation 6.2.2 is the required storage. Normally the hydrographs are not known in the design phase and are usually estimated. The following is a discussion of some of the methods used to estimate the storage in the design of detention facilities. For retention facilities the value of O(t) is equal to the sum of the evaporation and infiltration rates. During a storm these may be negligible and the required storage volume is therefore equal to the runoff volume.

6.2.1 – Rational Method

The rational method discussed in Section 3.2, computes a peak runoff rate only; however, it may be extended to compute volumes by multiplying the peak flow rate by the storm duration.

The peak flow rate permitted to leave the detention pond, O(t), is based on the contributing undeveloped area, A_U, undeveloped runoff coefficient, C_U, and rainfall intensity, i_U, associated with the time of concentration of the undeveloped basin. The return period for the rainfall intensity is fixed by local ordinance or by the design parameters of the larger downstream drainage network. In some cases, the allowable outflow may be even more limited due to the capacity of the succeeding network. Often this release rate is expressed in terms of cfs/acre and the rates range from 0.10 – 0.25. In either case, the outflow is assumed to remain constant for all storm durations, t_d. Therefore the volume at t_d is simply the product

of O(t) and t_d. This is illustrated in Figure 6.2.1 where the line $V_O(t_d)$ is the outflow volume vs. time.

The inflow rate, I(t), is calculated by using the contributing developed area, A_D, the developed runoff coefficient, C_D, and a rainfall intensity, i_d, corresponding to storm duration, t_d, and the return period for the detention pond design. Thus, for various durations, the peak flow and therefore the volume of runoff $V_I(t_d)$ may be computed. Figure 6.2.1 shows the curve of $V_I(t_d)$ vs. t_d.

The maximum difference between the inflow and outflow volumes is the required detention pond storage. This is shown in Figure 6.2.1 as S_{max}.

$$S(t_d) = (C_D i_d A_D - C_U i_U A_U)\frac{t_d}{12} \qquad (6.2.3)$$

where $S(t_d)$ is the required storage (acre-ft), C_D is the developed runoff coefficient, C_U is the undeveloped runoff coefficient, A_D is the area of developed watershed (acres), A_U is the area of undeveloped watershed (acres), i_d is the rainfall intensity (inches/hr) for the return period of the detention pond and duration t_d, i_U is the rainfall intensity corresponding to the time of concentration of the undeveloped watershed and a return period based upon either local ordinance or the capacity of the downstream system (inches/hr), and t_d is the storm duration which is varied to find the maximum peak storage (hrs).

The factor 1/12 in Equation 6.2.3 is used to convert cfs-hours to acre-feet.

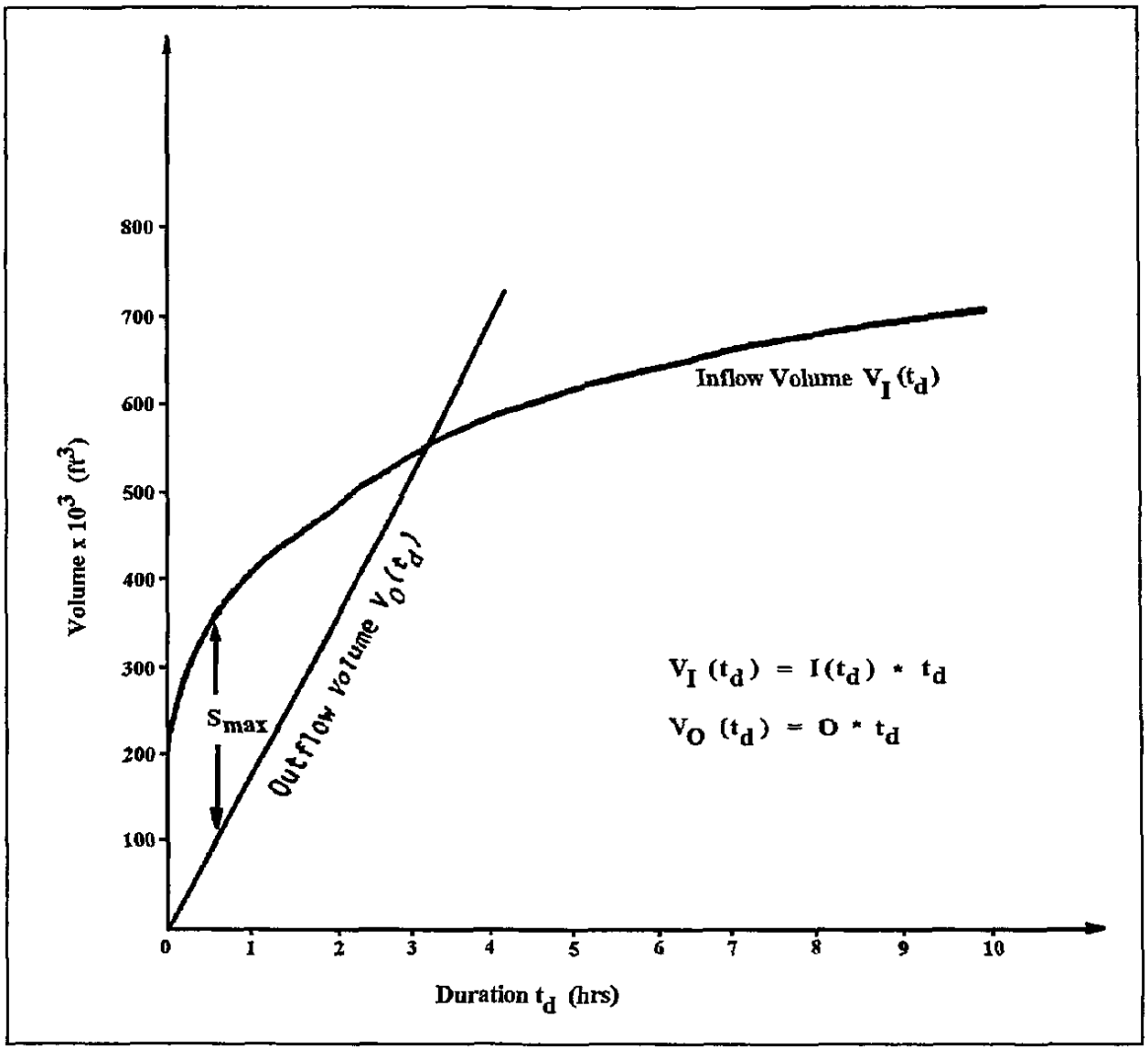


Figure 6.2.1
Graphical Representation of Storage Volumes as Determined by the Rational Method

As in the graphical approach, various storm durations, t_d, are selected and the largest value of $S(t_d)$ obtained is used to design the detention pond. Figure 6.2.2 is a worksheet which may be used for computations based on Equation 6.2.3.

The step by step procedure to compute the storage volume by using the rational method is as follows.

Step 1. Determine the area, A_U, runoff coefficient, C_U, and time of concentration for the undeveloped site. By using the appropriate intensity-duration-frequency curve determine the intensity, i_U, corresponding to the return period for the allowable outflow rate.

Step 2. Calculate the runoff (O(t)) from the undeveloped site. $O(t) = C_U i_U A_U$; or use the allowable release rate determined by other methods.

Step 3. Determine the developed runoff coefficient, C_D.

Step 4. Determine the rainfall intensities (i_d) for various durations (t_d), for the specified return period. Recommended durations are 10, 20, 30, 40, 50 min and 1, 1.5, 2, 3, 4, 5, 6, 7, 8, 9 and 10 hours.

Step 5. Determine the inflow rate to the detention pond. $I(t_d) = C_D i_d A_D$

Step 6. Compute the required storage for each duration,

$$S(t_d) = \frac{t_d}{12}[I(t_d) - O(t)] \qquad \text{(acre-ft)}$$

Step 7. Select the largest volume for designing the detention pond.

Various agencies have set guidelines for the selection of i_U, i_d, C_U and C_d. One agency, the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) (Metropolitan Sanitary District of Greater Chicago prior to 1989), uses the following criteria.

i_U based on a 3-year return period
i_D based on 100-year return period
$C_U \leq 0.15$
$C_D \geq 0.35$

In Example 6.2.1 these criteria are applied and the use of this procedure is demonstrated. A computer program which can be used to perform these computations is included at the end of the chapter. The calculations may also be performed conveniently using a spreadsheet.

Project: _____ **Detention Facility Design Return Period:** _____

Designer: _____ **Release Rate Return Period:** _____

Watershed Area:	acres
Time of Concentration (undeveloped):	minutes
Rainfall Intensity (i_U):	inches/hr
Undeveloped Runoff Coefficent (C_U):	
Undeveloped Runoff Rate (O):	cfs
Developed Runoff Coefficent (C_D)	

Storm Duration t_d (hours)	Rainfall Intensity i_d (inches/hr)	Inflow Rate $I(t_d)$ $(C_D i_D A_{D)}$ (cfs)	Outflow Rate O (cfs)	Storage Rate $I(t_d)$-O (cfs)	Required Storage (I(td)-O)$(t_d/12)$ (acre-feet)
0.17					
0.33					
0.50					
0.67					
0.83					
1.0					
1.5					
2.0					
3.0					
4.0					
5.0					
6.0					
7.0					
8.0					
9.0					
10.0					

Figure 6.2.2
Computation Sheet for Detention Storage Calculations Using the Rational Method

Example 6.2.1

A 10-acre parcel of land located in South Bend, Indiana which is currently a flat pasture with a clay and silt loam soil is to be developed to an area with the following characteristics: 20% parks, 50% single-family homes and 30% business. The overland travel path has a length of 150 feet and slope of 0.01 ft/ft. Determine the size of the detention pond required so that the 100-year runoff from the developed land will not exceed the 3-year runoff of the undeveloped land (MWRDGC guidelines). Use the rational method procedure outlined in Section 6.2.1.

Step 1: Undeveloped Runoff A = 10 acres

From Table 3.2.1, the runoff coefficient is 0.30. However, MWRDGC requires that the undeveloped runoff coefficient be less than or equal to 0.15. The overland flow time of concentration is computed by Kerby's Equation in Table 3.2.4.

$$t_c = 0.83\left[\frac{(150)(0.40)}{\sqrt{0.01}}\right]^{0.467} \quad ; \quad t_c = 16.46 \textit{ minutes}$$

Step 2: The peak undeveloped flow is found by the rational method. Following the procedure outlined in Section 2.2.4, the intensity for a three-year return period and 16.46 minute duration for Evansville, Indiana is found as follows.

c = 1.7204 $\alpha = 0.1753$ d= 0.485 $\beta = 1.6806$

$$i = \frac{c\,T_r^{\alpha}}{(t+d)^{\beta}} = \frac{1.7204(3)^{0.1753}}{\left(\frac{16.46}{60}+0.485\right)^{1.6806}} = 3.313 \textit{ inches/hour}$$

$$Q_U = (0.15)(3.31)(10) = 4.97 \text{ cfs}$$

Step 3: Compute the developed runoff coefficient. From Table 3.1, a composite runoff coefficient can be determined.

	Acres	*Type of Cover*	*Runoff Coefficient*	C_nA_n
	2	Parks	0.17	0.34
	5	Single Family Homes	0.40	2.00
	3	Business	0.60	1.80
TOTAL	10			4.14

$$C_D = \frac{\Sigma C_n A_n}{A_T} = \frac{4.14}{10} = 0.41 > 0.35$$

Step 4: Determine rainfall intensities for a 100-year return period for duration of 10, 20, 30, 40 and 50 minutes and 1, 1.5, 2, 3, 4, 5, 6, 7, 8, 9 and 10 hours for Evansville, Indiana using the following equation (where t is in hours):

For t ≤ 1 hour:

$$i = \frac{c\,T_r^{\alpha}}{(t+d)^{\beta}} = \frac{1.7204(100)^{0.1753}}{(t+0.485)^{1.6806}}$$

For t >1 hour:

$$i = \frac{c\,T_r^{\alpha}}{(t+d)^{\beta}} = \frac{1.2799(100)^{0.1872}}{(t+0.258)^{0.8252}}$$

t (hours)	**i (inches/hr)**	**t (hours)**	**i (inches/hr)**
0.17	7.92	3.00	1.14
0.33	5.40	4.00	0.92
0.50	3.96	5.00	0.77
0.67	3.04	6.00	0.67
0.83	2.42	7.00	0.59
1.00	1.98	8.00	0.53
1.50	1.90	9.00	0.48
2.00	1.55	10.00	0.44

Step 5: Determine the inflow rate, $I(t_d)$, and required storage at each of the above durations (See Figure 6.2.3).

Project: Example 6.21 **Detention Facility Design Return Period:** 100 **years**

Designer: TTB **Release Rate Return Period:** 3 **years**

Watershed Area:	10 acres
Time of Concentration (undeveloped):	16.46 minutes
Rainfall Intensity (i_U):	3.31 inches/hr
Undeveloped Runoff Coefficent (C_U):	0.15
Undeveloped Runoff Rate ($O=C_U i_U A_U$):	4.97 cfs
Developed Runoff Coefficent (C_D)	0.41

Storm Duration t_d (hours)	Rainfall Intensity i_d (inches/hr)	Inflow Rate $C_D i_D A$ (cfs)	Outflow Rate O (cfs)	Storage Rate $I(t_d)$-O (cfs)	Required Storage (I(td)-O)(t_d/12) (acre-feet)	
0.17	7.92	32.48	4.97	27.51	0.38	
0.33	5.40	22.15	4.97	17.18	0.48	← **Peak Storage**
0.50	3.96	16.22	4.97	11.25	0.47	
0.67	3.04	12.47	4.97	7.50	0.42	
0.83	2.42	9.94	4.97	4.97	0.34	
1	1.98	8.14	4.97	3.17	0.26	
1.5	1.90	7.80	4.97	2.83	0.35	
2	1.55	6.35	4.97	1.38	0.23	
3	1.14	4.69	4.97	*	*	
4	0.92	3.76	4.97	*	*	
5	0.77	3.16	4.97	*	*	
6	0.67	2.74	4.97	*	*	
7	0.59	2.42	4.97	*	*	
8	0.53	2.18	4.97	*	*	
9	0.48	1.98	4.97	*	*	
10	0.44	1.82	4.97	*	*	

* Since I(tD) - O< zero, there is no storage needed

Figure 6.2.3
Detention Storage Calculations for Example 6.2.1 Using the Rational Method

Under certain conditions it may be impossible to convey and collect all of the runoff from a given watershed. The result is that some runoff is discharged directly into the larger drainage network without being detained. To compensate for this unrestricted release, the allowable release rate, O(t), is lessened by that amount.

A shortcoming of this procedure is that the release rate is a constant and does not account for changing water surface elevations within the facility which in turn increases the head above and therefore the discharge through a restrictor. The storage volume determined will be smaller than that actually needed. A more accurate procedure is to employ a storage routing program which requires a stage-storage-discharge curve. Chapter 8 presents two computer programs that perform an iterative routing procedure that is more precise for simulating the inflows and outflow and the corresponding storage volume.

6.2.2 - Soil Conservation Service Hydrograph Method

In Section 3.3, methods were presented by which the time distribution of stormwater runoff could be found using rainfall data and certain basin parameters. These hydrographs, developed by the Soil Conservation Service, are easily used to calculate storage requirements for stormwater storage facilities.

As previously described, the difference between the inflow from the developed watershed and the allowable outflow from a detention pond, is the required storage volume. The outflow is determined using characteristics of the undeveloped watershed and a rainfall frequency equal to or less than the receiving system can handle, or as prescribed by local ordinance.

Inflow and outflow hydrographs are shown in Figure 6.2.4, in which the difference between the hydrographs, which is the required storage, is shown shaded. At point C, the inflow rate to the detention pond is equal to the outflow rate (e.g., the pond will start to empty). The outflow rate is assumed to remain constant in this figure, but actually this rate will depend upon the depth of water in the pond and the type of outlet structure (i.e. weir, orifice or pipe-flow). It should also be noted that for a detention pond the total inflow volume is equal to the total outflow volume.

The inflow hydrograph can be determined using the procedure outlined in Chapter 3. The outflow hydrograph is found by using the depth-storage relationship for the outflow structure. Using this information, the required storage volume can be found by calculating the area between the hydrographs. This is found by using a planimeter, digitizer, or by using Equation 6.2.5,

$$S(t_d) = \frac{\Delta t}{12} \sum_{t=\Delta t}^{t'} [Q_I(t) - Q_O(t)] \qquad (6.2.5)$$

where $Q_I(t)$ is the magnitude of the inflow hydrograph at time t, $Q_O(t)$ is the magnitude of the

outflow hydrograph at time t, Δt is the time interval used in calculations, t' is the time at which inflow = outflow (point C in Figure 6.2.4), and $S(t_d)$ is the total required storage (acre-feet).

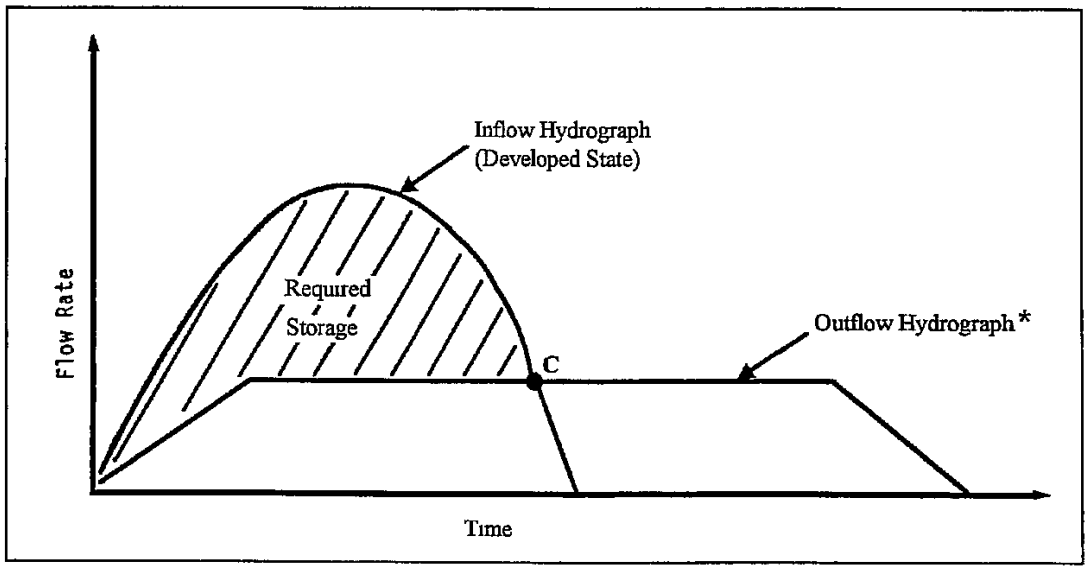


Figure 6.2.4
Inflow and Outflow Hydrographs for a Hypothetical Detention Pond

* Assumes a constant release rate. See Chapter 8 for computer applications with outflows dependent on pond stage.

An outline of the procedure used in determining the required storage with the hydrograph method is as follows.

Step 1. Calculate the curve numbers for the developed and undeveloped basins.

Step 2. Find the times of concentration, t_c, for the basin as outlined in Section 3.3.

Step 3. From t_c, calculate ΔD, q_p, and t_p using Equations 3.4.2 and 3.4.3, respectively, for the developed and undeveloped site.

Step 4. Determine the coordinates of the inflow and outflow unit hydrographs by using Figure 3.4.3 for a dimensionless unit hydrograph or by plotting a triangular unit hydrograph.

Step 5. From the design storm duration, depth, time distribution and frequency, calculate the cumulative rainfall at time interval ΔD for both the undeveloped and developed states.

Step 6. Using the basin curve number and ultimate abstraction, S, calculate the cumulative runoff, R(t), at each time interval ΔD by using the rainfall data from Step 5.

Step 7. Using Equation 3.4.8 or by summing up the coordinates for a triangular hydrograph, calculate the storm hydrographs.

Step 8. Using the peak flow from the undeveloped site as the peak outflow, calculate the outflow hydrograph as determined by the type of outflow structure.

Step 9. Calculate the required storage using Equation 6.2.5, using a planimeter, or digitizer.

The application of this method is illustrated in Example 6.2.2.

Example 6.2.2 (SCS Hydrograph Method)

For the 150 acre watershed in Indianapolis, Indiana described below, determine the required storage volume using the hydrograph method described in Section 6.2.2. The inflow is to be based on a 100 year, 1-hour storm and the outflow is to be restricted to a 10 year, 1-hour storm. The soil type is C. Use a triangular unit hydrograph and AMC II.

Undeveloped	Developed
75 acres Meadow (good condition)	50 acres Residential (1/2 acre lots)
45 acres Woods (good cover)	10 acres Gravel road
30 acres Pasture (good condition)	40 acres Parking lots
	50 acres Lawns (fair condition)
Travel Path of 5625 ft. over unpaved land with a 0.6% slope	Travel Path of 3060 ft. over a paved area with a slope of 1% and 9000 ft. of 24 in. storm sewer with a slope of 3% (assume pipe is flowing full and n = 0.013).

Step 1: Determine a composite curve number for both states.

UNDEVELOPED			
	CN	AREA	CN_n x A_n
Meadow (good condition)	71	75	5325
Woods (good cover)	70	45	3150
Pasture (good condition)	74	30	2220
Total		150	10,695

DEVELOPED			
	CN	AREA	CN_n x A_n
Residential (1/2 acre)	80	50	4000
Gravel Roads	89	10	890
Parking Lots	98	40	3920
Lawns (fair condition)	79	50	3950
Total		150	12,760

$$CN_U = \frac{\sum [CN_n \times A_n]}{A_T} = \frac{10{,}695}{150} = 71.3$$

$$CN_D = \frac{\sum [CN_n \times A_n]}{A_T} = \frac{12{,}760}{150} = 85.1$$

Step 2: Determine the time of concentration for both states.

Undeveloped: From figure 3.4.5 with a slope of 0.006 and unpaved area as the cover the velocity is found to be 1.25 ft/sec. The time of concentration is computed by:

$$t_c = \frac{L}{3600V} = \frac{5625}{3600(1.25)} = 1.25 \ \textit{hrs.}$$

Developed The time of concentration is the sum of the overland travel time and the channel travel time.

The overland travel time is found using Figure 3.4.5 with a 1% slope and a paved area surface (sheet flow)
V = 2.0 ft/sec.

$$t_t = \frac{L}{3600V} = \frac{3060}{3600(2.0)} = 0.43 \ hrs.$$

The channel travel time is computed by calculating the velocity from Manning's formula (Equation 4.2.1) for full pipe flow. The pipe is assumed to have a diameter of 2 feet.

$$V = \frac{Q}{A} = \frac{1.49}{n} R^{2/3} S^{1/2} = \frac{1.49}{0.013}\left[\frac{2}{4}\right]^{2/3} [0.03]^{1/2} = 12.5 \ ft/sec.$$

$$t_t = \frac{L}{3600V} = \frac{9000}{(3600)(12.5)} = 0.20 \ hrs.$$

$$t_c = 0.20hrs + 0.43hrs = 0.63 \ hrs.$$

Step 3: Calculate □D, t_p and q_p (A= 150 acres = 0.234 mi^2)

Undeveloped: $\Delta D = (0.133)t_c = (0.133)(1.25) = 0.17$ hrs (10 minutes)

$$t_p = \frac{\Delta D}{2} + 0.6t_c = \frac{0.17}{2} + 0.6(1.25) = 0.84 \ hrs.$$

$$q_p = \frac{484AQ}{t_p} = \frac{484(0.234)(1)}{0.84} = 134.83 \ cfs$$

Developed: $\Delta D = (0.133)(0.63) = 0.08$ hrs (5 minutes)

$$t_p = \frac{0.08}{2} + 0.6(0.63) = 0.42 \ hrs.$$

$$q_p = \frac{(484)(0.234)(1)}{0.42} = 269.66 \ cfs$$

Step 4: The coordinates of the triangular unit hydrographs are found by plotting t_p, q_p and t_b as shown in Figure 6.2.5.
The base time, $t_b = 2.67\ t_p$.

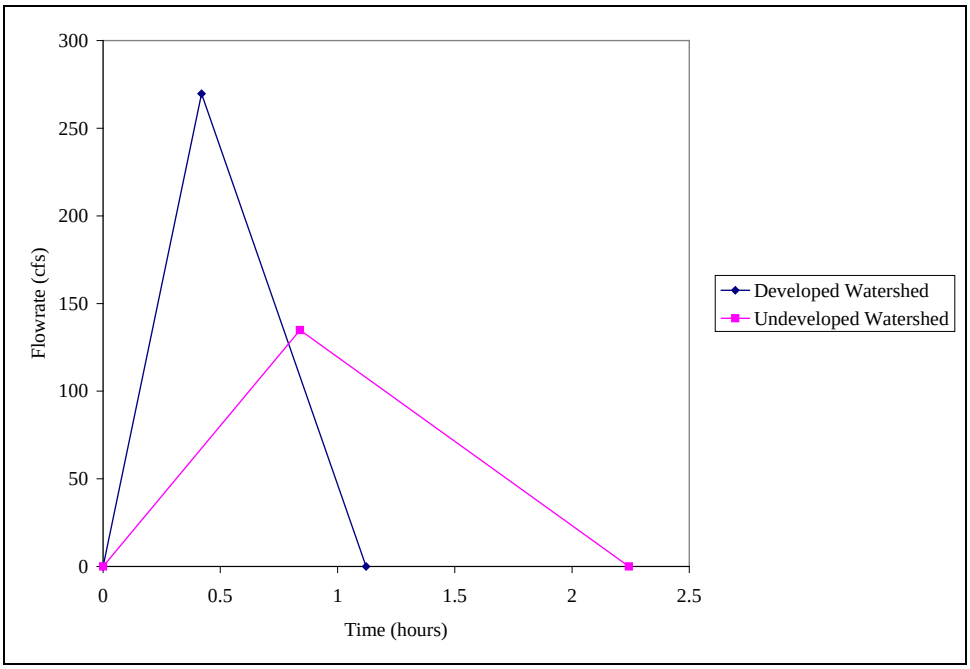


Figure 6.2.5
Triangular Unit Hydrographs for Example 6.2.2

Step 5: Use a Huff median (50% probability) first quartile distribution for Indianapolis (Table 2.1.5) and calculate the cumulative rainfall at 10-minute intervals for the undeveloped site and 5-minute intervals for the developed site.

The undeveloped 10-year, 1-hour depth and the developed 100-year, 1-hour depth for Indianapolis are determined from Equation 2.2.13

$$i_1^{10} = \frac{2.1048(10)^{0.1733}}{(1+0.470)^{1.1289}} = 2.03 \ \textit{inches}$$

$$i_1^{100} = \frac{1.5899(100)^{0.2271}}{(1+0.725)^{0.8797}} = 2.80 \ \textit{inches}$$

Cum. % of Storm Time	Time (min.)	Percent of Total Storm Rainfall			Increment of Rainfall (in.)	
		Cumulative	Incremental		Undeveloped	Developed
			(10 min)	(5 min)	Total=2.03"	Total=2.80"
0.00%	0	0.00	-	-	-	-
8.33%	5	17.00		17.00		0.48
16.67%	10	32.00	32.00	15.00	0.65	0.42
25.00%	15	48.00		16.00		0.45
33.33%	20	57.00	25.00	9.00	0.51	0.25
41.67%	25	63.00		6.00		0.17
50.00%	30	69.00	12.00	6.00	0.24	0.17
58.33%	35	75.00		6.00		0.17
66.67%	40	81.00	12.00	6.00	0.24	0.17
75.00%	45	87.00		6.00		0.17
83.33%	50	91.00	10.00	4.00	0.20	0.11
91.67%	55	96.00		5.00		0.14
100.00%	60	100.00	9.00	4.00	0.18	0.11

Step 6: Use the curve numbers for both the developed and undeveloped conditions and calculate the cumulative runoff at each interval. First find the ultimate abstraction for both conditions.

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

$$S = \frac{1000}{71.3} - 10 = 4.03 \ \textit{inches} \qquad (\textit{undeveloped})$$

$$S = \frac{1000}{85.1} - 10 = 1.75 \ \textit{inches} \qquad (\textit{developed})$$

Undeveloped

Time (min.)	Incremental Rainfall (inches)	Cumulative Rainfall P(t) (inches)	Cumulative Runoff R(t) (inches)	Incremental Runoff R(t) (inches)
0	---	0.00	0.00	---
10	0.65	0.65	0.01	0.01
20	0.51	1.16	0.03	0.02
30	0.24	1.40	0.08	0.05
40	0.24	1.65	0.14	0.07
50	0.20	1.85	0.21	0.07
60	0.18	2.03	0.28	0.07

$$R(t) = \frac{(P(t) - 0.2S)^2}{P(t) + 0.8S} = \frac{(P(t) - 0.81)^2}{P(t) + 3.22}$$

Developed

Time (min.)	Incremental Rainfall (inches)	Cumulative Rainfall P(t) (inches)	Cumulative Runoff R(t) (inches)	Incremental Runoff (inches)
0	---	0.00	0.00	---
5	0.48	0.48	0.01	0.01
10	0.42	0.90	0.13	0.12
15	0.45	1.34	0.36	0.23
20	0.25	1.60	0.52	0.16
25	0.17	1.76	0.63	0.11
30	0.17	1.93	0.75	0.12
35	0.17	2.10	0.88	0.12
40	0.17	2.27	1.00	0.13
45	0.17	2.44	1.13	0.13
50	0.11	2.55	1.22	0.09
55	0.14	2.69	1.34	0.11
60	0.11	2.80	1.43	0.09

$$R(t) = \frac{(P(t) - 0.35)^2}{P(t) + 1.40}$$

Step 7: Multiply the coordinates of the triangular unit hydrographs by each incremental runoff and compute the storm hydrograph by summing up the individual hydrographs. The storm hydrographs and construction are shown in Figures 6.2.6 and 6.2.7.

Step 8: The peak flow for the undeveloped site is found to be about 33 cfs. This is the maximum flowrate for the outflow hydrograph.

Step 9: For simplicity, assume that the outflow devices will regulate the flow such that a constant flow of 33 cfs will be released. Chapter 8 presents examples of the computer models available to simulate variable outflows. Calculate the volume between the hydrographs as shown in Figure 6.2.10. The calculations are shown in Figure 6.2.11.

Step 10: From Figure 6.2.8 and 6.2.9, the required storage is found to be 11.9 acre-feet. Figure 6.2.11 also displays the required storage graphically.

Step 11: For a detention pond with a 400' x 400' bottom and 3:1 side slopes, the relationship between volume and pond depth can be determined as shown below:

$$V = d\left[\frac{2(400 + 6d) + (400 \times 400)}{2}\right]$$

Solving by trial and error to find a depth which produces a volume of 519,321 ft^3 gives d = 6.5 ft.

Time	Unit	Excess Precipitation (in)								Storm
(min)	Hydrograph									Hydrograph
	(cfs)	0.00	0.01	0.02	0.05	0.07	0.07	0.07		(cfs)
0.00	0.00	0.00								0.00
10.00	26.75	0.00	0.00							0.00
20.00	53.50	0.00	0.27	0.00						0.27
30.00	80.26	0.00	0.54	0.54	0.00					1.07
40.00	107.01	0.00	0.80	1.07	1.34	0.00				3.21
50.00	133.76	0.00	1.07	1.61	2.68	1.87	0.00			7.22
60.00	119.45	0.00	1.34	2.14	4.01	3.75	1.87	0.00		13.11
70.00	103.43	0.00	1.19	2.68	5.35	5.62	3.75	1.87		20.46
80.00	87.41	0.00	1.03	2.39	6.69	7.49	5.62	3.75		26.97
90.00	71.39	0.00	0.87	2.07	5.97	9.36	7.49	5.62		31.39
100.00	55.38	0.00	0.71	1.75	5.17	8.36	9.36	7.49		32.85 ← Peak Flow
110.00	39.36	0.00	0.55	1.43	4.37	7.24	8.36	9.36		31.32
120.00	23.34	0.00	0.39	1.11	3.57	6.12	7.24	8.36		26.79
130.00	7.32	0.00	0.23	0.79	2.77	5.00	6.12	7.24		22.15
140.00	0.00	0.00	0.07	0.47	1.97	3.88	5.00	6.12		17.50
150.00			0.00	0.15	1.17	2.75	3.88	5.00		12.94
160.00				0.00	0.37	1.63	2.75	3.88		8.63
170.00					0.00	0.51	1.63	2.75		4.90
180.00						0.00	0.51	1.63		2.15
190.00							0.00	0.51		0.51
200.00								0.00		0.00
210.00										0.00

Figure 6.2.6
Computation of Storm Hydrograph for Undeveloped Site for Example 6.2.2

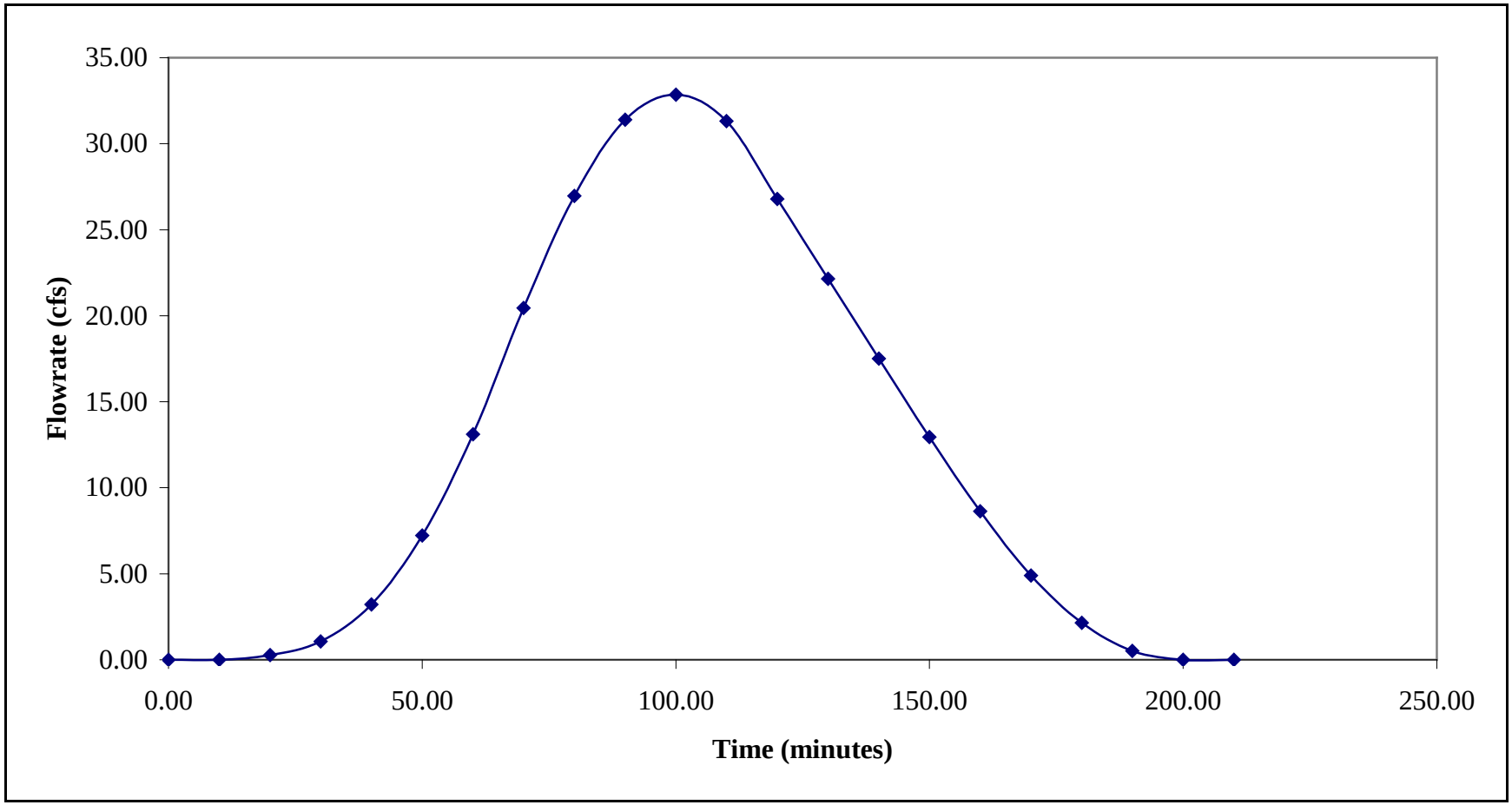


Figure 6.2.7
Storm Hydrograph for Undeveloped Site for Example 6.2.2

Time (min)	Unit Hydrograph (cfs)	Excess Precipitation (in)													Storm Hydrograph (cfs)
		0.00	0.01	0.12	0.23	0.16	0.11	0.12	0.12	0.13	0.09	0.11	0.09		
0.00	0.00	0.00													0.00
5.00	53.50	0.00	0.00												0.00
10.00	107.01	0.00	0.54	0.00											0.54
15.00	160.51	0.00	1.07	6.42	0.00										7.49
20.00	214.02	0.00	1.61	12.84	12.31	0.00									26.75
25.00	267.52	0.00	2.14	19.26	24.61	8.56	0.00								54.57
30.00	238.90	0.00	2.68	25.68	36.92	17.12	5.89	0.00							88.28
35.00	206.86	0.00	2.39	32.10	49.22	25.68	11.77	6.42	0.00						127.59
40.00	174.83	0.00	2.07	28.67	61.53	34.24	17.66	12.84	6.42	0.00					163.43
45.00	142.79	0.00	1.75	24.82	54.95	42.80	23.54	19.26	12.84	6.96	0.00				186.92
50.00	110.75	0.00	1.43	20.98	47.58	38.22	29.43	25.68	19.26	13.91	4.82	0.00			201.31
55.00	78.71	0.00	1.11	17.13	40.21	33.10	26.28	32.10	25.68	20.87	9.63	5.89	0.00		212.00
60.00	46.67	0.00	0.79	13.29	32.84	27.97	22.76	28.67	32.10	27.82	14.45	11.77	4.82		217.27
65.00	14.64	0.00	0.47	9.45	25.47	22.85	19.23	24.82	28.67	34.78	19.26	17.66	9.63		212.28
70.00	0.00	0.00	0.15	5.60	18.10	17.72	15.71	20.98	24.82	31.06	24.08	23.54	14.45		196.20
75.00			0.00	1.76	10.73	12.59	12.18	17.13	20.98	26.89	21.50	29.43	19.26		172.46
80.00				0.00	3.37	7.47	8.66	13.29	17.13	22.73	18.62	26.28	24.08		141.62
85.00					0.00	2.34	5.13	9.45	13.29	18.56	15.73	22.76	21.50		108.76
90.00						0.00	1.61	5.60	9.45	14.40	12.85	19.23	18.62		81.75
95.00							0.00	1.76	5.60	10.23	9.97	15.71	15.73		59.00
100.00								0.00	1.76	6.07	7.08	12.18	12.85		39.94
105.00									0.00	1.90	4.20	8.66	9.97		24.73
110.00										0.00	1.32	5.13	7.08		13.54
115.00											0.00	1.61	4.20		5.81
120.00												0.00	1.32		1.32
125.00													0.00		0.00
130.00															0.00

Figure 6.2.8
Computations for Storm Hydrograph for Developed Site for Example 6.2.2

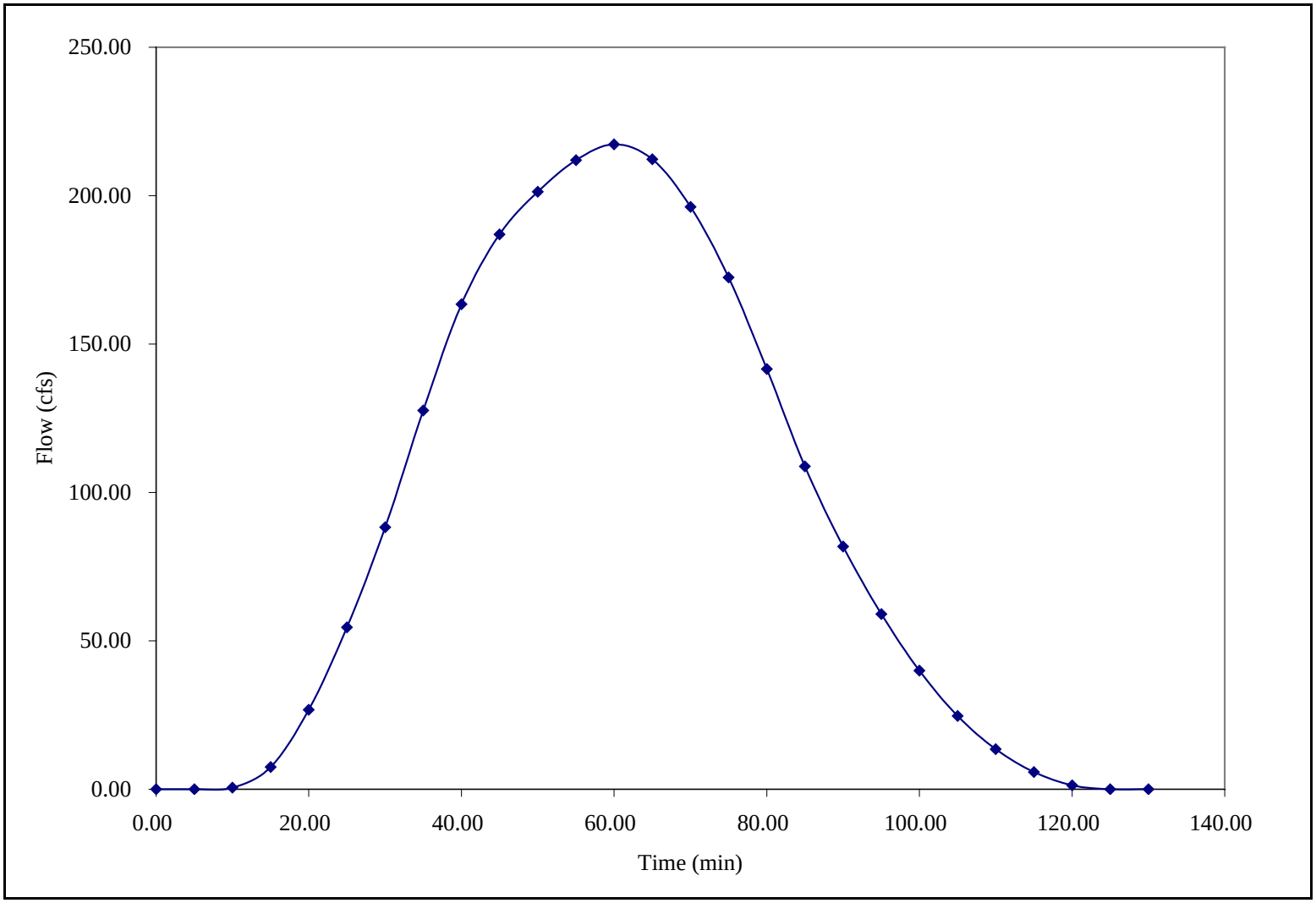


Figure 6.2.9
Storm Hydrograph for Developed Site for Example 6.2.2

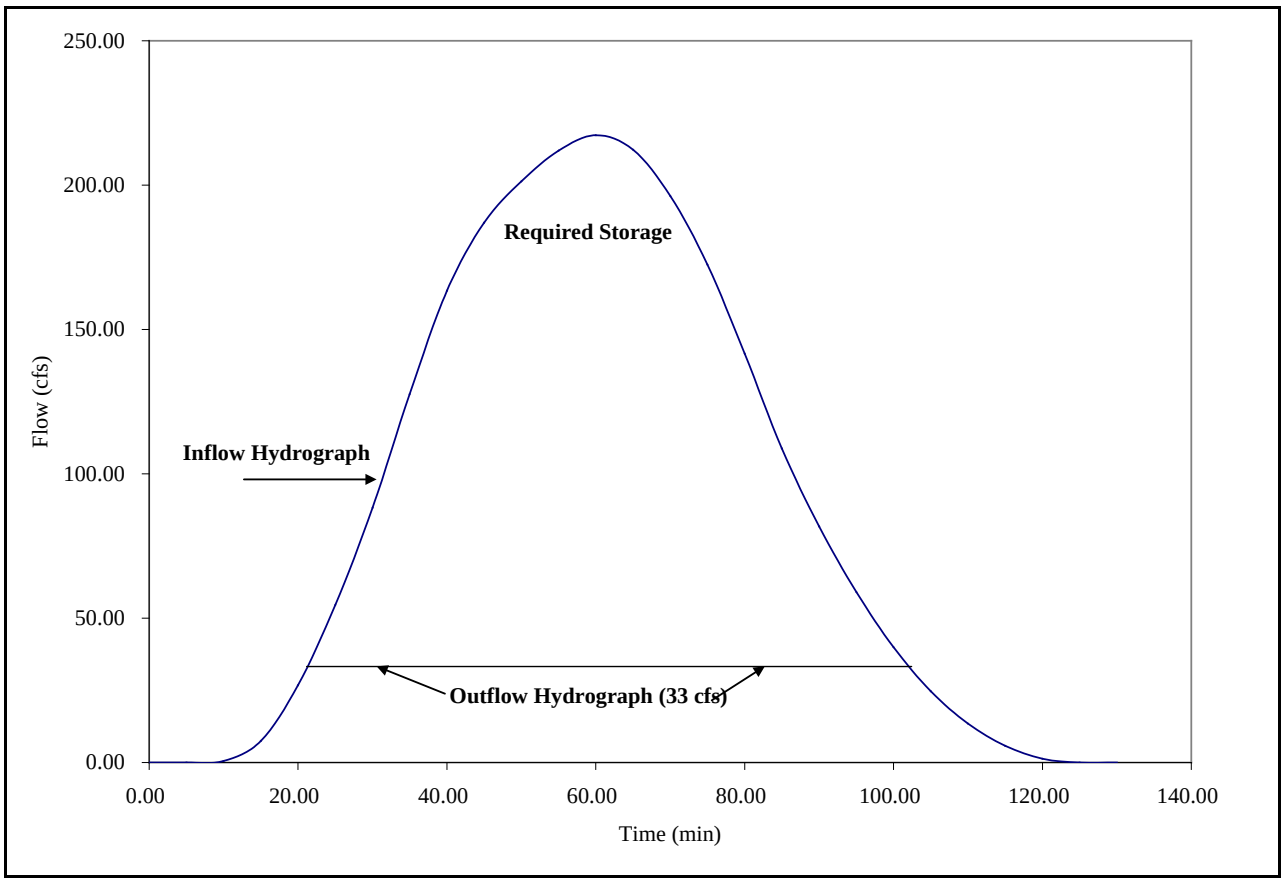


Figure 6.2.10
Storage Detention Graphic for Example 6.2.2

t (minutes)	Q_I (cfs)	Q_O (cfs)	Q_I-Q_O (cfs)	s =(Q_I-Q_O)* Δt (ft^3)	
21	33	33	0	0.00	
25	54.57	33	21.57	5176.80	
30	88.28	33	55.28	16584.46	
35	127.59	33	94.59	28376.50	
40	163.43	33	130.43	39128.06	
45	186.92	33	153.92	46176.79	
50	201.31	33	168.31	50492.23	
55	212.00	33	179.00	53699.08	
60	217.27	33	184.27	55281.29	
65	212.28	33	179.28	53783.97	
70	196.20	33	163.20	48960.77	
75	172.46	33	139.46	41839.08	
80	141.62	33	108.62	32585.44	
85	108.76	33	75.76	22729.32	
90	81.75	33	48.75	14625.99	
95	59.00	33	26.00	7799.45	
100	39.94	33	6.94	2082.38	
102	33	33	0	0.00	
			ΣS =	519321.614	ft^3
				11.9219838	ac-ft

Figure 6.2.11
Storage Determination for Example 6.2.2

6.3 - DESIGN OF STORAGE FACILITIES

General guidelines for the design of storage facilities are outlined in this section. Commonly used design criteria are presented for each type of facility. In the last part of this section the types of outlet structures used for these facilities and applicable equations are discussed. Application of the material in this section is illustrated by Example 6.3.1.

6.3.1 - Retention Ponds

Retention ponds should always be constructed for both aesthetic and practical use. Investigation of potential water quality problems associated with low flow in dry weather, direct pollution from surface runoff or high nutrient levels is necessary. High nutrient levels may cause growth of algae and subsequent eutrophication. If any of these problems arise with a retention pond, detention pond should be considered instead.

The following is a list of some design considerations for retention ponds:

- When possible, retention ponds should be stocked with fish. To provide a habitat suitable for aquatic life, it is recommended that the pond has a surface area of at least one-half acre and a minimum depth of at least 10 feet over at least 25% of the total area. The average depth of the remaining area should be at least 5 feet. This is needed to inhibit insect breeding and weed growth although periodic maintenance may still be necessary to completely control this growth. A minimum side slope of 3:1 is also recommended. A five foot safety ledge should be constructed at a depth of no more than 18 inches.

- It is recommended that the design include a means of keeping a design water level during periods of prolonged dry weather as a way of enhancing water quality. One method which has been suggested to augment the pond volume and solve these problems is the placement of a fire hydrant near the pond site.

- An emergency outflow spillway or weir should be installed to ensure that flows which may exceed the design capacity of the pond do not damage surrounding property.

- Fencing may be required to ensure the safety of children.

- All inlet openings larger than 6 inches should have bars or screens on them to protect children and large animals and to collect debris.

- Retention facilities can be developed to provide wetland communities rather than being deep holes. The grading can be accomplished to establish sedge meadow and erect emergent wetlands.

6.3.2 - Detention Ponds

Detention ponds should be designed for multi-purposes. In some parts of the country they are used as athletic fields or parks during dry weather. Regardless of its dry weather use, there are certain guidelines which must be followed to ensure that the pond is properly maintained and does not become a detriment or hazard.

- The pond should have no side slopes which are greater than 3 feet horizontal to 1 foot vertical. A five foot safety ledge should be constructed at a depth of no more than 18 inches.

- An underdrain system should be constructed to minimize the wetness of the pond bottom. This is to ensure that the bottom does not harbor insects or prohibit recreational dry weather use. This underdrain system may discharge directly into the receiving stream or into a wet well where it is then pumped out to the receiving stream.

- An alternative to the underdrain is the sloping of the pond bottom with at least a 2% grade from inlet to outlet. Another means of eliminating the wetness is to slope the pond to a lined channel which flows through the pond. This channel may also be used to convey dry weather flow.

- The pond bottom and side slopes should be finished with at least four inches of topsoil and seeded or sodded. This seed or sod should be capable of withstanding periodic flooding conditions.

- Outlets which discharge overland should be provided with rip-rap protection to prohibit erosion.

- An overflow spillway or weir should be placed at the high water elevation and a minimum of 6 inches of freeboard should be provided.

- Screens or bars should be placed over all inlets and outlets greater than 6 inches to protect children and large animals, and to collect debris.

- To maximize water quality benefits the pond inlet and outlet should be located on the opposite ends to provide the maximum settling time within the basin and encourage settling of suspended solids.

6.3.3 - Parking Lot Storage

Parking lots may be utilized in the temporary storage of stormwater in two forms. The first and most widely used method is the storage of runoff in depressed area. The accumulated runoff is allowed to accumulate and then released through a drain using restrictors. The

restrictors may be small diameter pipes, orifices, or small grates. This type of storage requires special design criteria as presented below.

- The storage areas of the parking lots should be restricted to remote areas or those which cause the least inconvenience to users.
- The maximum depth of water should not exceed 6 inches.
- The parking lot should be drained in at least 30 minutes after rainfall termination.
- Frequent maintenance is necessary to ensure that the drain openings do not plug up.
- Orifices less than four inches should be discouraged due to clogging.

Another way of using parking lots as a detention storage facility is by conveying the runoff from paved surfaces to grassed areas or seepage pits located adjacent to the lot. Criteria used in determining infiltration rates are discussed in Section 6.3.5.

6.3.4 - Rooftop Storage

The temporary storage of rainfall on rooftops can be an economical approach to temporarily store stormwater. Most current building codes in the northern states require that rooftops withstand live loadings of 30 to 40 pounds per square foot. This is equivalent to about 6 inches of water which is usually much greater than that required for normal detention on flat roofs.

There are obviously several problems associated with rooftop storage. Leakage, overflows, and structural damage are just some of the potential problems. To minimize these hazards and design the best facility, the following guidelines are recommended.

- Provide extra membrane liners to create a watertight seal on the roof.
- Place overflow drains at the design depth to reduce damage due to clogging of drains.
- Design the roof to structurally withstand a live load of at least 35 pounds per square foot.
- Design the roof so that the maximum depth with a flat roof is 3 inches and the maximum depth with a sloped roof is 6 inches.
- Space each drain to handle 10,000 square feet of roof area.

- Determine the allowable outflow rate from the rooftop and then use Table 6.3.1 to size the leader which is the pipe which takes the water from the roof vertically to storm drain, or the horizontal piping to release the water. Check that the depth does not exceed the 3 inches for a flat roof or 6 inches for the sloped roof.

Table 6.3.1
System Sizing Data (Blenderman, 1979)

Pipe Diameter (inches)	*Roof Drains and Vertical Leaders (gpm)*	*Horizontal Storm-Drainage Piping Slope (in/ft)*		
		1/8	*1/4*	*1/2*
	Flow Capacity (gpm)			
2	30	-	-	-
2.5	54	-	-	-
3	92	34	48	69
4	192	78	110	157
5	360	139	197	278
6	563	223	315	446
8	1208	479	679	958
10	-	863	1217	1725
12	-	1388	1958	2775
15	-	2479	3500	4958

6.3.5 - Infiltration of Stormwater

The temporary storage and subsequent infiltration of stormwater into the soil may be accomplished through the use of a basin, trench, or porous asphalt. These are normally used when the receiving stream cannot accept any additional runoff, where it is necessary to recharge the groundwater, or where a positive gravity outlet is not feasible. The rate at which the water can percolate through the soil depends upon the soil makeup and the ground water table.

The soil permeability is a measure of the ability of the soil to allow infiltration. Typical values of the coefficient of permeability, denoted by K, are given in Table 6.3.2. This coefficient can be used with Darcy's Law as presented in Equation 6.3.1.

Table 6.3.2
Coefficients of Permeability (Hydraulic Conductivity)

Typical Soil	Permeability K (ft/sec)	Relative Permeability
Coarse, Gravel	over 5.0×10^{-1}	Very Permeable
Sand, Fine Sand	5.0×10^{-1} - 5.0×10^{-3}	Medium Permeability
Silty Sand, Dirty Sand	5.0×10^{-3} - 5.0×10^{-5}	Low Permeability
Silt	5.0×10^{-5} - 5.0×10^{-7}	Very Low Permeability
Clay	less than 5.0×10^{-7}	Practically Impervious

$$Q' = A'Kh_L \qquad (6.3.1)$$

where Q' is the flow rate (cfs), A' is the cross-sectional area of soil through which the water flows (ft^2), K is the coefficient of permeability (Table 6.3.2)(ft/sec), and h_L is the headloss or the gradient over a flow distance L (it is recommended that a factor of safety of two be used, i.e. divide Q' in half)(ft/ft).

Equation 6.3.1, which is applicable for a fully saturated soil, is used to determine the required size of the ditch or basin. Figure 6.3.1 shows a typical recharge trench. The void space in the rock fill must have sufficient capacity to detain the difference between the incoming runoff, the water which is infiltrating during the entire storm, and the amount which may be released.

The procedure which is used in determining the storage requirement is outlined below:

Step 1: Calculate the volume of accumulated runoff for a given time increment, t_d, for the storm duration using the rational method, SCS curve number method, or hydrograph method. ($Q \times t_d$)(cubic feet).

Step 2: Calculate the allowable release volume which does not have to be infiltrated, if any. ($Q \times t_d$).

Step 3: Calculate the volume of the water flowing through the soil at time t_d. ($Q' \times t_d$) Q' is found from Equation 6.3.1.

Step 4: The difference between the inflow volume and the allowable release rate and percolated volume is the required storage.

Step 5: The largest required storage when all durations are considered is the design requirement.

Step 6: Design the basin or trench to have a void volume equal to the maximum required storage. Information concerning the required size of the rock fill and corresponding void ratios may be found in Design of Small Dams, USBR, 1987.

Porous asphalt may also be used in detaining the water and allowing it to percolate through the soil. This requires a special type of pavement: open graded permeable material (OGPM) with a gravel subbase. There are some installations using this material but most are in experimental stages. Questions about the rigidity and effects on water quality still remain.

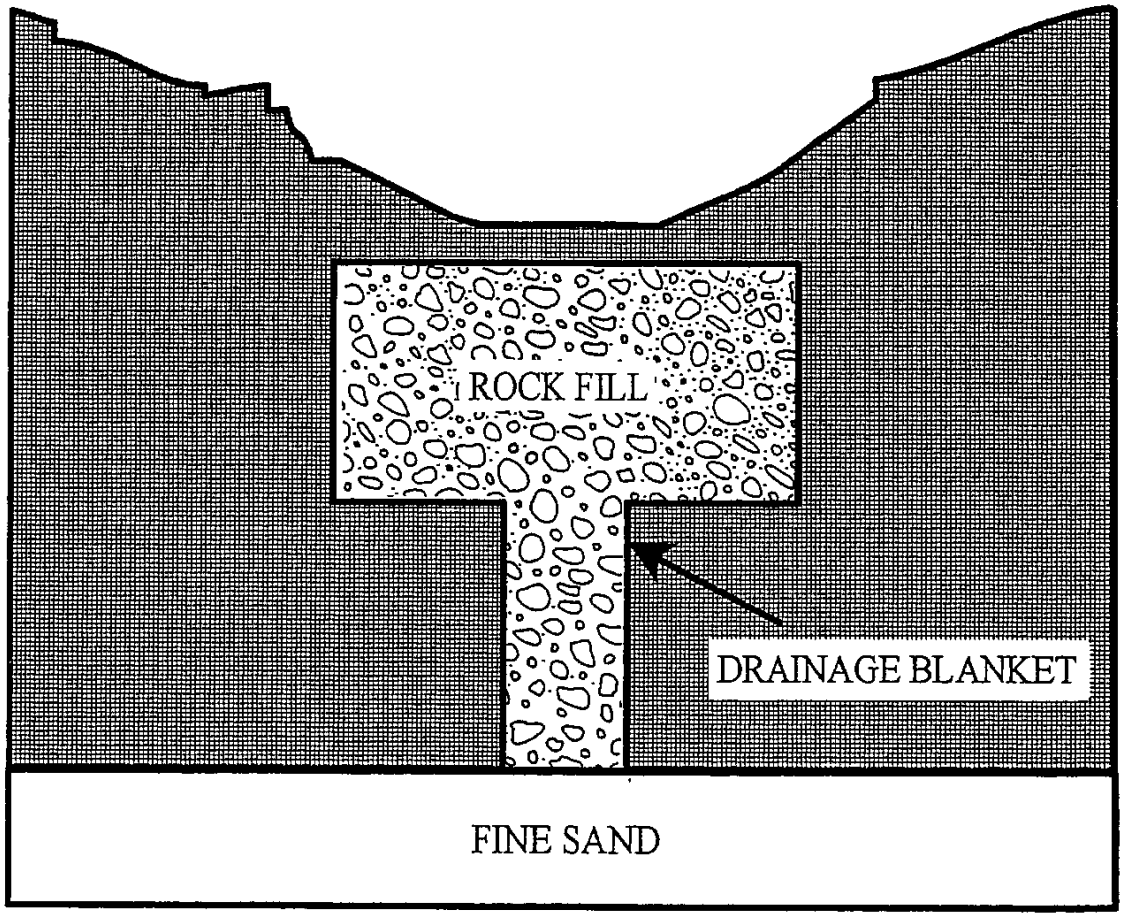


Figure 6.3.1
Design of On-Site Discharge Ditch (Karaca, 1980)
(The pervious drainage blanket shown above can be constructed of "Bidim" (by Monsanto) or equivalent).

6.3.6 - Outflow Control Devices

The design of any detention facility requires that the outflow be regulated to a maximum flowrate. This is usually accomplished by using standard calibrated devices. The three most commonly used devices are the orifice, weir, and pipe. The following is a description of each of the above and the governing equations.

ORIFICE: An orifice is a circular or rectangular opening of a prescribed shape and perimeter through which water flows. The flowrate depends upon the height of water, h_o, above the opening, and the type of orifice. The equation which is used to calculate the flow rate into an orifice is given below.

$$Q = C_d A_o \sqrt{2gh_o} \tag{6.3.2}$$

where C_d is the discharge coefficient, A_o is the area of the orifice (ft^2), g is the acceleration due to gravity (32.2 ft/sec^2), and h_o is the height of water above the center line of the orifice (ft).

Figure 6.3.2 is a typical circular orifice with the above variable defined and Figure 6.3.3 gives discharge coefficients for various types of orifices.

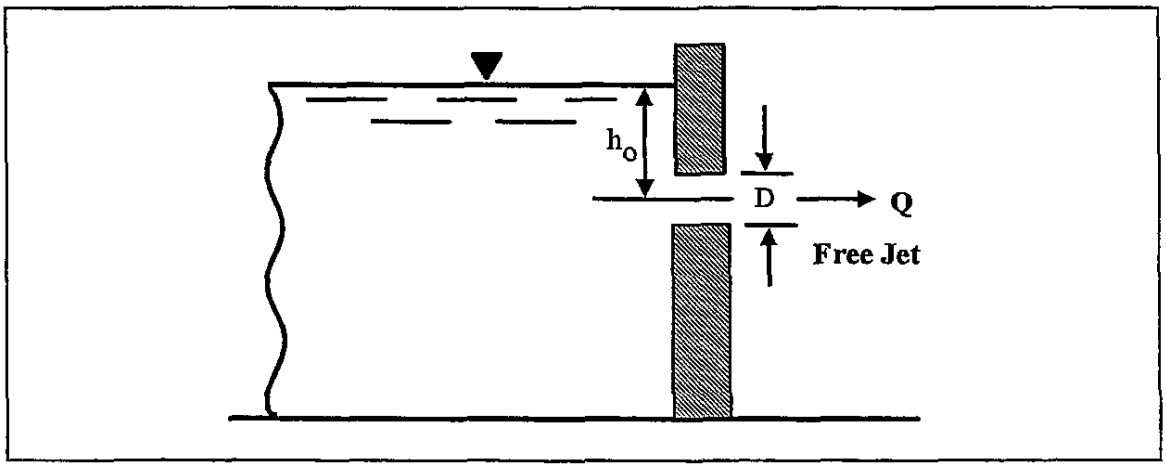
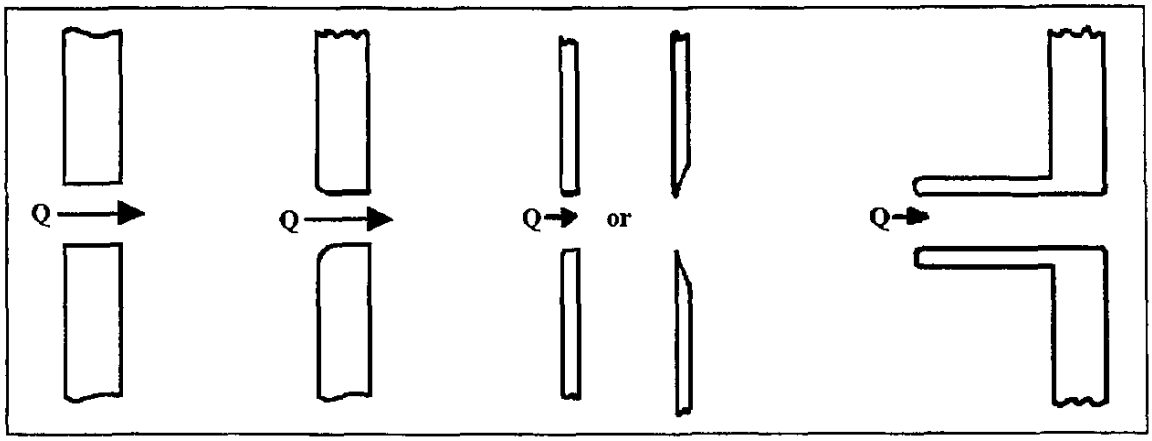


Figure 6.3.2
Typical Orifice



Square-edged	Round-edged	Sharp-edged	Projecting Sharp-edge
C_d= 0.79-0.82	C_d= 0.92-0.98	C_d= 0.58-0.64	C_d= 0.50

Figure 6.3.3
Discharge Coefficients C_d for Various Types of Orifices (Kuhl, 1977)

WEIR: A weir is a depression or cutout in a wall or channel through which water flows. Weirs are often used for controlling overflows. Two types of sharp-crested weirs, rectangular and triangular are discussed below. A typical rectangular weir is shown in Figure 6.3.4. The equation for flow though a rectangular weir is

$$Q = \frac{2}{3} C_d \sqrt{2g} L h_w^{3/2} \qquad (6.3.3)$$

where L is the length of the weir in feet, C_d is the discharge coefficient found by experiment or furnished by manufacturer, h_w is the height of the water surface about the weir opening in feet, and g is the acceleration due to gravity (32.2 ft/sec^2).

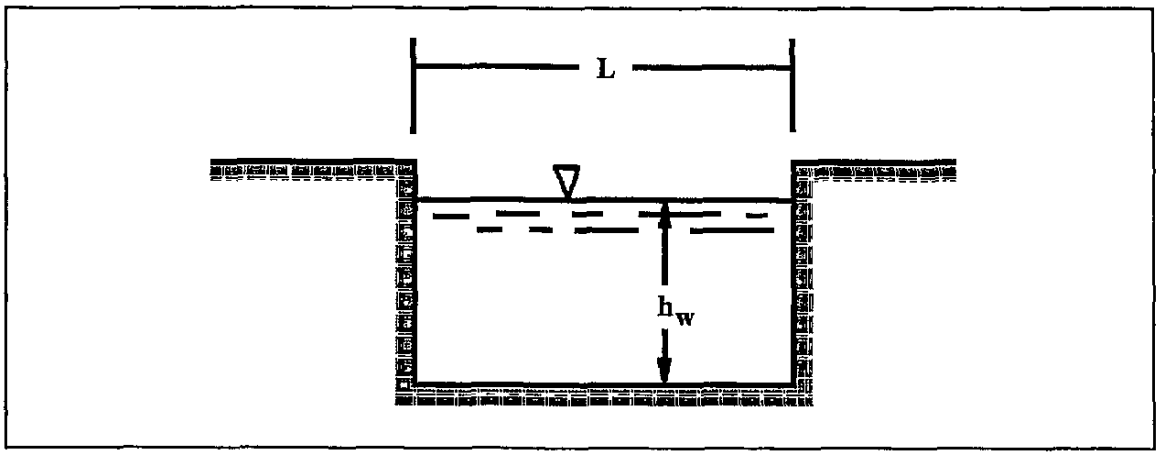


Figure 6.3.4
Typical Rectangular Weir

For a triangular weir shown in Figure 6.3.5, Equation 6.3.4 is used to calculate the flowrate,

$$Q = \frac{8}{15} C_d \sqrt{2g} \tan\theta h_x^{5/2} \qquad (6.3.4)$$

where θ is ½ the total weir angle and C_d, h_w, and g are the same as in Equation 6.3.3.

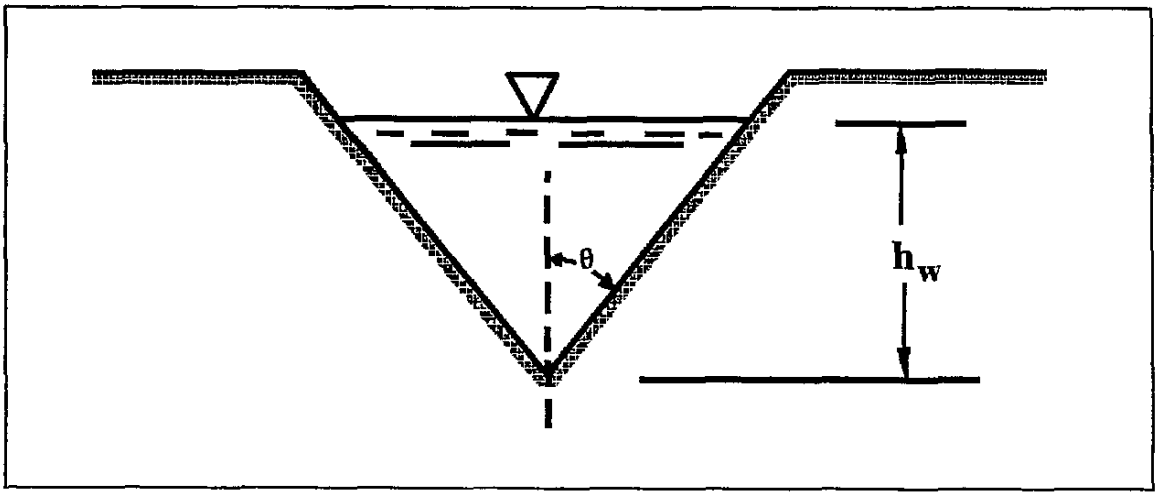


Figure 6.3.5
Typical Triangular Weir

PIPE FLOW: For a pipe flowing full, but not under pressure, Manning's Equation (Equation 4.2.1) may be used to calculate the flowrate. As soon as a depth of water develops above the pipe, the flowrate must be calculated using a modified form of the Manning equation and must include entrance and exit losses. Equation 6.3.5 is used to calculate the flowrate under these conditions,

$$Q = A_p \left[\frac{h_p}{\frac{K_e + K_o}{2g} + \frac{2.87 n^2 L}{D^{4/3}}} \right]^{1/2} \qquad (6.3.5)$$

where Q is the flowrate in cfs, A_p is the area of the pipe (ft^2), K_e is the entrance loss coefficient (given in Table 6.3.3), K_o is the outlet loss coefficient (usually taken as 1.0), D is the pipe diameter (ft), *n* is Manning's roughness coefficient (Table 4.2.1), L is the length of the pipe (ft) (Equation assumes a free jet @ exit), and h_p is the height of the water surface above the center of the pipe opening (ft).

All of the equations presented were to calculate the flowrate Q. However, many times the flowrate is known and the orifice opening, weir size or pipe diameter is the parameter actually required. In these cases the equations are rearranged solving for the unknown variable. Certain problems may arise when the computed result is not a commercially available pipe. When this occurs, the type of opening, discharge coefficient, or pipe roughness will have to be varied to obtain a commercially available size.

Table 6.3.3
Value of K_e (entrance losses)
(Bureau of Reclamation, 1987)

Entrance Condition	*K_e range*	*Average*
Square-edged inlets installed flush with vertical headwalls	0.43 to 0.70	0.50
Rounded Inlets installed flush with vertical headwalls, $r/D \leq 0.15$	0.08 to 0.27	0.10
Grooved or socket-ended concrete pipe installed flush with vertical headwall	0.10 to 0.33	0.15
Projecting concrete pipe with grooved or socket ends	-	0.20
Projecting steel or corrugated metal pipes	0.5 to 0.9	0.85

Example 6.3.1 (After MWRDGC)

For the developed watershed shown in Figure 6.3.6, calculate the required storage on the rooftop, parking lot and retention/ detention pond using the rational method and MWRDGC criteria and size their respective outlet structures. The time of concentration for the undeveloped site is 27 minutes. Use Evansville, Indiana rainfall data. Use an orifice for the detention pond and a concrete pipe with a length of 100 ft. with a square edged entrance for the parking lot. Determine the size of the pipe for the roof drain using Table 6.3.1.

Solution

For a time of concentration of 27 minutes, the rainfall intensity for a 3-year return period in Evansville, Indiana:

$$i = \frac{c\,T_r^{\alpha}}{(t+d)^{\beta}} = \frac{1.9533(3)^{0.1747}}{(\frac{27}{60}+0.522)^{1.6408}} = 2.479\ \textit{inches/hour}$$

Therefore the allowable outflow rate is:

$$O(t) = C_U i_U A_U = (0.15)(2.479)(650\times 650)\frac{1}{43{,}560} = 3.61\ \textit{cfs}$$

Note that 0.15 is the maximum allowable runoff coefficient.

Calculate the developed runoff coefficient:

Type of Cover	*Area Calculations*	*Area (ft^2)*	*Area (acres)*	*C*	C_nA_n
Pavement	150'x150' + 200'x650'	152,500	3.50	0.95	3.33
Roofs	300' x 200'	60,000	1.38	0.95	1.31
Pond	100' x 50'	5,000	0.11	1.0	0.11
Grass	9.7 acres - (3.50 + 1.38 + 0.11)		4.71	0.25	1.18
Total			9.7		5.93

$$C_D = \frac{\sum C_n A_n}{A_T} = \frac{5.93}{9.7} = 0.61$$

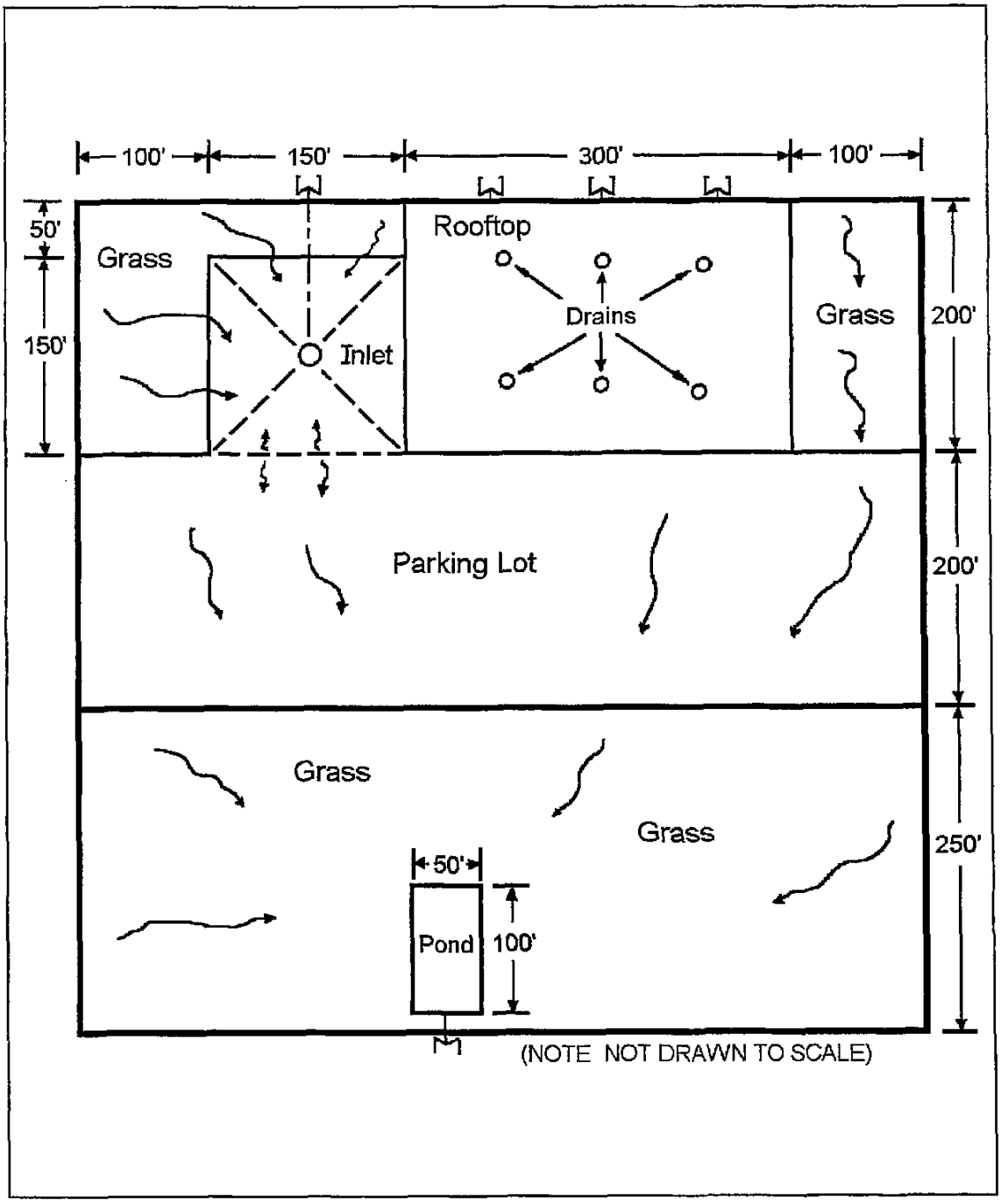


Figure 6.3.6
Hypothetical Watershed for Example 6.3.1 (After MWRDGC)

Project: Example 6.3.1 **Detention Facility Design Return Period:** 100

Designer: TTB **Release Rate Return Period:** 3

Watershed Area:	9.7 acres
Time of Concentration (undeveloped):	27 minutes
Rainfall Intensity (i_U):	2.479 inches/hr
Undeveloped Runoff Coefficent (C_U):	0.15
Undeveloped Runoff Rate ($O=C_U i_U A_U$):	3.61 cfs
Developed Runoff Coefficent (C_D)	0.6113402

Storm Duration t_d (hours)	Rainfall Intensity i_d (inches/hr)	Inflow Rate $I(t_d)$ (cfs)	Outflow Rate O (cfs)	Storage Rate $I(t_d)$-O (cfs)	Required Storage $(I(td)-O)(t_d/12)$ (acre-feet)	
0.17	4.36	25.88	3.61	22.27	0.31	
0.33	3.06	18.14	3.61	14.53	0.40	
0.50	2.28	13.54	3.61	9.93	0.41	← **Peak Storage**
0.67	1.78	10.57	3.61	6.96	0.39	
0.83	1.44	8.52	3.61	4.91	0.34	
1	1.19	7.04	3.61	3.44	0.29	
1.5	0.75	4.42	3.61	0.81	0.10	
2	0.52	3.08	3.61	*	*	
3	0.30	1.78	3.61	*	*	
4	0.20	1.18	3.61	*	*	
5	0.14	0.85	3.61	*	*	
6	0.11	0.65	3.61	*	*	
7	0.09	0.51	3.61	*	*	
8	0.07	0.42	3.61	*	*	
9	0.06	0.35	3.61	*	*	
10	0.05	0.30	3.61	*	*	

* Since I(tD) - O< zero, there is no storage needed

Figure 6.3.7
Detention Storage Calculations for the Entire Basin Using the Rational Method

Using the outflow rate and the developed runoff coefficient, the storage volume for the entire basin is calculated using the rational method and Figure 6.2.2. From the calculations shown by Figure 6.3.7, the maximum required storage is 0.41 acre-feet. This is the sum of the rooftop, parking lot and pond storage. Consequently, each facility must be designed separately so that the total release rate is less than or equal to 3.6 cfs and the total storage is 0.41 acre-feet.

The designer must decide how much runoff may be released from each facility. For this example, the rooftop will release 1.3 cfs, the pond 1.9 cfs and the parking lot, 0.4 cfs. In many cases a trial and error procedure is necessary to ensure that each facility does not exceed its maximum storage capacity.

Rooftop

The total area of rooftop is 1.38 acres and the developed runoff coefficient is 0.95. Using this information, the storage can be computed as shown in Figure 6.3.9. The peak storage is found to be 0.08 acre-feet. This volume will be detained in the "cells" shown below in Figure 6.3.4. Knowing the volume, the depth can be computed as follows.

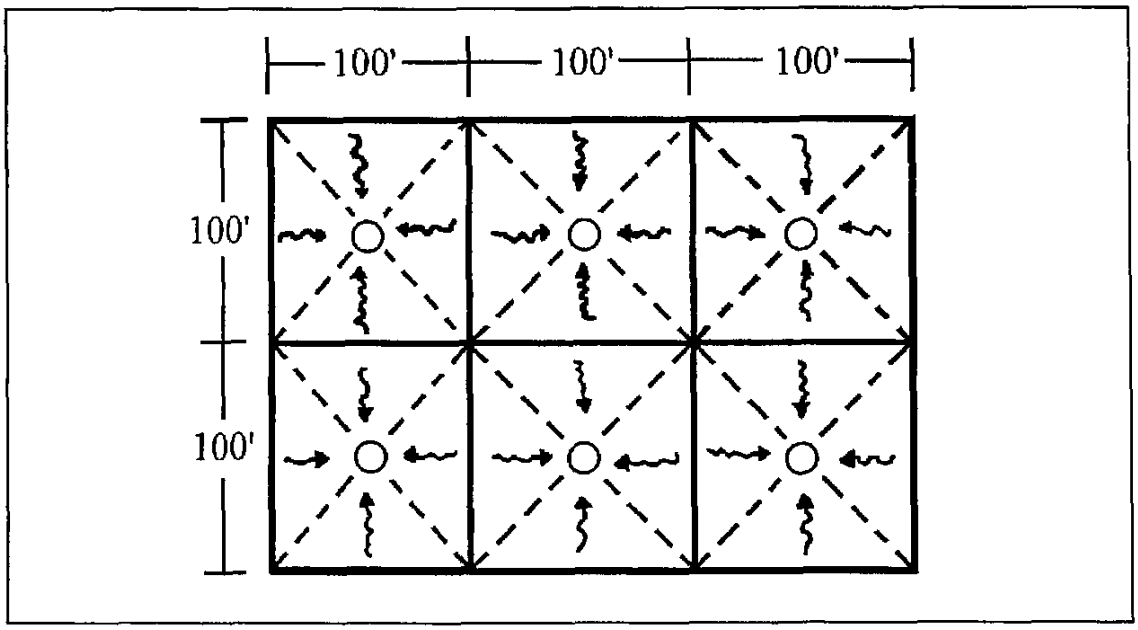


Figure 6.3.8
Roof Drain Configuration

Project: Example 6.3.1 **Detention Facility Design Return Period:** 100

Designer: TTB **Release Rate Return Period:** 3

Watershed Area:	1.38 acres
Time of Concentration (undeveloped):	--- minutes
Rainfall Intensity (i_U):	--- inches/hr
Undeveloped Runoff Coefficent (C_U):	---
Undeveloped Runoff Rate (O):	1.30 cfs
Developed Runoff Coefficent (C_D)	0.95

Storm Duration t_d (hours)	Rainfall Intensity i_d (inches/hr)	Inflow Rate I(t_d) (cfs)	Outflow Rate O (cfs)	Storage Rate I(t_d)-O (cfs)	Required Storage (I(td)-O)(t_d/12) (acre-feet)	
0.17	4.36	5.72	1.30	4.42	0.06	
0.33	3.06	4.01	1.30	2.71	0.08	← **Peak Storage**
0.50	2.28	2.99	1.30	1.69	0.07	
0.67	1.78	2.34	1.30	1.04	0.06	
0.83	1.44	1.88	1.30	0.58	0.04	
1	1.19	1.56	1.30	0.26	0.02	
1.5	0.75	0.98	1.30	*	*	
2	0.52	0.68	1.30	*	*	
3	0.30	0.39	1.30	*	*	
4	0.20	0.26	1.30	*	*	
5	0.14	0.19	1.30	*	*	
6	0.11	0.14	1.30	*	*	
7	0.09	0.11	1.30	*	*	
8	0.07	0.09	1.30	*	*	
9	0.06	0.08	1.30	*	*	
10	0.05	0.07	1.30	*	*	

* Since I(tD) - O< zero, there is no storage needed

Figure 6.3.9
Detention Storage Calculations for the Roof Top Using the Rational Method

The volume of each cell is

$$V = \frac{1}{3}Ad \therefore solving\ for\ d: \qquad d = \frac{3V}{A}$$

Since there are six "cells", the maximum depth required for each is:

$$d = \frac{3(0.08\,acre - ft)}{1.38\,acres} = 0.17\,ft$$

In summary, there are six drains needed. Each one must be able to handle a two-inch head and release

$$\frac{1.30\,cfs}{6} = 0.217\,cfs = 97.38\,gpm$$

From Table 6.3.1, it is found that a four-inch leader is required.

Parking Lot

The total area which contributes to the parking lot inlet is 1.15 acres. The developed runoff coefficient is found below:

Type of Cover	*Area Calculations*	*Area (ft^2)*	*Area (acres)*	*C*	C_nA_n
Pavement	150'x150'	22,500	0.52	0.95	0.49
Grass	100'x200' + 150'x50'	27,500	0.63	0.25	0.16
Total			1.15		0.65

$$C_D = \frac{\sum C_n A_n}{A_T} = \frac{0.65}{1.15} = 0.57$$

With the allowable release of 0.40 cfs, the storage can be calculated as shown in Figure 6.3.11, where the peak storage is found to be 0.05 acre-feet. The figure below shows the parking lot layout. The depth in the parking lot is

$$d = \frac{3V}{A} = \frac{3(0.05\,acre - ft)}{0.52\,acres} = 0.29\ ft.$$

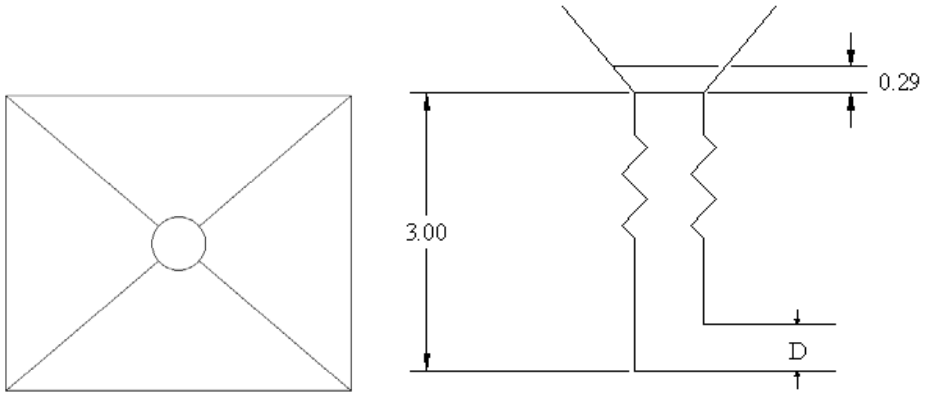


Figure 6.3.10
Drainage for Parking Lot Segment

Project: Example 6.3.1 **Detention Facility Design Return Period:** 100

Designer: TTB **Release Rate Return Period:** 3

Watershed Area:	1.15 acres
Time of Concentration (undeveloped):	--- minutes
Rainfall Intensity (i_U):	--- inches/hr
Undeveloped Runoff Coefficent (C_U):	---
Undeveloped Runoff Rate (O):	0.40 cfs
Developed Runoff Coefficent (C_D)	0.57

Storm Duration t_d (hours)	Rainfall Intensity i_d (inches/hr)	Inflow Rate $I(t_d)$ (cfs)	Outflow Rate O (cfs)	Storage Rate $I(t_d)$-O (cfs)	Required Storage (I(td)-O)(t_d/12) (acre-feet)	
0.17	4.36	2.86	0.40	2.46	0.03	
0.33	3.06	2.00	0.40	1.60	0.04	
0.50	2.28	1.50	0.40	1.10	0.05	← **Peak Storage**
0.67	1.78	1.17	0.40	0.77	0.04	
0.83	1.44	0.94	0.40	0.54	0.04	
1	1.19	0.78	0.40	0.38	0.03	
1.5	0.75	0.49	0.40	*	*	
2	0.52	0.34	0.40	*	*	
3	0.30	0.20	0.40	*	*	
4	0.20	0.13	0.40	*	*	
5	0.14	0.09	0.40	*	*	
6	0.11	0.07	0.40	*	*	
7	0.09	0.06	0.40	*	*	
8	0.07	0.05	0.40	*	*	
9	0.06	0.04	0.40	*	*	
10	0.05	0.03	0.40	*	*	

* Since I(tD) - O< zero, there is no storage needed

Figure 6.3.11
Detention Storage Calculations for the Parking Lot Using the Rational Method

If a 3-foot deep inlet is used, the total head to the centerline of the pipe is

$$(0.29 + 3.0 - \frac{D}{2})$$

The required pipe size is determined using Equation 6.3.5 and an iterative process:
(take n = 0.013)

$$0.5 = A_p \left[\frac{h_p}{\frac{K_e + K_o}{2g} + \frac{2.87 n^2 L}{D^{4/3}}} \right]^{1/2} = \frac{\pi D^2}{4} \left[\frac{3.29 - \frac{D}{2}}{\frac{1.43}{64.4} + \frac{2.87(0.013)^2(100)}{D^{4/3}}} \right]^{1/2}$$

D = 0.61 feet =7.37 inches

Pond

The total area of the basin which contributes to the pond is 7.17 acres. The runoff coefficient calculations are shown below:

Type of Cover	*Area Calculations*	*Area (ft^2)*	*Area (acres)*	*C*	C_nA_n
Parking Lot	200' x 650'	130,000	2.98	0.95	2.83
Pond	100' x 50'	5,000	0.11	1.0	0.11
Grass	100'x200' + 250'x650' -0.11 acres		4.08	0.25	1.02
Total			7.17		3.96

$$C_D = \frac{\sum C_n A_n}{A_T} = \frac{3.96}{7.17} = 0.55$$

Using the undeveloped runoff coefficient and an outflow rate of 1.9 cfs, the storage volumes can be calculated as shown in Figure 6.3.12. The required storage volume is 0.30 acre-ft. Since the permanent area is 5,000 ft^2, the volume required is determined using a minimum 3:1 slope.

If a depth of 4.75 feet is used, the outside perimeter of the pond will be 78.5 x 128.5 and the volume will be 0.82 acre-feet as shown below:

$$V = \left[\frac{(100 + 6(4.75)) \times (50 + 6(4.75)) + (100)(50)}{2} \right] 4.75 = 0.82\ ac - ft$$

Project: Example 6.3.1 **Detention Facility Design Return Period:** 100

Designer: TTB **Release Rate Return Period:** 3

Watershed Area:	7.17 acres
Time of Concentration (undeveloped):	--- minutes
Rainfall Intensity (i_U):	--- inches/hr
Undeveloped Runoff Coefficent (C_U):	---
Undeveloped Runoff Rate (O):	1.90 cfs
Developed Runoff Coefficent (C_D)	0.55

Storm Duration t_d (hours)	Rainfall Intensity i_d (inches/hr)	Inflow Rate $I(t_d)$ (cfs)	Outflow Rate O (cfs)	Storage Rate $I(t_d)$-O (cfs)	Required Storage (I(td)-O)(t_d/12) (acre-feet)	
0.17	4.36	17.21	1.90	15.31	0.21	
0.33	3.06	12.06	1.90	10.16	0.28	
0.50	2.28	9.01	1.90	7.11	0.30	← **Peak Storage**
0.67	1.78	7.03	1.90	5.13	0.28	
0.83	1.44	5.67	1.90	3.77	0.26	
1	1.19	4.68	1.90	2.78	0.23	
1.5	0.75	2.94	1.90	1.04	0.13	
2	0.52	2.05	1.90	0.15	0.02	
3	0.30	1.18	1.90	*	*	
4	0.20	0.78	1.90	*	*	
5	0.14	0.57	1.90	*	*	
6	0.11	0.43	1.90	*	*	
7	0.09	0.34	1.90	*	*	
8	0.07	0.28	1.90	*	*	
9	0.06	0.23	1.90	*	*	
10	0.05	0.20	1.90	*	*	

* Since I(tD) - O< zero, there is no storage needed

Figure 6.3.12
Detention Storage Calculations for the Detention Pond Using the Rational Method

Conclusion:

Type of Cover	*Storage (acre-feet)*	*Release Rate (cfs)*
Pond	0.30	1.9
Rooftop	0.08	1.3
Parking Lot	0.05	0.4
Total	0.43	3.6

```
AT
C
C     AUTHORS: DR. D.D. GRAY AND R.B. BOWERS
C     DATE STARTED: DEC. 22, 1986
C     LAST MODIFIED: APRIL 10, 1988
C
C     THIS PROGRAM USES THE MODIFIED RATIONAL METHOD TO
C     SIZE DETENTION STORAGE.
C     REFERENCE: HERPICC COUNTY STORM DRAINAGE MANUAL BY C.B. BURKE
C
C     ND = NUMBER OF DURATIONS CONSIDERED
      PARAMETER (ND=16)
      CHARACTER LABEL*80
      DIMENSION T(ND),RAIN(ND)
C
C     ARRAY T(ND) CONTAINS STORM DURATIONS IN HOURS
C     AS RECOMMENDED BY MSDGC.
      DATA T/0.17, 0.33, 0.50, 0.67, 0.83, 1.0, 1.5, 2.0, 3.0, 4.0,
     & 5.0, 6.0, 7.0, 8.0, 9.0, 10.0/
C
C     ARRAY RAIN(ND) CONTAINS INTENSITIES FOR THE CITY
C     AND THE RETURN PERIOD TR.
C     THE FOLLOWING INTENSITIES (7.6,5.5,...0.49) ARE FOR CHICAGO
      DATA RAIN/7.6, 5.5, 4.4, 3.7, 3.2, 2.8, 2.1, 1.7, 1.2, 1.0,
     & 0.84, 0.73, 0.65, 0.58, 0.53, 0.49/
C
C     TR IS THE RETURN PERIOD
      DATA TR/100./
C
C     FOR EACH CASE THERE ARE 2 CARDS OF INPUT DATA.
C
C       CARD 1 (A80):
C          LABEL = DESCRIPTIVE TITLE
C       CARD 2 (6F10.0):
C             TU = UNDEVELOPED RETURN PERIOD (YEARS)
C           RUND = UNDEVELOPED RAINFALL INTENSITY (INCHES/HOUR)
C             CU = UNDEVELOPED RUNOFF COEFFICIENT
C             AU = UNDEVELOPED AREA (ACRES)
C             AD = DEVELOPED AREA (ACRES)
C             CD = DEVELOPED RUNOFF COEFFICIENT
C
    1 READ(5,100,END=99) LABEL
  100 FORMAT(A80)
      READ(5,101) TU,RUND,CU,AU,AD,CD
  101 FORMAT(6F10.0)
      WRITE(6,200) LABEL
  200 FORMAT('1',//,1X,A80,/)
      WRITE(6,201) TU,RUND,CU,AU,AD,CD
  201 FORMAT(' UNDEVELOPED RETURN PERIOD (YEARS) = ',
     &6X,F10.0,/,' UNDEVELOPED RAINFALL INTENSITY (INCH/HOUR) = ',
     &F10.3,/,' UNDEVELOPED RUNOFF COEFFICIENT = ',12X,F10.3,/,
     &' UNDEVELOPED AREA (ACRES) = ',18X,F10.3,/,
     &' DEVELOPED AREA (ACRES) = ',20X,F10.3,/,
     &' DEVELOPED RUNOFF COEFFICIENT = ',14X,F10.3,/)
C     CALCULATE ALLOWABLE OUTFLOW USING THE RATIONAL METHOD
      OUT=CU*AU*RUND
      WRITE(6,202) OUT
  202 FORMAT(' ALLOWABLE OUTFLOW RATE (CFS) = ',14X,F10.3,//)
      WRITE(6,203)
```

```
  203 FORMAT('  DURATION   INTENSITY   INFLOW',5X,' OUTFLOW'
     &,1X,' STORAGE RATE',1X,' STORAGE',/,2X,' HOURS'
     &,5X,' IN/HOUR',5X,' CFS',8X,' CFS',8X,' CFS',6X,' AC-FT',/)
      VOL=0.
C
C     CALCULATE STORAGE FOR EACH DURATION
C
      DO 10 I=1,ND
      FLOWIN=CD*AD*RAIN(I)
      DELQ=FLOWIN-OUT
      STOR=DELQ*T(I)/12.
      WRITE(6,204) T(I),RAIN(I),FLOWIN,OUT,DELQ,STOR
  204 FORMAT(1F7.2,5F12.2)
      IF(STOR.GT.VOL) VOL=STOR
   10 CONTINUE
      WRITE(6,205) VOL
  205 FORMAT(//,' MAXIMUM REQUIRED STORAGE (AC-FT) =',F12.2)
      GOTO 1
   99 CONTINUE
      STOP
      END
```

```
TEST CASE FOR MODIFIED RATIONAL METHOD
3.        1.8       0.15      13.4      13.4      0.36
SECOND TEST CASE FOR MODIFIED RATIONAL METHOD
3.        1.8       0.15      13.4      13.4      0.50
```

TEST CASE FOR MODIFIED RATIONAL METHOD

UNDEVELOPED RETURN PERIOD (YEARS) = 3.
UNDEVELOPED RAINFALL INTENSITY (INCH/HOUR) = 1.800
UNDEVELOPED RUNOFF COEFFICIENT = 0.150
UNDEVELOPED AREA (ACRES) = 13.400
DEVELOPED AREA (ACRES) = 13.400
DEVELOPED RUNOFF COEFFICIENT = 0.360

ALLOWABLE OUTFLOW RATE (CFS) = 3.618

DURATION HOURS	INTENSITY IN/HOUR	INFLOW CFS	OUTFLOW CFS	STORAGE RATE CFS	STORAGE AC-FT
0.17	7.60	36.66	3.62	33.04	0.47
0.33	5.50	26.53	3.62	22.91	0.63
0.50	4.40	21.23	3.62	17.61	0.73
0.67	3.70	17.85	3.62	14.23	0.79
0.83	3.20	15.44	3.62	11.82	0.82
1.00	2.80	13.51	3.62	9.89	0.82
1.50	2.10	10.13	3.62	6.51	0.81
2.00	1.70	8.20	3.62	4.58	0.76
3.00	1.20	5.79	3.62	2.17	0.54
4.00	1.00	4.82	3.62	1.21	0.40
5.00	0.84	4.05	3.62	0.43	0.18
6.00	0.73	3.52	3.62	-0.10	-0.05
7.00	0.65	3.14	3.62	-0.48	-0.28
8.00	0.58	2.80	3.62	-0.82	-0.55
9.00	0.53	2.56	3.62	-1.06	-0.80
10.00	0.49	2.36	3.62	-1.25	-1.05

MAXIMUM REQUIRED STORAGE (AC-FT) = 0.82

SECOND TEST CASE FOR MODIFIED RATIONAL METHOD

UNDEVELOPED RETURN PERIOD (YEARS) = 3.
UNDEVELOPED RAINFALL INTENSITY (INCH/HOUR) = 1.800
UNDEVELOPED RUNOFF COEFFICIENT = 0.150
UNDEVELOPED AREA (ACRES) = 13.400
DEVELOPED AREA (ACRES) = 13.400
DEVELOPED RUNOFF COEFFICIENT = 0.500

ALLOWABLE OUTFLOW RATE (CFS) = 3.618

DURATION HOURS	INTENSITY IN/HOUR	INFLOW CFS	OUTFLOW CFS	STORAGE RATE CFS	STORAGE AC-FT
0.17	7.60	50.92	3.62	47.30	0.67
0.33	5.50	36.85	3.62	33.23	0.91
0.50	4.40	29.48	3.62	25.86	1.08
0.67	3.70	24.79	3.62	21.17	1.18
0.83	3.20	21.44	3.62	17.82	1.23
1.00	2.80	18.76	3.62	15.14	1.26
1.50	2.10	14.07	3.62	10.45	1.31
2.00	1.70	11.39	3.62	7.77	1.30
3.00	1.20	8.04	3.62	4.42	1.11
4.00	1.00	6.70	3.62	3.08	1.03
5.00	0.84	5.63	3.62	2.01	0.84
6.00	0.73	4.89	3.62	1.27	0.64
7.00	0.65	4.35	3.62	0.74	0.43
8.00	0.58	3.89	3.62	0.27	0.18
9.00	0.53	3.55	3.62	-0.07	-0.05
10.00	0.49	3.28	3.62	-0.33	-0.28

MAXIMUM REQUIRED STORAGE (AC-FT) = 1.31

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Chapter 7 - STORM SEWER SYSTEM DESIGN

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Chapter 7 - STORM SEWER SYSTEM DESIGN

LIST OF PARAMETERS

A	Area of watershed under consideration (acres).
C	Ratio of peak runoff rate to average rainfall rate over the time of concentration
d	Diameter of the culvert (ft)
H	Headwater depth (ft)
i	Rainfall intensity (in/hr)
i_t^{Tr}	Rainfall intensity corresponding to time, t, and recurrence interval, T_r
L	Length (ft)
Q	Peak discharge (cfs)
Q_p	Capacity of the pipe flowing full (cfs)
Q_r	Runoff from rational method (cfs)
S	Slope (ft/ft)
t_c	Time of concentration (minutes)
t_t	Travel time (minutes)
V	Velocity (ft/sec)
y_c	Critical depth (ft)
y_t	Tailwater depth (ft)

Chapter 7 - STORM SEWER SYSTEM DESIGN

7.1 - INTRODUCTION

The design of a storm sewer system utilizes all the material contained in the previous five chapters. The rainfall data presented in Chapter 2 is used with the runoff prediction methods in Chapter 3 to size the pipes or channels using Manning's Equation (Chapter 4, Equation 4.2.1). When inlets and gutters are to be included in the design or when storage facilities are mandated or desired, Chapters 5 and 6 may be used. Each of the above mentioned are essential components of the system and must be carefully considered.

This chapter presents an outline of the interactive components and references other parts of the manual for information pertaining to each component. Methods which are commonly employed for the design of storm sewers and the basic principles of culvert design are presented. Design criteria applicable for the state of Indiana are outlined and the various materials used for storm sewers and culverts are listed. Examples at the end of the chapter illustrate applications.

7.2 - METHODS EMPLOYED IN THE SIZING OF STORM SEWERS

In Chapter 3, methods were presented by which the quantity of precipitation which becomes stormwater runoff can be computed. The rational method, Soil Conservation Service Curve Number Method and synthetic hydrograph methods were all discussed. Since the curve number method computes only a volume and the hydrographs require a considerable amount of computation, they are not usually employed in complex storm sewer designs unless computer programs are available. Consequently, the rational method, by virtue of its simplicity and ease of application, is the most frequently used method throughout the world (UNESCO, 1977).

However, because of the shortcomings of the rational method and the rapid development and availability of personal computers, programs have been developed which incorporate, in varying detail, hydraulic-hydrologic principles in an attempt to better simulate the rainfall-runoff process. This section discusses the rational method as employed in storm sewer design and one of several available computer programs for storm sewer design.

Table 7.2.1

Components in Storm Sewer Design

Description	Reference
Pipes	
(a) Materials, types	Section 7.5
(b) Sizing	Section 7.2
(c) Design criteria	Section 7.4
(d) Roughness coefficients	Section 4.2, Table 4.2.1
Culverts	
(a) Materials, types	Section 7.5
(b) Sizing	Section 7.3
(c) Design criteria	Section 7.4
(d) Roughness coefficients	Section 4.2, Table 4.2.1
Curbs and Gutters	
(a) Flow in gutters	Section 5.2
(b) Gutter inlets	Section 5.3.1
(c) Curb inlets	Section 5.3.2
(d) Combined inlets	Section 5.3.3
Storage Facilities	
(a) Retention pond	Section 6.3.1
(b) Detention pond	Section 6.3.2
(c) Parking lot storage	Section 6.3.3
(d) Rooftop storage	Section 6.3.4

7.2.1 - Sizing Storm Sewers with the Rational Method

The theory and assumptions of the rational method were discussed in Chapter 3 (Section 3.2). The method is very easily extended for the design of a storm sewer system and for this reason is widely used.

As presented in Chapter 3, the rational formula is written as:

$$Q = CiA \qquad (7.2.1)$$

where Q = peak discharge (cfs), C = ratio of peak runoff rate to average rainfall rate over the time of concentration, i = rainfall intensity (inches/hr), and A = area of watershed under consideration (acres).

The principal idea of the rational method is that the peak rate of surface runoff, Q, from a subbasin is proportional to the subbasin area, A, and the average rainfall intensity, i, over a period of time just sufficient for all parts of the subbasin to contribute to the outflow (time of concentration). The runoff coefficient, C, is supposed to reflect all those characteristics of the

watershed such as imperviousness and antecedent moisture condition, which affect the runoff rate.

Runoff coefficients for rural areas may be found in Table 3.2.1 and urban areas in Table 3.2.2. The time of concentration can be found using either Table 3.2.4 or an appropriate estimate. For a design return period, the rainfall intensity can be found using the method discussed in Sections 2.2.4 and 2.2.5, with the storm duration equal to the time of concentration.

When applied to a watershed with many subbasins, the formula is used to size each sewer pipe separately, considering all the upstream areas contributing to that pipe. Figure 7.2.1 has been presented to aid in the computations; Table 7.2.2 explains the headings for each column.

Examples 7.6.1 and 7.6.2 illustrate the use of the rational method for storm sewer design using Figure 7.2.1.

7.2.2 - Computer Programs for Storm Sewer Design

With the rapid development of personal computers, computations that once took many hours to complete are reduced to a few seconds of computer time. Consequently, it is now feasible to investigate the performance of alternate designs and the effect of varying design parameters.

One very popular and useful computer program for storm sewer design is the Illinois Urban Drainage Area Simulator (ILLUDAS) (Terstriep and Stall, 1974). This program allows for the design of up to 999 pipes or channels given the lengths and slopes of each reach. It requires no more data than that needed for the rational method but it enables the designer to investigate the effects of variations of storm frequency, duration, antecedent moisture condition and the temporal distribution of the rainfall with relative ease. In addition, the sizing of storage facilities is easily accomplished. This program which incorporates the basic principles of hydraulics and hydrology may be obtained along with the Users Manual from the Illinois State Water Survey, Urbana, Illinois (Bulletin #58).

A couple of computer programs used to design storm sewers are contained in the HYDRAIN system (FHWA, 1992). The Hydrain package contains seven programs; HYDRAIN, HYDRO - Hydrology, HYDRA - Storm drains, HYCLV - Culverts, WSPRO - Step backwater and bridge hydraulics, HY-8 - Culverts, and HYCHL - Roadside channels. The programs have editors that assist in developing input files and give explanations of the desired data.

Project ____________ Date ____________ Sheet ______ of ______

Engineer ____________ Design Storm ____________ Mannings *n* ____________

Line Number	Upstream Manhole	Downstream Manhole	Length (ft)	Cj	Aj (acres)	CjAj	ΣCjAj	tj (min)	tcum (min)	i (inches/hour)	Q (cfs)	Pipe Diameter (inches)	Pipe Slope (%)	Pipe Capacity (cfs)	Velocity (ft/sec)	Travel Time (min)	Rim Elevation Upstream (ft)	Rim Elevation Downstream (ft)	Invert Elevation Downstream (ft)	Invert Elevation Upstream (ft)	Pipe Cover Upstream (ft)	Pipe Cover Downstream (ft)
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23

Figure 7.2.1
Storm Sewer Design Sheet-Rational Method

Table 7.2.2
Storm Sewer Design Sheet - Rational Method, Explanation of Column Headings in Figure 7.2.1

Column 1	Identification number of the particular reach.
Column 2 & 3	Identification numbers of the upstream and downstream structures.
Column 4	Length of the reach under consideration.
Column 5	Runoff coefficient of the subbasin. This may be a weighted composite based on the type of cover in the subbasin.
Column 6	Area of the subbasin, in acres, entering the upstream structure.
Column 7	Product of the runoff coefficient and the subbasin area.
Column 8	Sum of the runoff coefficients and area products contributing to the pipe under consideration.
Column 9	Inlet time for the subbasin under consideration (time of concentration).
Column 10	Maximum inlet time and total travel time for the water from the most distant subbasin to contribute (the longest travel time when all are considered).
Column 11	Rainfall intensity for the storm design frequency and storm duration equal to the time of concentration (Column 10).
Column 12	Peak flowrate for the reach under consideration, $Q = i\Sigma CA$.
Column 13	Selected pipe diameter.
Column 14	Pipe slope between structures. This may be the slope of the ground or may be the slope necessary to achieve minimum velocity.
Column 15	Full flow capacity of the pipe with diameter selected in Column 13 and slope in Column 14, determined using Manning's Equation.
Column 16	Full pipe velocity found by $V = Q/A$.
Column 17	Travel time in the reach found by $t_t = L/60V$.
Column 18 & 19	Rim (ground) elevations of the upstream and downstream structures, respectively.
Column 20 & 21	Invert elevations of the pipe at the upstream and downstream structures found by subtracting the fall of the sewer from the upstream invert.
Column 22 & 23	Pipe cover for the upstream and downstream pipes. Found by subtracting the top of the pipe elevation from the rim elevation.

7.3 - HYDRAULICS OF CULVERTS

Culverts are special types of pipes employed in the conveyance of stormwater runoff because of the many types of flow which may occur. The flow is dependent upon the inlet geometry, slope, size, roughness and approach or tailwater conditions. In order to properly determine the quantity of flow which will be conveyed, all of the above need to be considered.

The various types of culvert flow which may be encountered are shown in Figure 7.3.1. If the outlet is submerged or if the outlet is not submerged but the headwater is high and the barrel is long, the culvert will flow full as shown in Types 1 and 2. A culvert which has the characteristics of Type 2 is termed "hydraulically-long". On the other hand, a culvert which is not long enough for flow to develop is called "hydraulically-short" and is categorized as Type 3. The culvert size, slope, entrance geometry and headwater and tailwater conditions determine whether a culvert is "hydraulically-long" or "-short". Charts are available to roughly distinguish between the two types. (Chow, 1988)

If the outlet and inlet are not submerged, and the tailwater is deeper than the critical depth (see Chapter 4, Section 4.2.2) the flow in the culvert is subcritical as shown in Figure 7.3.1 as Type 4. Two types of possible flow conditions exist when the tailwater depth is less than critical depth. The first type occurs when the depth of flow in the culvert is greater than the critical depth. For this condition the culvert is said to have "outlet-control", while a pipe depth less than critical depth will have supercritical flow and is under "inlet-control". These two conditions are shown as Types 5 and 6 in Figure 7.2. Table 7.3.1 summarizes each type in outline form.

Table 7.3.1
Summary of Culvert Flow (after Chow, 1988)

Condition	Flow Computations	Type (Figure 7.3.1)
A. Outlet Submerged	full pipe flow, dependent on inlet geometry, etc.	1
B. Outlet Not Submerged		
1. Headwater greater than critical value		
(a) Culvert hydraulically long	full pipe flow	2
(b) Culvert hydraulically short	orifice pipe flow	3
2. Headwater less than critical value		
(a) Tailwater higher than critical depth	weir flow	4
(b) Tailwater lower than critical depth		
(i) Subcritical Slope	weir flow	5
(ii) Supercritical Slope	weir flow	6

The sizing procedure which is commonly employed is a method outlined in "Hydraulic Charts for the Selection of Highway Culverts" (Hydraulic Engineering Circular No. 5, 1965) and "Capacity Charts for the Design of Highway Culverts" (Hydraulic Engineering Circular No. 10, 1965), both by the Federal Highway Administration. The appropriate chart is dictated by the parameters of roughness, slope, headwater depth, tailwater depth, length and either inlet or

outlet control. Rather than presenting the numerous charts in this manual, the reader is referred to the above circulars for the computation of culvert capacities. The circulars are presented in the computer program HY-8 (see Chapter 4, Section 4.5) which facilitates the lengthy computations required for the sizing procedure.

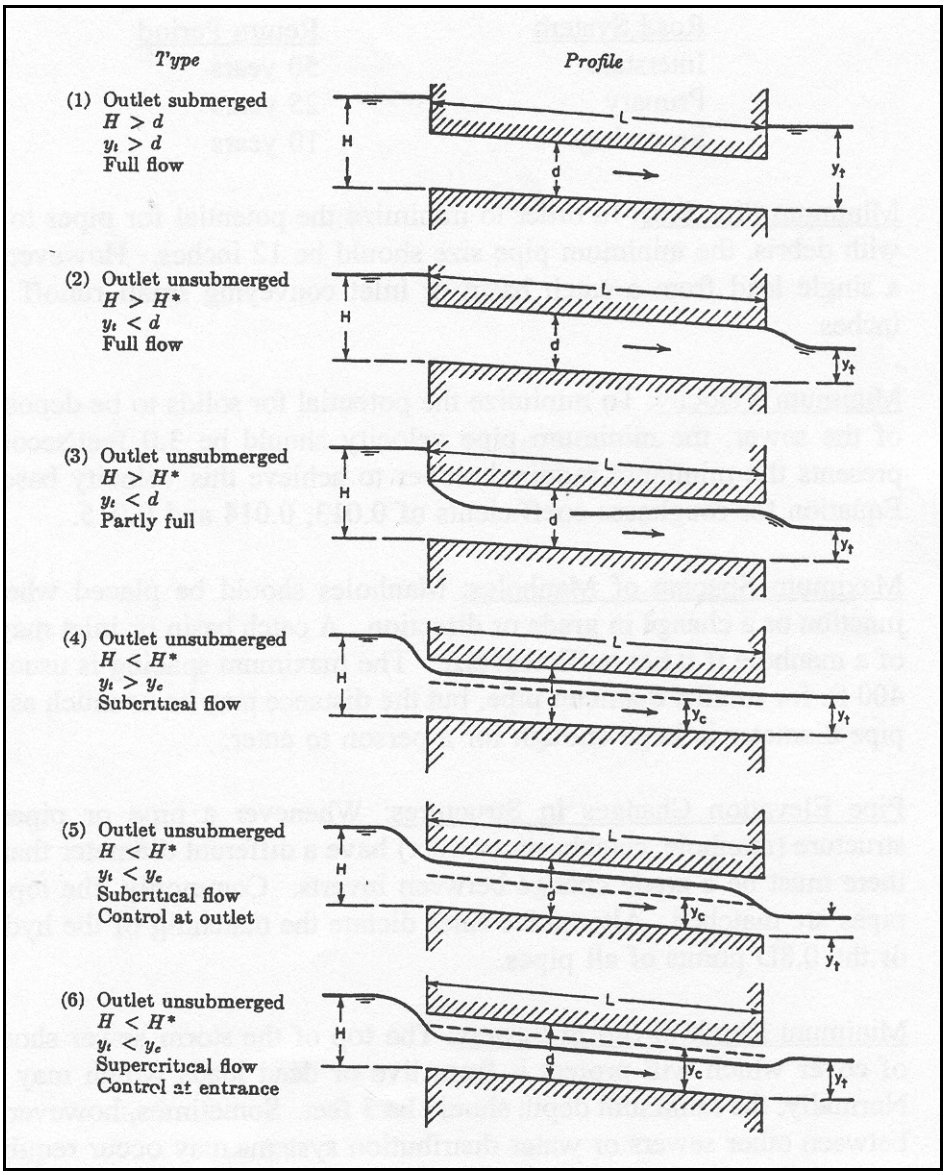


Figure 7.3.1
Types of Culvert Flow (Chow, 1988)

7.4 - CRITERIA FOR STORM SEWER DESIGN

This section outlines some criteria for the design of storm sewers.

1. Return Period: The minimum return period which should be used is 5 - 10 years for urban storm sewer systems. When there is a greater possibility of property damage, larger frequencies should be used. For roadways, the Indiana Department of Highways recommends the following return periods for culvert cross-drains and any type of facility in an underpass or depressed section of highway.

Road System	Return Period
Interstate	50 years
Primary	25 years
Secondary	10 years

2. Minimum Pipe Size: In order to minimize the potential for pipes to become clogged with debris, the minimum pipe size should be 12 inches. However, the diameter of a single lead from a catch basin or inlet conveying small runoff rates may be 10 inches.

3. Minimum Velocity: To minimize the potential for solids to be deposited in the invert of the sewer, the minimum pipe velocity should be 3.0 feet/second. Table 7.4.1 presents the minimum required slopes to achieve this velocity based on Manning's Equation for roughness coefficients of 0.013, 0.014 and 0.015.

4. Maximum Spacing of Manholes: Manholes should be placed whenever there is a junction or a change in grade or direction. A catch basin or inlet may be used instead of a manhole if it has sufficient size. The maximum spacing is usually between 300-400 ft. for smaller diameter pipe, but the distance may be as much as 500 ft. when the pipe diameter is large enough for a person to enter.

5. Pipe Elevation Changes in Structures: Whenever a pipe or pipes which enter a structure (manhole, catchbasin or inlet) have a different diameter than the outlet pipe, there must be a grade change between inverts. Commonly, the top elevation of the pipes are matched. Alternative rules dictate the matching of the hydraulic grade line or the 0.8D points of all pipes.

6. Minimum Depth of Storm Sewers: The top of the storm sewer should have a depth of cover which will protect it from live or dead loads which may be placed on it. Normally, the minimum depth should be 3 feet. Sometimes, however, a grade conflict between other sewers or water distribution systems may occur requiring the depth to be less than 3 feet. In this case extra strength pipe or special trench backfill procedures must be employed. The reader is referred to "Design Manual - Concrete Pipe" (Concrete Pipe Association of Indiana, 1974) or Modern Sewer Design, (American Iron and Steel Institute, 1980) for information regarding pipe loadings.
7. Minor and Major Systems: The design of a storm sewer system should include an

investigation of the impact of a high-frequency storm (i.e. 100 years) on the minor and major system. The minor system is comprised of the storm sewers, inlets, catchbasins etc. The major system includes the portion of the watershed which will become operative once the capacity of the minor system is exceeded. This includes the streets or other depressed areas which will act as open channels and convey the excess stormwater runoff to a natural watercourse or storage facility. If the resulting depth in these areas poses a danger to life or property, the capacity of the minor system should be increased.

Table 7.4.1
Minimum Pipe Slopes Necessary to Ensure a 3 ft/sec Velocity

Pipe Diameter	Percent Slope (ft/ft x 100)		
	$n = 0.013$	$n = 0.014$	$n = 0.015$
10	0.555	0.643	0.739
12	0.435	0.505	0.579
15	0.323	0.375	0.430
18	0.253	0.294	0.337
21	0.206	0.239	0.275
24	0.173	0.200	0.230
27	0.148	0.171	0.196
30	0.128	0.149	0.171
36	0.101	0.117	0.134
42	0.082	0.095	0.109
48	0.069	0.079	0.091
54	0.059	0.068	0.078
60	0.051	0.059	0.068
66	0.045	0.052	0.060
72	0.040	0.046	0.053
78	0.036	0.042	0.048

7.5 - MATERIALS USED FOR STORM SEWERS

This section briefly reviews some commonly used pipe materials for storm sewer systems. Table 7.5 lists the pipe materials commonly used for storm sewers and the sizes in which they are commercially available. The selection of one type over another is based on considerations of the durability of materials, installation cost and structural requirements.

Table 7.5.1
Commercially Available Materials and Sizes for Sewer Pipe

Nominal Diameter (inches)	CONC	RCCP	CSP	ESCP	ABS	PVC	CPVC	CI	VCC	CD	CAA
4	X		X	X		X	X		X	X	
6	X		X	X		X	X		X	X	X
8	X		X	X	X	X	X		X	X	X
10	X		X	X	X	X	X		X	X	X
12	X	X			X	X		X	X	X	X
14								X			
15	X	X			X	X			X	X	X
16								X		X	
18	X	X				X		X	X	X	X
20								X		X	
21	X	X				X			X	X	X
24	X	X				X		X	X	X	X
27		X				X			X		
30		X		X		X		X	X	X	X
33		X		X		X			X		
36		X		X		X		X	X		X
39											
42		X						X			X
48		X						X			X
54		X									X
60		X									X
66		X									
72		X									
78		X									
84		X									
90		X									
96		X									
102		X									
108		X									

Table 7.5.1 (cont'd)

Symbol Description

X	Available in pipe size
CONC	Concrete pipe
RCCP	Reinforced concrete culvert and storm drain & sewer pipe
CSP	Concrete sewer and storm drain & culvert pipe
ABS	Acrylontrile - Butadiene - Styrene composite sewer piping
PVC	Polyvinyl chloride (PVC) pipe
CPVC	Corrugated poly (vinyl Chloride) (PVC) pipe with smooth interior
CI	Cast iron pipe
VCC	Vitrified clay culvert
CD	Clay draintile (extra quality)
CAA	Corrugated aluminum alloy

NOTE: Verify pipe loading condition and consult with the pipe manufacture regarding pipe class and wall thickness.

7.6 - EXAMPLE PROBLEMS

Example 7.6.1

Determine the required pipe sizes for the hypothetical watershed shown in Figure 7.6.1 and the peak flowrate out of the watershed using the rational method and the design criteria presented in Section 7.5. The time of concentration, runoff coefficient and area are given for each subbasin. Also, the ground elevations at each structure and the distances between structures are shown. Assume the watershed is located near Fort Wayne, Indiana. Use Intensity-Duration-Frequency Equations (Section 2.2.4) to determine the rainfall intensities and match the tops of all pipes at structures.

Using Intensity-Duration-Frequency Equations, the formula to determine the rainfall intensity is:

For $t \leq 1$ hour:

$$i = \frac{c\,T_r^{\alpha}}{(t+d)^{\beta}} = \frac{2.0030(10)^{0.1655}}{(t+0.516)^{1.4643}}$$

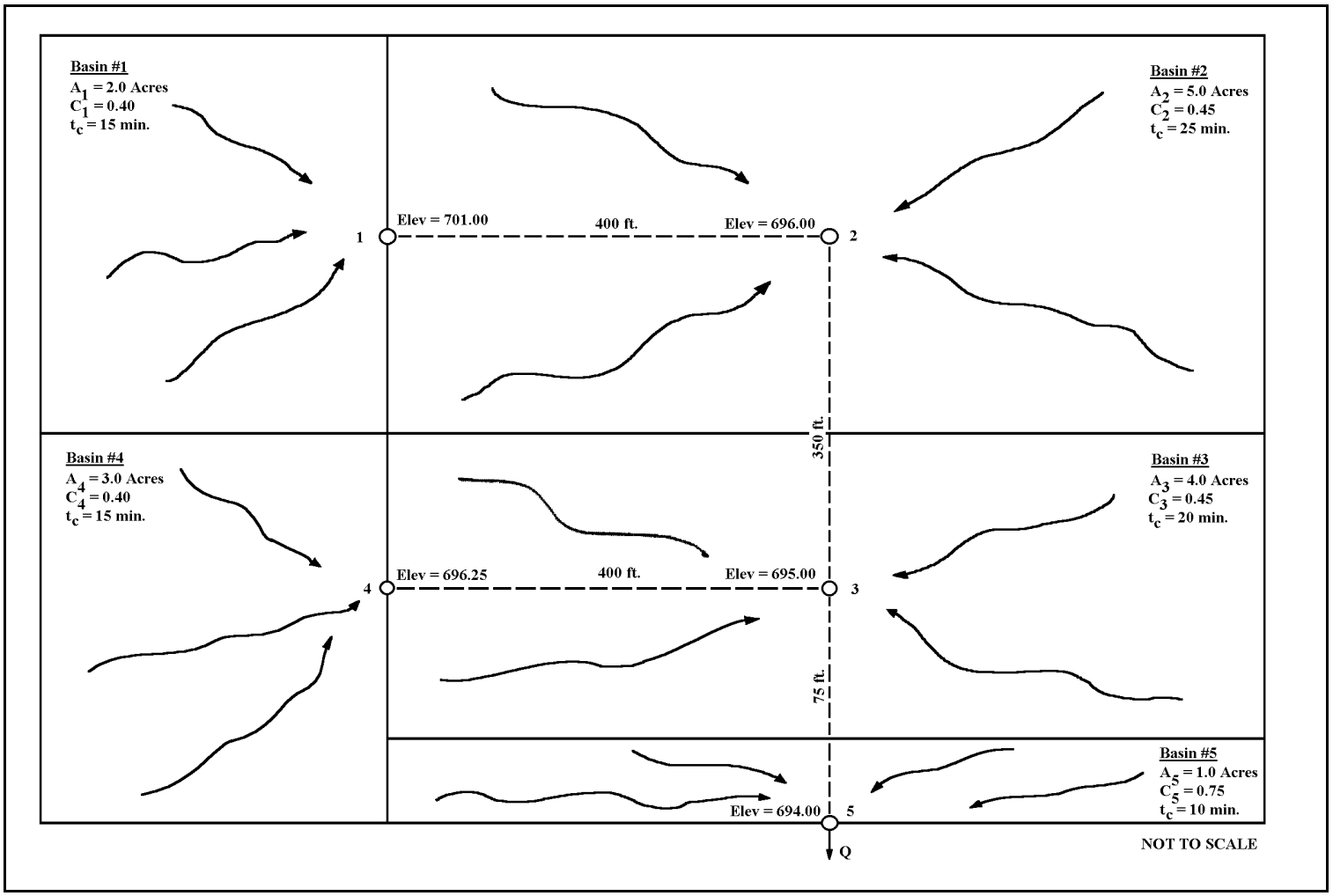
For $t > 1$ hour:

$$i = \frac{c\,T_r^{\alpha}}{(t+d)^{\beta}} = \frac{1.4381(10)^{0.1878}}{(t+0.525)^{0.8616}}$$

The calculations are shown in Figure 7.6.2. The peak flowrate from the entire watershed is found to be 21.35 cfs. An explanation of each column and entry value for line number 3 is listed in Table 7.6.1.

It should be pointed out that the depths could have been lessened if the pipe had a larger diameter necessitating a smaller slope. There is always a trade-off between increased pipe costs with the larger diameter and the decrease in excavation costs or vice versa. This is where some economic analysis comes into play. A summary of methods to evaluate these costs was reported by Han, Rao and Houk (1980).

Figure 7.6.1
Sketch of Hypothetical Watershed for Example 7.6.1



Project: Example 7.6.1 Date: Nov-09 Sheet: 1 of 1

Engineer: TTB Design Storm: 10 Years Mannings *n*: 0.013

Line Number	Upstream Manhole	Downstream Manhol	Length (ft)	C_j	A_j (acres)	C_jA_j	ΣC_jA_j	t_j (min)	t_{sum} (min)	i (inches/hour)	Q (cfs)	Pipe Diameter (inches)	Pipe Diameter (feet)	Pipe Slope (%)	Pipe Capacity (cfs)	Velocity (ft/sec)	Travel Time (min)	Rim Elevation Upstream (ft)	Rim Elevation Downstream (ft)	Invert Elevation Upstream (ft)	Invert Elevation Downstream (ft)	Pipe Cover Upstream (ft)	Pipe Cover Downstream (ft)
1	1	2	400	0.40	2.0	0.80	0.80	15	15	4.33	**3.47**	12	1.00	1.25%	3.98	5.07	1.31	701.00	696.00	696.00	691.00	4.00	4.00
2	2	3	350	0.45	5.0	2.25	3.05	25	25	3.25	**9.90**	24	2.00	0.29%	12.09	3.85	1.52	696.00	695.00	691.00	690.00	3.00	3.00
3	4	3	400	0.40	3.0	1.20	1.20	15	15	4.33	**5.20**	18	1.50	0.31%	5.87	3.32	2.01	696.25	695.00	690.00	688.75	4.75	4.75
4	3	5	75	0.45	4.0	1.80	6.05	20	26.52	3.12	**18.89**	24	2.00	1.33%	26.12	8.31	0.15	695.00	694.00	688.75	687.75	4.25	4.25
5	5	out	---	0.75	1.0	0.75	6.80	10	26.67	3.11	**21.15**	---	---	---	---	---	---	694.00	---	687.75	---	---	---

Figure 7.6.2
Storm Sewer Design Sheet-Rational Method, Example 7.6.1

Table 7.6.1
Storm Sewer Design Sheet - Rational Method, Example 7.6.1 - Explanation of Entries

Column Number	Value	Description	Explanation
3	3	Downstream Manhole	From Figure 7.6.1
4	400	Length between structures	From Figure 7.6.1
5	0.4	C_j Runoff Coefficient for subbasin #4	From Figure 7.6.1
6	3	A_j Area (acres) of subbasin #4	From Figure 7.6.1
7	1.2	C_jA_j Product of area and Runoff Coefficient for subbasin #4	From Figure 7.6.1
8	1.2	ΣC_jA_j Sum of all Area-Runoff Coefficient products contributing to line #3	1.20 because no other contributing areas
9	15	t_j Inlet time (minutes) for subbasin #4	From Figure 7.6.1
10	15	t_{cum} The longest travel time (minutes) to upstream structure #3.	This is the larger of the subbasin inlet time or the time of all upstream areas to contribute
11	4.33	i Rainfall intensity (inches/hour)	For Fort Wayne Indiana at the duration of t_{cum} at a ten-year return period
12	5.2	Q Peak Flow (cfs) into line #3	Found by Q = iΣCjAj
13	30	Pipe Diameter (inches) selected to convey Q	Found by Trial and Error, Pipe size must convey at least design capacity found in 12
14	2.5	Pipe Diameter (feet) selected to convey Q	Convert inches to feet to ease future calculations
15	0.31	Percent Slope S_o	Existing conditions found by Figure 7.6.1 or Minimum slope for chosen pipe diameter Table 7.4.1
16	5.75	Full Pipe Capacity (cfs)	Found by using Manning's Equation for a circular pipe when flowing full, must be greater than 12
17	4.68	Velocity of flow (ft/sec) in pipe	V=Flow/Area
18	1.42	Travel Time (minutes)	Travel Time= Length/Velocity for water to flow from Manhole 4 to 3
19	696.25	Rim Elevation Upstream (feet)	From Figure 7.6.1
20	695	Rim Elevation Downstream (feet)	From Figure 7.6.1
21	690	Invert Elevation Upstream (feet)	Line # 2 Invert Elevation Downstream
22	688.75	Invert Elevation Downstream (feet)	Invert Elevation Upstream -(Length * Slope)
23	4.5	Pipe Cover Upstream (feet)	Rim Elevation Upstream-Invert Elevation Upstream-1.75
24	4.5	Pipe Cover Downstream (feet)	Rim Elevation Downstream-Invert Elevation Downstream-1.75

Example 7.6.2

This problem involves the design of the storm sewer system for Fair Oaks Estates, using the rational method. This subdivision, investigated by Burke and Gray (1979) is described below.

Fair Oaks Estates is a subdivision which was constructed in Carol Stream, Illinois, a suburb west of Chicago. (Some of the inputs have been varied from the as-built conditions.) The soil is classified as Type B and the topography is basically flat. The total area of the subdivision, which includes 30 lots and is 28% impervious, is 13.4 acres. The subdivision layout and the planned drainage system is shown on Figure 7.6.3. Inlets are located in the low spots and at the corners of intersections. The drainage subbasins which contribute to the inlets are also delineated in Figure 7.6.3. The storm sewer discharges into a detention pond located in the northeast corner.

The design requirements are:

1. The storm sewer must pass a ten-year storm, based on the intensity-duration-frequency curve for Chicago, Illinois.
2. The top of the pipe must have 3.5 ft. cover.
3. The full-pipe velocity must be greater than 2 ft/sec and less than 12 ft/sec.
4. The minimum pipe diameter is 12 inches.
5. The pipe is commercial reinforced concrete pipe with a Mannings n of 0.013.
6. The outfall pipe invert must be at or above 756.00 ft MSL.
7. When the pipe size changes at a structure, the top of the pipes shall be matched.
8. Subbasin inlet times shall be assumed.

From the plan drawings and topographic map, the ground elevations and pipe lengths were determined. The rational method was used to determine the required pipe diameters and slopes. The calculations are shown in Figure 7.6.4. An explanation of each column and entry value for line number 3 is listed in Table 7.6.2.

The schematic of the drainage system designed with the rational method is shown in Figure 7.6.5. The peak flow is found to be 16.90 cfs and the time of concentration for the basin is 27.78 min. The minimum pipe is 12 inches and the maximum diameter is 21 inches. The minimum pipe cover is 3.5 feet and the maximum is 4.74 feet. Velocities fall in the range of 2.04 ft/sec to 6.68 ft/sec.

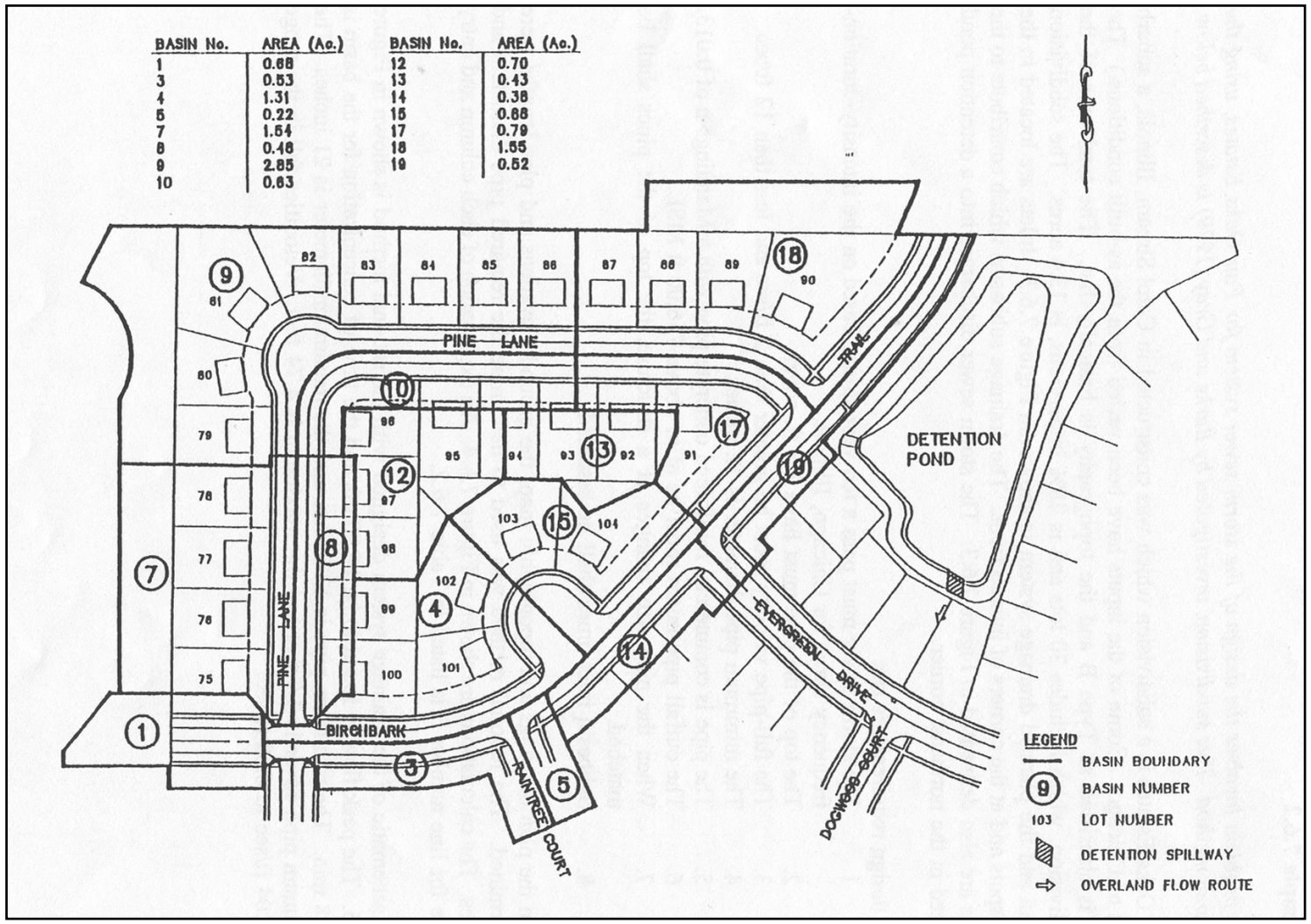

BASIN No. AREA (Ac.)
1 0.68
3 0.53
4 1.31
5 0.22
7 1.54
8 0.46
9 2.85
10 0.63
BASIN No. AREA (Ac.)
12 0.70
13 0.43
14 0.38
15 0.68
17 0.79
18 1.55
19 0.52
PINE LANE
TRAIL
DETENTION POND
PINE LANE
BIRCHBARK
RAINTREE COURT
EVERGREEN DRIVE
DOGWOOD COURT
LEGEND
BASIN BOUNDARY
BASIN NUMBER
LOT NUMBER
DETENTION SPILLWAY
OVERLAND FLOW ROUTE

Figure 7.6.3
Fair Oaks Estates Hydrologic System

PROJECT Fair Oaks Estates DATE 3-1-81 SHEET 1 OF 1

ENGINEER C. B. Burke DESIGN STORM Ten Years MANNINGS *n* 0.013

Line Number	Upstream Manhole	Downstream Manhole	Length (ft)	C_j	A_j (Acres)	C_jA_j	ΣC_jA_j	t_j (min)	t_{cum} (min)	i $\left[\frac{\text{inches}}{\text{hour}}\right]$	Q (cfs)	Pipe Diameter (inches)	Pipe Slope (%)	Pipe Capacity (cfs)	Velocity (ft/sec)	Travel Time (min)	Rim * Elevation Upstream	Rim * Elevation Downstream	Invert * Elevation Upstream	Invert * Elevation Downstream	Pipe Cover Upstream	Pipe Cover Downstream
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
1	21	20	22	0.28	0.63	0.18	0.18	20	20.0	4.00	0.7	12	0.002	1.60	2.03	0.18	73.93	73.96	69.43	69.39	3.5	3.6
2	20	19	134	--	--	--	0.18	--	20.2	3.99	0.7	12	0.024	5.55	7.06	0.76	73.96	66.14	69.39	61.61	3.6	3.5
3	19	18	76	0.44	0.53	0.23	0.41	20	20.9	3.94	1.6	12	0.010	3.62	4.62	0.27	66.14	65.88	61.61	60.83	3.5	4.0
4	18	17	290	--	--	--	0.93	--	21.2	3.92	3.6	15	0.005	4.93	4.02	1.20	65.88	64.30	60.58	58.90	4.0	4.1
5	17	16	195	--	--	--	3.64	--	27.1	3.50	12.9	21	0.006	13.00	5.41	0.60	64.30	63.59	58.65	57.10	4.1	4.7
6	16	15	30	--	--	--	4.87	--	27.7	3.47	16.9	21	0.011	16.90	7.02	0.07	63.59	outfall	57.10	56.75	4.0	--
7	14	13	28	0.32	2.85	0.91	0.91	25	25.0	3.66	3.3	12	0.008	3.33	4.24	0.11	68.27	68.27	63.77	63.53	3.5	3.7
8	13	12	115	0.49	0.63	0.31	1.22	20	25.1	3.66	4.4	15	0.006	5.38	4.38	0.44	68.27	67.23	63.28	62.48	3.7	3.5
9	12	11	26	--	--	--	1.99	--	25.5	3.63	7.2	15	0.016	8.19	6.68	0.07	67.23	66.82	62.48	62.06	3.5	3.5
10	11	10	300	0.25	0.70	0.17	2.16	20	25.6	3.62	7.8	18	0.005	7.82	4.42	1.13	66.82	65.85	61.81	60.16	3.5	4.1
11	10	9	76	0.25	0.43	0.11	2.27	20	26.7	3.54	8.0	18	0.014	12.46	4.55	0.28	65.85	64.08	60.16	59.09	4.1	3.5
12	9	17	52	0.38	0.88	0.33	2.76	20	27.0	3.51	9.6	18	0.008	9.72	5.49	0.16	64.08	64.30	59.09	58.65	3.5	4.1
13	8	7	32	0.32	1.31	0.42	0.42	20	20.0	4.00	1.6	12	0.002	1.71	2.18	0.25	65.48	65.48	60.98	60.91	3.5	3.5
14	7	18	23	0.47	0.22	0.10	0.52	20	20.2	3.98	2.0	12	0.003	2.11	2.69	0.14	65.48	65.88	60.91	60.83	3.5	4.0
15	6	5	45	0.45	0.79	0.35	0.35	20	20.0	4.00	1.4	12	0.002	1.60	2.03	0.37	63.34	63.50	58.84	58.74	3.5	3.7
16	5	4	44	0.37	1.55	0.57	0.92	20	20.3	3.97	3.6	12	0.010	3.65	4.64	0.16	63.50	63.38	58.74	58.28	3.7	4.1
17	4	16	25	0.50	0.52	0.26	1.18	20	20.5	3.96	4.6	12	0.017	4.67	5.95	0.07	63.38	63.59	58.28	57.85	4.1	4.7
18	3	2	28	0.33	1.54	0.51	0.51	20	20.0	4.00	2.0	12	0.003	2.05	2.61	0.18	70.16	70.16	65.66	65.57	3.5	3.5
19	2	12	144	0.54	0.48	0.26	0.77	20	20.1	3.98	3.0	12	0.019	5.01	6.38	0.38	70.16	67.23	65.57	62.73	3.5	3.5
20	1	9	56	0.39	0.15	0.15	0.15	20	20.0	4.00	0.6	12	0.002	1.60	2.03	0.46	64.20	64.08	59.07	59.59	3.5	3.5

* Add 700 ft. to all elevations.

Figure 7.6.4
Storm Sewer Design Sheet - Rational Method,
Example 7.6.2

Table 7.6.2
Storm Sewer Design Sheet - Rational Method, Example 7.6.2 - Explanation of Entries

Column Number	Entry Value	Explanation - Comments
1	3	Line Number
2	19	Upstream Manhole
3	18	Downstream Manhole
4	76	Length between structures (From Figure 7.6.4)
5	0.44	C_j Runoff Coefficient for subbasin. This is a weighted composite based on the type of cover in the subbasin.
6	0.53	A_j Area (acres) of subbasin
7	0.23	C_jA_j Product of area and Runoff Coefficient for subbasin (0.44 x 0.53 = 0.23)
8	0.41	ΣC_jA_j Sum of all Area-Runoff Coefficient products contributing to line #3 (0.18 + 0.23 = 0.41)
9	20	t_j Time of concentration (minutes) for subbasin. (Assumed to be 20 min.)
10	20.9	t_{cum} The longest Travel Time (minutes) from the most distant basin to contribute. (20 + 0.18 + 0.76 = 20.94 min.)
11	3.94	i Rainfall intensity (inches/hour) for Chicago, Illinois for a ten-year return period and a storm duration of 20.94 min. (3.94 inches/hour)
12	1.6	Q Peak Flow (cfs) into line #3, found by $Q = i\Sigma C_jA_j$, Q = 3.94 (0.41) = 1.61 cfs.
13	12	Pipe Diameter (inches) selected to convey Q
14	0.010	Pipe Slope (pct) at which the pipe selected above is placed.
15	3.62	Full Pipe Capacity (cfs) of the pipe with the selected diameter and slope determined by Manning's Equation. (This must be greater than or equal to the peak flowrate, Q)
16	4.62	Velocity of flow (ft/sec) in pipe (Q/A = V)
17	0.27	Travel Time (minutes) for the water to flow from Structure 19 to Structure 18. (t = L/60V = 0.27 min.)
18	766.14	Rim Elevation Upstream
19	765.88	Rim Elevation Downstream
20	761.61	Invert Elevation Upstream
21	760.83	Invert Elevation Downstream
22	3.53	Pipe Cover Upstream (Rim elev. - Top of Pipe elev.)
23	4.05	Pipe Cover Upstream (Rim elev. - Top of Pipe elev.)

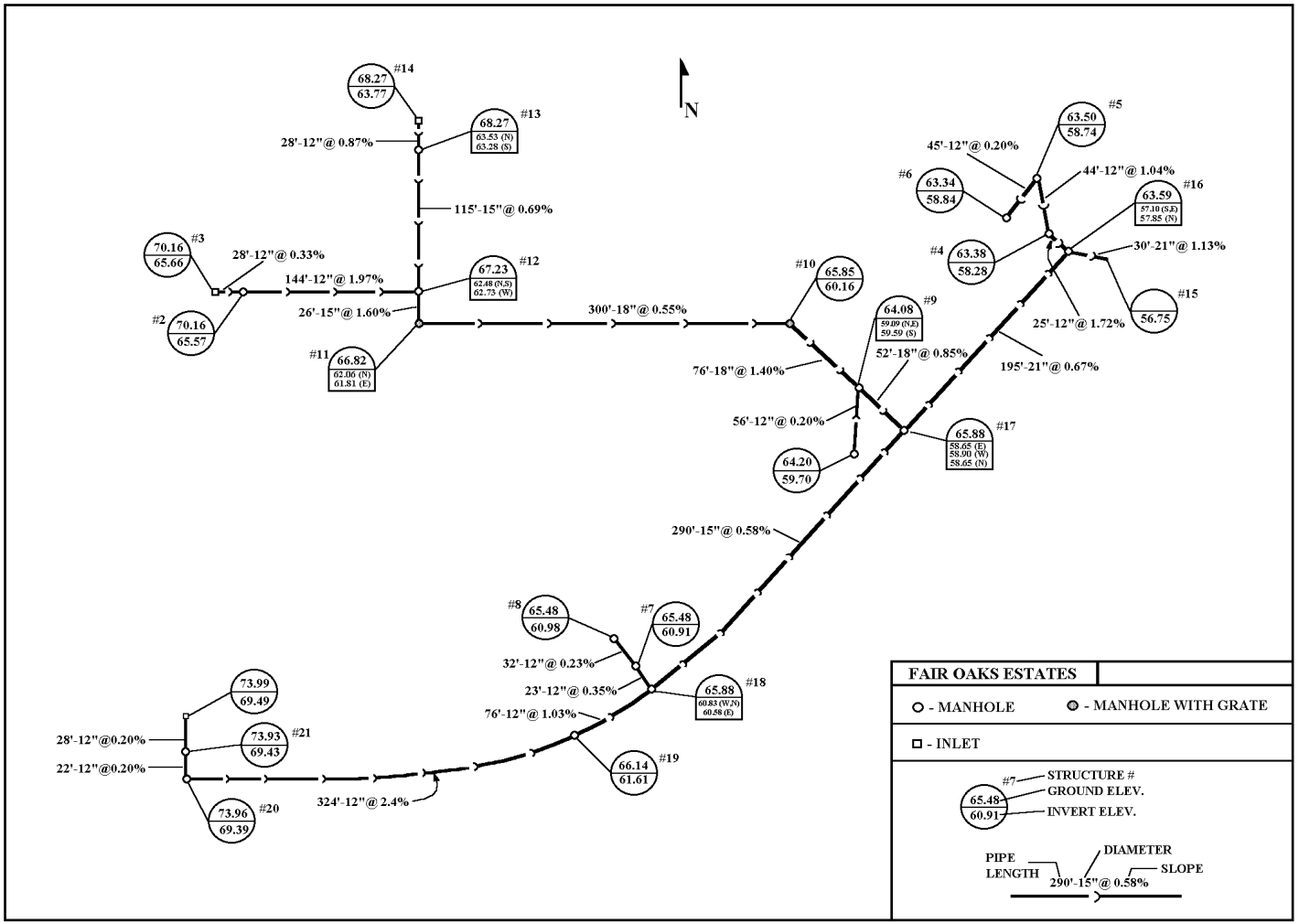


Figure 7.6.5
Layout of Storm Sewer Network System Designed by the Rational Method, Example 7.6.2

Table 7.6.3
Results of the Rational Method of Sewer System Design for Example 7.6.2 Using a 10-Year Return Period

Sewer Reach	Length (ft)	Drainage Area (acres)	C	Q_r * (cfs)	D (inches)	S (%)	V (ft/sec)	Q_p * (cfs)	Q_p / Q_r
21-20	221	0.63	0.28	0.71	12	0.2	2.03	1.60	2.26
20-19	324	-	-	0.71	12	2.40	7.06	5.55	7.82
19-18	76	0.53	0.44	1.61	12	1.03	4.62	3.62	2.25
8-7	32	1.31	0.22	1.68	12	0.23	2.18	1.71	1.00
7-18	23	0.22	0.47	2.07	12	0.35	2.69	2.11	1.02
18-17	290	-	-	3.65	12	0.58	4.02	4.93	1.35
14-13	28	-	-	3.33	12	0.87	4.24	3.33	1.00
13-12	115	0.63	0.49	4.46	15	0.69	4.38	5.38	1.21
3-2	28	1.54	0.33	2.04	12	0.33	2.61	2.05	1.00
2-12	144	0.48	0.54	3.06	12	1.97	6.38	5.01	1.64
12-11	26	-	-	7.22	15	1.60	6.68	8.19	1.13
11-10	300	0.70	0.25	7.82	18	0.55	4.42	7.82	1.00
10-9	76	0.43	0.25	8.04	18	1.40	4.55	12.46	1.55
1-9	56	0.39	0.39	0.61	12	0.20	2.03	1.60	2.63
9-17	52	0.88	0.38	9.69	18	0.85	5.49	0.72	1.00
17-16	195	-	-	12.91	21	0.67	5.41	13.00	1.00
6-5	45	0.79	0.45	1.40	12	0.20	2.03	1.60	1.14
5-4	44	1.55	0.37	3.65	12	1.04	4.64	3.65	1.00
4-16	25	0.52	0.50	4.67	12	1.71	5.95	4.67	1.00
16-15	30	-	-	16.90	21	1.13	7.02	16.90	1.00

* Q_r = Runoff from rational method
Q_p = Capacity of the pipe flowing full

Chapter 7 - REFERENCES

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Chapter 8 - COMPUTER APPLICATIONS

As illustrated in the preceding chapters, the design of stormwater infrastructure requires many lengthy computations. The task of performing hydrologic and hydraulic computations is greatly simplified using computer models. Three examples of computer models used to perform these tasks are WinTR-20, HEC-HMS, and Win TR-55. These programs are widely used and have been incorporated into many local and state regulations related to stormwater management.

This chapter utilizes the techniques learned in Chapters 2, 3 and 6 - precipitation, determination of peak runoff and volume, and stormwater storage. The chapter includes three examples for both models which illustrate the application of these programs. The same examples are used for both WinTR-20 and HEC-HMS.

8.1 - WinTR-20 PROGRAM

8.1.1 - Explanation of WinTR-20 and its Uses

The TR-20 computer program was prepared by the Soil Conservation Service (SCS) in 1977 and updated in 1982 to assist in the hydrologic evaluation of storm events for water resource projects. In 1998, the Natural Resources Conservation Service (NRCS) created WinTR-20 to replace to older DOS version of TR-20 (NRCS, 2004). The program is a single event model which computes direct runoff resulting from any synthetic or natural rainfall event of a specified duration. There is no provision for recovery of initial abstraction or infiltration during periods without rainfall. The program develops runoff hydrographs from excess precipitation and routes the flow through stream channels and reservoirs. It combines the routed hydrograph with those from tributaries and computes the peak discharges, their times of occurrence and the water elevations at any desired cross section or structure. The program provides the analysis of up to nine different rainstorm distributions over a watershed under various combinations of land treatment, flood control structures, diversions, and channel modifications. Such analysis can be performed on as many as 200 reaches and 99 structures in any one continuous run (NRCS, 2004). The program can be obtained from the NRCS electronically at:

http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-models-wintr20.html

8.1.2 - Summary of WinTR-20 Input Structure

The input data requirements are surprisingly few, with the amount of data depending on the complexity of the problem to be solved. If actual rainfall events are not going to be used, the depth of precipitation is the only meteorological input. For each subarea, the drainage area, runoff curve number, and the time of concentration are required; the antecedent soil moisture condition (i.e. AMC I, II, or III) can be specified, although the SCS now recommends only AMC II, the so-called average runoff condition (SCS, 1986). For reach routing, WinTR-20 utilizes the

Muskingum-Cunge method. This method requires the user to input such as reach length and elevation, discharge, area, top width and friction slope table.

Input

In WinTR-20, there are many different Data Sections in which you can input information. Depending on the specific watershed that is being modeled, certain sections may not need to be completed. The following examples illustrate how to use many of the different features of the program. For more detailed information regarding the program, a user's manual can be found on the NRCS website at:

http://www.wsi.nrcs.usda.gov/products/W2Q/H&H/tools_models/WinTR20.html

Calculations

There are a few calculations that should be completed before modeling a watershed in WinTR-20. For a large watershed it may be necessary to divide the watershed into subbasins. Each subbasin is determined by finding the different outlet points or design points within the watershed, then finding the area contributing to those points.

1) **Area** - The area of each subbasin must be calculated in square miles (mi^2). This can be done with a digitizer, a planimeter, or simply by counting squares.

2) **Curve Number** - The Curve Number (CN) must be computed for each subbasin. As described in Chapter 3, the runoff curve number is a dimensionless number of 100 or less that relates runoff to the soil-cover complex of the watershed. Higher curve numbers mean greater runoff. The tables for calculating the curve number can be found in **Table 3.8**. A worksheet, from the TR-55 manual (NRCS, 1986), for calculating the curve number is provided (Appendix D) which multiplies a subbasin's area(s) times the curve number(s) for the respective subbasin in order to compute a weighted value.

3) **Time of Concentration** - Following the curve number, the time of concentration (t_c) is calculated for each subbasin. The time of concentration is defined as the time required for a hypothetical particle of water to flow hydraulically from the most distant point in the watershed to the outlet point or design point. The t_c is calculated using NRCS TR-55 methodology, which is explained in Appendix D. This appendix also contains blank worksheets used for computing the time of concentration. The methodology divides flows into three types: sheet flow, shallow concentrated flow, and channel flow. The travel time must be computed for each type of flow that is found. These times are then added together to find the total t_c.

8.1.3 - WinTR-20 Example Problems

A watershed located near Lafayette, Indiana is modeled to assess the watershed peak runoff rates for existing conditions. The first example determines the runoff from one subbasin. The second example routes the discharge from the subbasin through a culvert. Lastly, the third example incorporates a reach and additional subbasin to determine the peak discharge from the watershed at the outlet. The examples are successive and the delineation of the watershed is shown on Exhibit 8.2.1.

Example 8.1.1

1. Description

Referring to Exhibit 8.2.1, Subbasin 1 (SA1) is modeled to determine the peak runoff for present conditions using the 12-hour, 100-year rainfall depth. This area comprises the entire watershed that is on the west side of Interstate 65 before it passes under the Interstate. By this example, it is intended to explain the basic input for any Win TR-20 run.

2. Hydrological Input Data

The cumulative rainfall data is the Huff Point Second Quartile rainfall distribution. To find the 12-hour rainfall depth for the 100-year recurrence interval, Chen's method (Chapter 2) is consulted. The point rainfall depth is computed as 5.48 inches for the Lafayette area.

3. Calculations

> **Note**: All references made to Tables and Figures in the following calculations refer to the NRCS TR-55 tables and figures in Appendix D.

Area - The area of subbasin 1 is computed to be 461 acres or 0.72 mi^2, by the use of a digitizer.

Curve Number - In order to determine the curve number (CN) representing this subbasin, the existing soil group can be found from the SCS County Soils Map, or if it is presently developed use the corresponding CN. There are many different uses for this subbasin which are used to calculate a composite curve number. For example, 13.8 acres (3.0%) of the subbasin is commercial area. From Table 3.3.3 the curve number for commercial area on hydrologic soil group C is 94. Similarly, the other land uses are located on Tables 3.3.3 and 3.3.4 and their respective curve numbers are found. The total composite curve number and percentage of area is 8423, which corresponds to a composite CN of 84. Table 8.1.2 shows these calculations presented in a tabular format.

Table 8.1.1
NRCS TR-55 CN Calculations for Example 8.1.1

Runoff Curve Number

Project: Ross Ditch- EXAMPLE 8.1.1 By: _____ Date: 7/12/2006
Location: Tippecanoe County Checked: _____ Date: 7/12/2006

Circle One: (EXISTING) PROPOSED Description: SUBBASIN 1 (SA1)

Soil Name and Hydrologic Group (Appendix A)	Cover Description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	Curve Number Table 2-2	Curve Number Fig. 2-3	Curve Number Fig. 2-4	Area ☐ acres ☐ sq. mi. ☒ %	Product of Curve Number and Area
C	Row Crops, Good	82			61.7%	50.6
C	Commercial	94			3.0%	2.8
C	Brush, Fair	70			3.3%	2.3
C	Open Space, Good	74			2.6%	1.9
	Impervious Water	100			17.4%	17.4
	Impervious Roads	98			1.4%	1.4
B	Row Crops, Good	75			9.0%	6.8
B	Open Space, Good	61			0.4%	0.2
B	Residential, 1 acre	68			1.2%	0.8
				Totals =	100.0%	84.23

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{84.23}{1} = 84.23$

Use CN = **84**

Time of Concentration - Finding the path of a hypothetical particle of water to flow hydraulically from the most distant point of Area 1 to the point of discharge reveals 100 ft. of sheet flow and 2200 ft. of shallow concentrated flow over unpaved surface. Table 8.1.2 shows the calculations for the time of concentration using the NRCS TR-55 worksheet. The total time of concentration for subbasin 1 is 1.11 hrs.

Table 8.1.2
SCS TR-55 T_c Calculations for Example 8.1.1

Time of Concentration (T_c) or Travel Time (T_t)

Project: Ross Ditch- EXAMPLE 8.1.1
Subarea: Subbasin 1 By: _____ Date: _____
Location: Tippecanoe County Checked: _____ Date: _____
File: _____

Circle One: (EXISTING) PROPOSED Description: _____

SHEET FLOW

Segment ID		I					
Surface Description (table 3-1)		Cultiv.					
Manning's roughness coeff., n		0.17					
Flow Length, L (total L £ 100')	(ft)	100					
Two-yr 24-hr rainfall, P2	(in)	3.00					
Land slope, s	(ft/ft)	0.005					
$T_t = (0.007(nL)^{0.8})/(P_2^{0.5} s^{0.4})$	(hr)	0.32	+			=	0.32 hr

SHALLOW CONCENTRATED FLOW

Segment ID		II								
Surface Description (paved or unpaved)		unp								
Flow Length, L	(ft)	2200								
Watercourse slope, s	(ft/ft)	0.00225								
Average velocity, V	(ft/s)	0.77								
$T_t = L / 3600 V$	(hr)	0.79	+		+		+		=	0.79 hr

CHANNEL FLOW

Segment ID		III				
Cross-sectional flow area, a	(ft^2)					
Wetted perimeter, Pw	(ft)					
Hydraulic radius, r = a/Pw	(ft)					
Channel slope, s	(ft/ft)					
Manning's roughness coeff., n						
$V = (1.49 r^{0.667} s^{0.5}) / n$	(ft/s)					
Flow length, L	(ft)					
$T_t = L / 3600 V$	(hr)		+		=	hr

Watershed or subarea T_c or T_t = **1.11** hr

4. Computer Input

The following section demonstrates how to correctly input the data.

1) Once WinTR-20 is properly installed and started, you may select the FILE tab from the top and from the drop down menu please chose New WinTR-20 File. This action can be seen in Figure 8.1.1 (a) below.

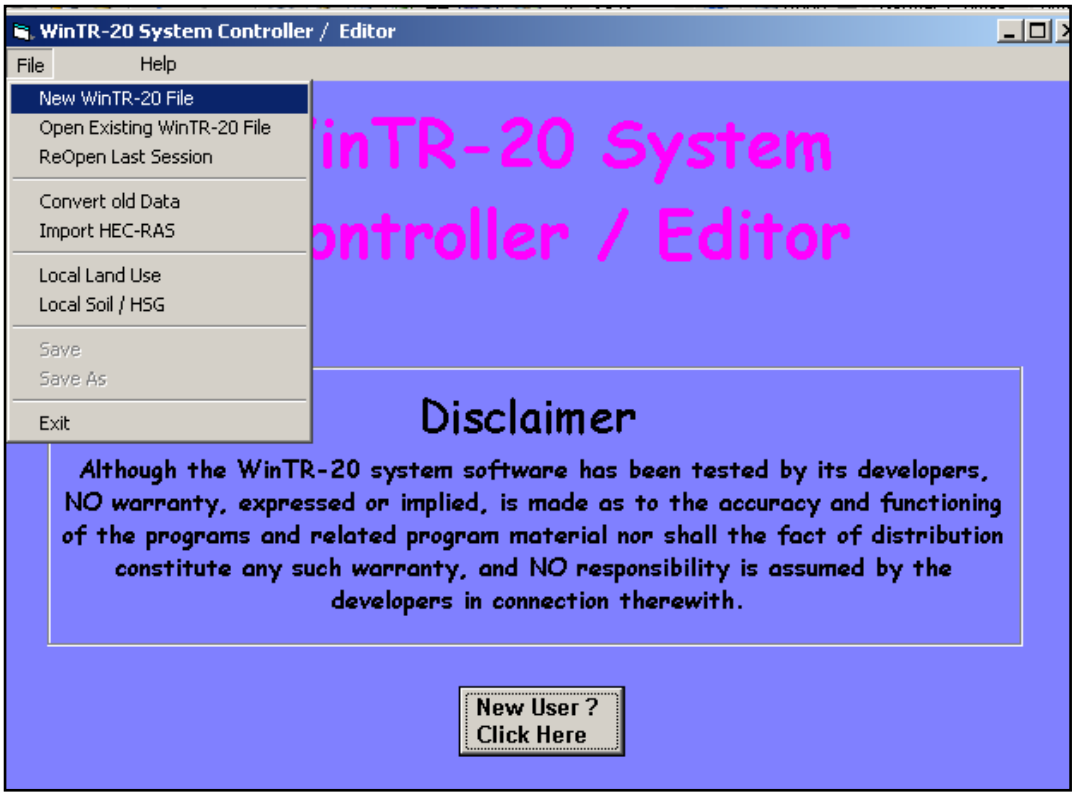


Figure 8.1.1 (a)
New File Creation in Win TR-20

2) The WinTR-20 Identifier Screen will appear once the selection detailed above is made. Either English or Metric units can be used, depending on what is appropriate for the specific project. The minimum hydrograph value can also be modified, the default value is 0.0. The watershed description is the last component to this screen. Once the proper selections have been completed, Accept Changes (Close) should be selected.



Figure 8.1.1 (b)
Project Description and Output Options

3) The Controller/Editor screen will now appear. The only Data Section Name that should have an X next to it is WinTR-20: Version 1.0. This X signifies that the WinTR-20 Identifier screen was completed. Although there are many data sections, it is not necessary to enter data for each section This example uses Duration Increment, Global Output, Rainfall Distribution, Storm Analysis, Structure Rating and Sub-Area. Please begin by selecting the Sub-Area Data Section.

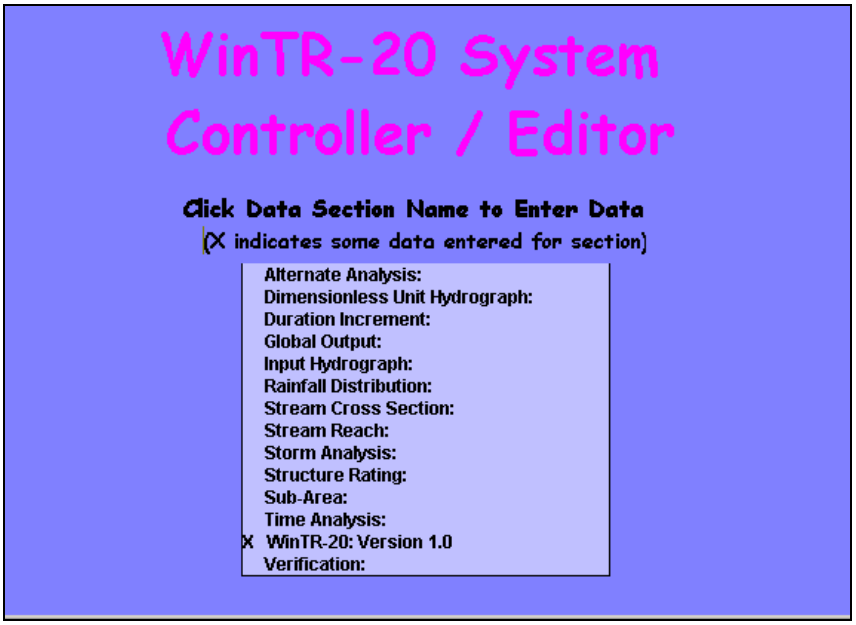


Figure 8.1.1 (c)
Win TR-20 Data Section Menu

4) The Sub-Area screen is used to input basic information about the watershed. If the

watershed is very large, it may be necessary to divide the area in to multiple sub-basins. For this example, we only have 1 subbasin and label it Area 1.OUTLET is entered in the Sub-Area Reach Identifier because the sub-area joins a stream system at the watershed outlet. The area in square miles, the weighted curve number and time of concentration are entered into the appropriate spaces. For this example, the CN and Tc previously calculated in Tables 8.1.1 and 8.1.2 were used. In order to complete this screen, select Accept Changes (Close). This action will bring the user back to the Controller/Editor screen. Please select the Duration Increment Section.

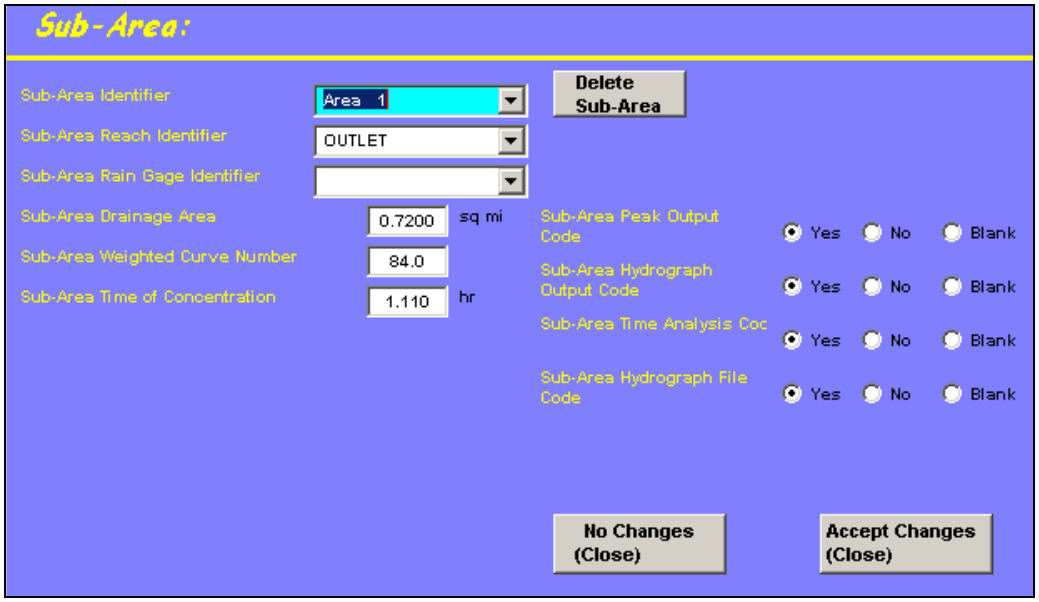


Figure 8.1.1 (d)
Subbasin Information Entry for Area 1

5) The Duration Increment screen allows the user to specify the durations required for the specific project. Again, when completed please select Accept Changes (Close). Now select the Rainfall Distribution section.

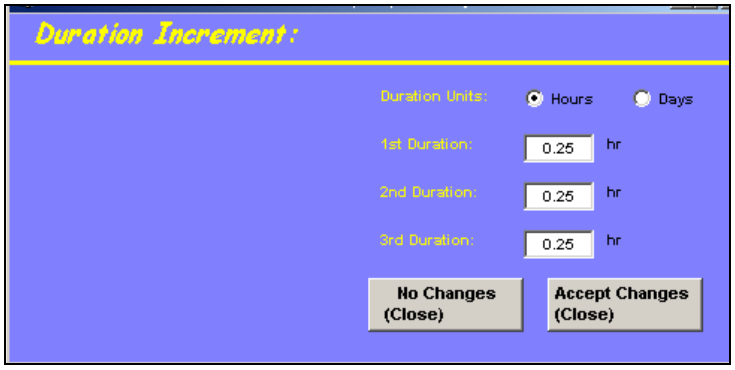


Figure 8.1.1 (e)
Duration Increment for Example 8.1.1

6) The Rainfall Distribution screen allows the user to enter information regarding an actual or theoretical storm event. Multiple rain tables may be entered. For the example, the Huff

2nd Quartile Distribution is used for the 12-hour duration storm event that is to be used in this example. The rainfall table identifier is a name assigned to the data to easily identify it later. There are rainfall distribution tables that are built-in to WinTR-20. A complete list of these distributions can be found within the program and in the user's manual. When the information has been properly entered select Accept Changes (close).

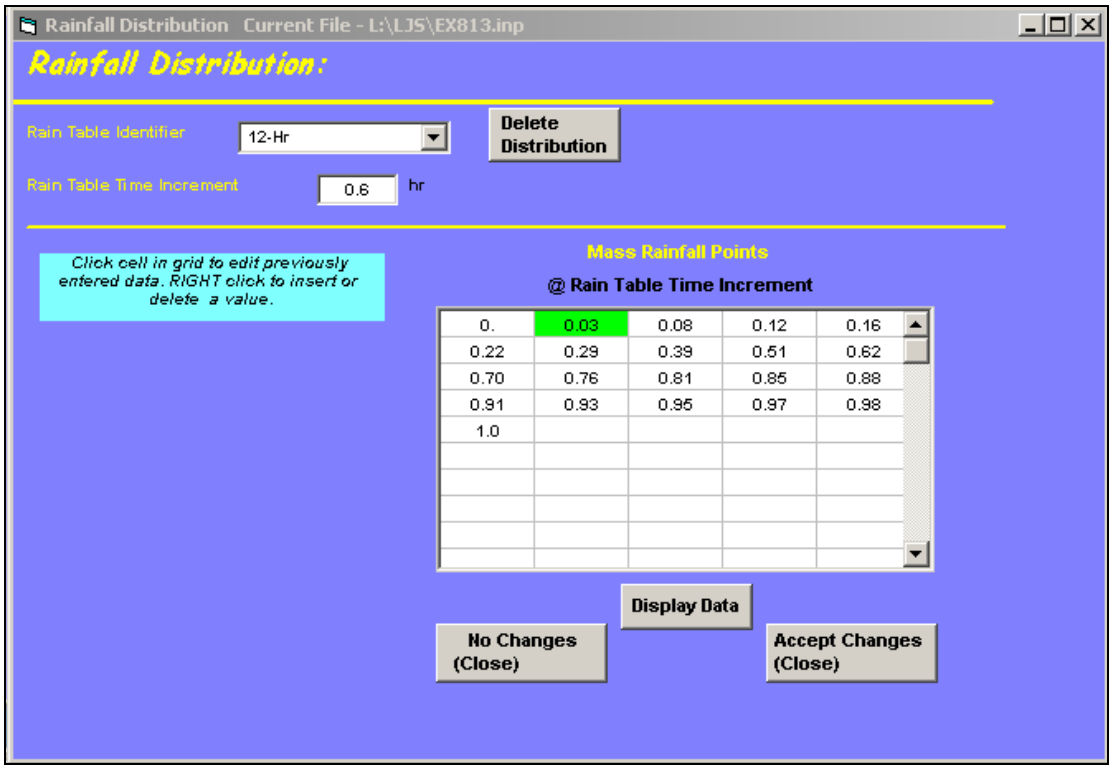


Figure 8.1.1 (f)
Huff 2nd Quartile Rainfall Distribution for 12-Hour Storm Event

7) The next data section to complete is the Storm Analysis section. WinTR-20 allows the user to enter multiple storms. This example only requires one storm and it has been labeled "100-YR,12HR." Since there is only 1 rain gage or one set of rain data being used, it is not necessary to complete the Rain Gage Identifier. Earlier in the calculations, it has been determined that the point rainfall depth for this location is 5.48 inches. Next, the proper Rainfall Table Identifier is chosen. The "12-Hr" table created in the previous step was selected. Finally the Antecedent Runoff Coefficient is chosen to be two because it assumed to be average conditions. Accept Changes (Close) is selected to exit the window.

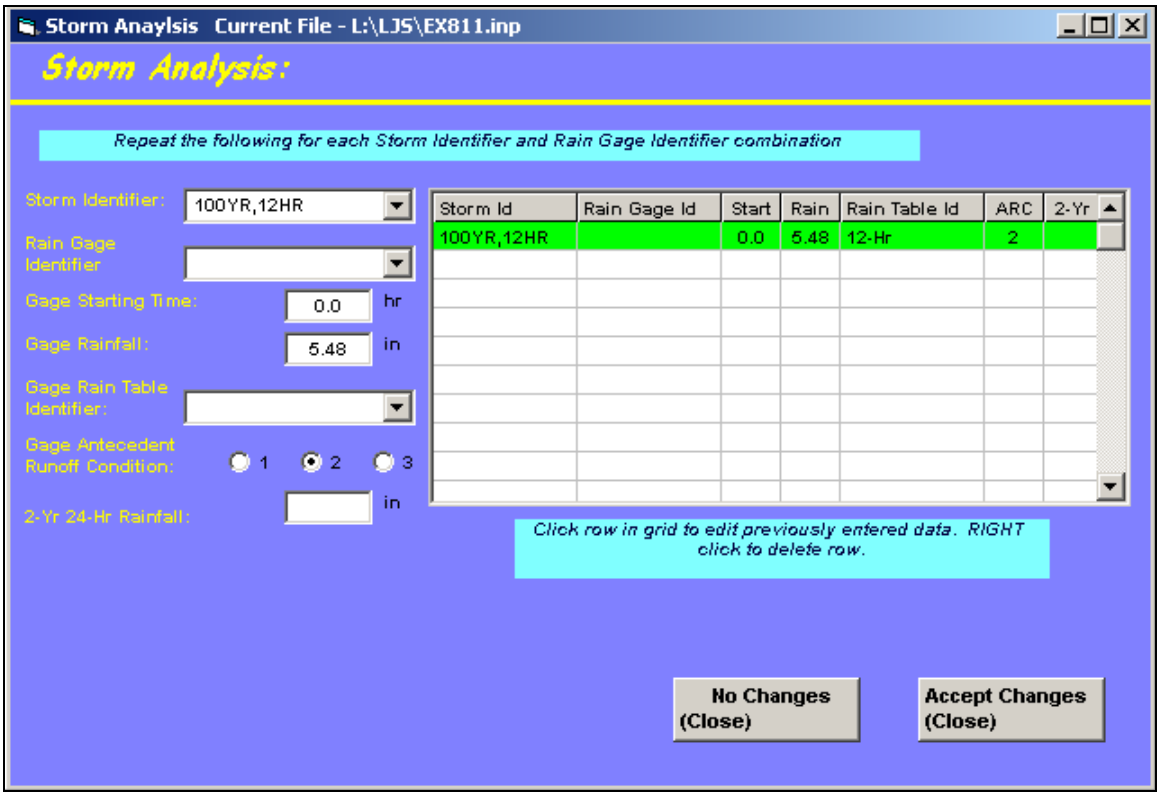


Figure 8.1.1 (g)
Rainfall Input for 100-Year 12-Hour Storm Event for Lafayette

8) Finally, the Global Output data section is selected. The Hydrograph Print Precision section is left blank, which is set to default to one decimal place. The Minimum Hydrograph Display Flow is the smallest hydrograph flow point, which is set at 0.01. The print time increment can also be selected. Also the final specifications for defaults have been set. Accept Changes (Close) has been selected to exit the window.

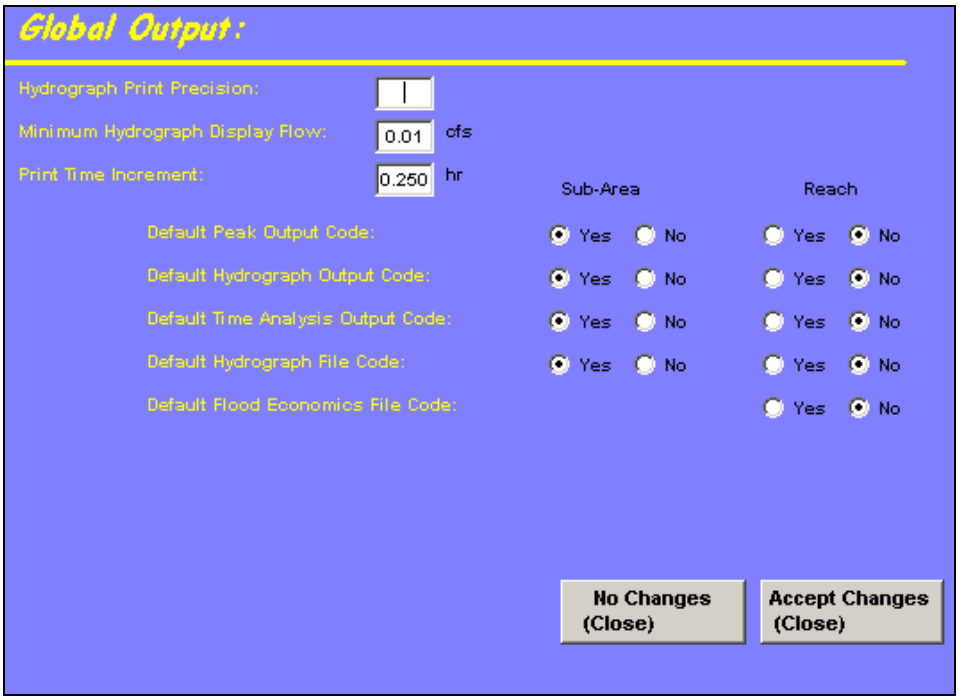


Figure 8.1.1 (h)
Output Control for Example 8.1.1

9) Now that the correct information has been entered, the project needs to be saved. To do this select File and pull down to Save. This action can be seen in the following image.

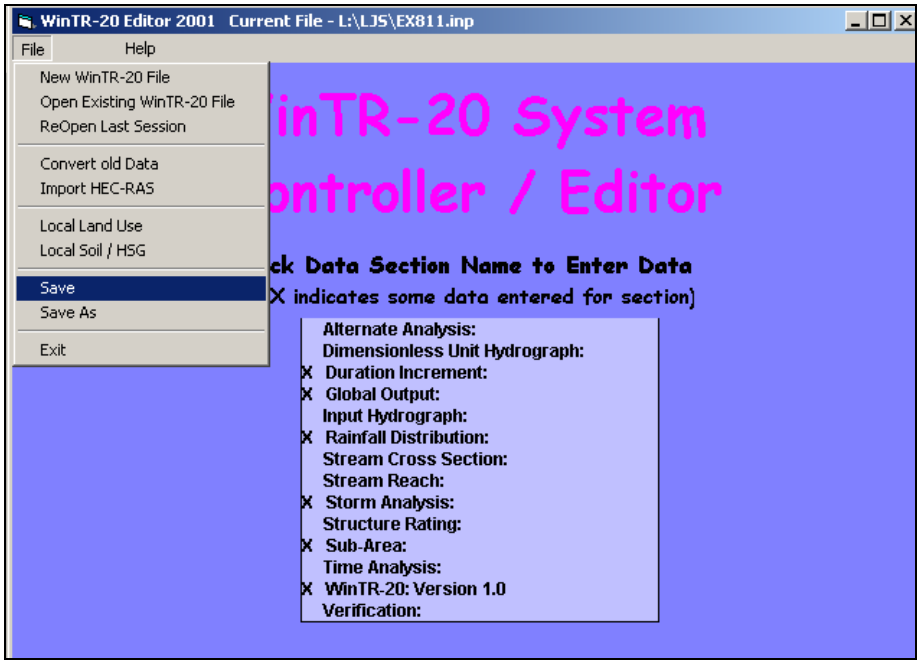


Figure 8.1.1 (i)
Completion of Required Input Data

10) Notice that when the model is saved, new tabs at the top appear, specifically the Run tab.

Please select the run tab and a new window will appear with desired output.

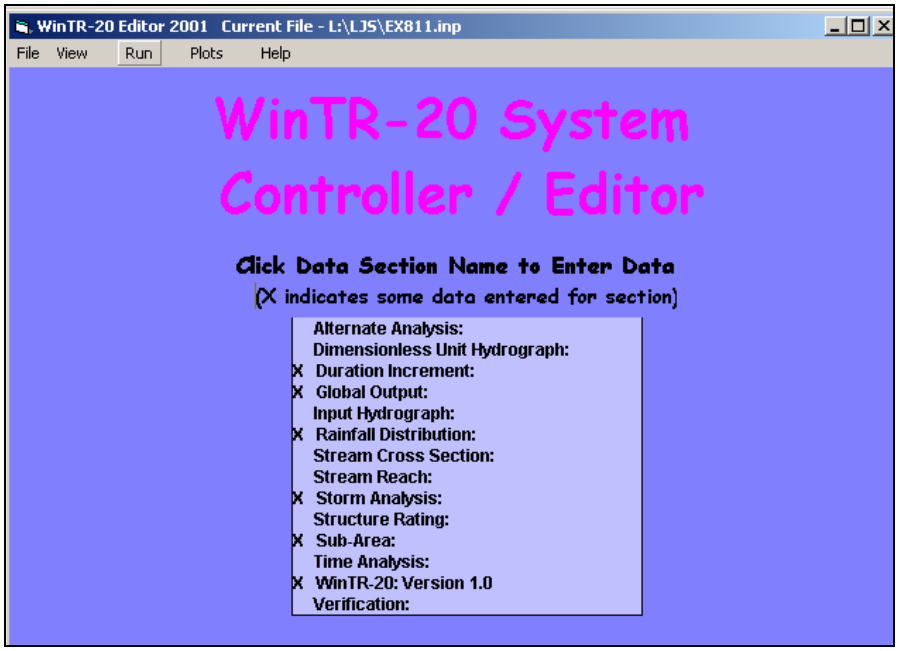


Figure 8.1.1 (j)
The Run Tab Appears When Model is Saved

5. Output Summary/Explanation

The output from this model can be seen in Figure 8.1.2. From the following output, it can be seen that for a 12-hour duration rainfall event with a 100-year recurrence interval, the amount of excess rainfall from the subbasin is approximately 3.712 inches. The peak discharge occurs after 5.63 hours and has a flow rate of 372.73 cfs.

EXAMPLE 8.1.1-ROSS DITCH WATERSHED-TIPPECANOE COUNTY
EXISTING CONDITIONS-SUBBASIN 1

Name of printed page file:
L:\LJS\EX811.out

STORM 100YR,12HR

Area or Reach Identifier	Drainage Area (sq mi)	Rain Gage ID or Location	Runoff Amount (in)	Peak Flow Elevation (ft)	Peak Flow Time (hr)	Peak Flow Rate (cfs)	Peak Flow Rate (csm)
Area 1	0.720		3.712		5.63	372.73	517.68
OUTLET	0.720		3.712		5.63	372.73	517.68

Figure 8.1.2
Win TR-20 Output for Example 8.1.1

Example 8.1.2

1. Description

The runoff from Subbasin 1 flows under Interstate 65. However, because the culverts are small relative to the flow, there is ponding of runoff. To represent this, the runoff from Subbasin 1 is routed through twin culverts under Interstate 65. A stage-storage-discharge relationship is required for the structure, and the attenuation effects are evaluated. Table 8.1.3 shows the stage--discharge relationship obtained from the HY-8 example in Chapter 4 (Example 4.5.1). The storage is determined from digitizing the existing contours of the subbasin and interpolating.

Table 8.1.3
Stage-Storage-Discharge Relationship for Culverts and Pond in Example 8.1.2

Stage (feet)	*Discharge (cfs)*	*Storage (acre-ft)*
654.17	0	0.0
654.75	5	98.1
655.08	10	117.2
656.10	15	195.6
656.16	20	201.5
656.29	30	214.2
656.43	40	228.0
656.59	50	243.7

2. Computer Input

Since the watershed information and storm event is the same, the previous file will be used to begin.

1) The file from example 1 is opened by selecting File, Open Existing WinTR-20 File. In order to avoid confusion, the Watershed Description is changed. In order to close this window select Accept Changes (Close).

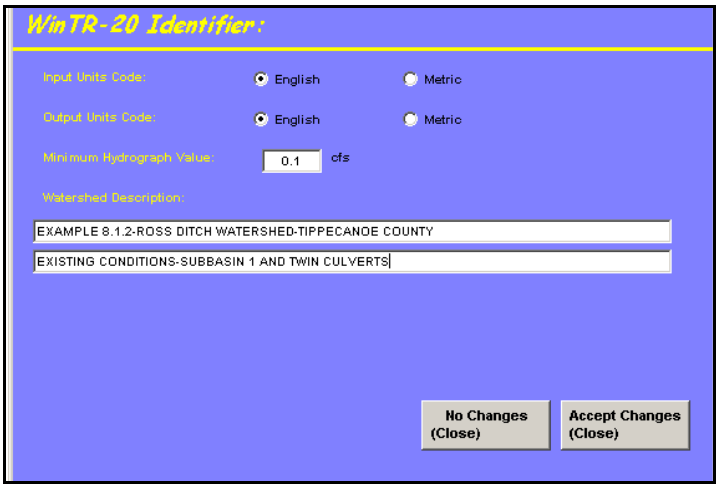


Figure 8.1.3 (a)
Change Description From Previous Example

2) The following screen will appear. Now it is necessary to create a new file for this example. Go to File, Save As and save the example in the desired location with a new file name to differentiate it from the first example.

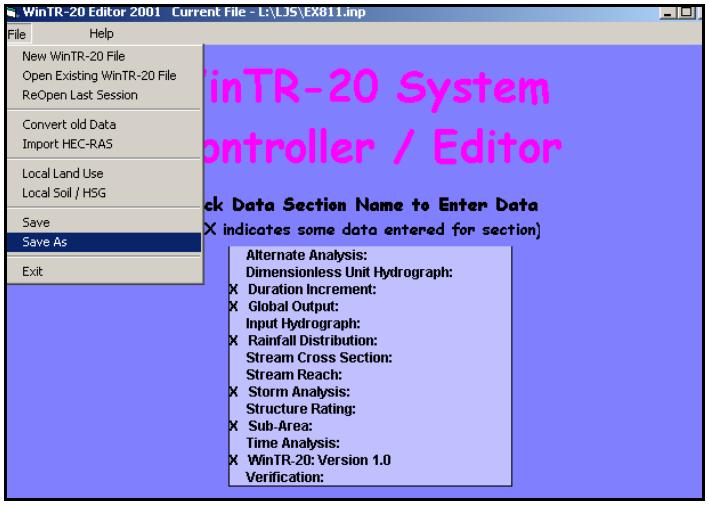


Figure 8.1.3 (b)
Save and Rename as Example 8.1.2

3) In order to model reservoir routing, a structure will be added by selecting the Structure Rating data section. Using the information provided, the staged-storage-discharge relationship can be entered into the data section. The starting elevation is the elevation at zero discharge and zero storage.

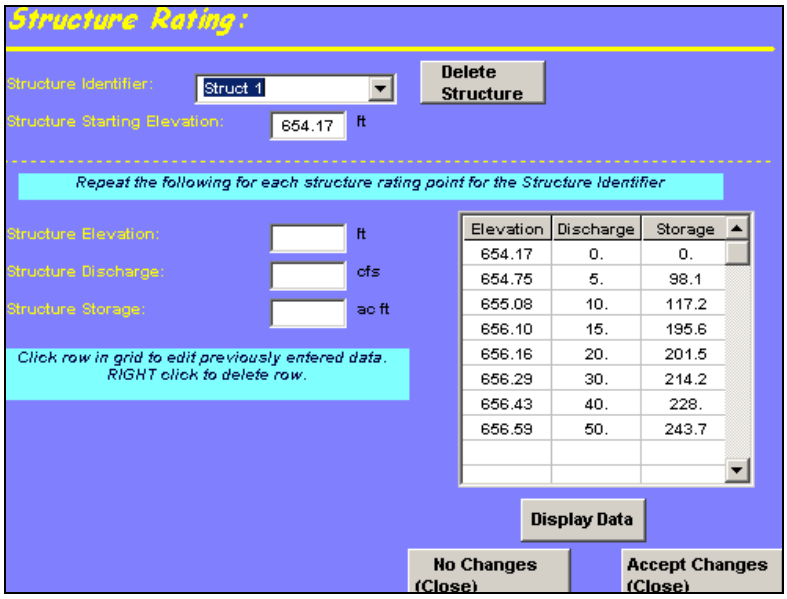


Figure 8.1.3 (c)
Stage-Storage-Discharge Relationship for Structure 1

4) Now the Stream Reach data section is selected. The reach associated with the structure routing is given an identifier and the Stream Receiving Reach Identifier is listed as the OUTLET.

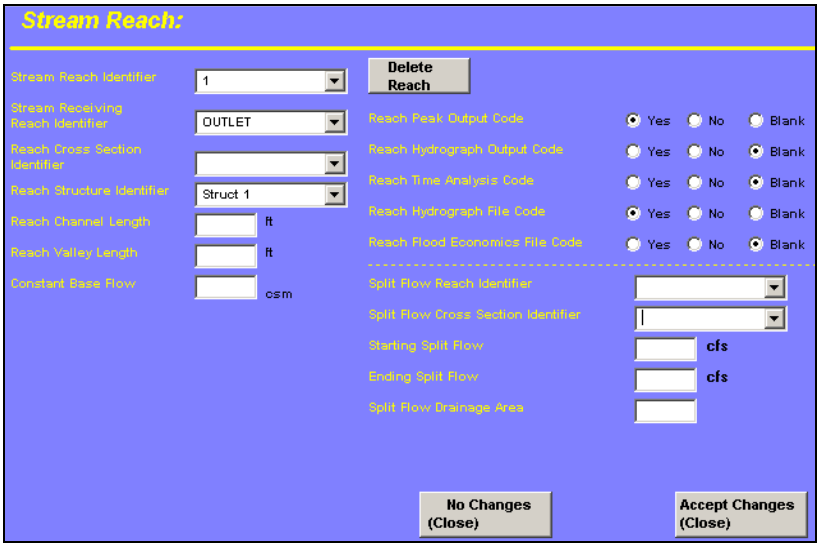


Figure 8.1.3 (d)
Assign the Structure Routing to the Reach

3. Output Summary/Explanation

As shown in Figure 8.1.4, the output for this example shows the peak runoff rate for Subbasin 1 and the peak flow and elevation through the culverts. The culvert elevation is checked to determine if the size is sufficient to avoid overtopping. The peak discharge elevation is 655.33 feet for the 100-year storm. This indicates that that the flow does not overtop I-65 for the storm

event modeled.

Table 8.1.4
Summary of Example 8.1.2

LOCATION	*PEAK DISCHARGE (cfs)*	*RUNOFF AMOUNT (inches)*	*WATER SURFACE ELEVATION (feet)*	*TIME TO PEAK (hrs)*
Subbasin 1	372.73	3.712	-	5.63
Culverts	11.24	3.707	655.33	13.27

EXAMPLE 8.1.2-ROSS DITCH WATERSHED-TIPPECANOE COUNTY
EXISTING CONDITIONS-SUBBASIN 1 AND TWIN CULVERTS

Name of printed page file:
L:\LJS\EX812.out

STORM 100YR,12HR

Area or Reach Identifier	Drainage Area (sq mi)	Rain Gage ID or Location	Runoff Amount (in)	Peak Flow Elevation (ft)	Peak Flow Time (hr)	Peak Flow Rate (cfs)	Peak Flow Rate (csm)
Area 1	0.720		3.712		5.63	372.73	517.68
1	0.720	Upstream	3.712		5.63	372.73	517.68
1	0.720	Downstream	3.707	655.33	13.27	11.24	15.61
OUTLET	0.720		3.707		13.27	11.24	15.61

Figure 8.1.4
WinTR-20 Output for Example 8.1.2

Example 8.1.3

1. Description

The entire watershed is analyzed to determine the peak runoff from the 0.87 square miles. This example incorporates the area from Example 8.1.2 and routes the outflow from the culverts through a ditch and adds the runoff from subbasin 2 (SA 2), as shown on Exhibit 8.1.

The hydrologic input data from the previous examples is used. This refers to the rainfall data for the Huff Second Quartile rainfall distribution and the 5.48 rainfall depth corresponding to a 12-hour 100-year recurrence interval in Lafayette. In addition, the 12-hour 10-year and 24-hour 10 and 100-year rainfall events are computed to demonstrate the simplicity of adding additional events.

2. Calculations

The calculations for subbasin 2 are performed in the same manner as subbasin 1 was calculated.

The area is digitized and the curve numbers are computed using the appropriate land use. The time-of-concentration is calculated using SCS TR-55 methodology. Tables 8.1.5 and 8.1.6 contain the information for the computation of the CN and t_c, respectively.

Table 8.1.5
Curve Number Calculation for Subbasin 2

Drainage Area	*Area (%)*	*Land Use (Hydrologic Group)*	*CN*	*Area * CN*
2	38.0	Row Crop, Good (B)	75	2850.0
	5.3	Open Space, Good (B)	61	323.3
	1.4	Woods, Good (B)	55	77.0
	1.7	Brush, Good (B)	48	81.6
	8.0	Impervious, Roads	98	784.0
	26.0	Row Crop, Good (C)	82	2132.0
	14.2	Open Space, Good (C)	74	1050.8
	3.4	Woods, Good (C)	70	238.0
	2.0	Brush, Good (C)	65	130.0
TOTAL	100			7666.7
			Composite Curve Number	77

Table 8.1.6
SCS-TR55 t_c Calculations for Subbasin 2

Time of Concentration (T_c) or Travel Time (T_t)

Project: Ross Ditch- EXAMPLE 8.1.1
Subarea: Subbasin 1 By: _____ Date: _____
Location: Tippecanoe County Checked: _____ Date: _____
File: _____

Circle One: (EXISTING) PROPOSED Description: _____

SHEET FLOW

Segment ID		I					
Surface Description (table 3-1)		Woods					
Manning's roughness coeff., n		0.4					
Flow Length, L (total L £ 100')	(ft)	100					
Two-yr 24-hr rainfall, P2	(in)	3.00					
Land slope, s	(ft/ft)	0.01					
$T_t = (0.007(nL)^{0.8})/(P_2^{0.5}\, s^{0.4})$	(hr)	0.49	+			=	0.49 hr

SHALLOW CONCENTRATED FLOW

Segment ID		II								
Surface Description (paved or unpaved)		unp								
Flow Length, L	(ft)	2600								
Watercourse slope, s	(ft/ft)	0.008								
Average velocity, V	(ft/s)	1.44								
$T_t = L / 3600\ V$	(hr)	0.50	+		+		+		=	0.50 hr

CHANNEL FLOW

Segment ID		III				
Cross-sectional flow area, a	(ft^2)					
Wetted perimeter, Pw	(ft)					
Hydraulic radius, r = a/Pw	(ft)					
Channel slope, s	(ft/ft)					
Manning's roughness coeff., n						
$V = (1.49\ r^{0.667}\ s^{0.5}) / n$	(ft/s)					
Flow length, L	(ft)					
$T_t = L / 3600\ V$	(hr)		+		=	hr

Watershed or subarea T_c or T_t = 0.99 hr

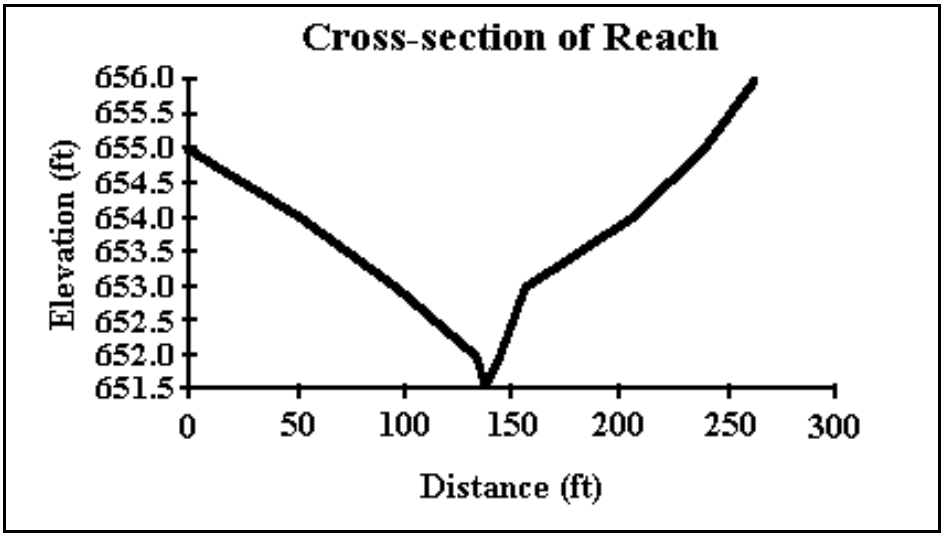


Figure 8.1.5
Cross-section of Reach through Subbasin 2

3. Computer Input

Since this example builds further on the previous examples, follow the same renaming and Save As procedure as in Example 8.1.2.

1) In order to add the second sub-basin, select the Sub-Area data section to begin.

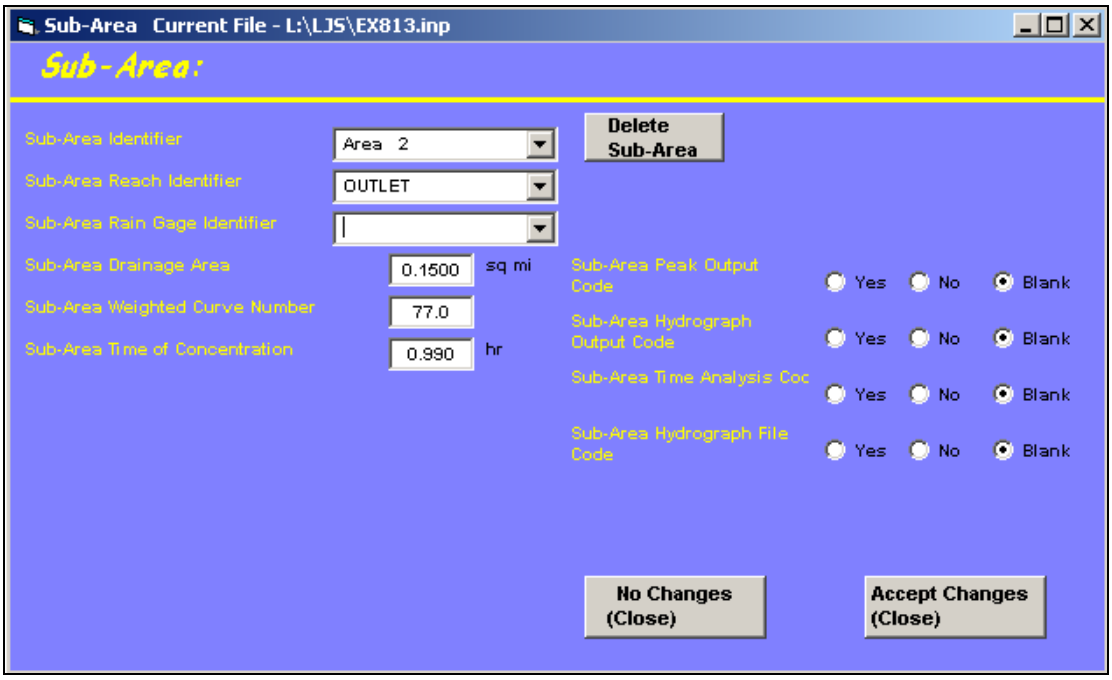


Figure 8.1.6 (a)
Subbasin Information Entry for Area 2

2) Next, under the Stream Cross Section data section, enter the reach information for the 1500-foot long reach described in Figure 8.1.5.

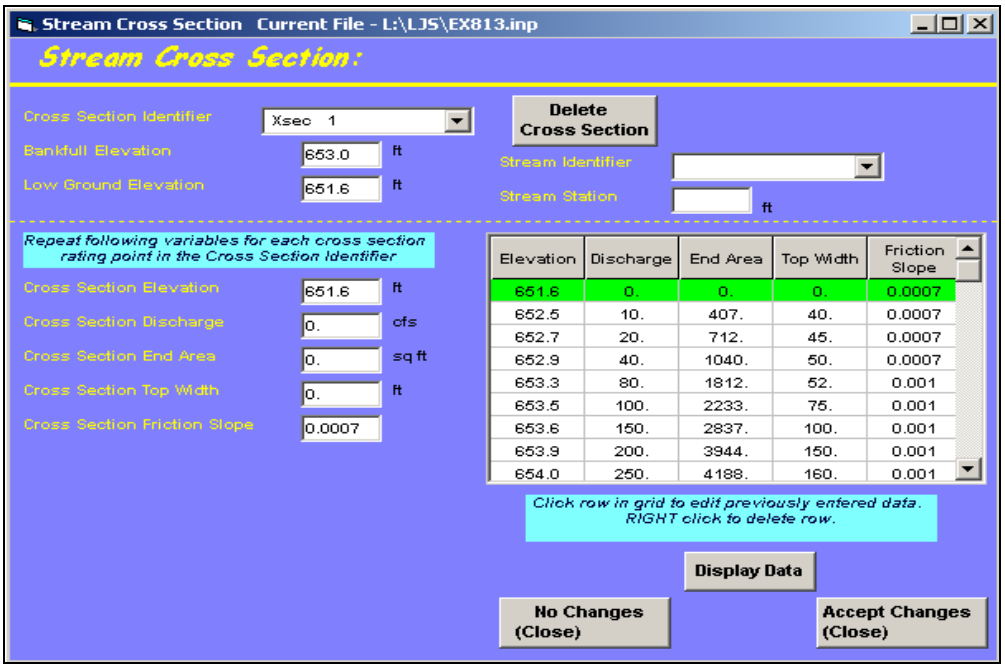


Figure 8.1.6 (b)
Reach Information for Reach in Figure 8.1.5

3) Next, identify a new stream reach for the reach route. Note that the receiving stream, OUTLET, is the same for Subbasin 2 and Reach 2.

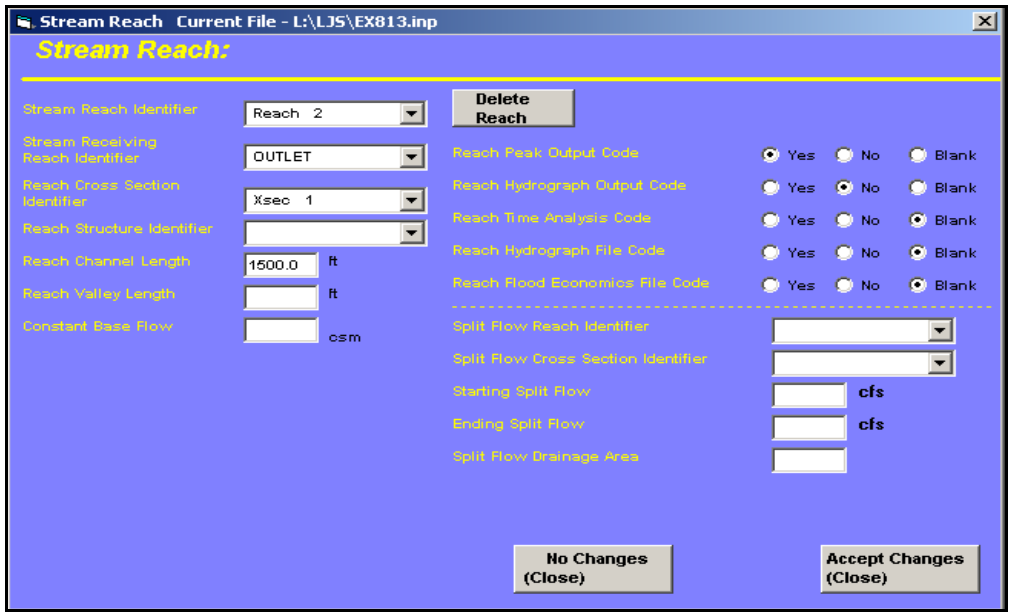


Figure 8.1.6 (c)
Assign Reach Route to Stream Reach

1. Output Summary/Explanation

Now that the necessary information has been entered the example is run similar to the

previous example. Save the model and then select Run.

STORM 100YR,12HR

Area or Reach Identifier	Drainage Area (sq mi)	Rain Gage ID or Location	Runoff Amount (in)	Peak Flow Elevation (ft)	Peak Flow Time (hr)	Peak Flow Rate (cfs)	Peak Flow Rate (csm)
Area 1	0.720		3.712		5.63	372.73	517.68
Reach 1	0.720	Upstream	3.712		5.63	372.73	517.68
Reach 1	0.720	Downstream	3.707	655.33	13.27	11.24	15.61
Reach 2	0.720	Upstream	3.707	652.52	13.27	11.24	15.61
Reach 2	0.720	Downstream	3.707	652.42	13.83	9.08	12.61
Area 2	0.150		3.029		5.64	65.21	434.76
OUTLET	0.870		3.590		5.70	67.30	77.35

Figure 8.1.7
Win TR-20 Output for Example 8.1.3

Table 8.1.7
Summary of Results for Example 8.1.3 for the 12-Hour 100-Year Event

Area	*Area (mi^2)*	*Time of Peak (hours)*	*Peak Discharge*
Subbasin 1	0.72	5.63	372.73
Culverts (outlet)	---	13.27	11.24
Reach	---	13.83	9.08
Subbasin 2	0.15	5.64	65.21
Outlet	0.87	5.70	67.30

8.2 - HEC-HMS PROGRAM

8.2.1 - Explanation of HEC-HMS and its Uses

HEC-HMS is a flood hydrograph package developed by the U.S. Army Corps of Engineers, Hydrologic Engineering Center (HEC), which is used to estimate runoff from precipitation data. The size and complexity of the basins simulated vary from small urban basins to large river systems. HEC-HMS can be used to (Bedient & Huber, 1988):

1. Estimate unit hydrographs, loss rates, and streamflow routing parameters from measured data;
2. Simulate streamflow from historical or design rainfall data;
3. Compute damage frequency curves and expected annual damages for various locations and multiple flood control plans;
4. Simulate reservoir outflows for dam safety analysis.

In addition, the program has several other features which may be used to compute snowmelt simulation, parameter estimation, dam safety analysis, pumping and diversion schemes, multiple-flood and multiple-plan analyses, and simulation of precipitation depth-area relationships.

In this section, the program is applied to several cases with emphasis on a couple of the features. Further information on the capabilities of HEC-HMS the manual and software can be obtained by writing to: The Hydrologic Engineering Center, U.S. Army Cops of Engineers, Attention: Training Division, 609 Second Street, Davis, CA 95616. A free copy of the HEC-HMS software and user's manual is available on the U.S. Army Corps of Engineers website at: http://www.hec.usace.army.mil/software/hec-hms/.

8.2.2 - Summary of HEC-HMS Input Structure

When developing a precipitation-runoff model the boundaries of the basin must be identified initially. Most often the basin is subdivided into smaller subbasins depending on the study objectives, drainage pattern and other factors. It is important to identify points where runoff information is needed. The model can be structured to produce hydrographs at any desired location. As different areas of a large basin may have different hydrologic response characteristics, it is important to select an appropriate computational time interval and subdivide the area so that lumped parameters provide a reasonable depiction of the watershed.

There are several methods in HEC-HMS to compute surface runoff. These are: (1) the unit hydrograph input directly, (2) the Clark or the Synder synthetic unit hydrographs, (3) SCS method (curve number method and SCS unit hydrograph), and (4) kinematic wave for overland hydrograph. Losses may be computed by using one of the four methods shown in Table 8.2.1.

Table 8.2.1
Estimation of Losses in HEC-HMS (Bedient & Huber, 1988)

METHOD	*DESCRIPTION*
Initial and Constant	Initial loss volume is satisfied and then constant loss rate begins.
HEC exponential	Loss function is related to antecedent moisture condition and is a continuous function of soil wetness.
SCS curve number	Initial loss is satisfied before calculating cumulative runoff as a function of cumulative rainfall.
Green and Ampt	Infiltration rate is computed as a function of soil infiltration capacity governed by Richards' equation.

Flood routing is used to simulate flood wave movement through river reaches and reservoirs. Most of the flood routing procedures are based on the continuity equation and some relationship between flow and storage. Some of the methods available in HEC-HMS are: (1) Muskingum-storage coefficient plus travel time through each reach, (2) normal-depth channel routing - Manning's equation is used to compute a table of storage versus outflow values for use in Modified Puls routing (described below), (3) Kinematic wave - outflow from each reach based on depth of flow in continuity and Manning's equations. There are other methods, not discussed here, which deviate slightly from these principle methods.

The modified Puls method, also known as Storage-Indication, utilized to route reservoirs, consists of a repetitive solution of the continuity equation (Equation 6.2.2) on the basis of assumptions that the reservoir water surface remains horizontal and that outflow is a unique function of storage. The continuity equation can be rearranged to isolate the two unknowns for the first time step, O_2 and S_2.

$$\frac{S_2}{\Delta t} + \frac{O_2}{2} = \left(\frac{S_1}{\Delta t} + \frac{O_1}{2} \right) - O_1 + \frac{I_1 + I_2}{2} \qquad (8.2.1)$$

Since I is known for all time steps, and O_1 and S_1 are known for the first time step, the right hand side of Equation 8.2.1 can be determined. Values of O_2 and S_2 are then used as input on the left-hand side, and the computation is repeated for the next time interval, and so on. A storage-indications curve is a plot of $[(S/\Delta t) + (O/2)]$ vs. O.

The procedure for applying the modified Puls method is as follows (Hoggan, 1989):

1. Determine a discharge rating curve for the reservoir outlet.
2. Determine reservoir storage that correlates with each elevation on the rating

curve for reservoir outflow.

3. Construct a storage indication vs. outflow curve: $[(S/\Delta t) + (O/2)]$ vs. O.
4. Route the inflow hydrograph through the reservoir on the basis of Equation 8.2.1.
5. Verify the results with data from historical events.

When the modified Puls method is used in HEC-HMS to do reservoir routing, the storage and outflow values are required input data. Storage-outflow input data may be specified in two ways: (1) as precomputed storage vs. elevation or discharge data and (2) as water surface area vs. elevation data.

A "cascade of reservoirs" may be used to depict storage routing in a river. The river is divided into a series of routing reaches represented by reservoirs, with the outflow from the first reservoir being the inflow to the second, the outflow from the second being the inflow to the third, and so on. Caution must be used when applying the modified Puls method to channel routing. The degree of attenuation introduced in the routed flood wave varies depending on the river reach lengths chosen, or alternatively, on the number of routing steps specified for a single reach.

When this method is applied to a reach, a hydrograph at an upstream location is routed to a downstream location. The storage is the volume in the channel under the water surface profile, and the outflow is the discharge in the channel at the downstream end of the reach. A storage-outflow relationship must be defined which is similar to that for reservoir routing, but the approach is different. In river reaches, storage-outflow relationships are determined from one of the following: (1) steady-flow profile computations, (2) observed profiles, (3) normal-depth calculations, (4) storage calculations based on inflow and outflow hydrographs, and (5) optimization techniques applied to inflow and outflow hydrographs.

1. HEC-HMS Model Structure

The project requirements for a HEC-HMS model can be broken down into the following:

- Meteorological Model (Precipitation, snowmelt, and evapotranspiration)
- Basin Model (Hydrologic elements interconnected to represent watershed)
- Control Specifications (Start time, end time, time interval)

Meteorological Model

The primary function of the meteorological model is to specify the precipitation events to be simulated by the basin model. HEC-HMS also allows the user to specify snowmelt and evapotranspiration data. There are several precipitation methods available in HEC-HMS, including:

- User-specified Hyetograph
- User-specified Gage Weights
- Inverse Distance Weighting

- Grid-Based Precipitation
- Frequency Based Design Storm
- Standard Project Storm (Eastern U.S.)
- SCS Hypothetical Storm
- No Precipitation

Basin Model

The Basin Model contains the physical elements of a watershed, such as: subbasins, reservoirs, reaches, pump stations, diversions, sinks, and dams.

Control Specifications

The Control Specifications for each model are mainly the start and end time for each simulation. The time increment for each simulation is also specified under Control Specifications.

8.2.3 - Example Problems

Example 8.2.1

1. Description

The runoff from Subbasin 1 (SA1) of the watershed shown on Exhibit 8.2.1 is estimated using the SCS method in HEC-HMS. The area of the subbasin is 461 acres or 0.72 sq. mi. The curve number and the time of concentration for this area are determined to be 84 and 1.11 hours, respectively. The precipitation data for the subbasin is a 12-hour 100-year event. From Chapter 2, for a 100-year 12-hour event for the Lafayette area, the rainfall depth is 5.48 inches. This is distributed over the subbasin incrementally.

2. Computer Input

Project Definition

From the main (PROJECT DEFINITION) screen, select **New Project** from **File** on the menu bar. This will open the NEW PROJECT screen. This project will be named "Example 8.2.1." This creates a new folder with the project name in the directory chosen that will store all of the project files. External files such as HEC-DSS files and background maps do not have to be stored in the project folder. Select U.S. Customary Units. Press the **Create** button to set up the new project and return to the PROJECT DEFINITION screen. To set the project defaults, select **Project Options** from the **Tools** on the menu bar. Under the **Loss** tab, select **SCS Curve Number**, under the **Transform** tab, select **SCS Unit Hydrograph**, under the **Baseflow** tab, select **No Baseflow**, under the **Routing** tab, select **Modified Puls**, under the **Precipitation** tab, select **Specified Hyetograph**, under **Evapotranspiration**, select **None** and under **Snowmelt**, select **None**.

Finally, inspect the various **Program Settings** menu under the **Tools** tab on the menu bar. This will set various items for the HEC-HMS program that will remain constant when you open the program. Also, open the **Help** screen to examine its structure and contents. Don't forget to save your project before closing this screen or exiting the program.

Hydrologic Schematic

To create the basin model, select **Components – Basin Model Manager** from the menu bar. From the Basin Model Manager window, click the **New** button. Enter the basin name "**Basin 1**" and click the Create button. Close the Basin Model Manager window and the new watershed appears in the *Watershed Explorer* window.

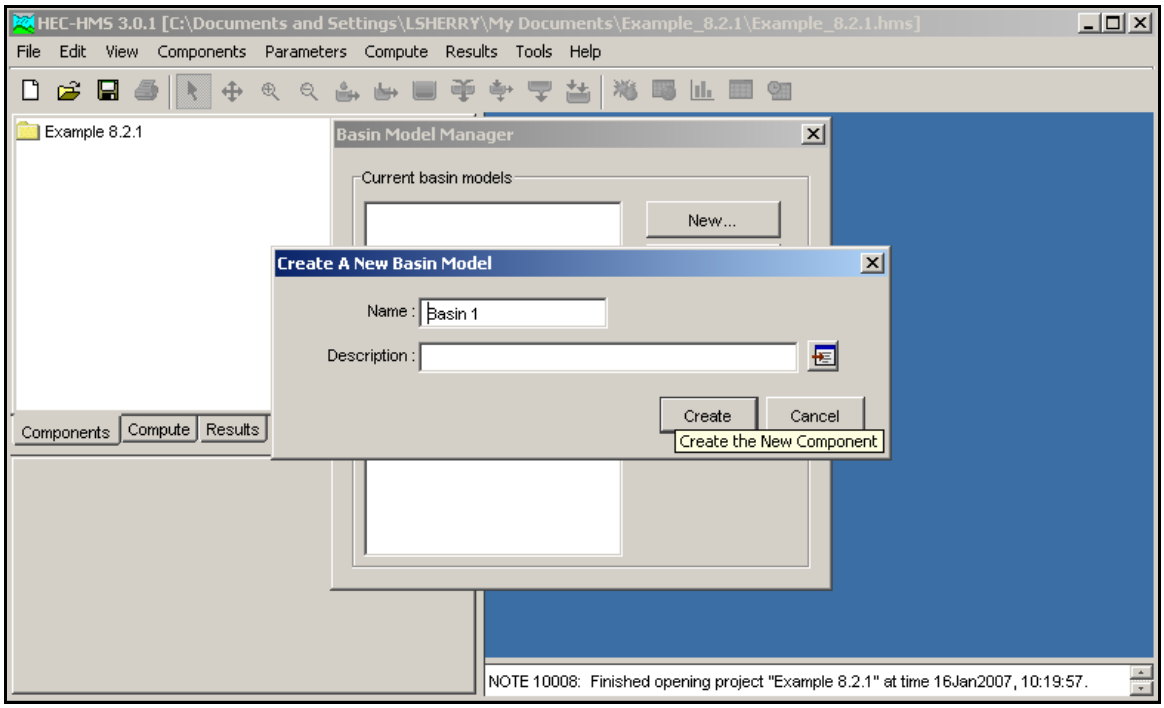


Figure 8.2.1 (a)
Creation of New Basin Model

Background maps may be placed on the desktop to provide context for the hydrologic elements that make up your basin. Background maps are optional because they do not interact with the program in any way. To build the model's schematic, open the basin model by double clicking on the basin model 'Basin 1" in the *Watershed Explorer* window. Using the tool bar above the Desktop, the basin components can be added to the model. This includes adding all basin components, such as subbasins, reaches, reservoirs, and junctions. As basin components are added to the map, they appear in the *Watershed Explorer* window. Once all of the basin components are added to the map, connections must be made between components to establish the direction of flow.

The basin model in this example (Basin 1) consists of only one subbasin. From the tool bar above the desktop, click on the Subbasin Creation Tool and then click on the watershed map. Name the subbasin, "Subbasin 1." To add the characteristics of the subbasin that are described in this problem, click on the plus sign next to Subbasin 1 in the *Watershed Explorer* window. As shown in Figure 8.2.1(b) below, under the Subbasin tab, the subbasin area (0.72 square miles) can be entered here. Also shown are the various methods that were set up previously using the Project Options tool. Under the **Loss** tab, the curve number of 84 for Subbasin 1 is entered. Under the **Transform** tab, the lag time of Subbasin is entered. Because the lag time is 0.6 times the time of concentration (1.11 hours), the lag time for Subbasin 1 is approximately 40 minutes.

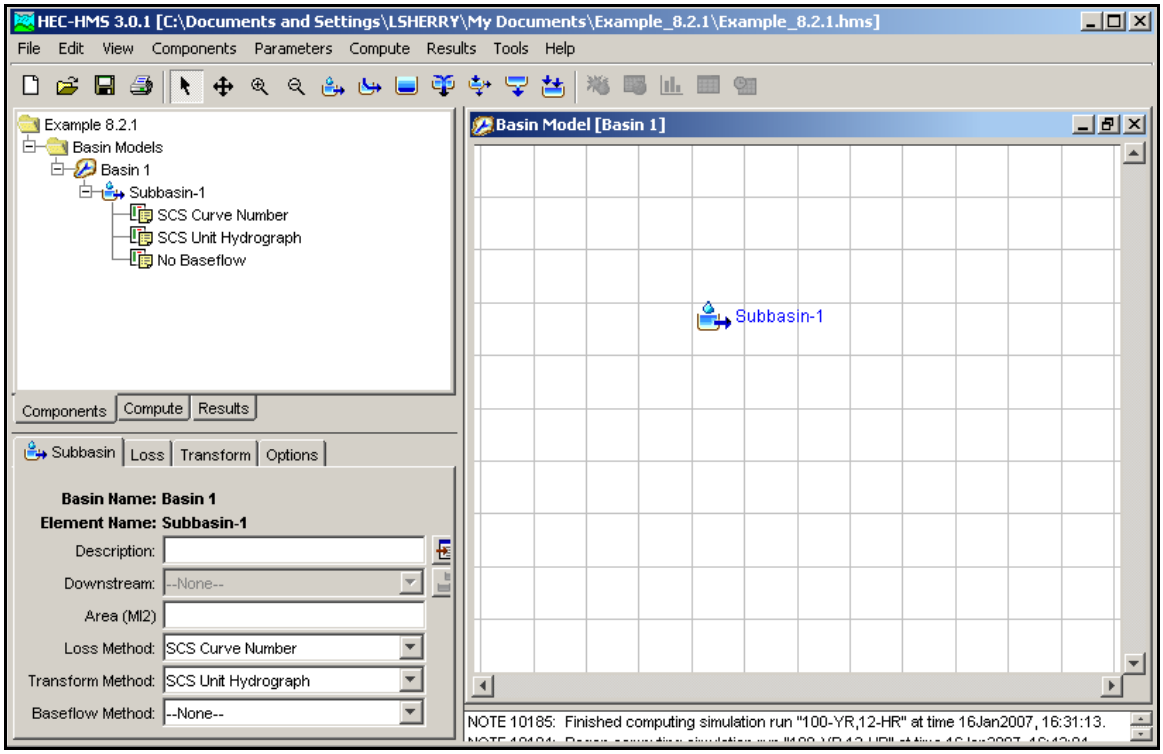


Figure 8.2.1 (b)
Adding Subbasin 1 to the Basin Model

Precipitation

Example 8.2.1 specifies that Subbasin 1 be analyzed for the 100-year, 12-hour storm event for Lafayette. The Huff 2nd Quartile distribution for the 100-year 12-hour storm event is tabulated below. Note that the Huff Distribution has been converted to 0.5-hour intervals. The current version of HEC-HMS does not allow input of rainfall data by percent cumulative storm duration, which is how the Huff Distributions have been tabulated. Rainfall data must be input at specified time increments. Therefore, some calculations may be required to convert the Huff Distributions of certain duration storm events into time increments. This has been done for the 100-year 12-hour storm event in Table 8.2.2.

Hour	Cumulative Rainfall Distribution
0	0.000
0.5	0.025
1	0.063
1.5	0.100
2	0.133
2.5	0.170
3	0.220
3.5	0.278
4	0.357
4.5	0.450
5	0.547
5.5	0.633
6	0.700
6.5	0.750
7	0.793
7.5	0.830
8	0.860
8.5	0.885
9	0.910
9.5	0.927
10	0.943
10.5	0.960
11	0.973
11.5	0.983
12	1.000

Table 8.2.2
Huff 2nd Quartile Distribution for 12-hour Storm Event

To enter the Huff 2nd Quartile data, select **Time Series Manager** from **Components** on the menu bar. Select Precipitation Gage as the Data Type. Click **New** and enter the name for the gage as "Huff2nd12Hr." A time series data folder now appears in the *Watershed Explorer* window. Note that even though this is a hypothetical design storm, it is entered as is it were rainfall collected at a gage. To enter the rainfall data, click the plus sign next to the Time-Series Data in the *Watershed Explorer* window. Now click on the plus sign next to the

Precipitation Gages to make the precipitation available. Click the plus sign next to the "Huff2nd12Hr" gage and the *Watershed Explorer* window displays all time windows for the precipitation gage. A default time window was added when the gage was created. Select the default time window in the *Watershed Explorer* window and this opens the Component Editor for this precipitation gage and time frame. The component editor has four tabs. Click on the "Manual Entry," the gage location can be left at the default. Change the "Units" to Cumulative Inches and the "Time Interval" to 30 minutes (see distribution in Table 8.2.2. Now click on the "Time Window" tab. Since this is a design storm, hypothetical times and dates can be used. Next, click on the "Table" tab and enter the cumulative rainfall distribution from Table 8.2.2. The "Graph" tab can be used to view the precipitation data graphically.

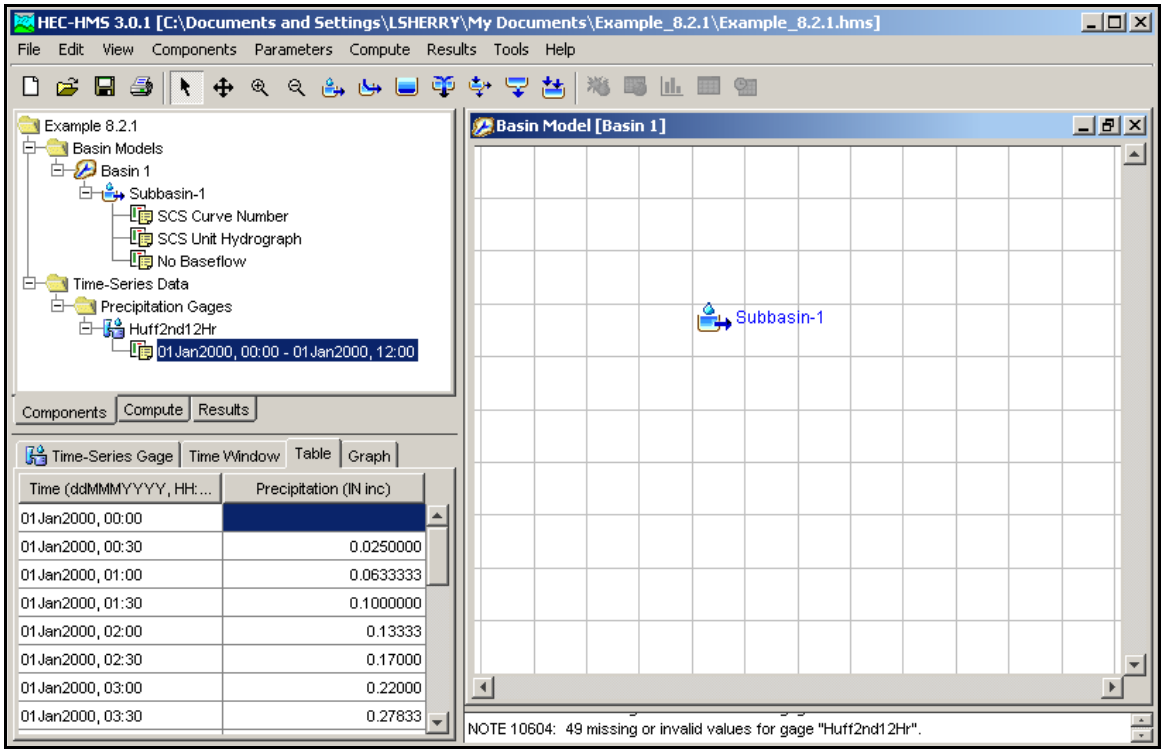


Figure 8.2.1 (c)
Entering the 2nd Quartile Huff Distribution for a 12-Hour Storm Event

To apply the rainfall distribution to Subbasin 1, click **Components-Meteorological Model Manager** from the menu bar. Click the **New** button and enter the name "100-YR, 12HR." Close the Meteorological Model Manager window after pressing the **Create** button. A meteorological model folder now appears in the *Watershed Explorer* window. Click the plus sign next to the **Meteorological Model** in the *Watershed Explorer* window to open the model that was just created. Click on "100-YR, 12HR" to open the *Component Editor* for the model. Click on the "Basins" tab and under "Include Subbasins" enter **Yes**. Click the plus sign next to the "100-YR, 12HR" model in the *Watershed Explorer* window to open the meteorological model options. Select the **Specified Hyetograph** to open the Component

Editor for the hyetograph. Select the precipitation gage (Huff2nd12Hr) that was created earlier. Finally, enter **5.48** in the "Total Depth (IN)" column.

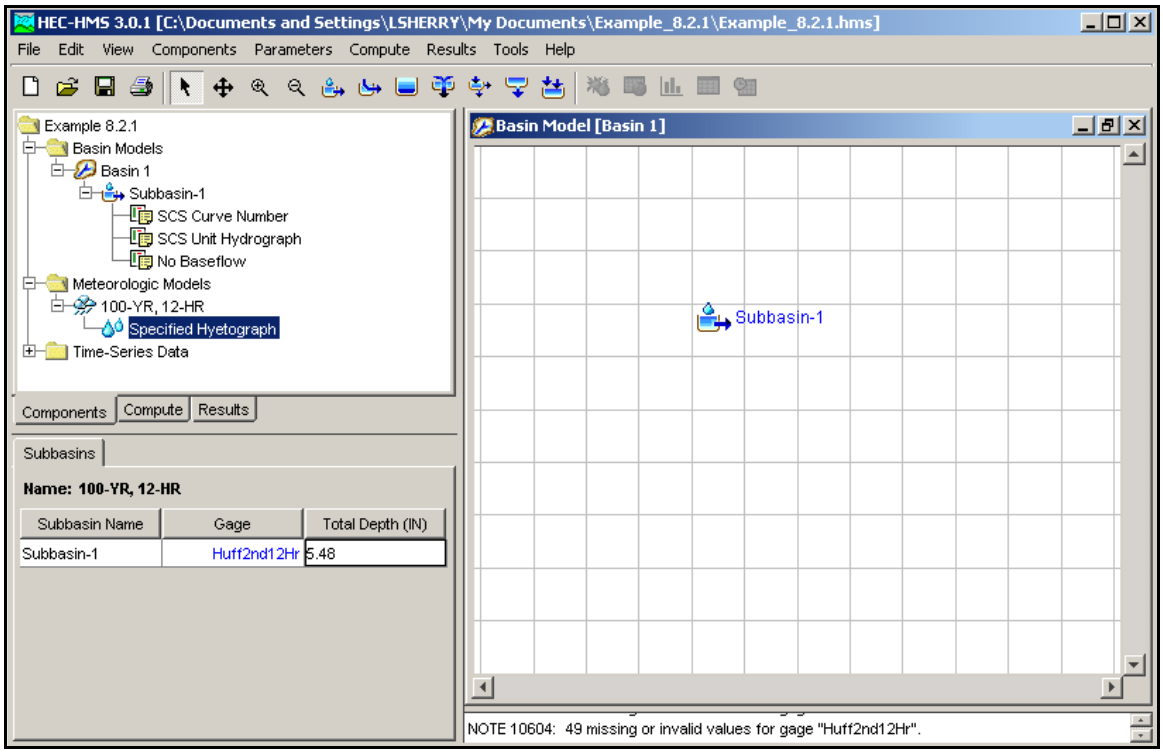


Figure 8.2.1 (d)
Assigning the Precipitation Gage to Subbasin 1

To summarize, the creation of the meteorological model is a two-step process. The hypothetical storm distribution is entered as a precipitation gage and then the gage is tied to each of the subbasins.

Control Specifications

Two of the three data sets needed to perform a simulation have now been completed, the basin model and the precipitation model. However, control specifications are still required prior to the execution of a model run. To enter the control specifications, select **Component – Control Specifications** on the menu bar from the main (PROJECT DEFINITION) screen. Create a new Control Specification titled "100YR, 12-HR." Return to the *Watershed Explorer* window and click on the plus sign next to **Control Specifications**. You will see the "100-YR, 12-HR" that was just created. Click on this and enter the following information on the input screen: For Start Date, enter 01JAN2000; for start time, enter 00:00; for end date, enter 01JAN2000; for end time, enter 18:00; for time increment, enter 5 minutes. The 18-hour simulation should capture the entire hydrograph for the 12-hour storm event at the outflow point.

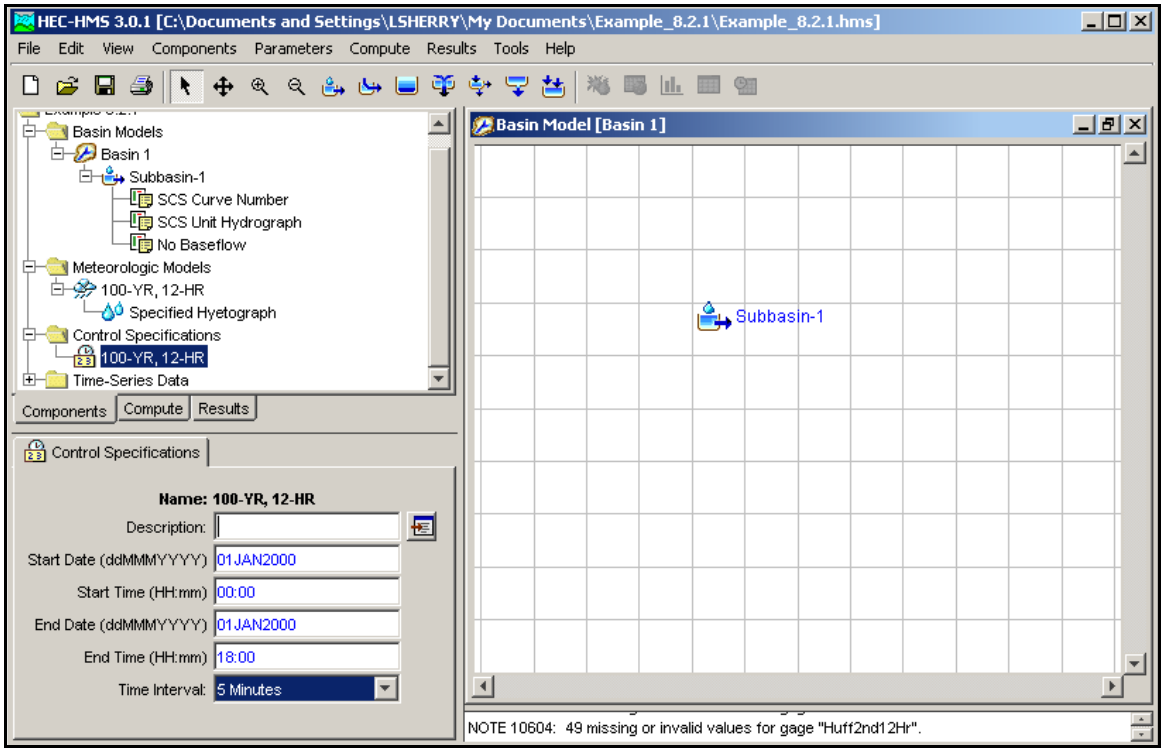


Figure 8.2.1 (e)
Entering the Control Specifications

Model Simulation

Before the model can be run, a simulation run must be created. Click on **Compute-Create Simulation Run** from the main menu. Change the name to "100-YR, 12-HR" and click **Next**. On the next screen is the basin model, which should already be highlighted. Click **Next**. The next screen asks us for the meteorological model. Click on "100-YR, 12-HR" and then click **Next**. The final screen asks us for the Control Specifications. Click on "100-YR, 12-HR." Go to the *Watershed Explorer* window and click on the "Compute" tab. Click on the plus sign next to Simulation Runs and you will see the 100-year 12-hour simulation run. Click on the simulation run and you will see the Basin Model, Meteorological Model, and Control Specifications under the "Simulation Run" tab.

To run the simulation, right-click on the 100-year 12-hour simulation in the *Watershed Explorer* window and then click on **Compute**. A COMPUTE screen will track the progress of the run, provide you with error notes when appropriate, and display a successful completion note if no fatal errors are encountered.

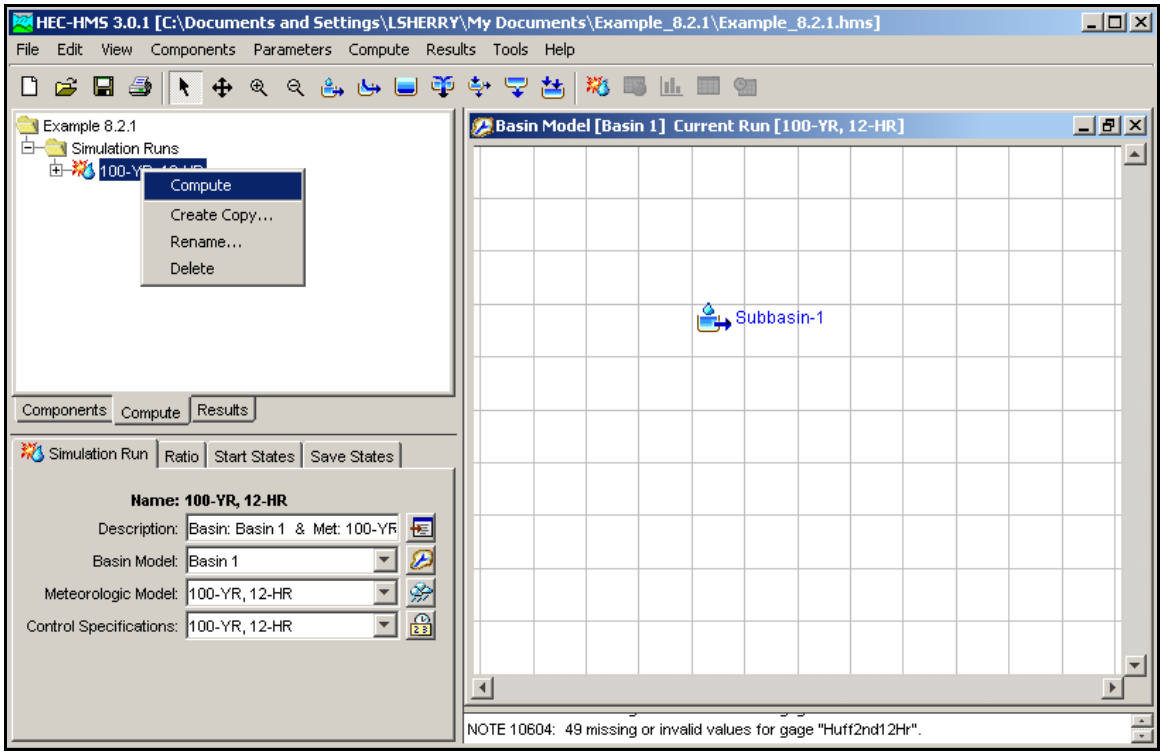


Figure 8.2.1 (f)
Performing the Model Simulation Run

3. Computer Output

A summary of the results for Subbasin 1 can be obtained by right-clicking on Subbasin 1 in the model schematic, selecting "View Results [100-YR, 12-HR]" and then "Summary Table." The results for this example are shown in Figure 8.2.2 below.

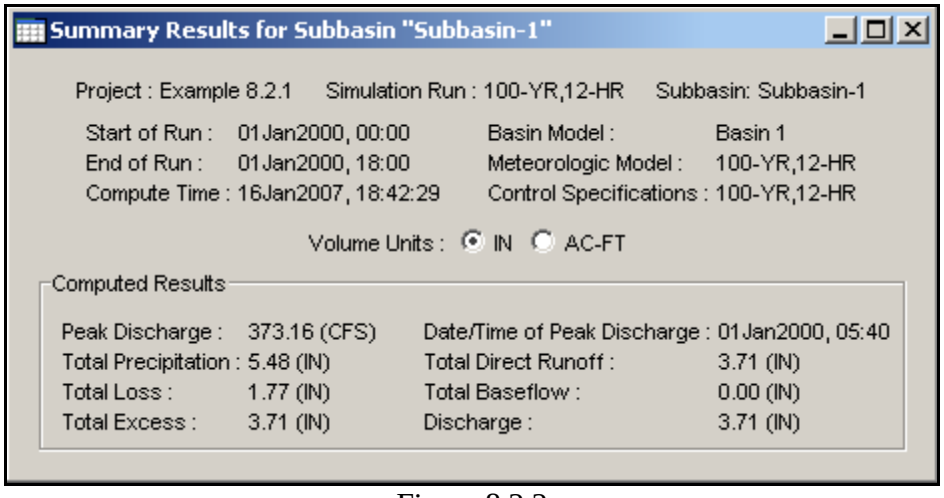


Figure 8.2.2

HEC-HMS Output for Example 8.2.1

4. Summary/Explanation

The results from Example 8.2.1 show that the peak flow for the present conditions from Subbasin 1 is 373 cfs. The time to peak is 5.67 hrs. These values are identical to those results obtained from Win TR-20 in Example 8.1.1 (Peak flow of 373 cfs at a time of 5.63 hrs.).

Example 8.2.2

1. Description

The flow from Subbasin 1 is routed through twin 38" by 60" culverts under Interstate 65.

2. Calculations

A stage-storage-discharge relationship is calculated for the culverts. This is shown in Table 8.1.4. The curve number and time of concentration for this example are the same as the previous example.

3. Computer Input

Using the input from the previous example, use the "Save As" command to save the previous HEC-HMS example as "Example 8.2.2."

Reservoir Routing

Using the toolbars located above the desktop, click on the Reservoir Creation Tool and add a reservoir named "Pond 1." Next, right-click on Subbasin 1, choose "Connect Downstream" and drag it to Pond 1. Pond 1 will be used to represent the stage-storage-discharge relationship for the pond and culverts.

Click on the **Components-Paired Data Manager** option from the menu. Create a new **Elevation-Storage Function** and call it "Pond 1." Go to the *Watershed Explorer* window and click on the plus sign in front of **Paired Data**. Click on Pond 1 and enter the elevation-storage information from Table 8.1.4.

Click on the **Components-Paired Data Manager** option from the menu. Create a new **Storage-Discharge Function** and call it "Pond 1." Go to the *Watershed Explorer* window and click on the plus sign in front of **Paired Data**. Click on Pond 1 and enter the storage-discharge information from Table 8.1.4.

Now the HEC-HMS input requirements for the reservoir routing are satisfied. Compute the

simulation for this example by selecting the Simulation Run created in the previous example.

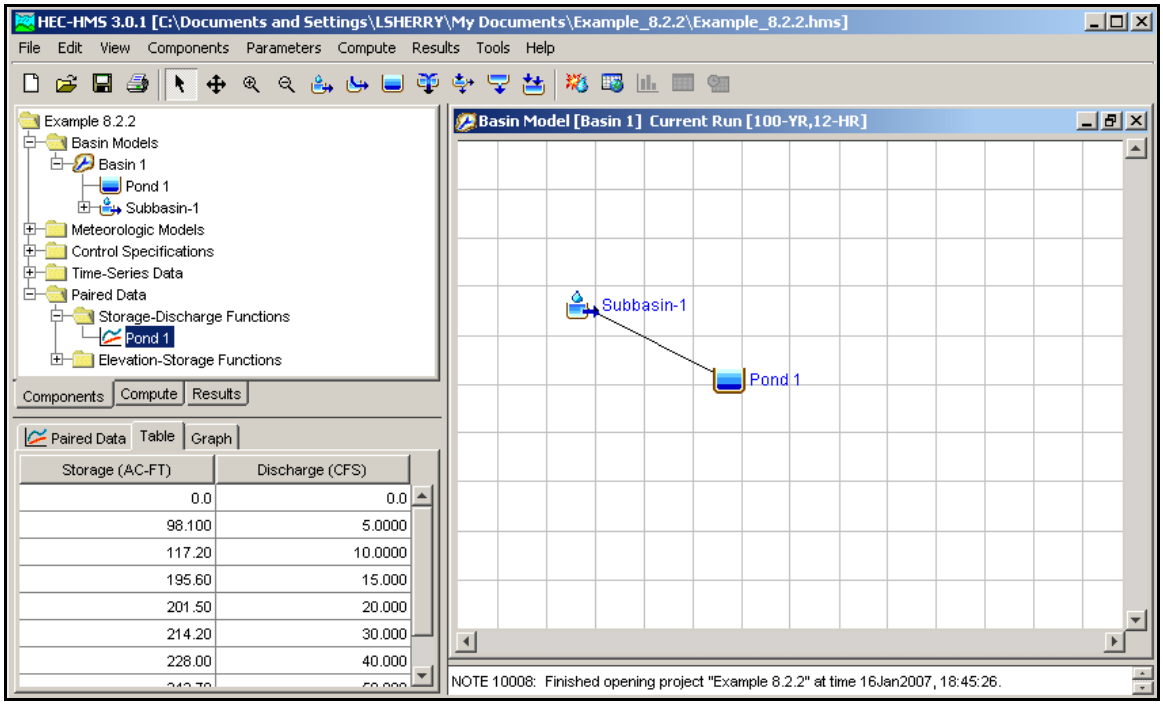


Figure 8.2.3
Entering Stage-Storage-Discharge for Pond 1

4. Output

The results are viewed exactly as in Example 8.2.1; right-click on Pond 1 in the model schematic, choose “View Results” and then click “Summary Table.” The summary table output for Pond 1 is shown in Figure 8.2.4 below.

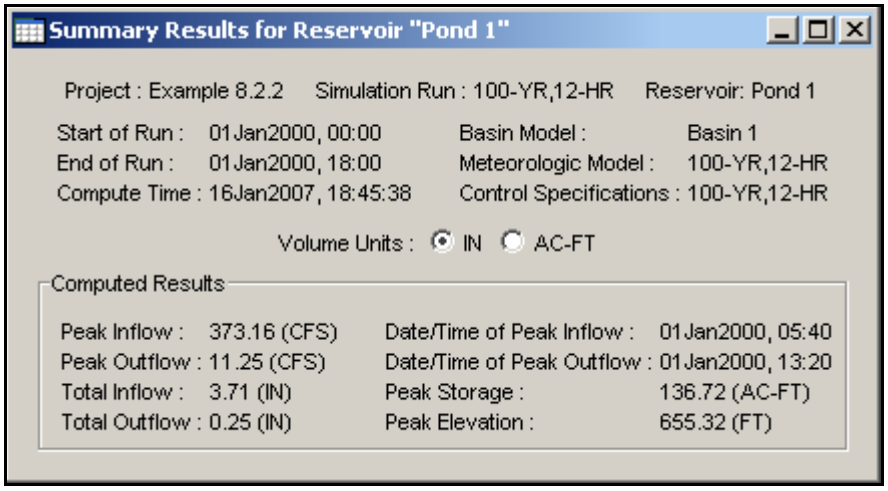


Figure 8.2.4
HEC-HMS Output for Example 8.2.2

5. Summary/Explanation

The flowrate from subbasin 1 is more than 30 times the outflow of the culverts because of the detention through the depressional area. The peak outflow from the culverts is 11 cfs at a peak time of 13.33 hrs. The culverts are large enough to pass the detained flow without overtopping. The peak elevation is 655.32 feet. These results are nearly identical to those found using Win TR-20 in Example 8.1.2.

Example 8.2.3

1. Description

The entire watershed is analyzed to compute the peak runoff hydrograph from the watershed area of 0.85 square miles. This example incorporates the same area as Example 8.2.2 and routes the outflow from the culverts through a ditch and adds the runoff from subbasin 2 to determine the peak runoff at the outlet of the watershed, State Road 26. The stage-storage-discharge relationship for the ditch is determined from Figure 8.1.5.

2. Calculations

The curve number and time of concentration are shown in Tables 8.1.6 and 8.1.7 for Subbasin 2. The area is calculated in square miles. All of these calculations are explained in Section 8.2.2. For the routing of the outflow from the culverts through the ditch to the discharge point of the basin, it is not necessary to calculate the stage-storage-discharge relation, as it was in Win TR-20.

3. Computer Input

Use the Reach Creation Tool and the Subbasin Creation Tool to add the second subbasin and the ditch to the hydrologic model. Name the second subbasin, “Subbasin 2,” and the ditch, “Reach 1.”

Because the outflow of the culverts is routed through the ditch, right-click on Pond 1 in the model schematic, and connect Pond 1 downstream to Reach 1. Using the Junction Creation Tool, add a junction named “Junction 1” to the model. Junction 1 will represent the location where the hydrographs of Reach 1 and Subbasin 2 will be combined. To combine the hydrographs, right-click on Subbasin 2 and connect it downstream to Junction 1.Do the same for Reach 1. The basin schematic for this example is shown in Figure 8.2.5.

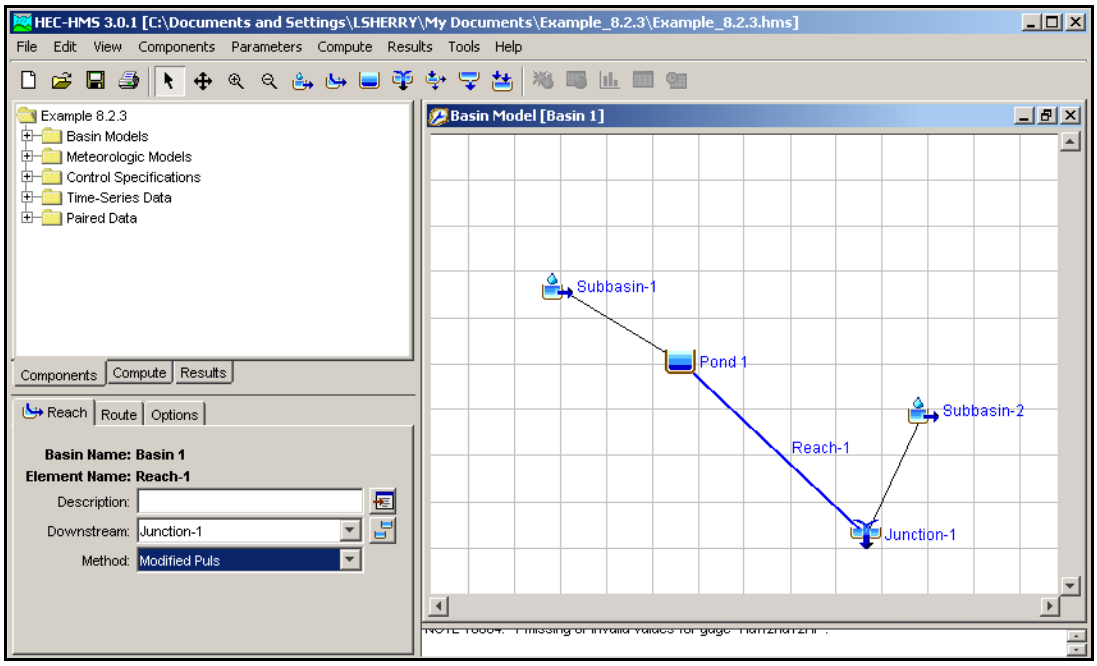


Figure 8.2.5
Basin Schematic for Example 8.2.3

Finally, using the stage-storage-discharge relationship from Example 8.1.3, enter the storage-discharge table for Reach 1. The storage-discharge table for Reach 1 is entered in the same manner as Pond 1 in Example 8.2.2. Because we are using the Modified-Puls method to perform the reach route, the storage-discharge relationship is the only required input for the reach.

4. Output

The output is shown in Figures 8.2.6 (a) and (b). For the final example, the input, hydrographs and summary table are shown.

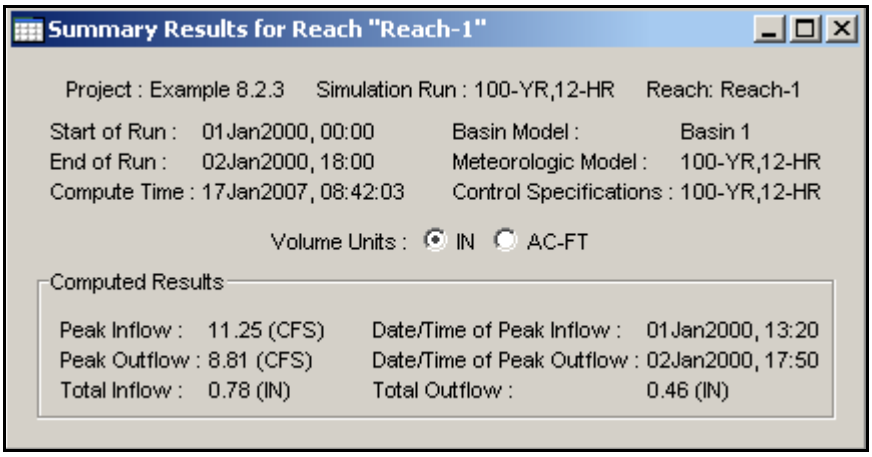


Figure 8.2.6 (a)
Summary Output for Reach 1

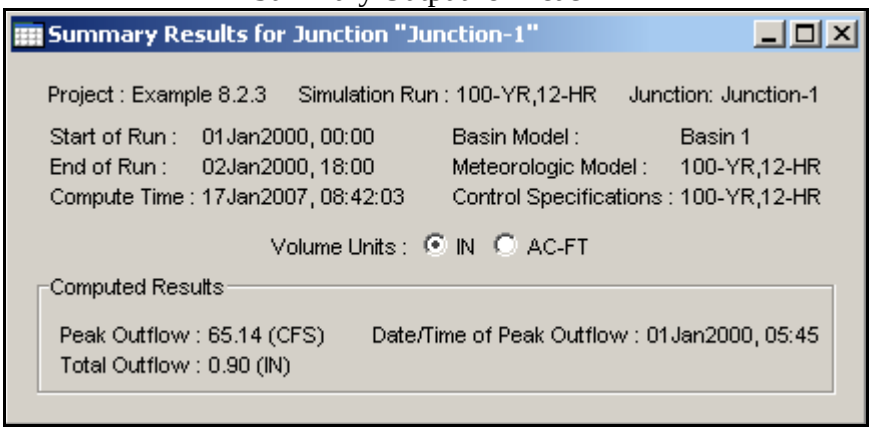
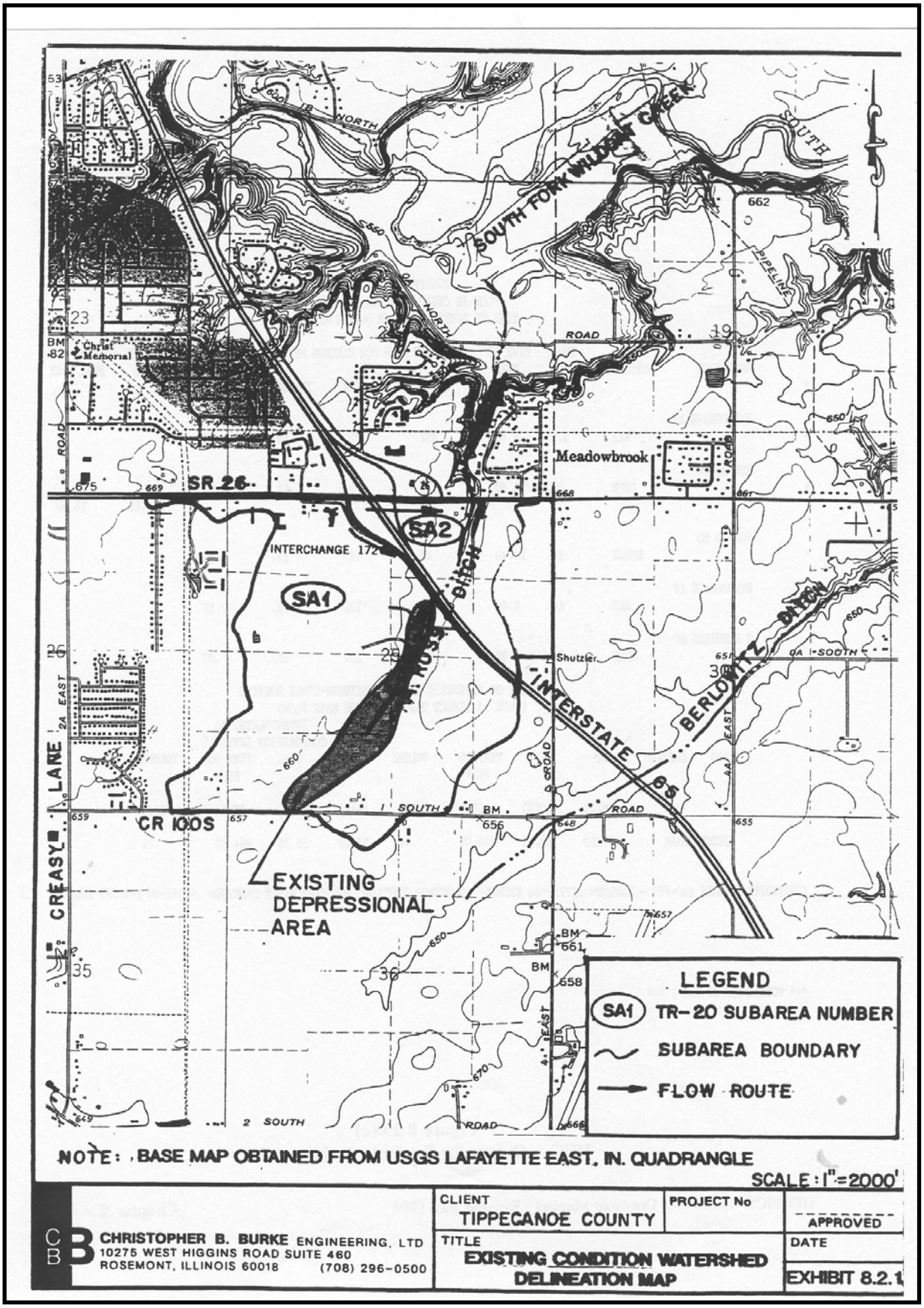


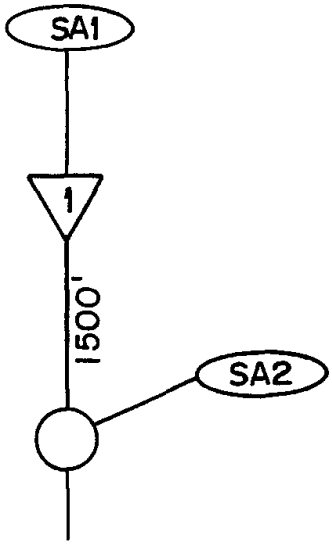
Figure 8.2.6 (b)
Summary Output for Junction 1

5. Summary/Explanation

The peak outflow from the watershed is 65 cfs and the time to peak is 5.75 hrs. As with the Win TR-20 example, the routing of the flow through the ditch to the outlet has insignificant effects on the time to peak. This outflow value is close to the value obtained from TR-20 which has a peak flow value of 67 cfs. The difference can be attributed to the channel routing methods used in the two programs: Muskingum-Cunge in Win TR-20 and Modified Puls in HEC-HMS.


SOUTH FORK WILDCAT CREEK
SR 26
INTERCHANGE 172
SA1
SA2
INTERSTATE 65
BERLOWITZ DITCH
Meadowbrook
Christ Memorial
Shutzler
CR 100S
CREASY LANE
EXISTING DEPRESSIONAL AREA
LEGEND
SA1 TR-20 SUBAREA NUMBER
SUBAREA BOUNDARY
FLOW ROUTE
NOTE: BASE MAP OBTAINED FROM USGS LAFAYETTE EAST, IN. QUADRANGLE
SCALE: 1"=2000'
CHRISTOPHER B. BURKE ENGINEERING, LTD
10275 WEST HIGGINS ROAD SUITE 460
ROSEMONT, ILLINOIS 60018 (708) 296-0500
CLIENT
TIPPECANOE COUNTY
PROJECT No
APPROVED
TITLE
EXISTING CONDITION WATERSHED DELINEATION MAP
DATE
EXHIBIT 8.2.1

A. ROSS DITCH
EXISTING CONDITION
TR-20 MODEL
FLOW SCHEMATIC



LEGEND:

SA1 TR-20 SUBAREA

COMBINE FLOWS

1500' REACH ROUTING

1 RESERVOIR ROUTING

CB CHRISTOPHER B. BURKE ENGINEERING, LTD 10275 WEST HIGGINS ROAD SUITE 460 ROSEMONT, ILLINOIS 60018 (708) 296-0500	CLIENT TIPPECANOE COUNTY	PROJECT No	APPROVED
	TITLE FLOW SCHEMATIC OF EXISTING CONDITIONS		DATE
			EXHIBIT 8.2.2

8.3 WIN TR-55 PROGRAM

Win TR-55 software was developed by the National Resource Conservation Service (NRCS) and is primarily used as a single event rainfall-runoff watershed model. Win TR-55 has the capability to perform both reach routing and reservoir routing for watersheds. However, the hydrologic analyses performed by Win TR-55 can only be done for a 24-hour duration storm event. In this section, the focus is on the program's ability to size a detention system.

A free copy of Win TR-55 and user manuals is available for download at the NRCS website: http://www.wcc.nrcs.usda.gov/products/W2Q/H&H/tools_motels/WinTR55.html

8.3.1 Win TR-55 Example Problem

Example 8.3.1

Using the data from Example 6.2.1, compute the detention storage requirement using the Win TR-55 computer program. Use AMC II, 24-hour 100-year rainfall depth, time of concentration of 20 minutes, and a SCS Type II distribution.

In summary:

CN	= 76
t_c	= 20 minutes
A	= 10 acres
O	= 4.97 cfs

The rainfall depth can be found with the information from Example 6.2.1 for Evansville, Indiana.

For t >1 hour:

$$i = \frac{c\,T_r^{\alpha}}{(t+d)^{\beta}} = \frac{1.3411(100)^{0.2166}}{(24+0.300)^{0.8154}} = 0.2697 \text{ inches/hr}$$

With an intensity of 0.2697 inches/hr, the corresponding depth is (24 x 0.2697) = 6.47 inches.

The first step is to enter the project description and subbasin information given in this example, as shown in Figure 8.3.1 (a).

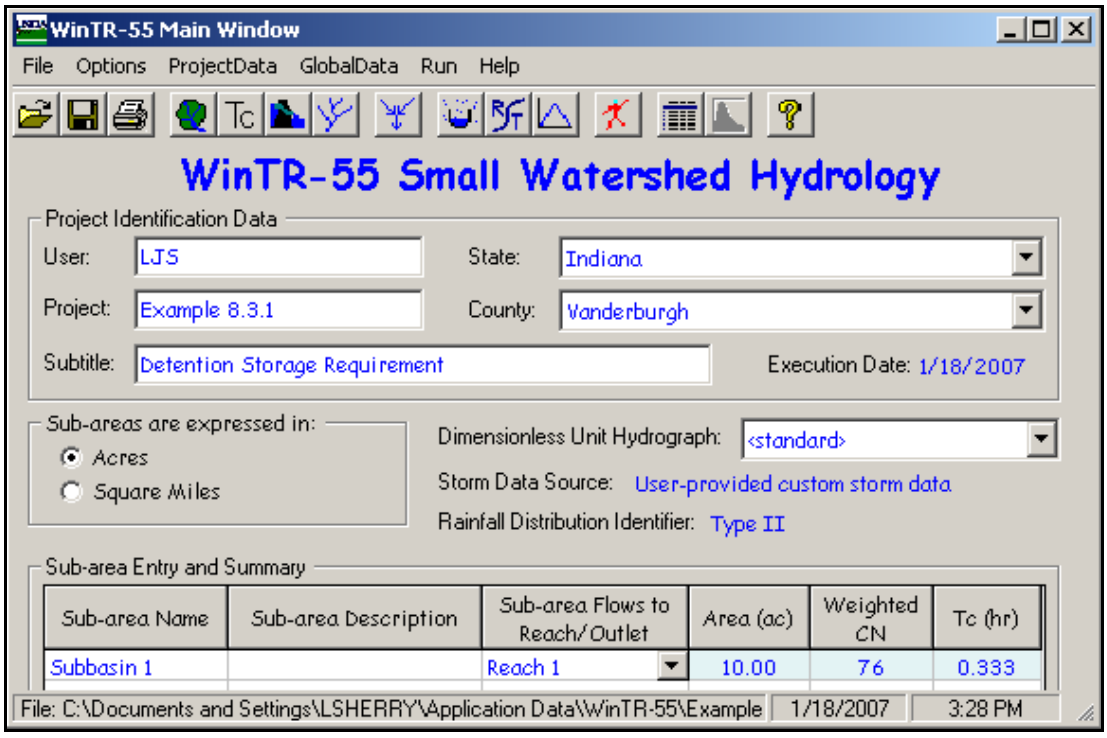


Figure 8.3.1 (a)
Entry of Project Description and Subbasin Characteristics

The next step is to enter the 100-year 24-hour rainfall depth to be used with the SCS Type II distribution, as shown in Figure 8.3.1 (b) below.

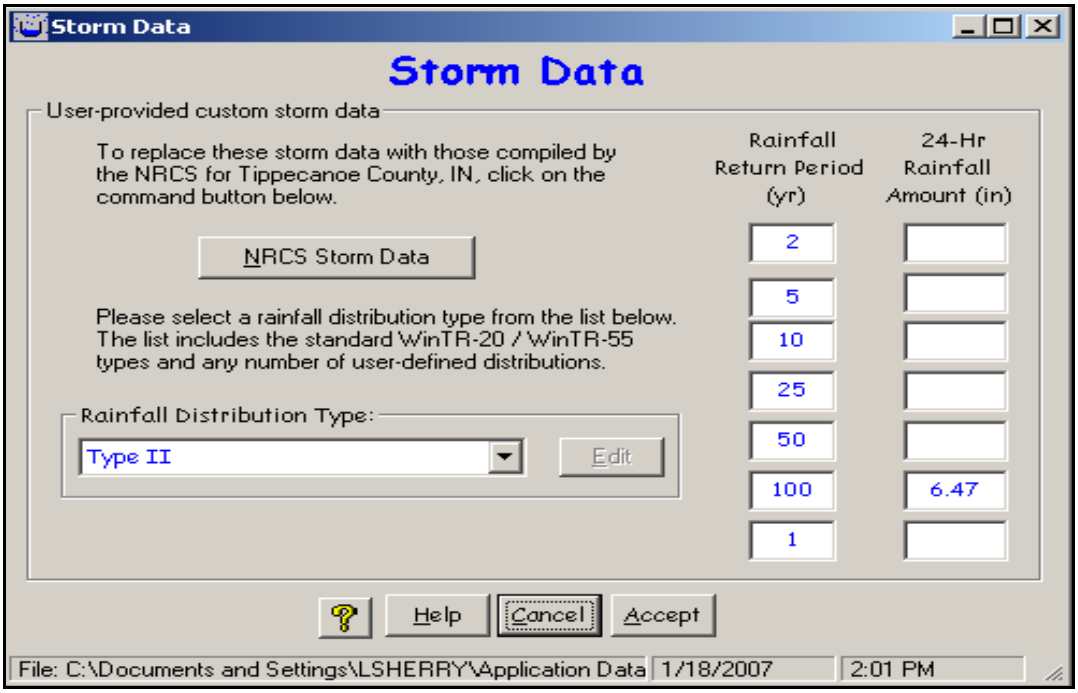


Figure 8.3.1 (b)

SCS Type II Distribution with 100-Year 24-Hour Rainfall

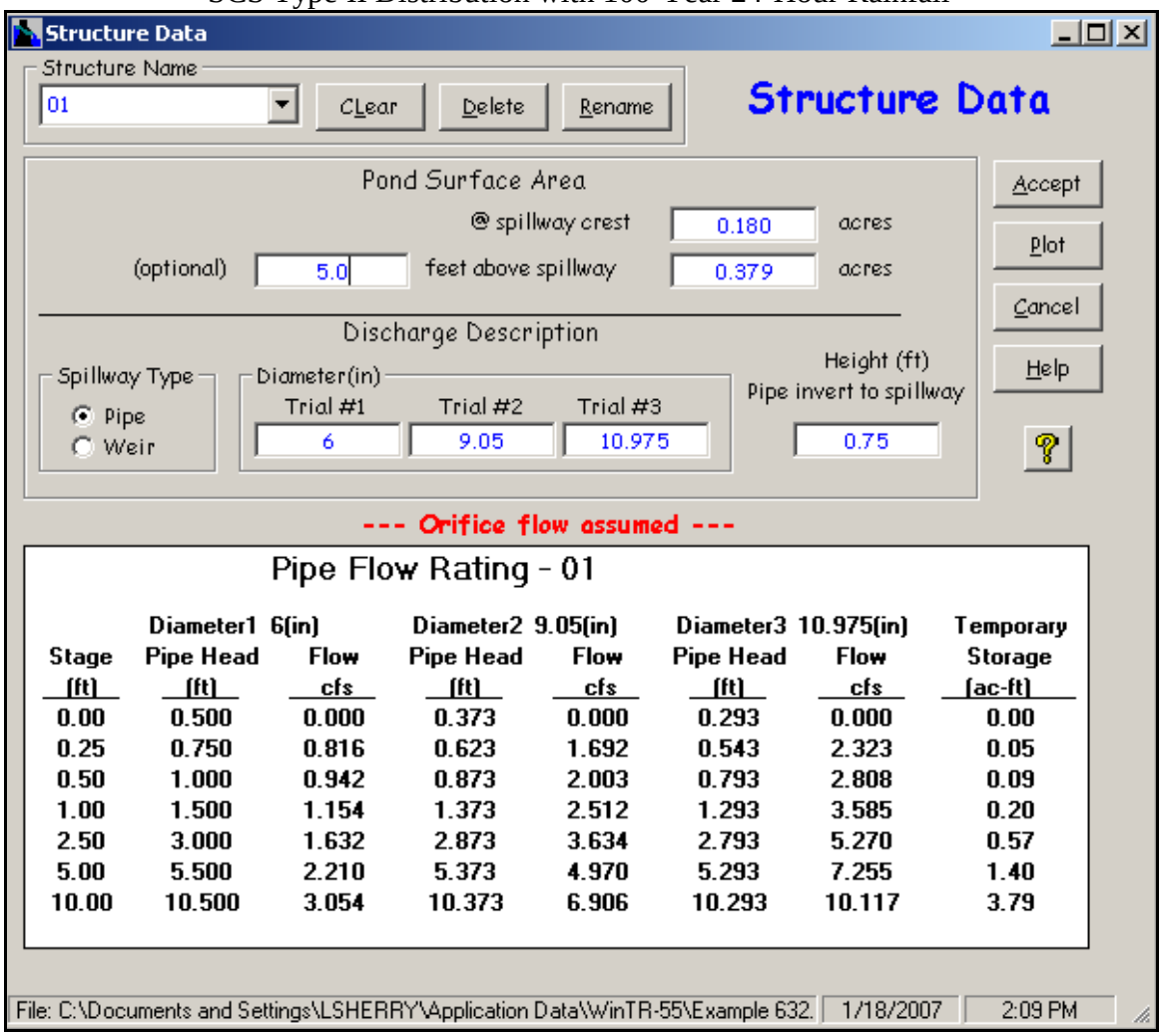


Figure 8.3.1 (c)
Input of Detention Pond Information

The final step is to enter the information to determine the size of the required detention pond. Assuming we have a detention pond with five feet of depth, the outlets to the pond are sized so that the maximum allowable release rate (4.97 cfs) is the actual release rate at when the pond is full. Win Tr-55 allows the user to enter up to three different sized outlets, referred to as "Trials," as shown in Figure 8.3.1 (c). Therefore, three different rating curves are generated for the three different outlet sizes. This is particularly useful for fine-tuning the restrictor size so that the allowable release rate can be met at the high water level of the detention pond. As shown in Figure 8.3.1 (c), the outlet size in Trial #2 (9.05 inches) was fine-tuned to release the allowable release rate of 4.97 cfs at a stage of five feet.

Now that the outlet size has been determined, the stage-area relationship for the pond must be entered so that the pond is releasing 4.97 cfs at its high water level of five feet. Assuming that the side slopes of the pond are at a 4:1 slope, the areas of the normal water

level of the pond (shown as spillway crest in Figure 8.3.1 (c)) and the area of the high water level of the pond (specified as 5.0 feet above spillway in Figure 8.3.1 (c)) are entered. The stage-storage relationship for each pond iteration is shown under the "Temporary Storage" column next to the rating curve for Trial #3. This is an iterative process that is complete when the Win TR-55 output shows exactly 4.97 cfs as the outflow of the detention pond. Figure 8.3.2 shows the Win TR-55 output for this example.

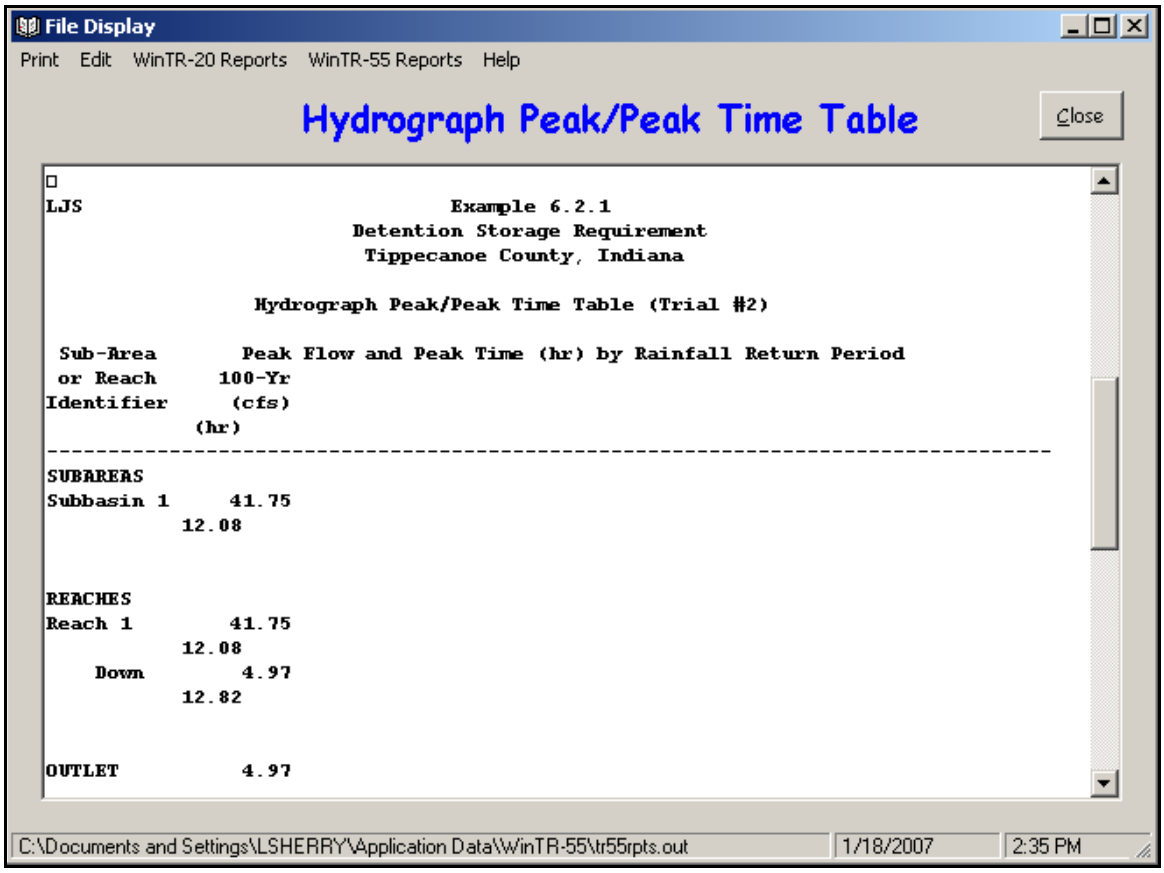


Figure 8.3.2
Win TR-55 Output for Example 8.3.1

Using Win TR-55, the peak inflow rate is 41.75 cfs for the site and using the detention storage procedure within the program, the volume is found to be 1.4 acre-feet. The difference between the rational method result of 0.51 acre-feet, from Example 6.2.1, and this example is because the Quick TR-55 computer program is less accurate as the q_o/q_I ratio approaches design limits. When the storage volume required is small, the shape of the outflow hydrograph is sensitive to the rate of rise of the inflow hydrograph (SCS, 1986).

Chapter 8 - REFERENCES

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9. Purdue, A.M., Jeong, G.D., and Rao, A.R., "Statistical Characteristics of Short Time Incremental Rainfall," Department of Public Works of Indianapolis/Purdue University, August 1992.

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CHAPTER 9 - STORMWATER QUALITY

9.1 - INTRODUCTION

Stormwater runoff has the potential to convey many undesirable hazardous materials such as heavy metals, nutrients, and sediment. These materials are transported by the runoff into the storm sewer system and eventually a natural body of water. This is known as nonpoint source pollution (NPS), and it is the leading cause of stormwater quality problems. Best Management Practices (BMPs) are measures that can be taken to reduce the conveyance of pollutants into the stormwater. By definition, BMPs consist of schedules of activities, prohibitions of practices, maintenance procedures, and other management practices to prevent or control pollutants in stormwater runoff (EPA, 1996). This chapter discusses temporary BMPs that are applicable to periods of construction, as well as permanent BMPs that function as part of any given storm sewer system.

For a more in-depth discussion of BMPs and their application, please refer to the *Indiana Storm Water Quality Manual*, developed by the Indiana Department of Environmental Management (IDEM). The manual is an up-to-date guide to stormwater quality practices and is available on-line at the following address: http://www.in.gov/idem/stormwater (IDEM, 2007). Additional BMP information and guidance is provided in the *International Stormwater BMP Database*, which is available on-line at the following address: http://www.bmpdatabase.org. The database, sponsored by such organizations as the American Society of Civil Engineers (ASCE) and the Environmental Protection Agency (EPA), is a database of BMP studies and study-related publications.

9.1.1 - Types of Stormwater Quality BMPs

There are many types of BMPs that relate to stormwater management and their use is well documented and quantified. Stormwater BMPs can be classified in two categories: (1) construction and (2) post-construction. Some BMPs are temporary measures that are used only to control pollutants during the construction phase of a project. These BMPs are usually in place to control sediment and erosion. BMPs can also function as permanent features of a storm sewer system. These BMPs are also designed to control sediment and erosion, but usually include filters that separate stormwater from materials such as heavy metals, nutrients, and even pathogens.

Runoff from construction sites and urban areas can adversely impact water bodies in a number of ways. For example, sediment from construction sites increases the turbidity of water, making it difficult for aquatic plants and fish to find food and sunlight. The addition of sediment to bodies of water also causes a decrease in storage and conveyance. Runoff from urban areas can contribute many harmful contaminants to water bodies. Among these contaminants are hydrocarbons, nutrients, salt, and bacteria. These are just a few examples of the negative impacts that can occur if there are no BMPs in place to control stormwater runoff.

9.2 - CONSTRUCTION STORMWATER BMPS

As mentioned above, construction sites and urban areas are major contributors of pollutants in stormwater. Several steps can be taken throughout the construction phase that will prevent pollutants from entering the storm sewer system and eventually a body of water. This section will present some commonly used BMPs to minimize pollution.

9.2.1 - Site Assessment

The first step in protecting stormwater quality is to identify the potential pollutant sources on a project site. Also, any wetlands located on-site should be identified and delineated. Although, sediment in stormwater runoff is perhaps the most commonly identified pollutant, there are many other potential pollutant sources, such as construction equipment and fuel storage locations. Regardless of the source, BMPs must be implemented to control the NPS that results from construction sites.

9.2.2 - Silt Fencing

Silt fencing is an example of a common temporary BMP that is used to intercept stormwater runoff that is carrying sediment. Silt fencing, which has been widely used for a couple of decades, consists of a barrier of geotextile fabric, or filter fabric, stretched across supporting posts. Silt fences are less expensive relative to other BMPs, and are very effective when installed properly. Figure 9.2.1 shows details of a silt fence installation.

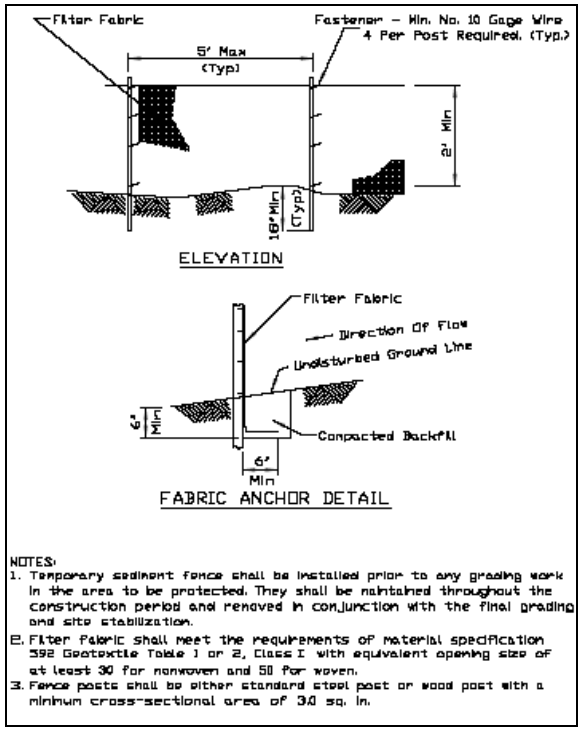


Figure 9.2.1
Typical Silt Fence Plan (NRCS, 2002)

9.2.3 - Rock Check Dams

In some cases ditches and swales may need erosion protection but cannot receive a non-erodible lining. If this occurs, a rock check dam could be implemented. A rock check dam is a small rock dam that is constructed across a grass swale or ditch. They are typically composed of rocks of varying size and sometimes use filter fabric as a stable base. A rock check dam can reduce the velocity in the stormwater runoff, trap sediment, and increase infiltration (if the swale or ditch has suitable soils). Implementing a channel with vegetative lining should be considered prior to the installation of a rock check dam. Rock check dams are typically more effective than straw bales and silt fences at stabilizing "wet weather ditches." Figure 9.2.2 shows details on the installation of a rock check dam.

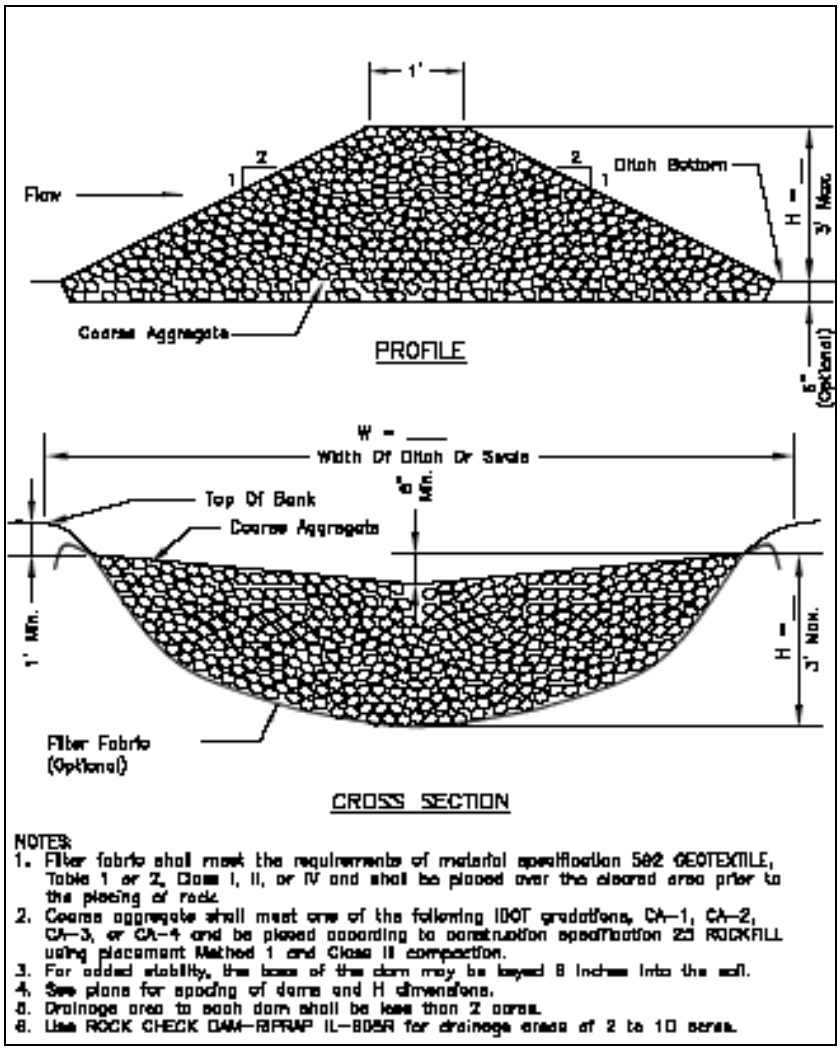


Figure 9.2.2
Details of Rock Check Dam Installation (NRCS, 2002).

9.2.4 - Inlet Protection

Typically, storm sewer systems associated with residential and commercial developments are installed before the actual construction of the development. In these cases, BMPs should be installed at the inlets to storm sewers to prevent any sediment from hydrologically disturbed areas from reaching the storm sewer system. There are many types of inlet protection, and each type is suitable for drainage areas less than one acre.

One type of inlet protection BMP is using fabric drop, or using a fabric barrier around an inlet. Before permanent stabilization of the entire area has occurred, fabric drop inlet protection allows the storm sewer to be functional. Fabric drop inlet protection is appropriate when draining small, flat areas where the peak flow rate tributary to the inlet will be shallow sheet flows of 1 cfs or less. Figure 9.2.3 below demonstrates the details of the installation of fabric drop inlet protection.

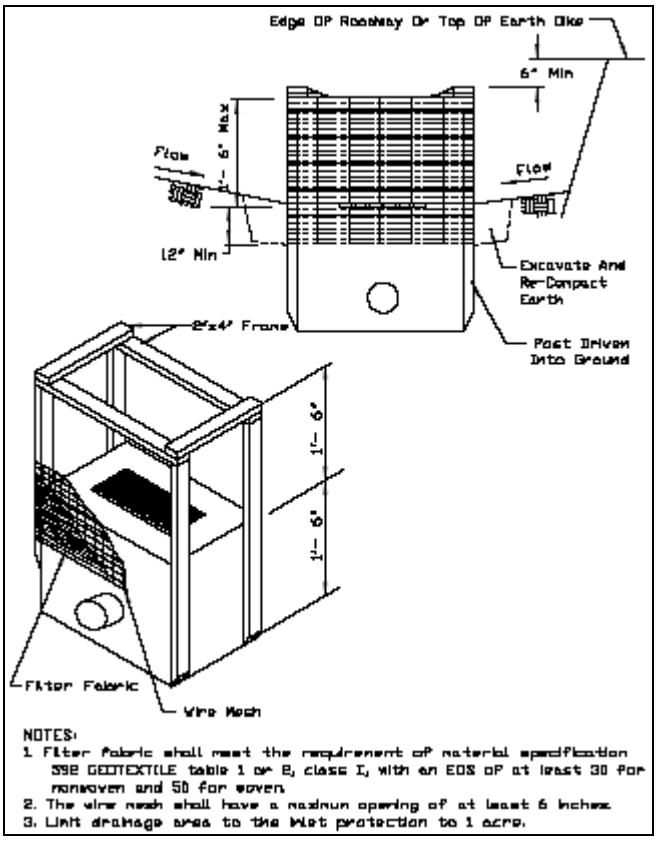


Figure 9.2.3
Details of Installation of Fabric Drop Inlet Protection (NRCS, 2002)

Another example of an inlet protection BMP is a gravel and wire mesh that can be used with either a drop inlet or a curb inlet. A gravel and wire mesh type of inlet protection should be used where heavy flows are expected and where a considerable amount of ponding will not cause an inconvenience. Gravel and wire mesh type inlet protection consists of a wire mesh that extends over the inlet opening. Gravel is then placed over the

wire mesh; a filter fabric is an optional addition. Figure 9.2.4 shows the details of the installation of gravel and wire mesh inlet protection.

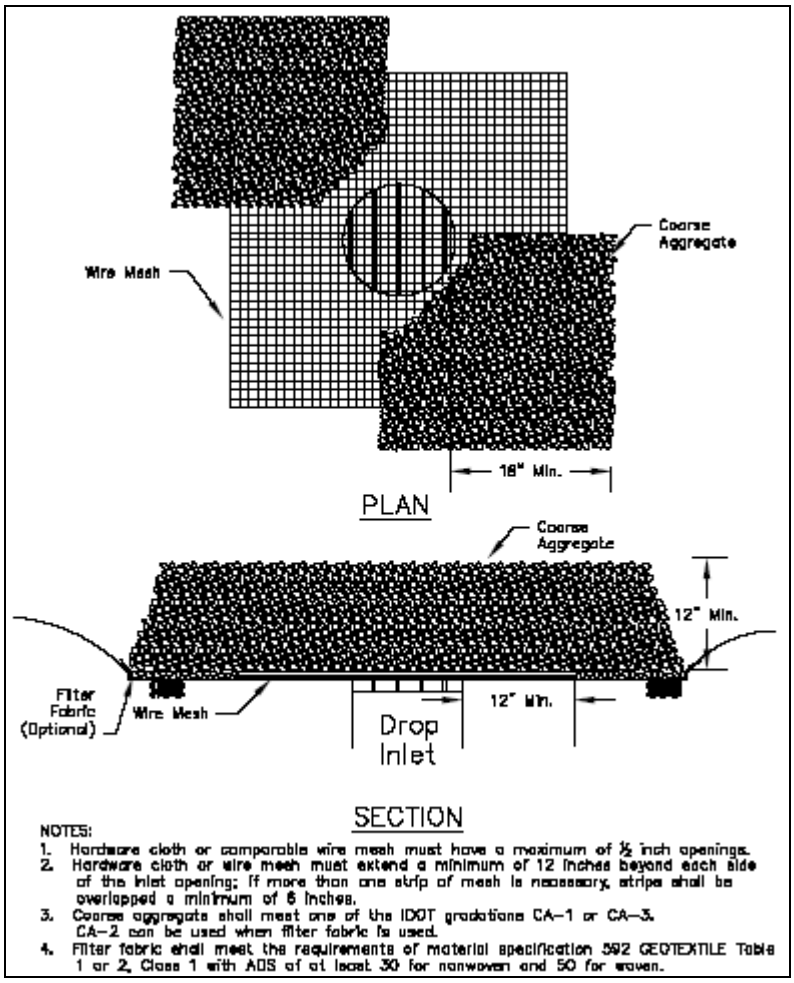


Figure 9.2.4
Details on the Installation of Gravel and Wire Mesh Inlet Protection (NRCS, 2002)

The types of inlet protection that are described above are just two of several inlet protection types. Each of these types of protection is discussed in detail in the *Illinois Drainage Manual*. Among the types of inlet protection are:

- Block and Gravel
- Excavated Drain
- Sod Filter
- Straw Bale Barrier

9.2.5 - Outlet Protection

In the previous section, BMPs to protect inlets to storm sewers were discussed. In this section, outlet protection BMPs will be examined. Outfalls from storm sewer systems, whether in the form of a tile drains, culverts, channels, or conduits can have much higher velocities than their receiving streams. The purpose of outlet protection is to reduce the velocity at the discharge point in order to prevent scour and erosion.

If the outfall of the storm sewer system is a tile drain, an outlet extension is a BMP that will provide a stable outfall. When small tile drains (less than 8 inches in diameter) discharge to an open ditch, a section of metal pipe can be attached to the tile drain outlet. The tile drain outlet is able to drain to a recessed area off the ditch that provided protection from bank erosion. Figure 9.2.5 below shows the details of a typical pipe outlet extension.

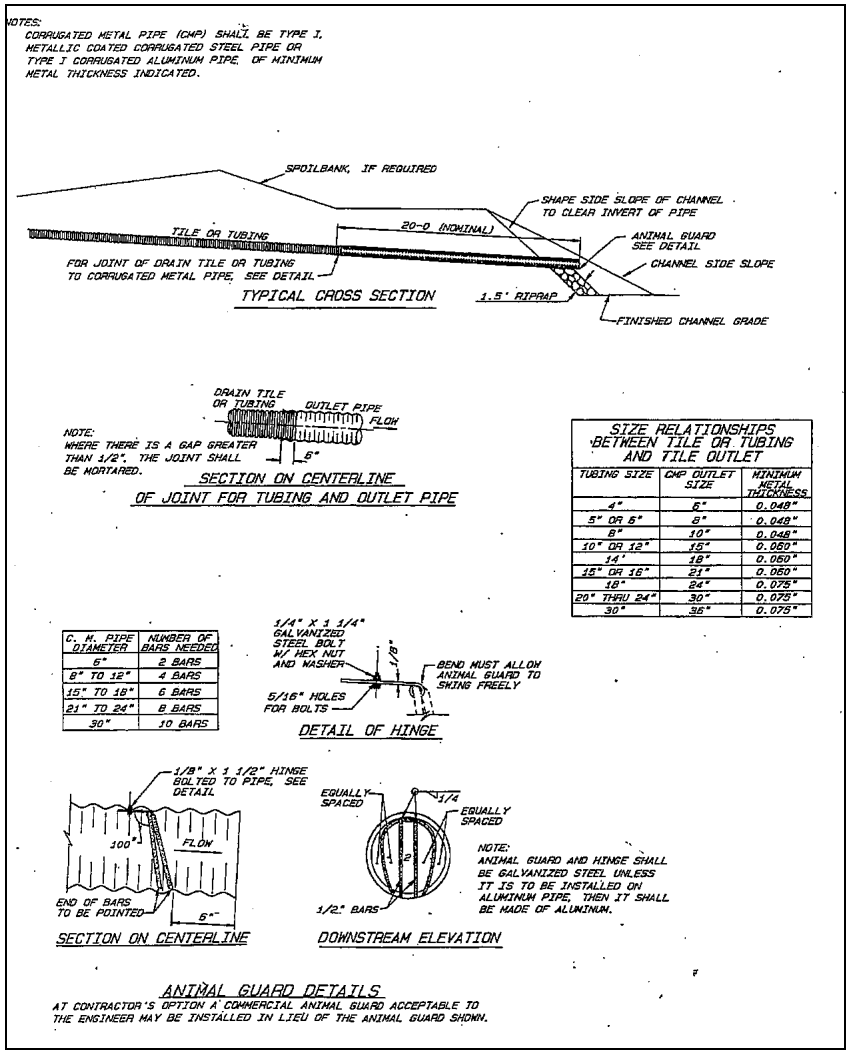


Figure 9.2.5
Specification for Typical Outlet Pipe Extension (NRCS, 2002)

Another type of outlet protection is designed to reduce the velocity, and therefore erosion, at the outlets of channels, culvert, and conduits. This BMP consists of placing a riprap-lined apron (rock outlet protection) at the outlet of the channel, culvert, or conduit. The purpose of this type of outlet protection is to reduce the velocity of the stormwater flows at the outlet. Geotextile fabric should be used as the foundation, and the length and width of the apron should be sized based on the design flow rate, outlet pipe diameter, and tailwater effects, if applicable. Figure 9.2.6 below shows installation details for a well-defined channel.

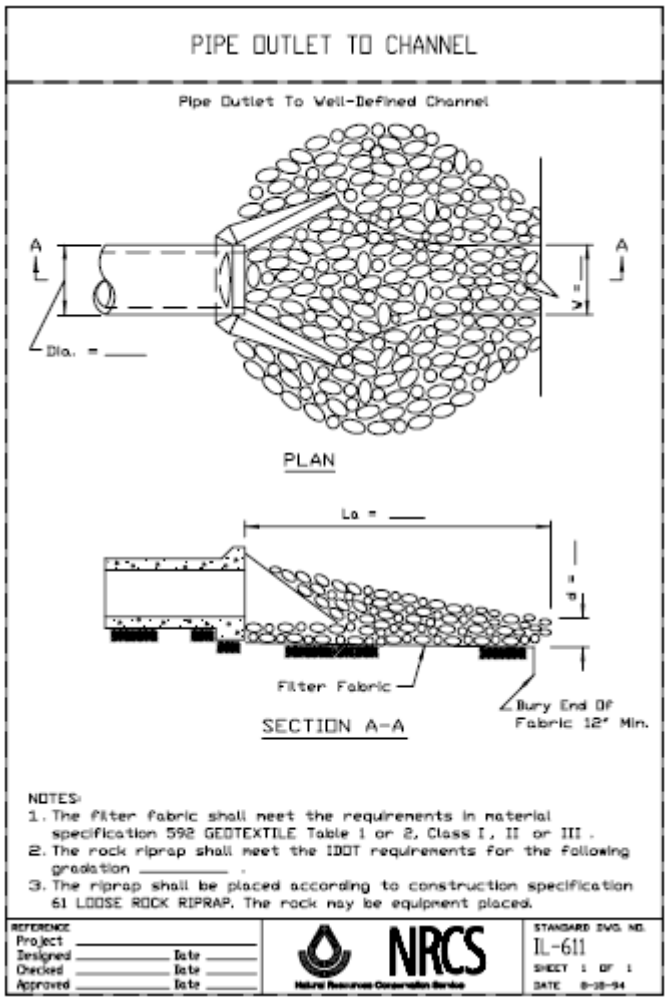


Figure 9.2.6
Riprap-lined Apron Installation (NRCS, 2002)

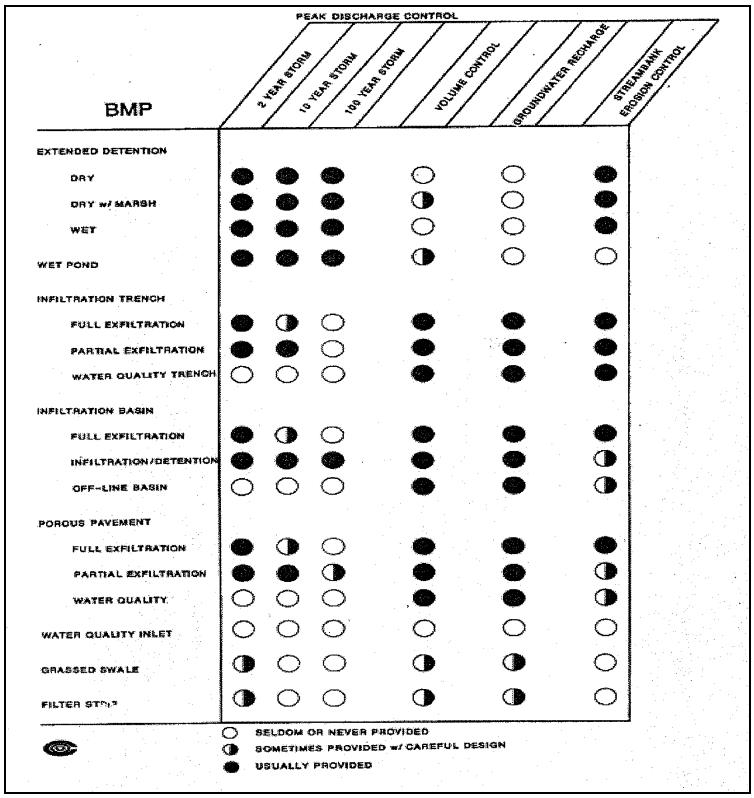


Figure 9.2.7
Commonly Used BMPs and Their Effect on Peak Discharge, Total Runoff Volume, Groundwater Recharge, and Streambank Erosion (Schueler, 1987).

9.3 - POST-CONSTRUCTION STORMWATER BMPS

The BMPs discussed in this chapter so far are temporary measures that protect the quality of stormwater during the construction phase. This section focuses on post-construction BMPs that are permanent components of a stormwater management system. Post-construction BMPs are designed to control sediment and erosion but also have the capability to filter out other undesirable materials such as hydrocarbons and nutrients.

9.3.1 - Level Spreaders

Level spreaders are a commonly implemented BMP that reduce concentrated flow to sheet flow and then passes it over a non-erodible area or vegetated filter strip. The resulting sheet flow has a lower erosion potential, and a higher potential for pollutant filtering. Level spreaders are used to control the runoff from impervious areas such as parking lots, but they can also be applied to diversion outlets. When dealing with runoff from impervious areas, the spreader and filter strip should be the same width as the impervious area that is being drained. For diversion outlet applications, the spreader should be designed according to the 10-year, 24-hour storm event. Table 9.3.1 describes the design criteria for level spreaders to be used with diversion outlets.

Table 9.3.1
Spreader Dimension Based on 10-Year, 24-Hour Flowrate (NRCS, 2002)

Design Flow (cfs)	Minimum Depth (ft)	Minimum Length (ft)
0-10	0.5	10
10-20	0.6	20
20-30	0.7	30

Figure 9.3.1 below provides the installation details for both types of level spreaders: flow from impervious area and diversion outlets.

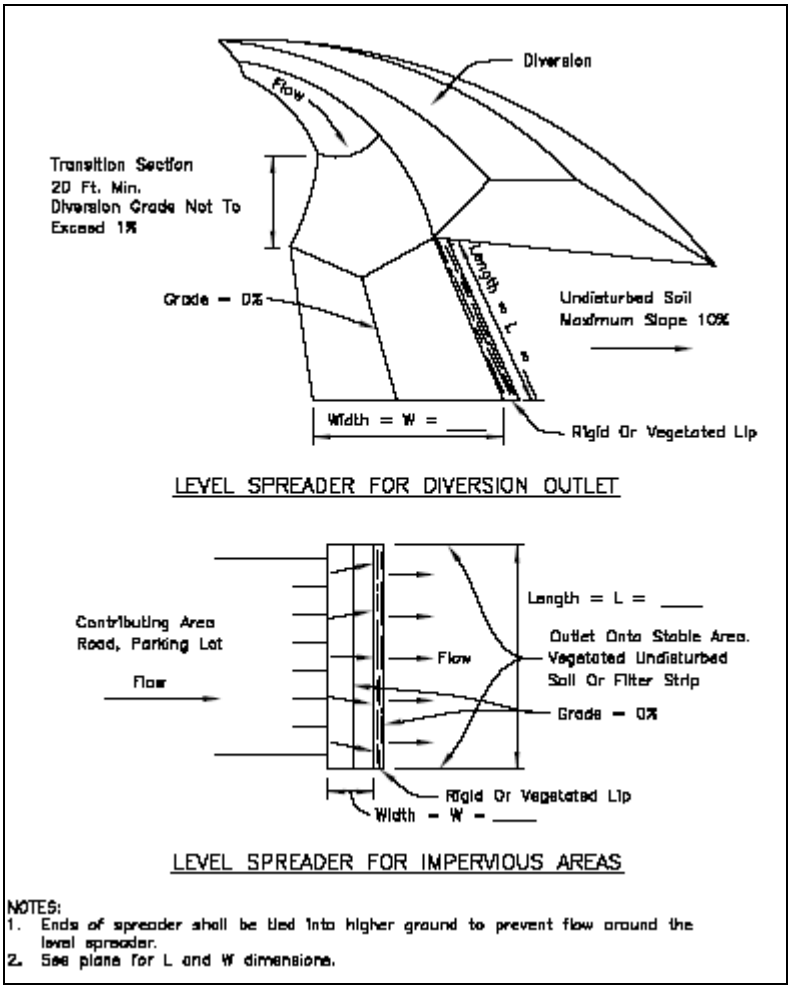


Figure 9.3.1
Level Spreader Installation Details (NRCS, 2002)

9.3.2 - Filter Strips

Filter strips are commonly used BMPs that serve as buffers between stormwater runoff in the form of sheet flow and the ultimate discharge point of the site (such as a creek, storm sewer, or lake). Filter strips are vegetated areas that can remove sediment and other pollutants from stormwater runoff and can also reduce the actual volume of stormwater runoff. Pollutants are removed from the stormwater runoff through absorption and vegetative uptake and runoff volume is reduced through ground infiltration.

The use of filter strips should be restricted to areas where stormwater runoff is discharged as sheet flow. In areas where stormwater runoff is discharged as concentrated flow, a level spreader may be used to convert the runoff to sheet flow before it reaches the filter strip. Filter strips should have slopes of 15% or less and should not have drainage areas greater than 5 acres. The dimensions of the filter strip are determined using the size of the drainage area. Figure 9.3.2 shows the installation details for the application of a filter strip.

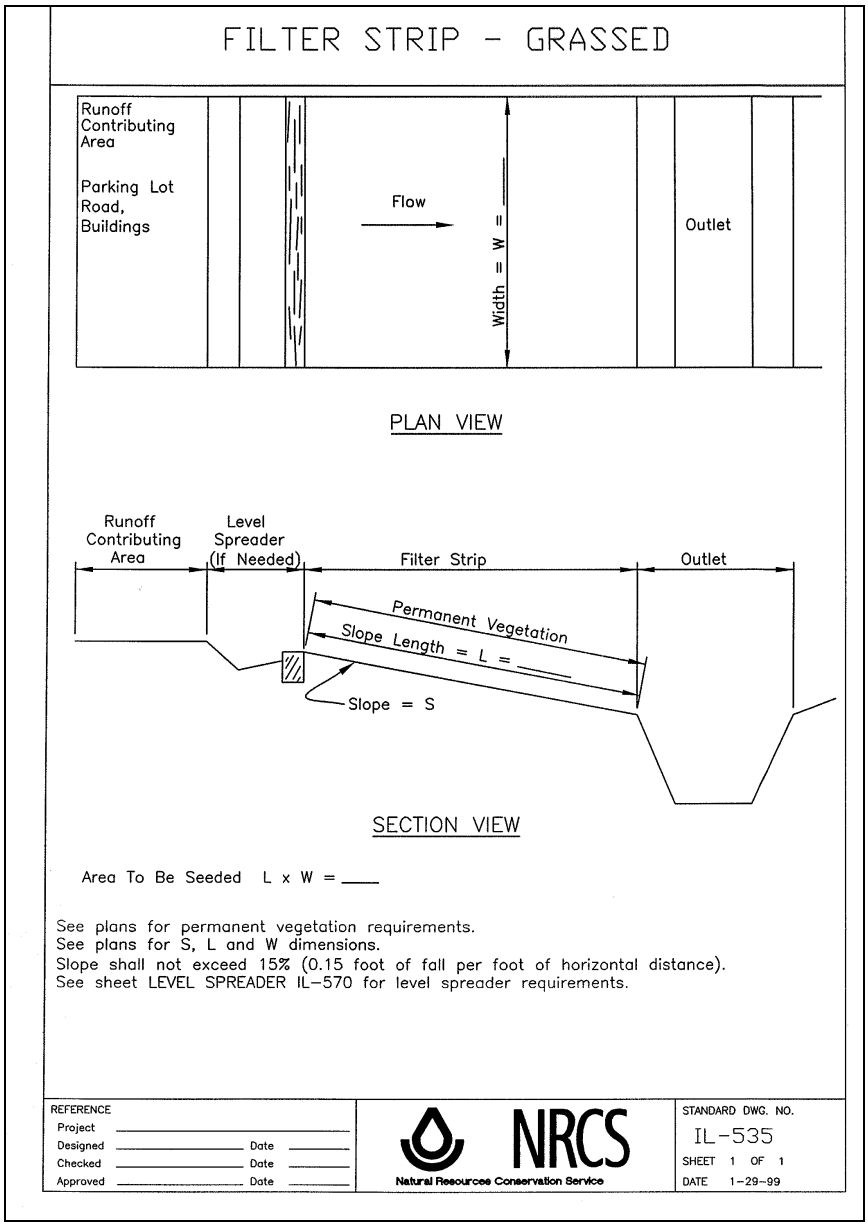


Figure 9.3.2
Filter Strip Installation Details (NRCS, 2002)

9.3.3 - Water Quality Inlets

This section focuses on relatively new structural BMPs that can treat stormwater by removing nonpoint source pollutants. Water Quality Inlets (WQIs) are inlet structures that have a series of chambers that can filter out substances such as oil, and allow sediment and other coarse materials to settle out. A typical WQI is shown in Figure 9.3.3.

WQIs can effectively control other nonpoint source pollutants such as hydrocarbons, and they have the ability to trap trash and other debris. The effectiveness of this type of BMP is directly related to the drainage area to which it is assigned. As with other BMPs discussed in this chapter, the recommended drainage area per WQI is one acre or less. Although WQIs are available as pre-manufactured units, they can be designed on-site as well to increase performance.

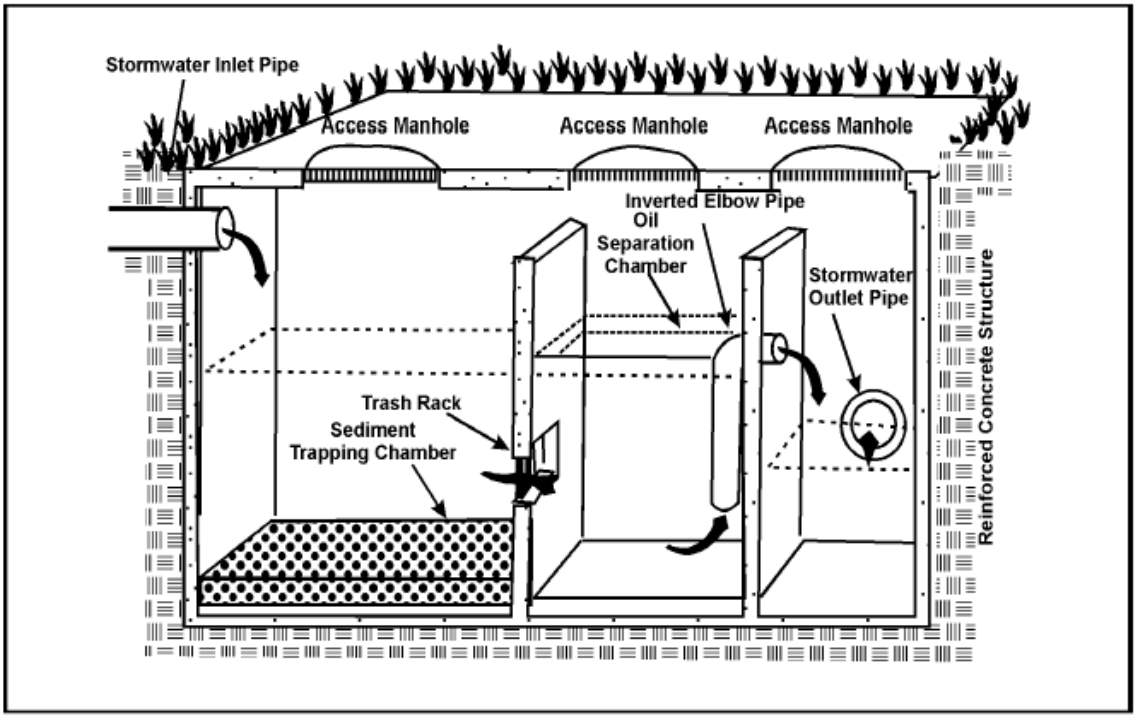


Figure 9.3.3
Typical WQI (EPA, 1991)

Routine maintenance has to be performed on WQIs to clean out sediment, trash and other debris, oil, and hydrocarbon by-products that have collected in the structure. The timeline of maintenance for a WQI is site-specific, and should be checked regularly. A properly maintained WQI should be relatively free of clogging and other separation problems.

Treatment Unit Description	Reference	TSS (%)	NO3-N (%)	Zn (%)	TPH-D (%)	Average Peak Flow Reduction (%)	Average Lag Time (Min.)
ADS Water Quality Unit	**UNH**	**66**	N/T	**74**	**47**	N/T	N/T
	www.ads-pipe.com	80	N/T	N/T			
ADS Infiltration Unit	**UNH**	**99**	N/T	**99**	**99**	**83**	**364**
Surface Sand Filter	**UNH**	**49**	**6**	**81**	**94**	**60**	**220**
	EPA: Sand Filters	70	N/T	45			
Sand Filter	Clayton & Schueler, 1996	85	N/T	71			
	Bell, W., et al, 1995	61–70	N/T	> 82			
Retention Pond	**UNH**	**81**	**64**	**92**	**61**	**85**	**554**
	EPA: Wet Detention Ponds	50–90	N/T	40–50			
	Winer, 2000	80 ± 27	43 ± 38				
Bioretention System	**UNH**	**97**	**44**	**99**	**99**	**85**	**615**
	EPA: Bioretention	90	N/T	N/T			
	Davis, et al, 1998	81	38				
	Winogradoff, 2001	N/T	N/T	87–99			
Aqua-Swirl and Aqua-Filter	**UNH**	**66**	**10**	**61**	**42**	N/T	N/T
	EPA website	84	N/T	N/T			
VortSentry	**UNH**	**29**	**37**	**42**	**53**	N/T	N/T
	Technical Bulletin 1	80	N/T	N/T			
V2B1 Structural System	**UNH**	**38**	**-43**	**35**	**40**	N/T	N/T
	www.env21.com	80					
Continuous Deflective Separation Unit	**UNH**	**41**	N/T	**26**	**26**	N/T	N/T
	various	52–84					
Gravel Wetland	**UNH**	**99**	**99**	**99**	**99**	**85**	**336**
	Clayton & Schueler, 1996	80–93	75–87	55–90			
Stone (Riprap) Swale	**UNH**	**52**	**-74**	**66**	**33**	N/T	N/T
Vegetated Swale	EPA: Vegetated Swales	81	38	71			
	Clayton & Scheuler, 1996	30–90	0–80	N/T			

Figure 9.3.4
Commonly Used BMPs and Their Approximate Benefit
(University of New Hampshire Stormwater Center, 2005)

Chapter 9 - REFERENCES

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LIST OF PARAMETERS

c, d, α, β	Regional coefficients for use in rainfall intensity-duration-frequency equations
A	Roof surface area
CN	Runoff Curve Number
C_W	Weir coefficient
D	Depth of green roof (in)
$\boldsymbol{\varepsilon}$	
E	Excess surface flow
f	Soil infiltration rate
h	Uniform height of surface water
I	The inflow rate of water (assumed to be equal to the product of the rainfall intensity over time increment ΔT) (in/hour or cm/hour)
i	Rainfall intensity
i	Time Step
K	Saturated Hydraulic Conductivity
O	Either the outlet pipes capacity or the rate of water infiltrating into permeable native soils (in/hour or cm/hour)
θ	Volumetric water content of green roof
θ_s	Water content of soil post rain fall
θ_{in}	Water content of soil pre rainfall
P	Rainfall depth
P_e	Excess rainfall from SCS Curve Number
Q	Infiltration rate
R	Runoff depth
S	Ultimate abstraction
t	Storm duration
T	Time Increment (hours)
T_p	Time to pond
T_r	Return period
W	Roof width
V	Barrel Volume
Z	Distance between the bottom of the basin and the water table

CHAPTER 10 - GREEN INFRASTRUCTURE

10.1 - INTRODUCTION

The term green infrastructure can be used to describe a number of concepts. The concept of green infrastructure can be used on a regional level to describe interconnected greenways and open spaces. Green infrastructure can be used on a neighborhood level to describe clustered development and the reduction of impervious surfaces. On the site level, green infrastructure has been applied to measures - some new, some old - that are focused on enhancing water quality and reducing the volume and rate of stormwater runoff (Jaffe et al, 2010). These measures are not necessarily capable of addressing flood flows, but rather, the more frequent storm events where there may be opportunities to trap and temporarily (or permanently) store stormwater runoff that would otherwise find its way into the combined sewers, storm sewers or waterways. These methods commonly utilize vegetation and soil to infiltrate runoff into the subsurface. The site level concept of green infrastructure is the focus of this chapter.

The various measures described in this chapter are those that have the most common applications and include porous pavements, green roofs, rain gardens and rain water harvesting systems. It has been recognized that all of these have water quality benefits as well as water quantity benefits. The water quantity benefits are the focus of this chapter. Water quality benefits associated with Green Infrastructure Treatment Trains are discussed in Section 10.7. This manual also differentiates between Best Management Practices (BMPs) and the measures described herein. The BMPs discussed in Chapter 9 are measures that have water quality benefits but no significant water quantity reduction benefits.

For each of the measures, a technical description is provided which conforms to both the Indiana and Illinois Department of Transportation specifications, followed by design considerations. The section will finish with an approach to quantifying the hydrologic benefits of the measures, and an example problem illustrating typical applications. At the end of the chapter, references are included that can be used by designers and planners who are considering implementing a green infrastructure program. Much of the information that is included in this chapter has been described elsewhere, but the focus of this chapter is to outline in one location the methods that are the most appropriate for planning, designing and permitting green infrastructure for stormwater quantity benefits.

10.2 - POROUS PAVEMENTS

Porous pavements are becoming more widely used as a component of stormwater management systems for conveyance, storage and/or groundwater recharge. Over the last 40 years the applications of porous pavements have been limited, but in the last decade there has been an increased awareness of the hydrologic benefits, and these materials are being considered as a part of green infrastructure.

Porous pavements allow rainwater to infiltrate into and through the surface, contrary to conventional pavement design, whose purpose is to minimize water infiltration by creating an

impervious barrier and diverting the rainfall into a collection system. Attention must be paid when designing porous pavement to carefully evaluate where the infiltrated rainwater is draining to and how much of the stormwater will be conveyed.

Benefits of porous pavements are increased stormwater infiltration, decreased surface runoff, improved water quality, and reduction in runoff velocity. Porous pavements are particularly important in filtering the first flush pollutants commonly observed at the beginning of a storm event. First flush pollutants are present on the land surface before the storm event and typically include car oil, gasoline, trash, road salt and suspended solids.

There are several locations in Illinois and Indiana where porous pavements have been used to demonstrate effectiveness and durability. The City of Chicago initiated the Green Alley program in 2006 and has converted hundreds of traditional alleys to Green Alleys, using permeable pavements and green lighting techniques (City of Chicago, 2010). U.S. Cellular Field, home to the Chicago White Sox, installed a 6-acre permeable paver parking lot in 2008, the largest installation to date (Davis, 2008). The Morton Arboretum (Lisle, Illinois) constructed a 5-acre parking lot in 2004 with permeable pavers, which is still meeting design infiltration capacity as of Fall 2015. These installations will be described herein as examples of where porous pavements have been successful.

Generally speaking, porous pavement design consists of three layers: the porous pavement surface, an aggregate base course and the underlying native soils. The pavement surfaces discussed in this manual are porous asphalt, pervious concrete and permeable pavers. This layer varies from 4” to 12” in depth depending on the type of surface and the anticipated loading. The aggregate base course typically is an 8” to 36” layer of aggregate with approximately 40% void space. The depth of this layer varies depending on the amount of storage desired and the anticipated loading.

All types of pavement must be designed to take into consideration the type of vehicle loading applied to the surface. Special consideration must be given to porous pavements along routes used by garbage and delivery trucks, or vehicles that frequently need to make sharp turns. Like any pavement, there are situations where improper construction techniques, improper aggregate base course preparation and lack of maintenance have resulted in surface failure. The design considerations included in this chapter address the lessons learned from these situations.

An underdrain may be required if the underlying soils have poor infiltration characteristics or if there is a high groundwater table on site. The underdrain must be shallow enough so that it can drain out by gravity. In a situation where an underdrain is required, it should be designed to prevent major ponding in the aggregate base course. Ponding may occur both laterally and horizontally. Underdrains need to be properly spaced to ensure proper drainage of the subsurface (Ferguson, 2005). To prevent the surrounding soils from entering into the aggregate base course, it is recommended that a pervious non-woven geotextile fabric is wrapped around the constructed concrete and over the uncompacted underlying soils.

Every one of the suppliers and many outside reviewers of this chapter have stressed the importance of proper maintenance of porous pavement which is noted in the manual. Routine maintenance is imperative to prevent clogging of the void space in the porous pavement. Low-pressure power washing followed by a vacuuming of the surface is recommended at least once a year, especially during the fall. Site drainage from adjoining landscaped areas should be designed to minimize direct runoff from these areas onto the porous asphalt (EPA, 2009). Porous pavement typically requires special winter weather maintenance considerations. Some types of porous surfaces cannot be treated with salt and other de-icing agents. Snow removal should also be performed with care to avoid damaging the pavement. Specific maintenance recommendations for the different types of surfaces are provided in the following sections. Note that each supplier has literature outlining proper installation and maintenance of their products.

The following are general design considerations that apply to all types of porous pavement. In the following sections, information on the different types of pavements is provided and specific considerations are outlined. The following sections also provide information on the use of the material, the applications of four different types of porous pavements and suggestions on how to quantify the hydrologic benefits of the porous pavements.

DESIGN CONSIDERATIONS FOR POROUS PAVEMENTS

1. Porous pavement should be used with underlying soils that have infiltration rates greater than 0.5 inches/hour to avoid stormwater surface ponding (EPA, 1999). If the infiltration rates are less than 0.5 inches/hour, a subsurface drainage pipe may be used to convey the stormwater away from the aggregate base course.

2. Routine maintenance such as power washing and street sweeping can prevent the void space from clogging with debris and sediment. Direct runoff from landscaped areas onto the porous pavement surface should be minimized.

3. Signage and other documentation that describes the porous pavement is important so that future owners or operators of the facility are aware and do not apply sealers or other chemicals that could damage the pavement or fill in the voids. The signage also offers a public education opportunity.

4. Consideration should be given to placing the pavement area within a stormwater easement if the installation is going to be part of a stormwater management system.

5. A 10-foot setback from down-gradient building foundations and a 100-foot setback from up-gradient building foundations are recommended to minimize impacts to the building foundation (EPA, 1999).

6. The porous pavement should be setback at least 100 feet from a potable water supply well (EPA, 1999).

7. Plastic or rubber shields on snow plow blades are recommended to reduce damage to the porous pavement during winter months (Obla, 2007). If a normal blade is to be used, it should be set a few inches higher than the porous pavement to prevent any damage to the surface.

10.2.1 - POROUS ASPHALT

Porous asphalt is standard hot-mix asphalt with reduced sand or fines that permits water infiltration. Guidelines have been developed by the Asphalt Pavement Association of Indiana (APAI) for applying porous asphalt on Indiana projects and include standards for material, mix design formula, mix design and general construction requirements. Traditional asphalt pavement design includes a sub-grade, followed by an aggregate base course and bituminous binder and surface course, which are designed for a specific loading and pavement life. The aggregate base course in permeable asphalt contains larger void space than traditional asphalt and is followed by lifts of the permeable asphalt. An installation of a typical porous asphalt parking lot is shown in Figure 10.2.1 and a typical cross-section is shown in Figure 10.2.2. Porous asphalt installations over an aggregate storage bed will reduce stormwater runoff volume, rate and pollutants. Porous asphalt can be a durable and cost competitive alternative to conventional asphalt when constructed and maintained properly (EPA, 2010).



Figure 10.2.1
Example of a Porous Asphalt Parking Lot

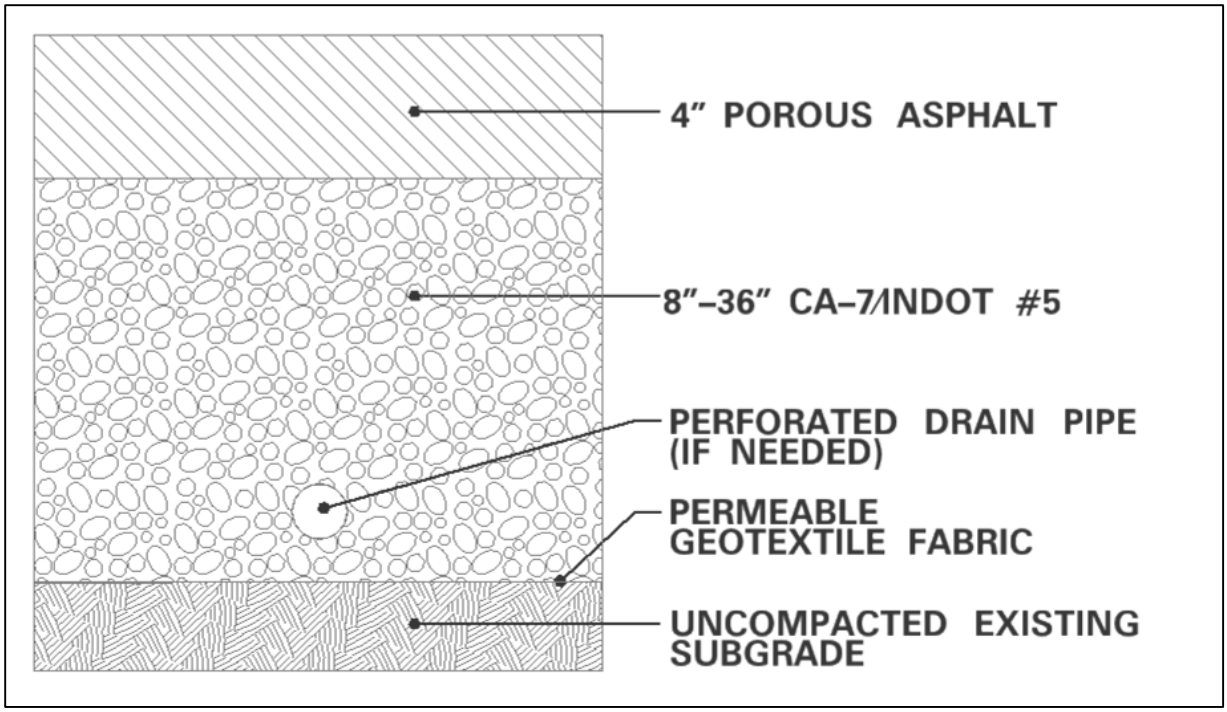


Figure 10.2.2
Typical Porous Asphalt Cross-Section Detail for Parking Lot

DESIGN CONSIDERATIONS FOR POROUS ASPHALT

1. For parking lots where occasional loading from delivery trucks and other types of short term heavy loads are anticipated, the typical section should include:

    a. 4" of Porous Asphalt
    b. 8 to 36" of Aggregate Base Course*

* The aggregate base course depth may vary depending on desired storage volume (Iowa, 2009).

2. For areas where more frequent loading from garbage trucks and other types of heavy vehicles is expected, the typical section should include:

    a. 6" of Porous Asphalt
    b. 8 to 36" of Aggregate Base Course*

* The aggregate base course depth may vary depending on desired storage volume (Iowa, 2009).

3. “Seal coating” of asphalt surfaces is a common maintenance technique used in the Midwest. The process of seal coating eliminates porous asphalt’s high infiltration capacity; therefore, seal coating or any similar maintenance techniques should never be performed on porous asphalt.

4. Terraced aggregate base course is recommended for porous asphalt surfaces with slopes greater than 2% to ensure maximum infiltration and to minimize scour and erosion of the aggregate base course (EPA, 2009).

10.2.2 - PERVIOUS CONCRETE

Pervious concrete, also referred to as porous, gap-graded or enhanced porosity concrete, is a concrete aggregate mixture that, when finished, has high permeability characteristics (Obla, 2007). This is accomplished by reducing the amount of fine aggregate that fills the voids between the coarser aggregate in normal Portland concrete cement mixes. In addition, the paste that typically fills the remaining voids is reduced, resulting in a concrete that has an average porosity of about 20% (Obla, 2007). This level of porosity is achieved by using coarse aggregate (CA) between 3/4” and 3/8”, which corresponds to Illinois Department of Transportation, CA-10 and Indiana Highway Department, INDOT-9 specifications. In normal applications of concrete, the amount of slump present is very important; however, it is not a performance specification for pervious concrete. Typically, pervious concretes achieve compressive strengths of about 2,500 psi. The typical specific weight of pervious concrete is 120 lb/ft^3. The American Society for Testing and Materials (ASTM) has developed standards to test density and void content of freshly mixed pervious concrete (ASTM C1688 / C1688M - 10a), as well as a test to determine the infiltration rate of in-place pervious concrete (ASTM C1701 / C1701M-09).

One benefit of pervious concrete is that it can be placed directly on a pervious aggregate base. Figure 10.2.3 is a typical pervious concrete cross-section that is commonly shown. The depth of the aggregate base is dictated by the volume of storage that is being provided, the depth to suitable soils and the localized ground water levels. An example of a pervious concrete parking lot is shown in Figure 10.2.4.

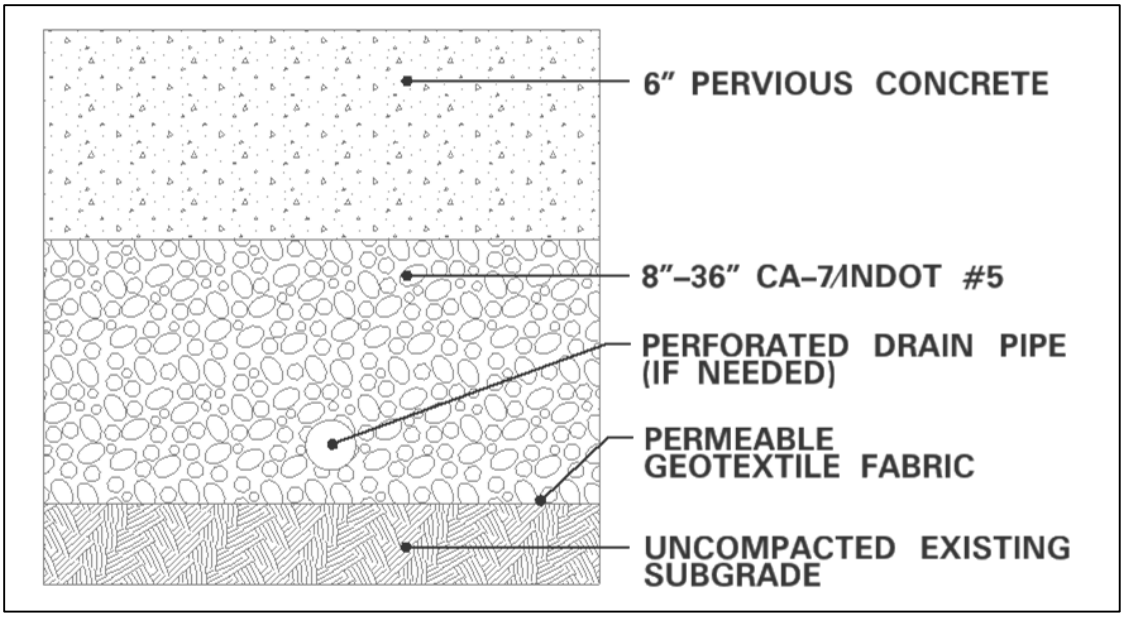


Figure 10.2.3
Typical Pervious Concrete Cross-Section Detail for Parking Lot



Figure 10.2.4
Example of Pervious Concrete Parking Lot (Lutey, 2011)

DESIGN CONSIDERATIONS FOR PERVIOUS CONCRETE

For any pavement surface, there are certain general design considerations that must be taken into consideration. Some are common to all porous pavements, while others are more specific to pervious concrete.

1. The general application of pervious concrete for parking lots that will be designed for loading from cars and trucks with less than 18,000 lbs axle load should be:

    a. 5 to 6" of Pervious Concrete
    b. 8 of 36" of Aggregate Base Course*

    * The aggregate base course depth is based on a design of the required storage needed in the subsurface. The 8" requirement is a minimum for structural support (National Concrete Pavement Technology Center, 2006). The aggregate base course depth may vary depending on desired storage volume.

2. The general application of pervious concrete for more frequent traffic and heavier loads, such as garbage trucks, is:

    a. 8 to 12" of Pervious Concrete
    b. 8 to 36" of Aggregate Base Course*

    * The aggregate base course depth is based on a design of the required storage needed in the subsurface. The 8" requirement is a minimum for structural support (National Concrete Pavement Technology Center, 2006). The aggregate base course depth may vary depending on desired storage volume.

3. An aggregate base course depth of 6 to 8" should be added to the design depth in areas with deep frost depths (Obla, 2007).

4. Vacuuming should be the primary method of maintenance on pervious concrete installations. Pressure washing can be used in cases where vacuuming is not able to restore the permeability of the surface (Lutey, 2011). Surface damage can be caused by high pressure power washing (Obla, 2007). Street sweeping may be beneficial to remove large debris from the surface, especially during the fall.

5. Avoid the direct application of any de-icing chemicals to the surface due to their corrosive nature to new, immature concrete (Lutey, 2011).

6. The slope of the surface pervious concrete should be minimized to limit damage from snow plows.

7. The slope of the sub-base of the pervious concrete installation should be less than 5% to ensure maximum infiltration and to minimize scour and erosion (Lutey, 2011).

10.2.3 - PERMEABLE PAVERS

Permeable pavers consist of concrete paver bricks which have gaps that are filled with stone chips to allow for the infiltration of stormwater into the subbase. An example of a permeable paver parking lot installation can be seen in Figure 10.2.5. Due to the variation in permeable paver design, the hydrologic minimum standard for a paver to be considered permeable is an infiltration rate of 50 in/hr, but can be as high as 120 in/hr (Unilock, 2011). A typical permeable paver cross-section is shown in Figure 10.2.6.



Figure 10.2.5
Example of Permeable Paver Parking Lot Installation at The Morton Arboretum, Lisle, IL

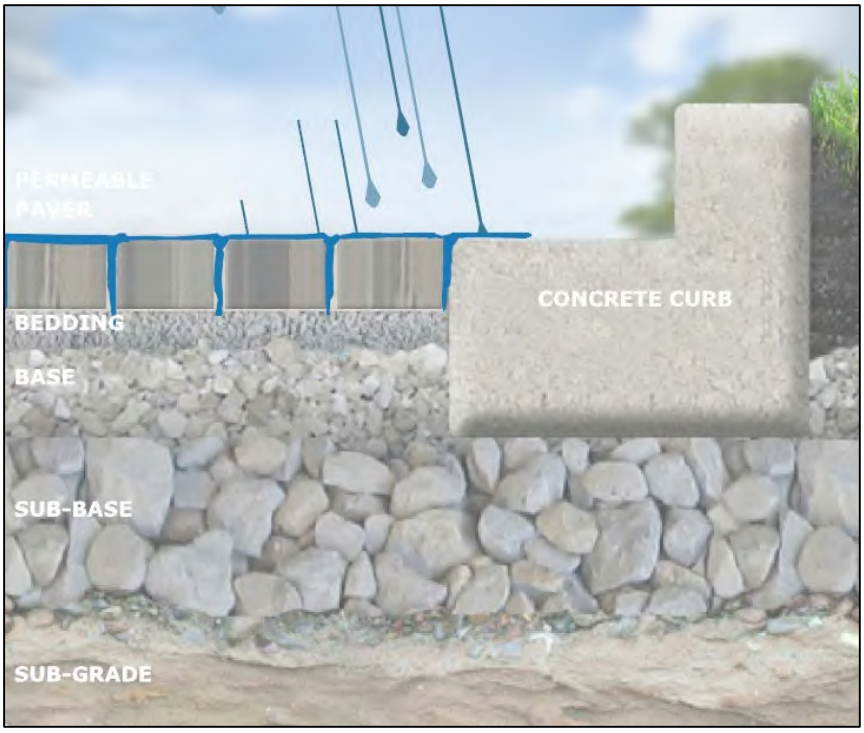


Figure 10.2.6
Example of a Permeable Paver Parking Lot Cross-Section (APT, 2011)

Maintenance of permeable pavers is paramount and can often be more difficult than some of the previously discussed permeable surfaces. Without proper maintenance, small debris can collect in the gaps between paver bricks. Additionally, it can be difficult to vacuum and power wash the surface without dislodging the chips that are used to fill the gaps.

The example of the Morton Arboretum was previously mentioned, but this example bears mentioning again. The performance of this parking lot has been monitored closely since construction and is performing to expectations. The parking lot design included heavier non-permeable pavers where bus traffic or delivery trucks were anticipated. The subsurface drainage of this particular parking lot also benefits from a sand/gravel layer to allow for rapid drainage of the infiltrated water. Even after ten years

DESIGN CONSIDERATIONS FOR PERMEABLE PAVERS

1. Detailed information for permeable paver design is shown Table 10.2.1. A typical permeable paver design includes the following:

    a. 3.15” Pavers*
    b. 1.5” of Setting Bed
    c. 8 to 36” of Aggregate Base Course**

* There are ASTM standards that all pavers must satisfy from a material, durability and strength perspective.
** The aggregate base course should be greater if it is to be used for the storage of stormwater (Unilock, 2011).

2. The permeable paver surface can be vacuumed to remove any clogs, but care should be taken to avoid removal of stone chips. Occasional weeding and spraying may be necessary to prevent growth in the gaps between the pavers. The stone chips may need to be replenished occasionally.

3. The surface of the permeable pavers should not be sloped greater than 5% (EPA, 1999).

Table 10.2.1
Suggested Permeable Paver Design (Unilock, 2011)

TYPICAL PAVEMENT STRUCTURE BY SUBGRADE SOIL TYPE

Loading Conditions	Material	Subgrade Soil Type									
		Granular Suitable as Borrow		Silty Sand (Silt and Fine Sand less than 40%)		Silty Sand (Silt and Fine Sand 40 - 60%)		Silt (Silt and Fine Sand greater than 60%)		Clay	
		in.	mm	in.	mm	in.	mm	in.	mm	in.	mm
Pedestrian Use	Pavers	2 3/8 - 2 ¾	60 - 70	2 3/8 - 2 ¾	60 - 70	2 3/8 - 2 ¾	60 - 70	2 3/8 - 2 ¾	60 - 70	2 3/8 - 2 ¾	60 - 70
	Bedding Sand	1 ¼	30	1 ¼	30	1 ¼	30	1 ¼	30	1 ¼	30
	Base	6	150	6	150	6	150	6	150	6	150
	Subbase										
Light Duty (Driveways, Car Parking Areas)	Pavers	2 3/8 - 3 1/8	60 - 80	2 3/8 - 3 1/8	60 - 80	2 3/8 - 3 1/8	60 - 80	2 3/8 - 3 1/8	60 - 80	2 3/8 - 3 1/8	60 - 80
	Bedding Sand	1 ¼	30	1 ¼	30	1 ¼	30	1 ¼	30	1 ¼	30
	Base	6*	150	8	200	6	150	6	150	6	150
	Subbase					12	300	20	500	20	500
Minor Residential Roads	Pavers	3 1/8	80	3 1/8	80	3 1/8	80	3 1/8	80	3 1/8	80
	Bedding Sand	1 ¼	30	1 ¼	30	1 ¼	30	1 ¼	30	1 ¼	30
	Base	6*	150	9	215	6	150	6	150	6	150
	Subbase					15	375	24	600	24	600
Residential and Collector Streets (25 to 500 ESAL/day)	Pavers	3 1/8	80	3 1/8	80	3 1/8	80	3 1/8	80	3 1/8	80
	Bedding Sand	1 ¼	30	1 ¼	30	1 ¼	30	1 ¼	30	1 ¼	30
	Base	6*	150	6	150	6	150	6	150	6	150
	Subbase			11	275	17	425	26	660	26	660
Medium to Heavy Industrial Areas (500 to 1000 ESAL/day)	Pavers	3 1/8	80	3 1/8	80	3 1/8	80	3 1/8	80	3 1/8	80
	Bedding Sand	1 ¼	30	1 ¼	30	1 ¼	30	1 ¼	30	1 ¼	30
	Base	6*	150	6	150	6	150	6	150	6	150
	Subbase			12	300	18	450	29	720	29	720
Heavy Industrial Areas (1000 to 1500 ESAL/day)	Pavers	3 1/8	80	3 1/8	80	3 1/8	80	3 1/8	80	3 1/8	80
	Bedding Sand	1 ¼	30	1 ¼	30	1 ¼	30	1 ¼	30	1 ¼	30
	Base	6*	150	6	150	6	150	6	150	6	150
	Subbase			14	340	20	490	30	740	30	740

* Minimum recommended base course thickness.

10.2.4 - INFILTRATION THROUGH POROUS PAVEMENTS

The main benefit of permeable pavements is the infiltration capacity of the pavement and the underlying subbase. Rainfall is allowed to percolate through the pavement rather than become surface runoff, as it would with traditional pavements. Although the water quality benefits of permeable pavements can be significant, as stated before, this chapter focuses on the water quantity benefits and how to quantify them.

In the majority of storm events, porous pavements have the infiltration capacity to absorb large quantities of water. Recent research has demonstrated that the surface is not the limiting factor and in fact can accommodate rainfall intensities which exceed the 100-year event (Burke, Papas, Rao, 2010, USEPA 2010). One study evaluated permeable pavers in a lab scenario and in the field, and concluded that the pavers produced little to no runoff. In the lab, the 10-year, 30-minute design storm (3.3 in/hr) was reproduced, and little to no runoff was observed. Table 10.2.2 summarizes the runoff observed in the field scenario (Burke, Papas, Rao, 2010).

Table 10.2.2
Runoff from a Permeable Paver Installation (Burke, Papas, Rao, 2010)

Event #	*Date*	*Storm Duration (hours)*	*Total Rainfall (inches)*	*Observed Parking Lot Runoff (inches)*
1	6/11/2009	3.5	0.34	0.01
2	6/16/2009	4.5	0.92	0.01
3	6/16/2009	1.5	0.19	0.00
4	6/19/2009	1.5	0.42	0.00
5	6/22/2009	1.5	0.18	0.00

A study by the U.S. Environmental Protection Agency (EPA) compared the infiltration capacity of pervious concrete, permeable pavers and porous asphalt. The three surfaces were all used in the same newly constructed parking lot, and measurements were taken in the field. The report concluded that although each surface had varying infiltration capacities, the rates were well above the design storm events.

Table 10.2.3 summarizes the observed mean infiltration rates from this study over a six-month period (November 2009 to April 2010).

Table 10.2.3
Infiltration Rates of Permeable Surfaces (Borst et al, 2010)

Surface	*Observed Mean Infiltration Rates (inches/hr)*
Pervious Concrete	1,575
Permeable Pavers	945
Porous Asphalt	80

10.2.5 - EVALUATION OF STORAGE UNDER POROUS PAVEMENTS

The storage volume of the voids below the porous pavement is one of the most important components of porous pavements. The storage volume is determined by the volume of the voids, the permeability of the underlying soils, and the type of subsurface conditions encountered. When a porous pavement, specifically pervious concrete, is constructed on compacted soil, the infiltration rate is 1.67 in/hour (Chopra, 2011). This compares to infiltration rates which can reach 100+ in/hr when the porous pavement is constructed on permeable subgrade.

The limiting hydrologic factor within a permeable surface system is the stone base, not the surface infiltration rate. The infiltrating rainfall will flow downward and fill voids in the stone. If the bottom of the excavation is a porous material like sand or stone, then a portion of the infiltrating rainfall can flow downward and into the permeable native soils. In this situation, the rainfall infiltrates into the groundwater as shown in Figure 10.2.7.

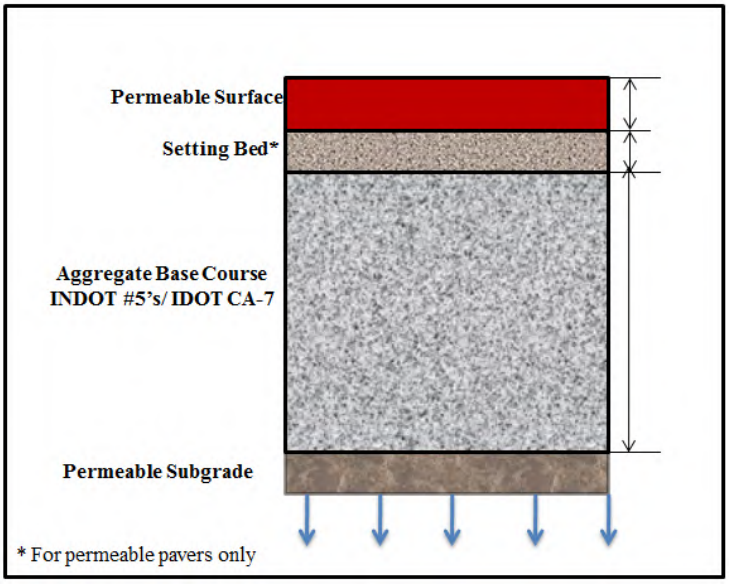


Figure 10.2.7
Permeable Subgrade Condition

If the material at the bottom of the excavation is clay or saturated soil, then the storage capacity is limited to the volume of the stone voids. An underdrain can be placed in the permeable pavement system to alleviate this problem (Figure 10.2.8). When an underdrain is installed, the infiltrating rainfall is attenuated and the subsurface acts similarly to a stormwater storage facility which temporarily stores the runoff. The aggregate base course under the permeable pavement acts like a porous soil which allows for infiltration until field capacity is reached.

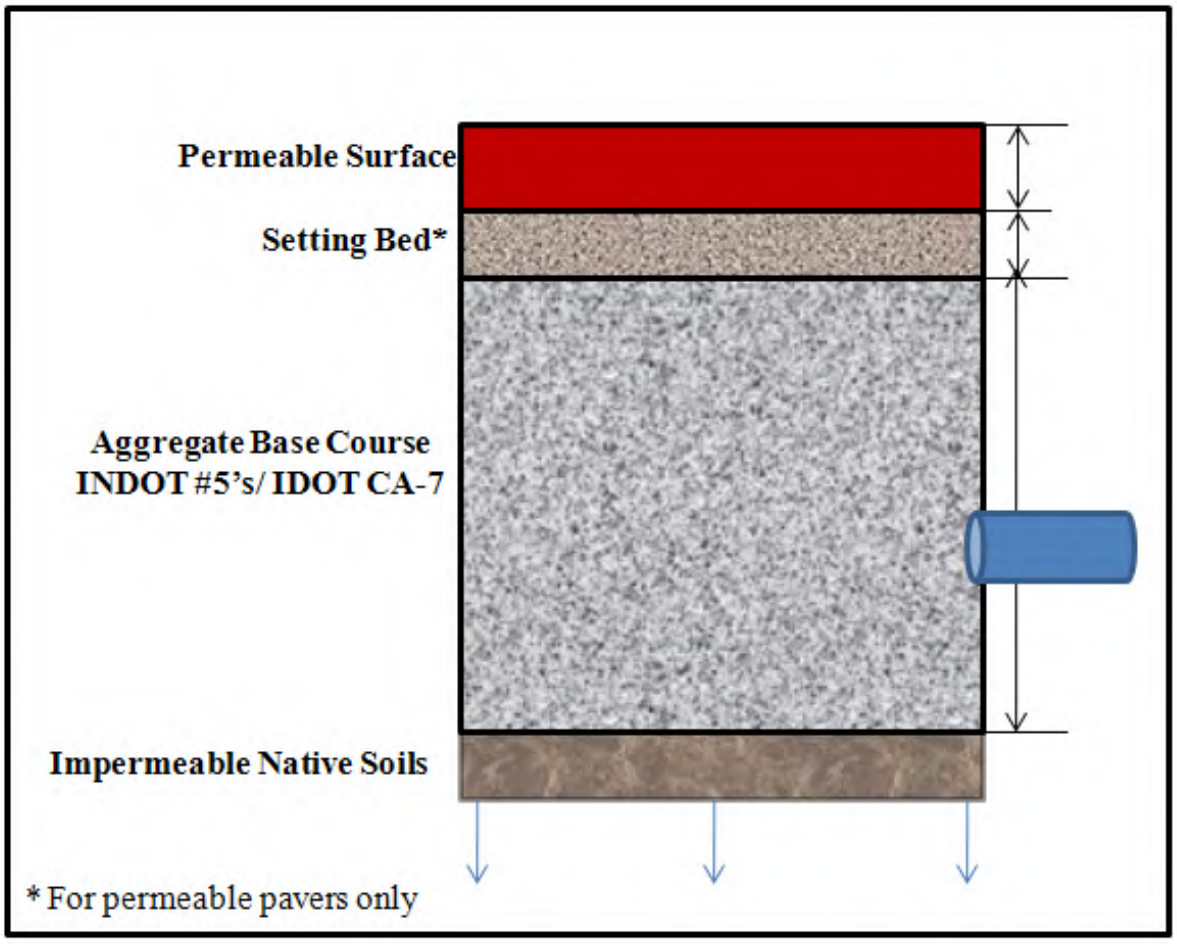


Figure 10.2.8
Impermeable Subgrade Condition

10.2.6 - QUANTIFICATION OF VOIDS

For the purposes of current application, the porosity is the ratio of voids to the total volume of aggregate base course. A 2011 report conducted by StormTech compared the porosity of various samples of angular crushed stone used for stormwater applications. The measured porosity varied from 35.3% to 54%, as shown in Table 10.2.4. The voids in the aggregate base course are where the infiltrated rainfall is temporarily stored. The greater the ratio of the voids, the more water can be stored in the aggregate base course. The porosity of the CA-7 (IDOT) and #5 (INDOT) aggregate have a value of 0.36.

Table 10.2.4
Measured Porosity of Angular Crushed Stone (StormTech, 2011)

Sample Type	*Data Source*	*Porosity*	*Bulk Density (lbs/ft^3)*
AASHTO # 4	StormTech lab	39.9%	94.3
AASHTO # 57	StormTech lab	45.4%	87.2
AASHTO # 4	StormTech lab	37.4%	103.0
AASHTO # 57	StormTech lab	38.7%	97.7
AASHTO # 57	NTH lab 50	50-51%	n/a
AASHTO # 57	NTH lab 50	50-52%	n/a
AASHTO # 3	NTH lab 53	53-54%	n/a
1 ½”	Anderson Eng. Cons.	41.9%	96.8
1 ½”	Anderson Eng. Cons.	35.3%	101.7
1 ½”	Anderson Eng. Cons.	37.8%	98.6
1 ½”	Anderson Eng. Cons.	41.3%	93.6
1 ½”	Anderson Eng. Cons.	38.2%	98.7
3/4”	Anderson Eng. Cons.	38.5%	100.3
3/4”	Anderson Eng. Cons.	38.9%	97.9
AASHTO #4	Universal Eng. Serv.	44.3%	78.6
AASHTO #57	Universal Eng. Serv.	43.2%	79.8
AASHTO #4	Universal Eng. Serv.	46.1%	70.8
AASHTO #57	Universal Eng. Serv.	42.8%	74.8
1 ½”	CTL Thompson TX	46%	90.5
1”	CTL Thompson TX	45%	91.6
1 ½”	CTL Thompson TX	48%	77.1

10.2.7 - HYDROLOGIC DESIGN CONSIDERATIONS

The subsurface conditions previously shown represent different hydrologic processes, resulting in the following modeling options when evaluating the water quantity benefits of porous pavements.

1. The curve number or runoff coefficient for porous surfaces used in the hydrologic analysis is less than the curve number used for impermeable surfaces (CN=98). This approach has been used by some communities and counties. In these situations the curve numbers were as follows for water quantity calculations:

   CN = 91

2. If the voids are situated on permeable soils or have an underdrain that has been properly sized to meet release rate requirements, full credit for the storage can be recognized and credited towards the required stormwater storage volume. The drainage of the water is important so that the storage is effective for all rainfall events. If the infiltrated rainfall cannot be drained out of the aggregate base course within 48 hours, the void ratios must be adjusted accordingly.

   As described in Section 10.2.4, a porosity value of no greater than 0.36 is suggested when estimating the void space. If the stone material being used has porosity less than 0.36, then 90% of the measured porosity should be used as a safety measure to account for any filling of the voids.

   Additional credit may be given for void storage but this is done on a case-by-case basis.

3. The third possible type of analysis is to treat the volume of voids below the permeable surface as a reservoir and to evaluate it using a reservoir-routing modeling approach, as shown in the following equation:

   $$I - O = \frac{\Delta S}{\Delta t}$$

   Where

   - I = The inflow rate of water (assumed to be equal to the product of the rainfall intensity over time increment Δt).
   - O = The outlet pipe's capacity or the rate of water infiltrating into permeable native soils. The outflow rate will also be masked by the flow attenuation of the stone layer. The velocities will be significantly less than that of an open storage facility.
   - S = Storage in voids.

A stage-storage relationship for the storage volume in the stone and either a stage-discharge relationship for the discharge pipe or a stage-discharge relationship for the underlying soil must be developed.

This third approach would require the use of Darcy's law and the information on the soil saturated hydraulic conductivity. Selection of values must account for the depth of groundwater and its influence on the level of saturation. This procedure is generally only applicable when the volume of voids is considerable and a large amount of offsite flow is discharging onto the permeable surface.

10.2 - EXAMPLE PROBLEM

Example 10.2.1

Determine the volume of storage provided below a porous pavement parking lot with dimensions of 200' by 200'and for aggregate base course depths of 6", 12" and 18". Assume a void ratio of 0.36 and an initial water content of zero. Neglect infiltration into the underlying soil. Assume that only the parking lot itself is draining onto the pavement surface (no runoff from adjoining areas) and that the infiltration rate of the porous pavement exceeds the rainfall intensity. The parking lot is located in Evansville, Indiana. Use the IDF equation presented in Chapter 2 to determine the 100-, 10- and 5-year, 24-hour rainfall depths.

Step 1: Using Equation 2.2.13 and the coefficients presented in Table 2.2.2 calculate the 24-hour rainfall depths for the 100-, 10- and 5-year storm events for Evansville.

$$i = \frac{cT_r^{\alpha}}{(t+d)^{\beta}}$$ Equation 2.2.13

$$P = i\ x\ d$$

P_{24}^{100} = 6.47 inches
P_{24}^{10} = 3.93 inches
P_{24}^{5} = 3.38 inches

Step 2: Calculate the stormwater runoff volume using Equation 3.3.1 and a CN of 98.

$$S = (\frac{1000}{CN}) - 10$$ Equation 3.3.2

S_{98} = 0.20

$$R(t) = \frac{\left((P(t)-0.2S)^2\right)}{P(t)+0.8S}$$ Equation 3.3.1

R_{24}^{100} = 6.23 inches
R_{24}^{10} = 3.70 inches

R_{24}^{5} = 3.15 inches

Multiply the runoff by the area of the parking lot to determine the total volume of the runoff.

$$Q_v = P(t)\ x\ \frac{ft}{12\ inches}\ x\ 200\ ft\ x\ 200\ ft$$

Q_{24}^{100} = 20,778 ft^3
Q_{24}^{10} = 12,319 ft^3
Q_{24}^{5} = 10,498 ft^3

Step 3: Compute the volume of storage provided below the parking lot for the proposed sub-base depths.

$$V_{6"} = 200\ ft\ x\ 200\ ft\ x\ 6"\ x\ \frac{ft}{12"}\ x\ 0.36$$

$V_{6"}$ = 7,200 ft^3
$V_{12"}$ = 14,400 ft^3
$V_{18"}$ = 21,600 ft^3

The depth of the aggregated base course in the porous pavement parking lot required to store the volume of the 24-hour, 100-year storm event is approximately 18".

10.3 - GREEN ROOFS

The Philadelphia Water Department defines a green roof as a roof that has been covered with living vegetation that will provide stormwater management and other benefits. Although it is typical for a green roof to be on top of a structure, it can in fact be at grade. One example of an at grade green roof is Millennium Park in Chicago as it covers a large parking structure. Green roofs are recognized as an important low impact development (LID) because of their many benefits (Peck & Bruce, 2007). Green roofs are also referred to as vegetated roofs in the literature.

The true history of green roofs is difficult to trace. The first homes and structures were constructed out of natural materials and would often include some sort of greenery on the roof. In modern times, Germans were the first to implement green roofs. In 2003, approximately 14% of all flat roofs in Germany had been converted green roofs (VanWoert et al., 2005). Over the past fifteen years, green roofs have started to emerge across the U.S. in anything from simple housing developments, to skyscrapers, to large industrial buildings.

There are numerous benefits to installing green roofs, including the aesthetic and air quality benefits (see Figure 10.3.1 for an example). Green roofs also provide insulation for the structure below, in addition to protecting it from the elements (severe temperature differences

and ultraviolet radiation). Green roofs also typically have much lower surface temperatures than traditional roofs, and reduce the heat island affect seen in many urban areas.

One of the important benefits is stormwater runoff mitigation, which is the focus of this section. Green roofs slow down the runoff from an otherwise impermeable surface. The type of green roof and the vegetation used have a significant impact on the extent of runoff mitigation. Post (2007) states that green roofs can reduce runoff by 65 to 94%. A study performed by Carter and Rasmussen (2006) found that the average stormwater retention of a green roof is approximately 78%.



Figure 10.3.1
Chicago City Hall Rooftop Garden (City of Chicago)

Two factors that affect the ability of the green roof's surface to infiltrate water are the slope and media depth. A study performed by VanWoert et al. (2005) compared various media depths and slopes to determine their effects on stormwater retention. Not surprisingly, the study found that a regular gravel ballast roof retained less water than a green roof. The study also found that there was not a significant difference in stormwater benefits between roofs with only growing media and roofs with growing media and vegetation. The slope was shown to have an effect on the amount of runoff that was retained, especially for low- to medium-event storms.

Standards for green roofs are evolving. These standards primarily address the structural loading and design aspects of their construction. Standards for properly quantifying stormwater runoff are sparse. A series of research papers produced by Carter and Rasmussen (2006) estimated

the NRCS Curve Number between 86 and 88, depending on the green roof, compared to 98 for a non-green roof. Taylor and Ganges (2004) outlined a method for quantifying runoff reduction of green roofs using theoretical Att-Kin storage coefficients.

When designing a green roof, another important design consideration is the loading capacity of the structure below. The types of loading conditions and roof-drain clogging scenarios are especially important to consider.

10.3 - EXAMPLE PROBLEMS

Example 10.3.1

An extensive green roof is to be constructed on top of an existing warehouse in Indianapolis. The roof area is approximately 9,500 ft^2. Use the NRCS Curve Number Method to compare the approximate runoff volume before and after the installation of the green roof for a 50-year, 2-hour storm event.

Table 10.3.3
Curve Number Estimates for Example 10.3.1

Roof Cover	*Curve Number*
Typical Roof	98
Green Roof	86

Using the IDF Equation given in Chapter 2, the rainfall intensity can be estimated and the rainfall depth, P, evaluated.

$$i = \frac{cT_R^{\propto}}{(t+d)^{\beta}}$$

$$i = \frac{1.5899 * 50^{0.2771}}{(2+0.725)^{0.8797}} = 1.95\ in/hr$$

$$P[t] = (1.95\ in/hr) * 2\ hrs = 3.89\ in$$

Now the NRCS Curve Number Method from Chapter 3 can be used to determine the amount of runoff.

$$S = \frac{1000}{CN} - 10$$
$$S_{\text{Typical roof}} = 0.20$$
$$S_{\text{Green roof}} = 1.63$$

Using equation 3.3.1, runoff for the two cases can be estimated.

Table 10.3.4
Results for Example 10.3.1

Roof Cover	*Runoff (in)*	*Total Runoff Volume (ft^3)*
Typical Roof	3.66	2,897
Green Roof	2.45	1,936

The change in roof cover results in a decrease in the runoff volume by approximately 33% when using the SCS curve number method.

Figure 10.3.2 below shows that as the return period increases, the green roof's ability to reduce surface runoff volume decreases.

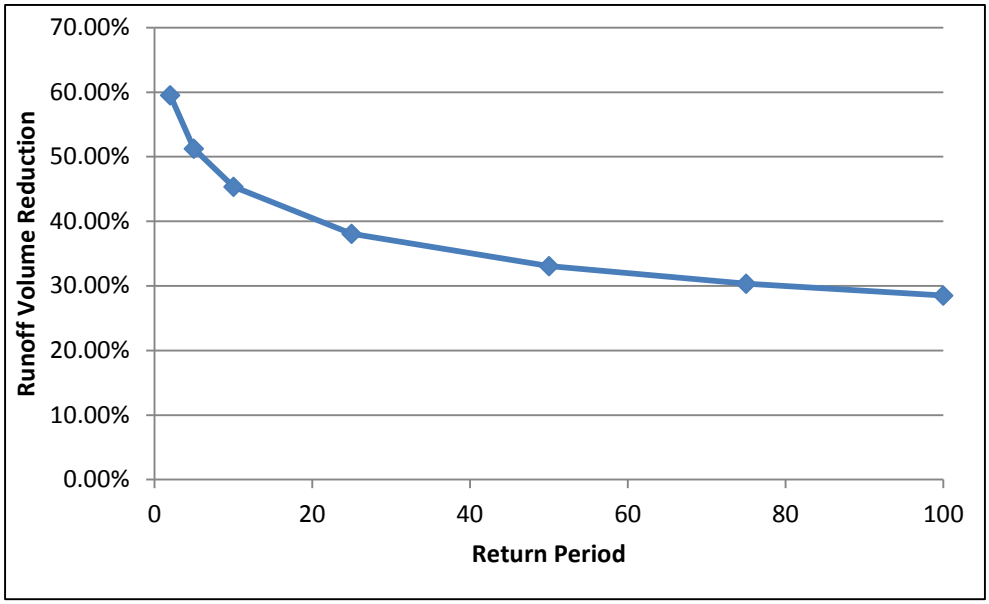


Figure 10.3.2
Runoff Volume Reduction versus Return Period

10.4 - INFILTRATION BASINS

An infiltration basin is an engineered temporary storage area that allows stormwater runoff to gradually infiltrate through a porous media or gravel stones. Akan and Houghtalen (2003) describe an infiltration basin as, “a water impoundment that does not have an outlet other than an emergency spillway.” The State of Iowa describes an infiltration basin as, “dry ponds constructed to allow infiltration to occur simultaneously with other treatment processes.” Often, infiltration basins may be located at the outlet point of a development to offer treatment before release or infiltration (Iowa Stormwater Management Manual, 2007).

There are a few variations on the traditional infiltration basin. A traditional infiltration basin (Figure 10.4.1) has a bottom consisting of soil or gravel. A naturalized basin (Figure 10.4.2), on the other hand, has natural plant species at the base or bottom of the basin.



Figure 10.4.1
Traditional Infiltration Basin (Improving Stormwater Basin Design, 2008)



Figure 10.4.2
Naturalized Infiltration Basin (Improving Stormwater Basin Design, 2008)

Infiltration basins have been an important aspect of responsible design for the past ten years. There are many benefits to infiltration basins. Both naturalized and traditional infiltration basins can improve water quality, increase groundwater infiltration, decrease stormwater runoff quantity and decrease stormwater runoff velocity. The plants in naturalized basins are able to promote water infiltration, thereby improving water quality. It should be noted that native and other easily-maintained plants should be preferred within the basin due to their deep root structure, which allows greater water infiltration and adaptability to a variety of climate

conditions. Additionally, infiltration basins should only be used at appropriate sites. A preliminary analysis should look at the existing soils, geology and groundwater conditions to see if they are favorable for infiltration basins.

Many municipalities and other agencies have developed design standards for infiltration basins. Often these standards can be found under Best Management Practices Standards. The material that lines the bottom of the basin will affect the amount of infiltration that will occur. Table 10.4.1 (also found in Chapter 6 of this manual) summarizes the average permeability associated with typical granular materials used in infiltration basin design.

Table 10.4.1
Soil Characteristics

Typical Soil	*Permeability, K (ft/sec)*	*Relative Permeability*
Coarse, Gravel	Over 5.0×10^{-1}	Very Permeable
Sand, Fine Sand	5.0×10^{1} to 5.0×10^{-3}	Medium Permeability
Silty Sand, Dirty Sand	5.0×10^{-3} to 5.0×10^{-5}	Low Permeability
Silt	5.0×10^{-5} to 5.0×10^{-7}	Very Low Permeability
Clay	Less than 5.0×10^{-7}	Practically Impervious

Over the lifetime of an infiltration basin, debris and other pollutants may clog void space within the soils. Proper maintenance is required to maximize the effectiveness of the basin. It is recommended that a maintenance plan be in place before construction. At a minimum, basic maintenance like trash and debris cleanup should be performed twice a year. If a naturalized basin is used, reseeding and planting may be necessary. Weedy species can over take the basin and compete with native vegetation if regular maintenance is not conducted. In addition, prolonged elevated water levels can drown native plants, especially during the establishment period in the first few years (Iowa Stormwater Management Manual, 2007).

Guo (2001) presents a number of equations that have been developed to properly design a traditional infiltration basin. Equation 10.4.1 is used to determine the minimum basin bottom area, A_{bottom} – based on the volume required – the distance between the bottom of the basin and the water table (Z), soil porosity (θ_s) and soil initial water content (θ_o). The volume required can be determined using a number of different methods, including those discussed in Chapter 6.

$$A_{bottom} \geq \frac{V_{required}}{Z(\theta_s - \theta_o)} \qquad \text{Equation 10.4.1}$$

Equation 10.4.2 is used to determine the maximum ponding depth. This calculation is dependent on the distance between the bottom of the basin and the water table, soil porosity and the initial water content.

$$h \leq Z(\theta_s - \theta_o) \qquad \text{Equation 10.4.2}$$

Equation 10.4.3 is used to determine the amount of time there will be ponding in the basin based on the height of water present (h) and the infiltration rate of the soil (f).

$$T_p = \frac{h}{f} \qquad \text{Equation 10.4.3}$$

Equation 10.4.4 can be used to determine the infiltration rate of the basin. Infiltration basins are typically designed to be a rectangular, square or circular shape. This equation is based on the soil infiltration rate and the area of the basin.

$$Q = fA \qquad \text{Equation 10.4.4}$$

10.4 - EXAMPLE PROBLEMS

Example 10.4.1

The following example will expand on results determined in Example 6.2.1. This example used the MWRDGC method to determine the required storage to be approximately 0.48 ac-ft. At this Indianapolis site, the soil has a porosity 0.35 and initial water content of 0.10. The site is approximately 12 feet above the water table. Determine the ponding depth, basin geometry (assuming a square), infiltration rate and total infiltration flow.

Step 1: The area of the basin bottom can be found by:

$$A_{bottom} \geq \frac{0.48\ ac - ft}{12\ ft * (0.35 - 0.10)}$$

$$\mathrm{A_{bottom}} \geq 0.16\ \mathrm{ac}$$

$$\mathrm{A_{bottom}} \geq 6{,}970\ \mathrm{ft^2}$$

$$h \leq 12\ (0.35 - 0.10)$$

$$h \leq 2.4\ ft$$

Step 2: This equates to an approximately 85-ft by 85-ft basin, with a maximum ponding depth of 2.4 feet. In order to satisfy the maximum drainage time of 24 hours, the infiltration rate, *f*, must be at least:

$$f = \frac{2.4\ ft * 12\frac{in}{ft}}{24\ hrs}$$

$$f = 1.2\ in/hr$$

Step 3: If the basin is designed as an 85-foot by 85-foot square, the infiltration volume rate can be estimated:

$$Q = fA$$

$$Q = 1.2\frac{in}{hr} * (85 * 85)\ ft^2$$

$$Q = 0.20\ cfs$$

Example 10.4.2

If a circular basin is used instead of a square basin, the radius – R – of the basin will be approximately 50 feet. What is the design infiltration volume rate if the minimum infiltration rate, *f*, is used?

$$Q = fA = f\pi R^2$$

$$Q = 1.5\frac{in}{hr} * \pi * (50\ ft)^2$$

$$Q = 0.27\ cfs$$

10.5 - RAIN GARDENS

Rain gardens are another type of green infrastructure that utilizes infiltration and vegetation to treat and absorb stormwater runoff. Rain gardens are defined as shallow depressions planted with specifically selected native vegetation to capture and treat stormwater runoff from rooftops, streets and parking lots (Tippecanoe County, 2012). Rain gardens can be used as a retrofit measure in low-lying areas that already collect runoff. They should be located at least 10 feet from existing building foundations and have the ability to overflow into a traditional stormwater management system. Pre-treatment is an important component to the rain garden as it will help to slow stormwater runoff entering the system and settle out large debris. Figure 10.5.1 displays a typical cross-section of rain garden and identifies the different components.

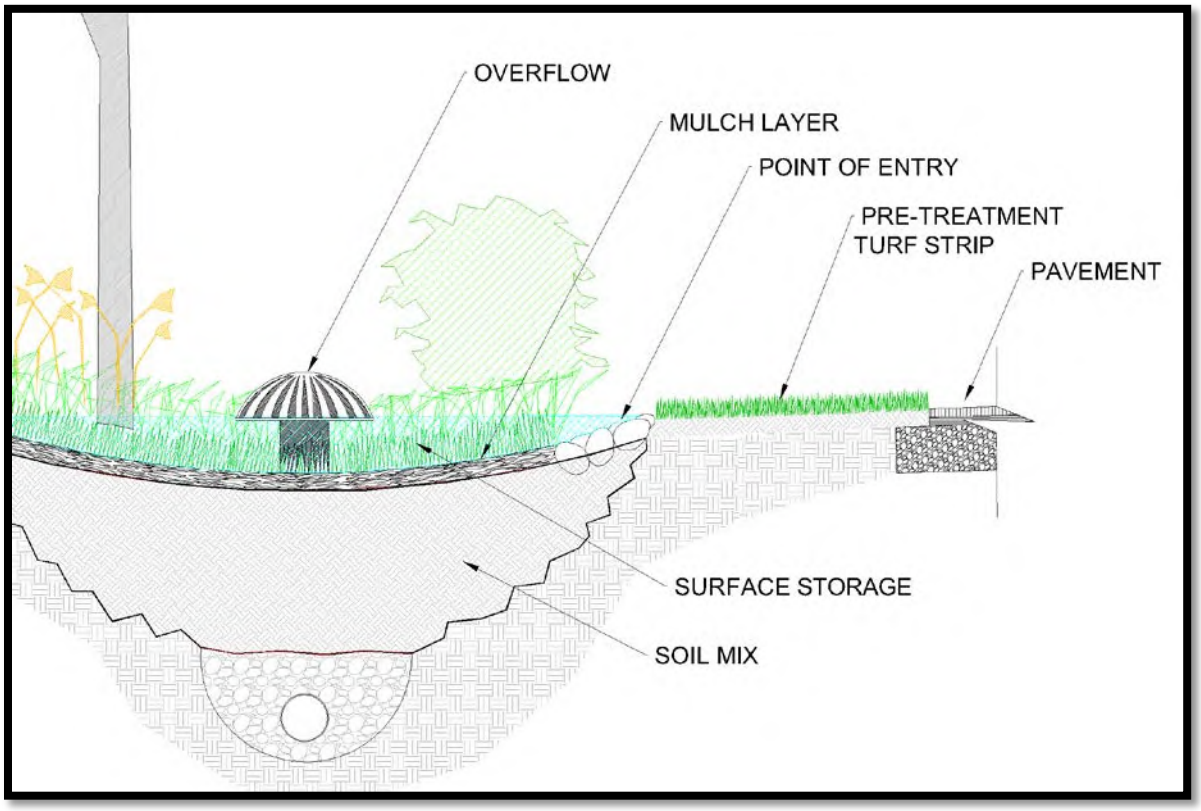


Figure 10.5.1
Typical Rain Garden Cross-Section

Research has been conducted in the Chicagoland area to evaluate the effectiveness of rain gardens. The Center for Neighborhood Technology (CNT) has developed a number of installation projects, which monitor the stormwater volume inflow and outflow. The demonstration projects revealed that the rain gardens could treat large volumes of stormwater runoff. At a test site titled St. Margaret Mary, 100% of stormwater runoff was captured and no outflow was observed (CNT USEPA, 2009). At a test site titled Bellwood stormwater up to a 2.68" storm event was captured on site (CNT IEPA, 2009).

Table 10.5.1 is taken from the NOAA's National Climatic Data Center 1981-2010 Climate Normals for the various locations in Indiana. This table shows the mean number of days a year that experience varying depths of precipitation. As an example, for the period analyzed at the Indianapolis International Airport, there was a mean of 129.2 days a year with over 0.01 inches of precipitation and a mean of 10.3 days a year over 1.00 inch of precipitation. This information may be used to estimate the mean number of days a year or the mean number of 24-hour storm events the rain garden will be overtop. Rain gardens are typically designed to manage storm

events with rainfall depths less than 1", which according to Table 10.5.1, 1" storage capacity corresponds to an approximately 88-95% of annual storm events may be captured.

Table 10.5.1
Precipitation Climate Normals – Mean Number of Days Annually Which Exceed Various Rainfall Depths (NOAA, 2011)

Rainfall Depth (inches)	*Indianapolis International Airport*	*Fort Wayne Airport*	*South Bend Regional Airport*	*Evansville Museum*
0.01	129.2	133.9	145.4	110.3
0.10	77.4	76.7	78.3	75.1
0.50	28.8	24.6	23.4	31.3
1.00	10.3	8.2	7.2	12.4

10.5 - EXAMPLE PROBLEMS

Example 10.5.1

A rain garden, which is 10' by 10' and 0.5' deep, is proposed for a residential site. If the area contributing to the rain garden is a mixture of roof drains and grassed areas as shown below, calculate the percent of all storm events that would be captured by the rain garden using Table 10.5.1. The rain garden is located in Ft. Wayne, Indiana and C soils should be assumed. In this case assume all of the roof runoff is directly drained to the rain garden.

Area of Roof = 500 ft^2 CN_{Roof} = 98
Area of Grass = 5,000 ft^2 CN_{Grass} = 80

Step 1: Calculate the stormwater runoff volume from both the grassed and roof area using Equation 3.3.1 for varying rainfall depths.

$$S = (\frac{1000}{CN}) - 10 \qquad \text{Equation 3.3.2}$$

S_{98} = 0.20 inches S_{80} = 2.50 inches

$$R(t) = \frac{((P(t)-0.2S)^2)}{P(t)+0.8S} \qquad \text{Equation 3.3.1}$$

In Equation 3.3.1, if, $P(t) \leq 0.2S$, then R(t) is equal to zero.

Table 10.5.2
Runoff Summary for Various Rainfall Depths

Rainfall Depth (inches)	*Runoff from Grassed Area (inches)*	*Runoff from Roof Area (inches)*
0.25	0.00	0.11
0.50	0.00	0.32
0.75	0.02	0.55
1.00	0.08	0.79

Multiply the runoff by the tributary area to determine the total volume of the runoff.

$$\mathbf{Q_v = R_{Grass}\ x\ \frac{1\ ft}{12\ in}\ x\ A_{Grass} + R_{Roof}\ x\ \frac{1\ ft}{12\ in}\ x\ A_{Roof}}$$

$Q_{0.25"}$ = 4.41 ft^3
$Q_{0.50"}$ = 13.25 ft^3
$Q_{0.75"}$ = 24.08 ft^3
$Q_{1.00"}$ = 37.12 ft^3

Step 2: Calculate the storage volume of a rain garden that is 10' by 10' and 6 inches deep using the Average End Area Method.

$$\text{Volume} = \frac{\text{Area of Top} + \text{Area of Bottom}}{2}\ \text{x Depth}$$

$$\text{Volume} = \frac{(10\ \text{ft x}\ 10\ \text{ft}) + (4\ \text{ft x}\ 4\ \text{ft})}{2}\ \text{x}\ 0.5\ \text{ft} = 29\ \text{ft}^3$$

Comparing the rain garden volume to the contributing stormwater runoff volume, the rain garden will begin to overtop for storm events with between 0.75" and 1.00" of rainfall depths. Using the Solver function in Microsoft Excel, the rainfall depth at which the rain garden will over top is 0.85". From Table 10.5.1, the rain garden will be able to treat and infiltrate water for 82% to 92% of storm events.

10.6 - RAINWATER HARVESTING

Rainwater harvesting has been typically used in areas with water scarcity issues to augment water supply, but it can also be used as a stormwater management technique. Rain barrels are typically 55-gallon containers that are placed at the end of a residential downspout as seen in Figure 10.6.1.

Cisterns are larger and can be specifically designed for each installation. The water stored in these systems can then be used for non-potable water use, such as car washing and gardening.

The water may also be used for toilet flushing and other indoor non-potable uses, but local regulations should be consulted.



Figure 10.6.1
Typical Rain Barrel (Koss, 2008)

The main benefit of rainwater harvesting is the reduction in stormwater runoff quantity, but a single 55-gallon rain barrel will only hold approximately 7.35 ft^3 of water. Although any reduction in stormwater quantity can be seen as a benefit, a single rain barrel has minimal effect on total stormwater reduction. However multiple rain barrels can have a greater impact on stormwater runoff reduction. Used in combination with other green infrastructure, rain barrels may additionally minimize stormwater runoff. Figure 10.6.3 shows the stormwater runoff reduction from a single 55-gallon rain barrel versus various rainfall depths for different-sized roofs. It should be noted that the barrels were assumed to be empty at the beginning of each rainfall event.

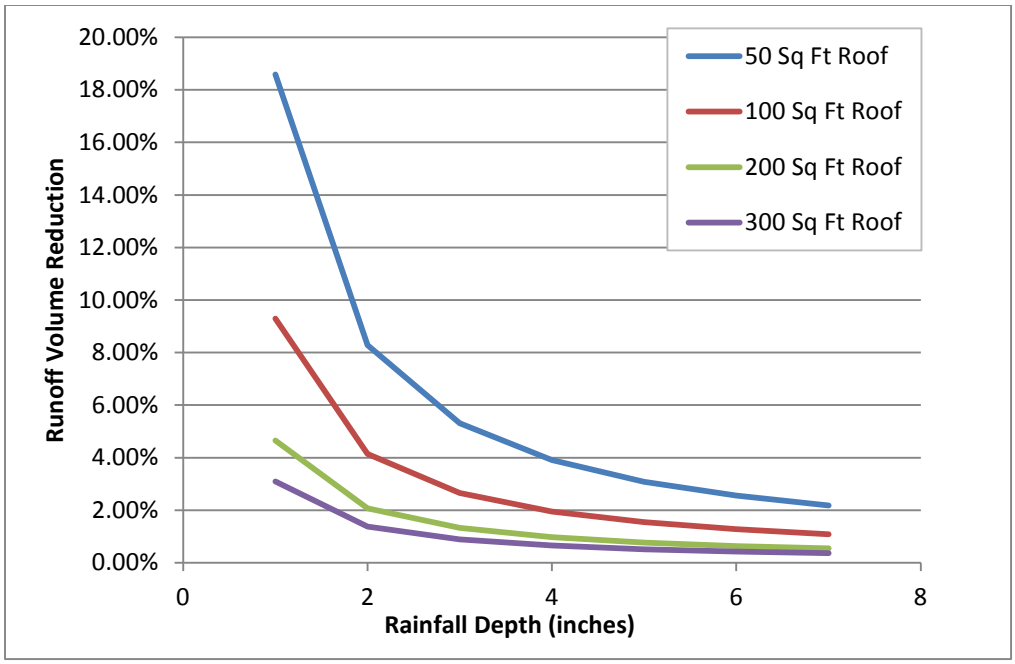


Figure 10.6.3
Runoff Reduction on Varying Sized Roofs Due to One Rain Barrel

Rain barrels and cisterns cannot be utilized efficiently when water freezes within the barrel. When a rain barrel or cistern is part of a stormwater management system, guidance or regulations on when it should be emptied after a storm event may be required to ensure that the storage is available during later storm events. In the case of large cisterns, emerging technology utilizes weather forecasts to increase the overall efficiency of the system.

10.6 - EXAMPLE PROBLEM

Example 10.6.1

For a 1,000-square foot roof, how many 55-gallon rain barrels are necessary to hold the resulting runoff from a 1"-storm event, assuming that the barrels are empty at the beginning of the storm? Use the NRCS Curve Number Method to determine the runoff volume.

$$55 \text{ gallons} \cong 7.35 \text{ ft}^3$$

$$\text{CN}_{\text{ROOF}} = 98$$

$$S_{\text{roof}} = 0.20$$

$$\text{R} = 0.79 \text{ inches} = 0.066 \text{ feet}$$

$$\text{V} = (0.066 \text{ ft} * 1000 \text{ sq ft})$$

$$\text{V} = 66 \text{ ft}^3$$

$$\#\text{ of barrels needed } = \frac{66 \text{ ft}^3}{7.4 \text{ft}^3}$$

Approximately nine rain barrels are needed.

10.7 - CREDIT FOR GREEN INFRASTRUCTURE PRACTICES

Many stormwater ordinances and regulations require that a specified volume of stormwater be retained on site, typically through the use of green infrastructure practices. In some cases there may be a provision that the volume retained by the green infrastructure practices may be credited towards the required detention volume. The following method can be used to ensure that detention facilities and outlet control structures are appropriately sized for the 100-year, 24-hour runoff volume while providing the volume control storage credit as a reduction in the overall curve number (CN) of the developed site. For example, the Cook County Watershed Management Ordinance (WMO) requires that one (1) inch of volume control be provided overall all proposed impervious area. A given watershed (site) with area, A_W, is required to provide detention stormwater storage using the NRCS procedure and is also required to retain stormwater storage of 1” over the impervious area.

The NRCS runoff equation is:

$$R_W = \frac{((P-0.2S)^2)}{P+0.8S}$$ Equation 3.3.1

Where,

R_W = runoff depth (in) from Area, A_W

P = rainfall depth used to calculate runoff (in),

S = potential maximum retention after runoff begins (in), and is calculated by:

$$S = (\frac{1000}{CN}) - 10$$ Equation 3.3.2

Where,

CN_W = runoff curve number for the watershed

The volume of runoff (acre-feet), V_W, from watershed A_W can then be calculated by:

$$V_w = R_W \text{ x } \frac{1 \text{ ft}}{12 \text{ in}} \text{ x } A_W$$

The total volume of runoff from the site can be reduced by the volume control required and the extra green infrastructure volume that may be provided:

$$V_{ADJ} = V_W - V_R - V_{GI}$$

where,

V_{ADJ} = adjusted runoff volume from site (acre-feet)

V_R = volume of volume control storage (1" over impervious area of development)

V_{GI} = volume of green infrastructure provided in addition to the required 1"

This reduced volume of runoff can be reflected in an overall reduction to the CN used in detention basin sizing by using:

$$\frac{V_{ADJ}}{A_W} = R_{ADJ} = \frac{(P-0.2S)^2}{(P+0.8S)}$$

Since R_{ADJ} is known, and P = 7.58" for the 100-year, 24-hour storm event in Cook County, we can solve for S, which then translates to the adjusted CN. The adjusted curve number (CN_{ADJ}) is then used to calculate the required detention volume for the site.

Example 10.8.1 provides a sample calculation for the CN reduction methodology for a site in Cook County based on the WMO requirements.

Example 10.8.1

For a 10-acre proposed residential area with a developed CN of 78, and 3 acres of impervious area, find the revised CN resulting from the 1" volume control provisions of the WMO.

Step 1: The future 100-year runoff volume for the proposed development without volume control can be calculated using the NRCS runoff equation.

$$R_W = \frac{(7.58" - 0.2S)^2}{(7.58 + 0.8S)}$$

$$S = \frac{1000}{78} - 10 = 2.82"$$

$$R_W = \frac{(7.58" - (0.2)(2.82"))^2}{7.58" + 0.8(2.82")}$$

$$R_w = 5.00"$$

Step 2: The total volume is therefore:

$$V_W = \frac{R_W}{12} \times A_W = \frac{5}{12} \times 10\ ac = 4.17\ ac - ft$$

Step 3: The volume associated with the total impervious area that must be stored is:

$$V_R = 3\ ac \times \frac{1\ ft}{12\ in} = 0.25\ ac - ft$$

For this example, $V_{GI} = 0$, so the adjusted runoff volume is:

$$V_{ADJ} = 4.17\ ac - ft - 0.25\ ac - ft = 3.92\ ac - ft$$

Step 4: And therefore:

$$12 \times \frac{V_{ADJ}}{A_W} = \frac{(P - 0.2S)^2}{(P + 0.8S)} = 4.70\ in$$

Since P = 7.58":

$$4.70\ in = \frac{(7.58" - 0.2S)^2}{(7.58" + 0.8S)}$$

Step 5: Solving this equation iteratively:

$$S = 3.28$$

and the adjusted CN, CN_{ADJ} = 75.32.

The curve number in this example is reduced from 78 to 75.32. This procedure reflects the stormwater volume reduction and allows for hydrologic routing through proposed stormwater management facilities.

This methodology also allows for the quantification of runoff volume reduction benefits for green infrastructure projects. When using this methodology careful consideration should be taken with the storm event(s) being analyzed. Since the runoff volume varies by storm event, the effect of the green infrastructure volume, and therefore the reduction in curve number, will also vary by storm event.

10.8 - GREEN INFRASTRUCTURE TREATMENT TRAINS

Although the focus of this chapter is the runoff volume reduction benefits of green infrastructure, there are inherent water quality benefits associated with these practices, especially when used in series or in combination with each other.

Green infrastructure methods can be used in combination to reduce stormwater runoff in even greater quantities. The following examples pair green infrastructure techniques to demonstrate this example.

10.9 - EXAMPLE PROBLEMS

Example 10.9.1

Use the NRCS Curve Number Method to determine the reduction of runoff from a site that has a 750 square foot green roof and two 55-gallon rain barrels. The site currently has a typical impermeable roof and no rain barrels. Table 10.9.1 summarizes the runoff volume from the current roof.

Table 10.9.1
Runoff from Regular Roof

Rainfall Depth (in)	*R(t) (inches)*	*Runoff Volume (ft^3)*
1	0.79	49.43
2	1.77	110.90
3	2.77	173.02
4	3.77	235.32
5	4.76	297.70
6	5.76	360.12

Using a CN of 86 for the green roof and two rain barrels to collect any excess runoff, Table 10.9.2 summarizes the runoff from the proposed conditions.

Table 10.9.2

Runoff from Proposed Conditions

Rainfall Depth (in)	*R(t) (inches)*	*Runoff Volume (ft^3)*	*Runoff Volume Provided in 2 Rain Barrels (ft^3)*	*Total Runoff Volume (ft^3)*
1	0.20	12.35	14.71	0.00
2	0.85	53.06	14.71	38.36
3	1.66	103.90	14.71	89.20
4	2.55	159.14	14.71	144.44
5	3.47	216.69	14.71	201.98
6	4.41	275.59	14.71	260.88

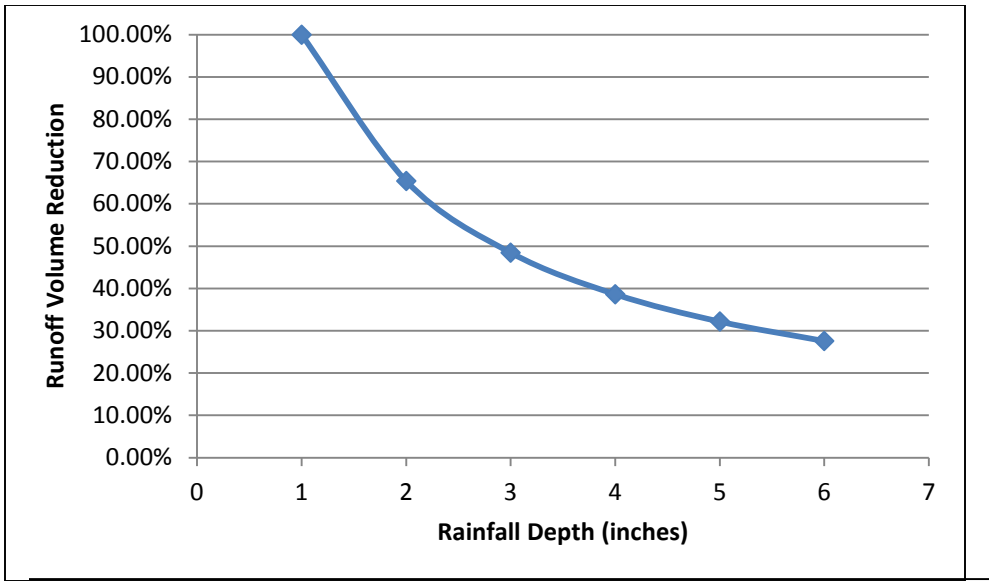


Figure 10.9.1
Surface Runoff Volume Reduction

Figure 10.9.1 summarizes the surface runoff reduction that the proposed conditions provide. For a 1" rainfall event, all of the runoff is absorbed if the rain barrels are empty at the beginning of the storm. As the rainfall depth is increased the reduction is substantially diminished. It may be concluded that the proposed infrastructure can reduce surface runoff significantly for smaller storm events, but for larger events the reduction decreases to approximately 30%.

Example 10.9.2

Using the results of Example 10.5.1 determine the increased number of storm events captured if the rain garden is combined with an underground storage tank of 500, 1,000 and 2,000 gallons located below the rain garden.

Step 1: Convert the underground storage tanks volume from gallons to cubic feet.

7.48 gallons	=	1.00	ft^3
500 gallons	=	66.8	ft^3
1,000 gallons	=	133.7	ft^3
2,000 gallons	=	267.4	ft^3

Step 2: Compare the additional volume from the storage tanks to the volume required in Example 10.5.1.

Table 10.9.3
Runoff Volume Reduction from Rain Garden and Storage Tanks

Volume of Storage Tank	*Total Storage Volume Available in Rain Garden and Storage Tank*	*Rainfall Depth & Associated Runoff Volume Reduction (in)*				
		0.5	*1.0*	*1.5*	*2.0*	*3.0*
500 Gallons	96	100%	100%	56%	31%	15%
1,000 Gallons	163	100%	100%	94%	53%	26%
2,000 Gallons	296	100%	100%	100%	96%	47%

The proposed green infrastructure systems are capable of storing 100% of the stormwater runoff volume associated with storm events less than 1". The 2,000 gallon cistern and rain garden are capable of storing up to the 1.5" storm event. Table 10.9.3 displays the runoff volume reduction benefits of the systems up to the 3" storm event.

Example 10.9.3

A residential dwelling is located on a 15,000 ft^2 lot. The driveway is 45 feet long and 20 feet wide, the house footprint is 1,800 ft^2 and the remainder of the lot is grass on a HSG C soil. Using a rain garden, porous pavement and rain barrels (at each corner of the building), determine how much of the total runoff from the lot can be detained. Assume that the site can be graded to facilitate the drainage to the proposed green infrastructure components.

Step 1: Determine the runoff generated from each surface using the NRCS Curve Number Method.

$$S = (\frac{1000}{CN}) - 10$$ Equation 3.3.2

$$R = \frac{((P-0.2S)^2)}{P+0.8S}$$ Equation 3.3.1

In Equation 3.3.1, if $P \leq 0.2S$, then R is equal to zero.

$$Q_V = R \text{ x } \frac{1 \text{ ft}}{12 \text{ in}} \text{ x } A$$

Table 10.9.4
Runoff Volume (ft^3) for Corresponding Rainfall Depths

Surface	*Area (ft^2)*	*CN*	*Rainfall Depth (in)*						
			0.25	*0.50*	*1.00*	*1.50*	*2.00*	*2.50*	*3.00*
Roof	1,800	98	15.88	47.68	118.64	192.02	266.15	340.61	415.24
Driveway	900	98	7.94	23.84	59.32	96.01	133.08	170.31	207.62
Grass	12,300	79	0.00	0.00	71.91	265.03	535.54	858.36	1,218.18
Total	*15,000*	---	*58.04*	*71.92*	*249.87*	*553.06*	*934.77*	*1,369.28*	*1,841.04*

Step 2: Determine the available storage from the green infrastructure system. The following assumptions should be made for the system:

- A porous pavement will be used on the entire driveway and it will have a 2 foot aggregate base course with 0.36 void space.
- Typical 55-gallon rain barrels will be used on 4 corners of the home.
- The rain garden will have the same volume as the one presented in Example 10.5.1.

$$\text{Porous Pavement Storage Volume} = 45 \text{ ft x } 20 \text{ ft x } 24 \text{ inches x } \frac{\text{ft}}{12 \text{ inches}} \text{ x } 0.36$$
$$= 648 \text{ ft}^3$$

$$\text{Rain Barrel Storage Volume} = 4 \text{ x } 55 \text{ gallons x } \frac{\text{ft}^3}{7.4 \text{ gallons}} = 29.7 \text{ ft}^3$$

$$\text{Rain Garden Storage Volume} = 29 \text{ ft}^3 \text{ (see Example 10.5.1)}$$

Table 10.9.5
Total Runoff Volume Reduction from Green Infrastructure System

Rainfall Depth	*0.25*	*0.50*	*1.00*	*1.50*	*2.00*	*2.50*	*3.00*
Total Runoff Reduction	100%	100%	100%	100%	76%	52%	38%

The total storage volume provided from the green infrastructure measures described above is 706 ft^3. The calculations above assume that the roof drains will first fill the rain barrels and then overflow and be directed towards the porous pavement. The yard will first drain into the rain garden and then overflow and be directed towards the porous pavement.

10.9 - CONCLUDING REMARKS

The purpose of this chapter is to provide guidance on commonly used green infrastructure practices, the important considerations associated with their design and the water quality and quantity benefits that these practices can provide. Because they promote the infiltration of stormwater, green infrastructure practices can filter out the pollutants in runoff, and can also provide meaningful reductions in the volume of stormwater runoff.

As illustrated throughout this chapter, due to the limited volumes typically provided in green infrastructure practices, the stormwater runoff volume reductions are more significant for smaller, more frequent storm events. The volume reduction benefits associated with green infrastructure will decrease as the magnitude of the storm event increases. Green infrastructure practices should be designed with these limitations in mind, and should not be viewed as a substitute for traditional stormwater detention design. Green infrastructure practices should be used to complement traditional stormwater volume control measures to improve water quality and reduce surface runoff volume.

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Appendix A - STATISTICAL ANALYSIS

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Appendix A - STATISTICAL ANALYSIS

LIST OF PARAMETERS

α, u	Functions of the mean and standard deviation in Gumbel's cumulative distribution function
b	Variable in Gumbel's cumulative distribution function
CDF	Cumulative Distribution Function
g	Skewness
J	Risk
k	Gumbel frequency factor
k´	Log Pearson frequency factor as a function of the skewness coefficient and return period
m	Number of occurrences
n	Number of discrete samples
$P(E_1)$	Probability of a single event
$P(E'_1)$	Probability of an event not occurring
s_d	Standard deviation
s'_d	Standard deviation of the logarithmic transformed values
T_r	Return Period
x	Discrete value of a sample or event
X	Magnitude of an event
x_m	Mean of the data
$\bar{x}$	Mean of the data

Appendix A - STATISTICAL ANALYSIS

This appendix is presented as a summary of the probabilistic and statistical principles used in the analysis of hydrological data. The general concepts are presented and the mode of application outlined. The reader is referred to the references at the end of the chapter for additional information.

A.1 - GENERAL CONCEPTS

In the statistical analysis of a process, the process is termed 'continuous' if it can assume any value between the limits of that process or 'discrete' if the process is confined to specified, incremental values. Examples of a continuous process are rainfall, storm runoff and river flows. An example of a discrete process is the number of times a batter in baseball reaches base safely.

In the study of hydrology we are usually concerned with average and extreme values and their variability. This information is utilized in the design of dams, storage facilities, storm sewers, etc.

A.2 - STATISTICAL PARAMETERS

A.2.1 The Mean

The mean of a set is often called the average. It is a measure of the central tendency of a distribution. This is illustrated in Figure A.3.1, where the areas to the right and left of line A-A are equal. A curve such as that shown in Figure A.3.1 is a continuous probability distribution function. For discrete samples, the mean is given by:

$$\bar{x} = x_m = \frac{1}{n}\sum_{i=1}^{n} x_i \qquad \text{(A.2.1)}$$

where x_m is the mean of the x_i data set.

The log mean is written as:

$$\log x_m = \sum_{i=1}^{n} \frac{\log x_i}{n} \qquad \text{(A.2.2)}$$

A.2.2 - Standard Deviation

The standard deviation of a sample is indicative of the scatter or the average distance about the mean. These deviations are calculated as $(x_1 - x_m)$, $(x_2 - x_m)$. . . $(x_n - x_m)$, where x_m is the mean. The average distance is the standard deviation which is written as:

$$s_d = \left[\frac{1}{n-1} \sum_{i=1}^{n} (x_i - x_m)^2 \right]^{1/2} \qquad s'_d = \left[\frac{1}{n-1} \sum (\log x_i - \log x_m)^2 \right]^{1/2} \tag{A.2.3}$$

The skewness indicates the weighting of the distribution to either side of the mean. It is determined by:

$$g = \frac{1}{s_d^3} \frac{n}{(n-1)(n-2)} \sum_{i=1}^{n} [x_i - x_m]^3 \tag{A.2.4}$$

The skewness is zero for symmetrical distributions, greater than zero for right skewness and less than zero for left skewness. The probability distribution in Figure A.3.1 has positive or right skewness.

A.3 - PROBABILITY CONCEPTS

The probability of a single event, E_1, is defined as the relative number of occurrences of the event after a sufficiently large number of trials. Therefore, the probability of event E_1, $P(E_1)$, is given as m_1/n for m_1 occurrences of event E_1 in n trials, if n is sufficiently large. The value m_1 represents the frequency of the event and m_1/n is the relative frequency or probability. The following illustrates this concept.

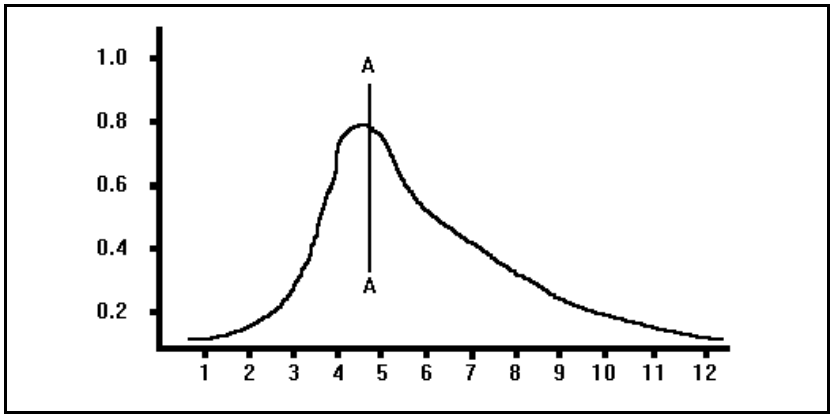


Figure A.3.1
Probability Distribution Function

Table A.3.1 presents the yearly data for the annual peak flow in the Wabash River at Lafayette, Indiana for a period of 69 years. This data represents a continuous process but for convenience it can be discretized in groups of 10,000 cfs intervals.

Table A.3.1 Annual Peak Flows for the Wabash River at Lafayette, Indiana (1924 – 1992)

Year	Peak Discharge (cfs)	Year	Peak Discharge (cfs)
1924	59,800	1959	87,600
1925	63,300	1960	38,100
1926	57,700	1961	54,700
1927	64,000	1962	45,000
1928	63,500	1963	60,000
1929	38,000	1964	57,600
1930	74,600	1965	36,000
1931	13,100	1966	64,100
1932	37,600	1967	65,300
1933	67,500	1968	67,100
1934	21,700	1969	64,900
1935	37,000	1970	41,700
1936	93,500	1971	30,000
1937	58,500	1972	38,500
1938	63,300	1973	39,500
1939	74,400	1974	48,800
1940	34,200	1975	34,700
1941	14,600	1976	43,400
1942	44,200	1977	31,400
1943	131,000	1978	49,500
1944	73,300	1979	48,800
1945	46,600	1980	46,200
1946	39,400	1981	43,200
1947	41,200	1982	55,300
1948	41,300	1983	59,100
1949	62,000	1984	40,000
1950	90,000	1985	79,200
1951	50,600	1986	40,000
1952	41,900	1987	13,800
1953	35,000	1988	32,600
1954	16,500	1989	39,500
1955	35,300	1990	50,800
1956	30,000	1991	75,200
1957	52,000	1992	32,700
1958	97,000		

Plotting the occurrences in each 10,000 cfs intervals gives the histogram shown in Figure A.3.2. In the selection of an interval, it is best to choose the smallest one possible. In fact, if the increment is infinitesimal and the record continuous, the bar graph would become a smooth curve similar to Figure A.3.1.

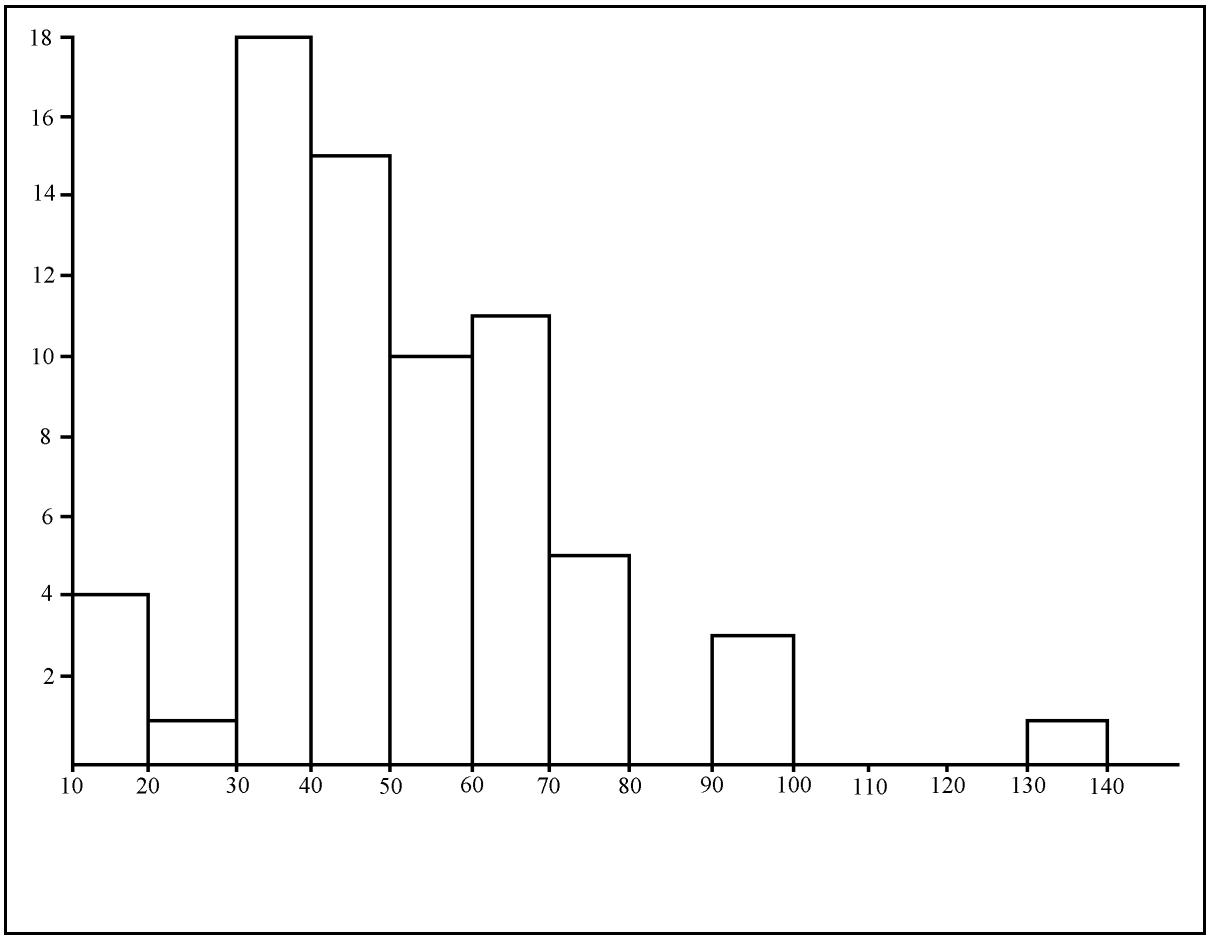


Figure A.3.2
Histogram of the Annual Maximum Flows of the
Wabash River at Lafayette, Indiana (1924 – 1992)

From Figure A.3.2, it can be concluded that on the average, the frequency of an annual peak flow with a magnitude between 30,000 and 40,000 cfs is 18 times in 69 years, where 18 is previously mentioned as m_1.

If the number of occurrences of this event is divided by the sample size of 69, the relative frequency of the occurrence of that event class can be determined. The relative frequency for flows with a magnitude between 30,000 and 40,000 cfs is therefore 18/69 or 0.26. But since the number of samples is small, it is only an estimate of the true probability.

If the frequencies of each one of the intervals in Figure A.3.2 are cumulatively summed up and graphed, the curve shown in Figure A.3.3 is found. This curve is a cumulative distribution function (CDF) and shows that the proportion of annual peak flows which were observed to have a magnitude less than or equal to 50,000 cfs, for example, is 0.55.

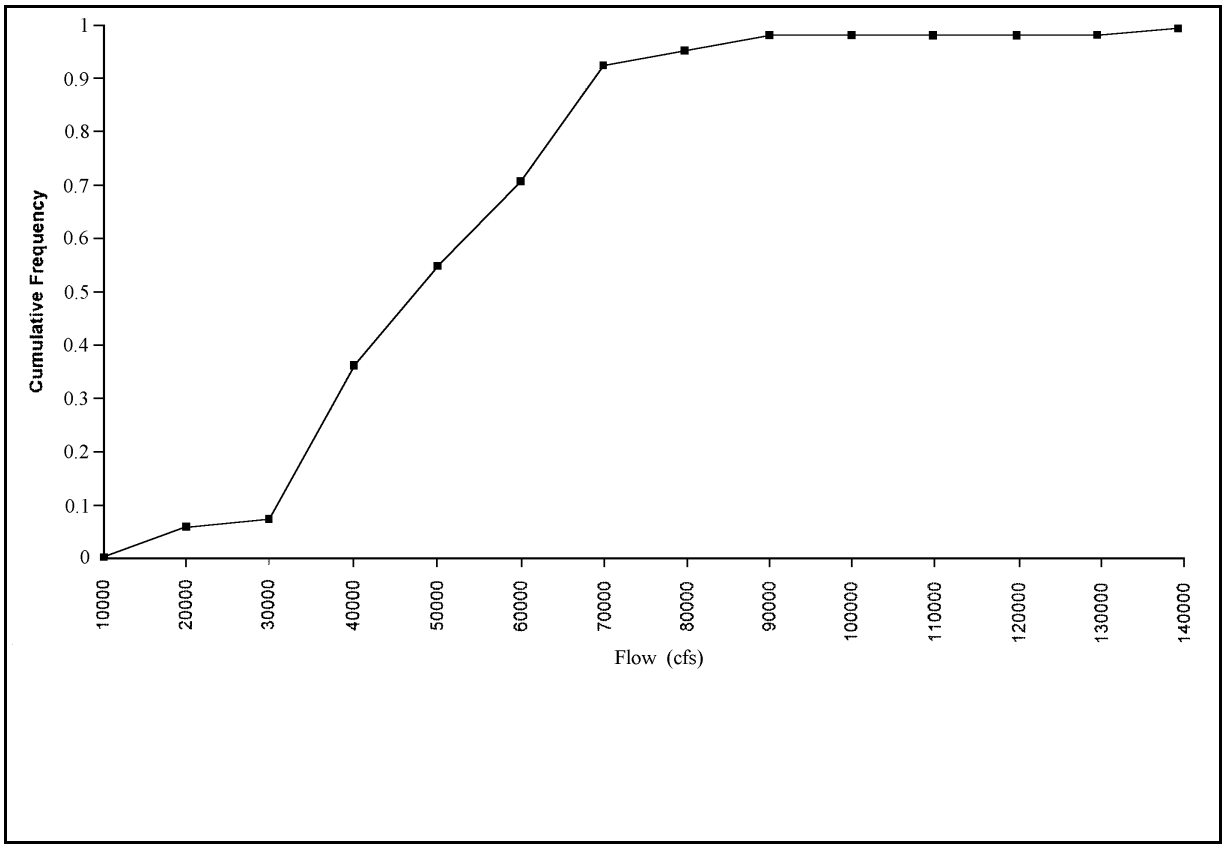


Figure A.3.3
Cumulative Distribution of Maximum Annual Flows of the
Wabash River at Lafayette, Indiana (1924 – 1992)

The probability of an event occurring can be said to equal the relative frequency, and since the probability of an event certain to occur is one and an impossible event has probability zero, we can express the probability of E_1 as falling in the range:

$$0 \le P(E_1) \le 1 \qquad \text{(A.3.1)}$$

It is observed from Figure A.3.3 that the sum of the probability estimates of the events is equal to one.

$$P(E_1) + P(E_2) + P(E_3) + \ldots P(E_n) = \sum_{i=1}^{n} P(E_i) = 1 \qquad \text{(A.3.2)}$$

The probability of an event not occurring $P(E'_1)$ plus the probability of that event occurring $P(E_1)$ is one. This can be written as:

$$P(E'_1) + P(E_1) = 1 \qquad \text{(A.3.3)}$$

As an example, take the probability estimate of a flood greater than 50,000 cfs. In this case it would be 1 – 0.55 or 0.45.

The information presented in the last two sections provides the manual user with a basic understanding of the statistical and probability techniques used in hydrology.

It should be pointed out that the probability estimate used in this section was given as m/n, where m was the number of occurrences of an event and n was the total number of observations. In later sections the plotting procedures utilize the expression $P = m/(n + 1)$ or $T_r = (n + 1)/m$. The reason for the (n + 1) term rather than n is beyond the scope of this manual. For now, it can be thought of as a correction to compensate for limited sample data so that the population distribution will be more representative.

A.4 - PROBABILITY DISTRIBUTIONS

This section presents two distributions which are used to describe the distribution of hydrological phenomenon. Only the basics of each distribution are presented. If more detail is desired, the user is referred to standard texts on the subject or references at the end of this Appendix.

A.4.1 – Gumbel's Extreme Value Distribution

The Gumbel distribution has been successfully used to represent minimum or maximum values of a given sample. The distribution is asymmetrical like the one shown in Figure A.3.1 with a log-normal distribution and constant skewness of 1.1396. The probability for an event being less than or equal to a given magnitude is given by the cumulative distribution function (CDF) and is written as:

$$P(X \le x) = F(x) = \exp[-\exp[-\alpha(x-u)]] \quad \text{(A.4.1)}$$

where α and u are functions of the mean and standard deviation. (X is the given magnitude, x is the event.)

The expression $-\alpha(x-u)$ can be estimated by:

$$b = -\alpha(x-u) = \frac{(x - x_m + 0.45\, s_d)}{0.78\, s_d} \quad \text{(A.4.2)}$$

As an example, consider the data shown in Table A.3.1. The mean for this data was 51,000 cfs and the standard deviation is 21,000 cfs. Therefore, for a flood of 50,000 cfs, b = (50,000 – 51,000 + 0.45(21,000))/(0.78 x 21,000) = 0.516. Therefore, F(X) for a 50,000 cfs flood is exp[-exp(-0.516)] = 0.55. This checks with the observed value of 0.55 shown in Figure A.3.3. The probability of the event being exceeded is 1 – 0.55 or 0.45. The return period is then given as 1/0.45 or 2.22 years.

A simplified method for estimating the magnitude of a flood using the Gumbel distribution for a given return period and sample size was presented by Chow, based on Equation A.4.3.

$$x = x_m + k s_d \quad \text{(A.4.3)}$$

In Equation A.4.3, k is a frequency factor which may be found by using Equation A.4.4 for a very large sample size distribution, or Table 2.2.1, page 2-19, for a sample size up to 100.

$$k = \frac{\sqrt{6}}{\pi}\left[0.5772 + \ln\left[\ln\frac{T_r}{T_{r-1}}\right]\right] \quad \text{(A.4.4)}$$

Example 2.2.1 in Chapter 2 demonstrates the use of Equation A.4.3 along with the procedures used in the plotting of the Gumbel distribution.

A.4.2 – Log Pearson Type III Distribution

This distribution is usually the recommended standard method for analyzing hydrologic data (Water Resources Council, 1981). The distribution is skewed and the data is transformed by X = log x to reduce the skewness. The mean, standard deviation, and skewness of this transformed data are then used to fit the distribution. The formula used for the distribution is somewhat involved and will not be presented here.

The simplified method used in the analysis is to use the following equation.

$$\log x = \log \bar{x} + k' s'_d \quad \text{(A.4.5)}$$

In Equation A.4.5, s'_d corresponds to the standard deviation of the logarithmic transformed

values, and term k´ is a frequency factor which is a function of the skewness coefficient and return period. Table 2.3 presents these factors as a function of return period. Examples and other discussion of the distribution are found in Chapters 2 and 3. The graphical approach to the analysis is identical to the Gumbel distribution except a different plotting paper is used.

A.5 – RISK

In Section A.3 it was shown that the frequency P(E) represents an average value. Using a flood as an example, we observe a flood magnitude associated with P(E) = 0.25 has on average 25 percent chance each year of being equaled or exceeded. It also implies that the flood would occur on the average once in 4 years [1 / 0.25]. This is called the return period T_r.

We can determine the probability that the flood will not occur in a year by:

$$P(\overline{E}) = 1 - P(E) = 1 - \frac{1}{T_r} \qquad \text{(A.5.1)}$$

It follows that the probability that E will not occur for n successive years is:

$$P_1(E_m) \times P_2(E_m) \times \ldots P_n(E_m) = P(E_m)^n = \left[1 - \frac{1}{T_r}\right]^n \qquad \text{(A.5.2)}$$

The risk of an event is the probability that the event will occur at least once in n years. As presented by Viessman et al. (1989), the risk J is equal to:

$$J = 1 - \left[1 - \frac{1}{T_r}\right]^n = 1 - \left[P(E_m)\right]^n \qquad \text{(A.5.3)}$$

Table A.5.1 presents a tabular solution of Equation A.5.3 for various levels of risk and the expected design life of the structure. For a structure with a design life of 30 years and accepted risk of 10%, the structure should be designed with a return period of 250 years as determined below.

$$0.10 = 1 - \left[1 - \frac{1}{T_r}\right]^{30} \rightarrow \quad 0.90^{\frac{1}{30}} = 1 - \frac{1}{T_r}$$

$$0.9965 - 1 = \frac{-1}{T_r} \rightarrow \quad T_r = 285 \text{ years}$$

Table A.5.1
Return Period Associated with Various Degrees of Risk and
Expected Design Life (after Viessman et al., 1989)

Risk %	Expected Design Life (years)							
	2	5	10	15	20	25	50	100
75	2.00	4.02	6.69	11.0	14.9	18.0	35.6	72.7
50	3.43	7.74	14.9	22.1	29.4	36.6	72.6	144.72
40	4.44	10.3	20.1	29.9	39.7	49.5	98.4	196.3
30	6.12	14.5	28.5	42.6	56.5	70.6	140.7	281
25	7.46	17.9	35.3	52.6	70.0	87.4	174.3	348
20	9.47	22.9	45.3	67.7	90.1	112.5	224.6	449
15	12.8	31.3	62.0	90.8	123.6	154.3	308	616
10	19.5	48.1	95.4	142.9	190.3	238	475	950
5	39.5	98.0	195.5	292.9	390	488	976	1949
2	99.5	248	496	743	990	1238	2475	4950
1	198.4	498	996	1492	1992	2488	4975	9953

The following pages contain rainfall depth maps for the entire continental United States. They are referenced in Chapter 2, Section 2.2.5 for determining rainfall depths for locations where Intensity-Duration-Frequency curves have not been developed.

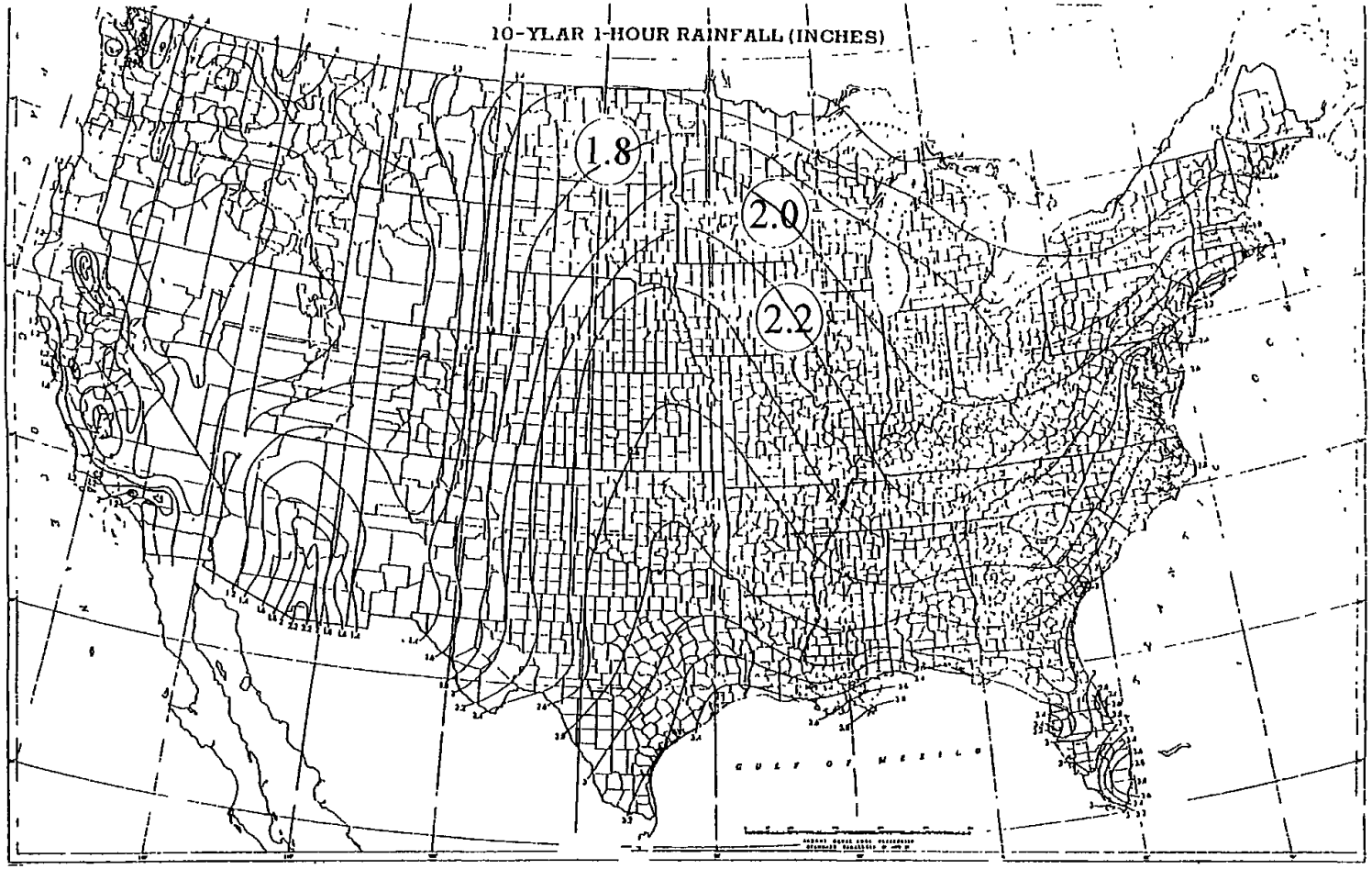


Figure A.6.1
10-Year 1-Hour Rainfall Depth Map (U.S. Weather Bureau Technical Paper 40)

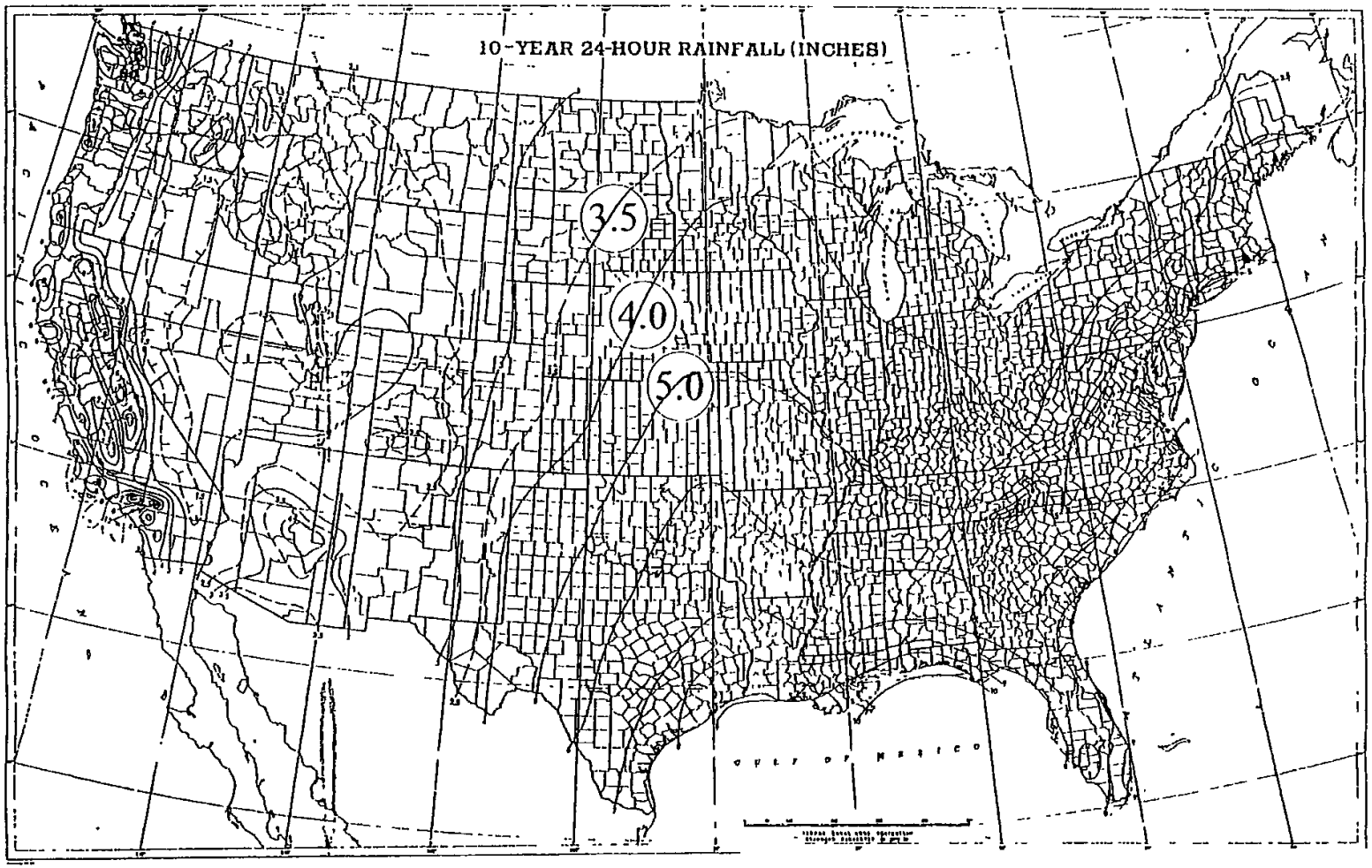


Figure A.6.2
10-Year 24-Hour Rainfall Depth Map (U.S. Weather Bureau Technical Paper 40)

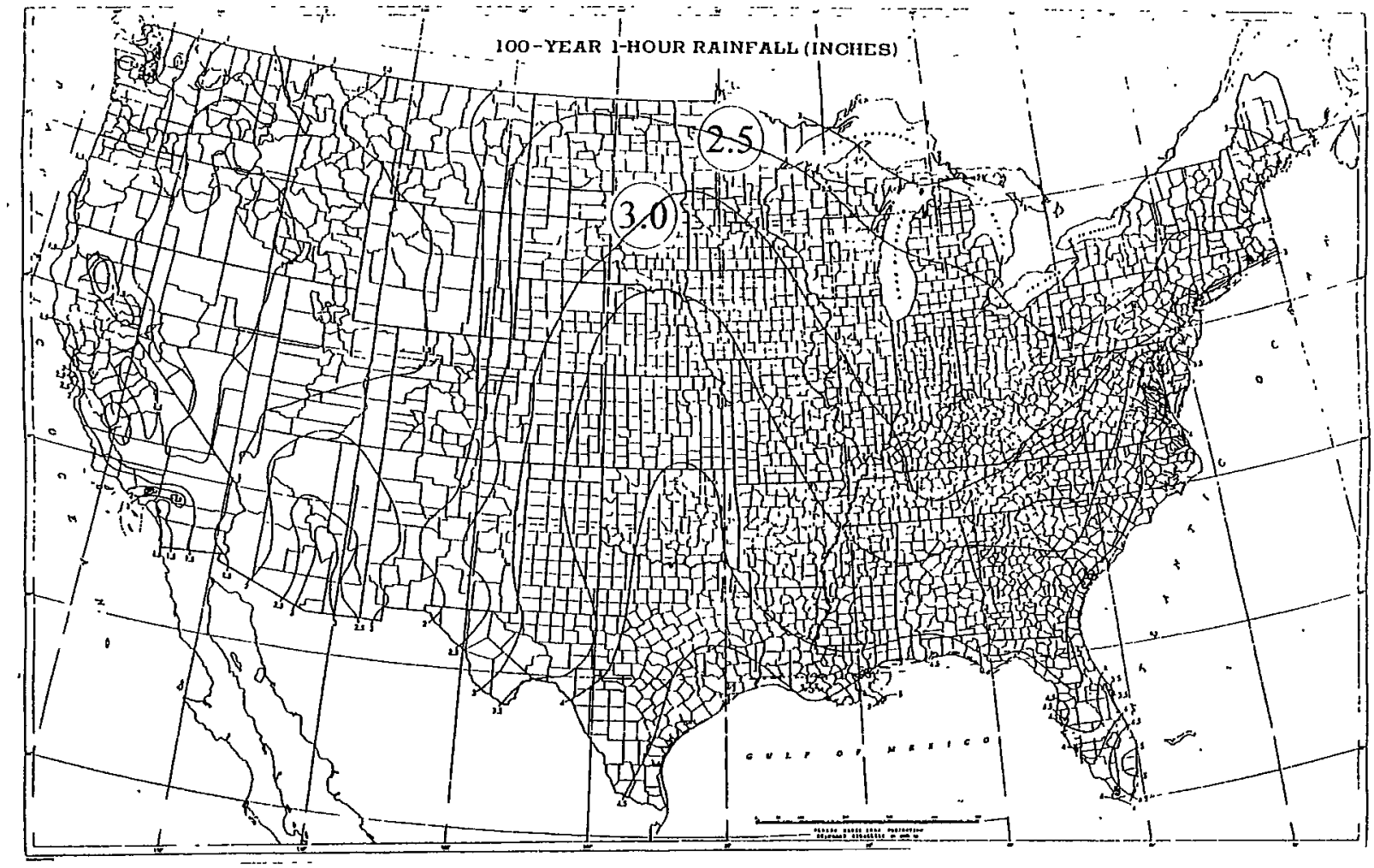


Figure A.6.3
100-Year 1-Hour Rainfall Depth Map (U.S. Weather Bureau Technical Paper 40)

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Appendix B - FUNDAMENTALS OF HYDRAULICS

LIST OF PARAMETERS

A	Area (ft^2)
C	Coefficient for Hazen-Williams equation
C_d	Coefficient of discharge
D	Diameter of pipe (ft)
D´	Hydraulic depth (ft)
EGL	Energy grade line
ε	Equivalent roughness (ft)
f	Friction factor
g	Acceleration due to gravity (32.2 ft/sec^2)
γ	Specific weight of the fluid (lbs/ft^3)
h	Vertical distance from the fluid surface (ft)
h_f	Pipe friction
h_L	Head loss due to pipe friction (ft)
h_m	Minor head losses (ft)
h_w	Height of water above weir crest (ft)
H_A	Mechanical energy head added to the system
H_R	Mechanical energy head removed from the system
HGL	Hydraulic grade line
L	Length of pipe (ft)
L_e	Equivalent length of pipe (ft)
m	Mass flowrate (slugs/sec)
n	Manning's roughness coefficient
ν	Kinematic viscosity of the fluid (ft^2/sec)
P	Hydrostatic pressure (lbs/ft^2)
P´	Wetted perimeter (ft)
Q	Flowrate (cfs)
R	Hydraulic radius (ft)
R_e	Reynolds number
ρ	Fluid density ($slugs/ft^3$)
S	Slope (ft/ft)
T	Top-width (ft)
V	Velocity (ft/sec)

Appendix B - FUNDAMENTALS OF HYDRAULICS

This appendix presents the fundamental principles of hydraulics and the basic concepts behind flow in pipes and open channels. These augment the materials presented elsewhere in the manual.

B.1 - CONSERVATION OF MASS

For steady flow (flow which does not vary with time), the principle of conservation of mass states that the flow of mass into a control volume must equal the flow of mass out of control volume for a given increment of time. This may be written between any two points within the control volume as:

$$m = \rho_1 A_1 V_1 = \rho_2 A_2 V_2 \qquad \text{(B.1.1)}$$

where subscripts 1 and 2 denote two sections in the flow, m = mass flow rate (slugs/sec), ρ = fluid density (slugs/ft^3), V = average velocity across the section (ft/sec), and A = area normal to the section (ft^2).

For fluids such as water which are incompressible, the density remains a constant ($\rho_1 = \rho_2$). Thus, the continuity equation is obtained.

$$Q = A_1 V_1 = A_2 V_2 \qquad \text{(B.1.2)}$$

where Q = volumetric flowrate (ft^3/s) (cfs).

B.2 - PRESSURE

The hydrostatic (stationary fluid) pressure varies directly with the fluid depth:

$$P = \gamma h \qquad \text{(B.2.1)}$$

where P = hydrostatic pressure (lbs/ft^2), γ = specific weight of the fluid (lbs/ft^3), and h = vertical distance from the fluid surface (ft).

This relationship is shown graphically in Figure B.2.1.

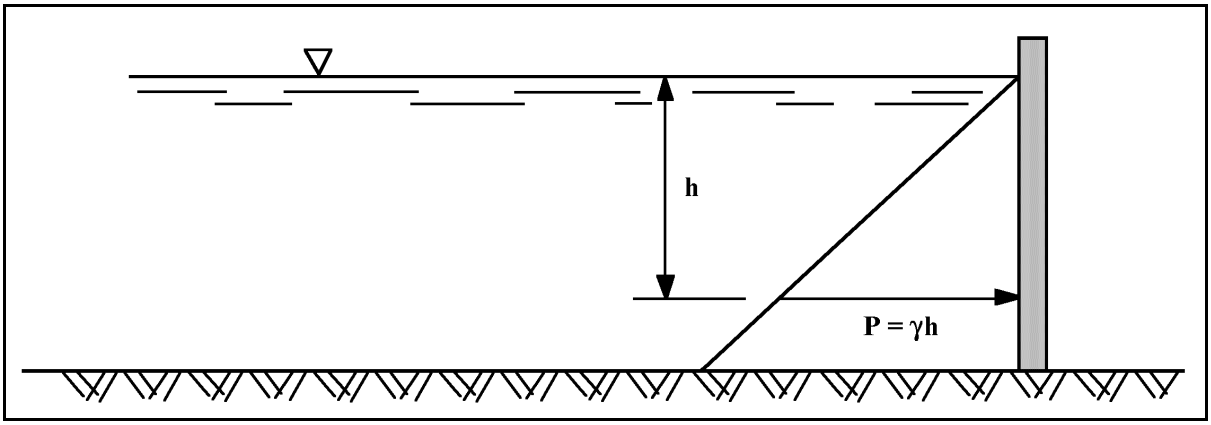


Figure B.2.1
Graphical Representation of Pressure Variation with Depth

The pressure determined by Equation B.2.1. and shown in Figure B.2.1, is termed gage pressure. Gage pressure is pressure taken from a reference scale with atmospheric pressure as zero. When the pressure is measured with respect to a perfect vacuum, it is called absolute pressure. Table B.2.1 shows the difference between the two pressure reference planes when the pressure is expressed in pounds per square inch (psi). Gage pressure is denoted as psig and absolute as psia.

Gage Pressure (psig)	Description	Absolute Pressure (psig)
20	-----	34.7
0	Normal Atmospheric Pressure	14.7
-14.7	Perfect Vacuum	0

Table B.2.1
Gage and Absolute Pressure

B.3 - ENERGY

The three types of energy which moving fluids may possess are: kinetic, potential and pressure. These are represented by the following relationships.

- Kinetic Energy/weight = $\frac{V^2}{2g}$ (ft-lbs/lbs) or (ft)

  where g = acceleration due to gravity (32.2 ft/sec^2)

- Potential Energy/weight = z (ft-lbs/lb) or (ft)

where z = elevation above some arbitrary datum

- Pressure Energy/weight = $\frac{P}{\gamma}$ (ft-lbs/lb) or (ft)

The energy of a fluid flowing across two sections (1 and 2) in a fluid continuum can be equated by writing the energy equation.

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma} + z_1 + H_A = \frac{V_1^2}{2g} + \frac{P_2}{\gamma} + z_2 + H_R + h_L \qquad \text{(B.3.1)}$$

where H_A = mechanical energy head (ft) added to the fluid system between points 1 and 2 (i.e. pump), H_R = mechanical energy head (ft) removed from the fluid system between points 1 and 2 (i.e. turbine), and h_L = head loss (ft) due to pipe friction (h_f) and obstructions such as valves, bends, etc. (minor losses, h_m) ($h_L = h_f + h_m$) between points 1 and 2.

The energy grade line (EGL) can be drawn graphically to represent the energy loss or gain along a pipe or channel. This is shown in Figure B.3.1 along with the hydraulic grade line (HGL), which is the sum of the pressure head and potential energy head [$P/\gamma + z$].

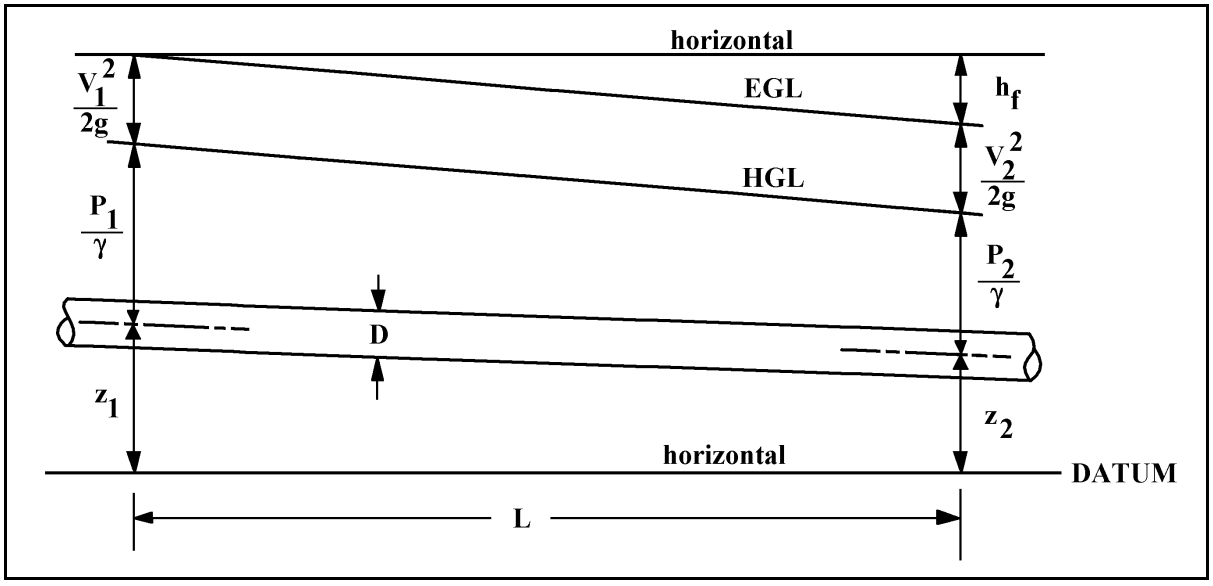


Figure B.3.1
Graphical Representation of Energy Grade Line and Hydraulic Grade Line for Pipe Flow

B.4 - FLOW IN PIPES UNDER PRESSURE

Pipe flow problems can usually be solved using the continuity and energy equation along with some method to evaluate the h_f term. There are basically two methods by which h_f may be determined: the Darcy-Weisbach and Hazen-Williams equations.

B.4.1 - Darcy-Weisbach Equation

This equation is very general and may be applied to any fluid. It is written as:

$$h_f = f \frac{L}{D} \frac{V^2}{2g} \qquad \text{(B.4.1)}$$

where L = length of pipe (ft), D = pipe diameter (ft), and f = friction factor.

The friction factor, f, may be found using the Moody Diagram (Figure B.4.1) by calculating the Reynolds number, R_e, and the relative roughness, ε/D, where ε = the equivalent roughness of the pipe wall, (ft), given in Table B.4.1, D = pipe diameter (ft), $R_e = VD/\nu$, and ν = kinematic viscosity of the fluid (ft^2/sec) (values for water are given in Table B.4.2).

The friction factor is determined by entering the graph along with the ε/D curve and moving left along the curve until the curve intersects a vertical line for the given R_e (from bottom axis). Read the left vertical axis to determine the friction factor.

The Darcy-Weisbach Equation can be rewritten to find the diameter and flowrate as presented in Equations B.4.2 and B.4.3.

$$D = \left[\frac{8 L Q^2 f}{\pi^2 g h_f} * f \right]^{1/5} \qquad \text{(B.4.2)}$$

$$Q = \left[\frac{\pi^2 D^5 g h_f}{8 L f} \right]^{\frac{1}{2}} \qquad \text{(B.4.3)}$$

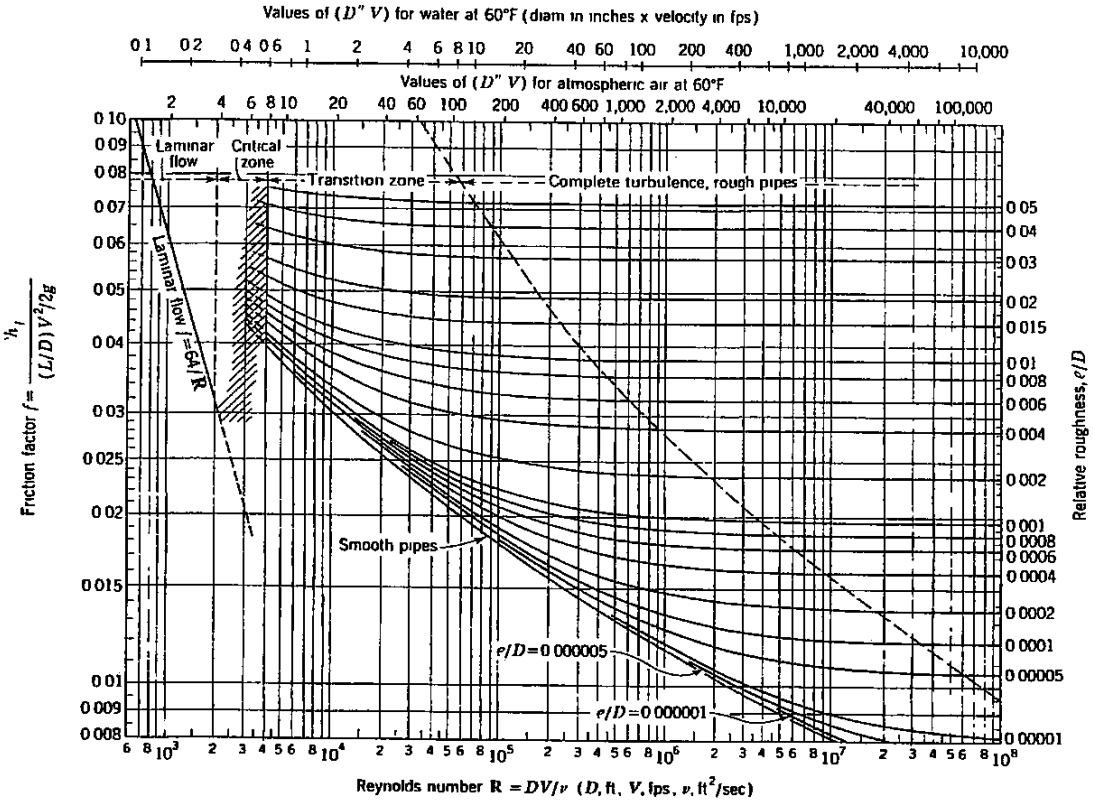


Figure B.4.1
Moody Diagram (Daugherty et al., 1985)

Another approach in estimating *f* is to use the explicit formula developed by Swamee and Jain (1976).

$$f = \frac{1.325}{\left[\ln\left(\frac{\varepsilon}{3.7D} + \frac{5.74}{R_e^{0.9}}\right)\right]^2} \qquad \text{(B.4.4)}$$

valid for $10^{-6} \le \frac{\varepsilon}{D} \le 10^{-2}$ $\quad 5000 \le R_e \le 10^8$

Table B.4.1
Equivalent Roughness

Pipe Material	ε (feet)
Riveted steel, few rivets	0.003
Riveted steel, many rivets	0.030
Concrete, finished surface	0.001
Concrete, rough surface	0.010
Wood-stave, smooth surface	0.0006
Wood-stave, rough surface	0.003
Cast iron, new	0.00085
Galvanized iron, new	0.00050
Asphalted cast iron, new	0.00040
Commercial steel, new	0.00015
Wrought iron, new	0.00015
Drawn tubing, new	0.000005
(glass, brass, copper, lead)	(essentially "smooth")

Table B.4.2
Values of Kinematic Viscosity for Water

Temp. ❍F	Kinematic Viscosity (ft2/sec)
40	1.67×10^{-5}
50	1.40×10^{-5}
60	1.21×10^{-5}
70	1.05×10^{-5}
80	9.15×10^{-6}
90	8.39×10^{-6}
100	7.36×10^{-6}

B.4.2 - Hazen-Williams Equation

This equation is only applicable for water at normal temperatures and for pipes with a diameter greater than 2 inches and a velocity less than 10 feet per second. The various forms of the equation are given below:

$$h_f = 10.6\,(Q_{mgd}/C)^{1.85}\ L/(D)^{4.87} \quad \text{(B.4.5)}$$

$$h_f = 4.73\,(Q_{cfs}/C)^{1.85}\ L/(D)^{4.87} \quad \text{(B.4.6)}$$

$$Q_{cfs} = 0.432\ C\ D^{2.63}\ (h_f/L)^{0.54} \quad \text{(B.4.7)}$$

$$Q_{mgd} = 0.279\ C\ D^{2.63}\ (h_f/L)^{0.54} \quad \text{(B.4.8)}$$

where subscripts cfs and mgd denote flow in cubic feet per second and million gallons per day, respectively.

C = coefficient which is dependent upon surface roughness (see Table B.4.3).

Table B.4.3
Values of C for the Hazen-Williams Equation

Type of Pipe	Condition		C
Cast Iron	New	All sizes	130
	5 years old	12" and Over	120
		8"	119
		4"	118
	10 years old	24" and Over	113
		12"	111
		4"	107
	20 years old	24" and Over	100
		12"	96
		4"	89
	30 years old	30" and Over	90
		16"	87
		4"	75
	40 years old	30" and Over	83
		16"	80
		4"	64
	50 years old	40" and Over	77
		24"	74
		4"	55
Welded Steel	Values of C the same as for cast iron pipes, 5 years older		
Riveted Steel	Values of C the same as for cast iron pipes, 10 years older		
Wood Stave	Average value, regardless of age		120
Concrete or Concrete-lined	Large sizes, good workmanship, steel forms		140
	Large sizes, good workmanship, wooden forms		120
	Centrifugally spun		135
Vitrified	In good condition		110
New High-Density Polyethylene Pipe			155
New PVC pipe			150
Transite (6")			140

B.4.3 - Minor Losses

Minor losses are losses due to fittings, obstructions, expansions or contractions. The most convenient method of quantifying these losses is the equivalent length technique. This technique expresses the minor loss in terms of an equivalent length of pipe which would have the same loss. This is expressed as:

$$h_m = f \frac{L_e}{D} \frac{V^2}{2g}$$

where L_e is the equivalent length of pipe.

Table B.4.4 contains a list of equivalent lengths expresses as L_e/D. Knowing the diameter of the minor loss, the equivalent corresponding length can be found.

B.4.4 - Flow in a Series Pipe System

For a pipe system which is in series, the discharge in each pipe is identical (assuming no inflow or outflow). Therefore, the total energy loss in the system is the sum of the head loss in each pipe.

$$h_f = h_{f1} + h_{f2} + ... + h_{fn}$$

This type of problem is solved by calculating the flow and headloss in each pipe and summing all the losses.

Table B.4.4
Representative Equivalent Length in Pipe Diameters (L/D) of Various Valve Fittings
(Flow of Fluids Through Valves, Fittings and Pipe,
Technical Paper 410, The Crane Company, 1969)

Description of Product				Equivalent Length In Pipe Diameters (L/D)
Globe Valves	Stem Perpendicular to Run	With no obstruction in flat, bevel, or plug type seat	Fully open	340
		With wing or pin guided disc	Fully open	450
	Y-Pattern	(No obstruction in flat, bevel, or plug type seat)		
		– With stem 60 degrees from run of pipe line	Fully open	175
		– With stem 45 degrees from run of pipe line	Fully open	145
Angle Valves		With no obstruction in flat, bevel, or plug type seat	Fully open	145
		With wing or pin guided disc	Fully open	200
Gate Valves	Wedge, Disc, Double Disc, or Plug Disc		Fully open	13
			Three-quarters open	35
			One-half open	160
			One-quarter open	900
	Pulp Stock		Fully open	17
			Three-quarters open	50
			One-half open	260
			One-quarter open	1200
Conduit Pipe Line Gate, Ball, and Plug Valves			Fully open	3
Check Valves	Conventional Swing		0.5† Fully open	135
	Clearway Swing		0.5† Fully open	50
	Globe Lift or Stop; Stem Perpendicular to Run or Y-Pattern		2.0† Fully open	Same as Globe
	Angle Lift or Stop		2.0† Fully open	Same as Angle
	In-Line Ball	2.5 vertical and 0.25 horizontal†	Fully open	150
Foot Valves with Strainer		With poppet lift-type disc	0.3† Fully open	420
		With leather-hinged disc	0.4† Fully open	75
Butterfly Valves (8-inch and larger)			Fully open	40
Cocks	Straight-Through	Rectangular plug port area equal to 100% of pipe area	Fully open	18
	Three-Way	Rectangular plug port area equal to 80% of pipe area (fully open)	Flow straight through	44
			Flow through branch	140
Fittings	90 Degree Standard Elbow			30
	45 Degree Standard Elbow			16
	90 Degree Long Radius Elbow			20
	90 Degree Street Elbow			50
	45 Degree Street Elbow			26
	Square Corner Elbow			57
	Standard Tee	With flow through run		20
		With flow through branch		60
	Close Pattern Return Bend			50

B.4.5 - Flow in a Parallel Pipe System

For a parallel pipe system, the headloss in each pipe is the same and the total flow is equal to the sum of the flow through the individual pipes.

$$Q_{TOTAL} = Q_1 + Q_2 + Q_3 + ... Q_n$$
$$h_{f1} = h_{f2} = h_{f3} = h_{fn}$$

This type of problem is solved by knowing the total flowrate and assuming a distribution of flow in the various pipes. Then the headloss is calculated for each pipe. If the losses are not equal, the flows are adjusted until they converge.

B.5 - FLOW IN OPEN CHANNELS

An open channel is a conveyance system in which the liquid stream is not completely enclosed by solid boundaries, thus the stream has a free surface subjected only to atmospheric pressure. Since the pressure is zero (gage pressure) at the surface, the energy available to cause fluid flow is due to change in elevation from one section to another in the channel. Therefore, the driving force for the flow is that component of the liquid weight which is along the slope of the channel. This driving force is resisted by a shearing force transmitted from the boundaries to the liquid.

B.5.1 - Geometric Properties of Open Channels

In order to properly evaluate the flow in open channels, certain geometric properties need to be defined. These properties which are defined below are given in Table 4.1.1 for various types of channels.

A =the cross-sectional water area (ft^2)
P´ =the wetted perimeter (ft) (the length of the cross-section boundary which is in contact with the fluid)
R =the hydraulic radius (ft) (R = A/P´)
T =the top-width (ft) (the distance across the free surface of the cross section)
D´=the hydraulic depth (ft) (D´ = A/T)

B.5.2 - Flow Calculations for Open Channels

For steady, uniform flow (flow in which the flowrate and water depth do not change from one section to another) of water the Manning Equation is used to calculate the flowrate or velocity. It may be written as:

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2} \qquad \text{(B.5.1)}$$

where n = roughness coefficient (given in Table 4.2.1).

Equation B.5.1 can be manipulated so that any of the unknown values can be found using known values.

Sometimes it is useful to separate the elements of Equation B.5.1 which depend upon the channel geometry from those which are normally known. This is accomplished by using the section factor $AR^{2/3}$.

$$AR^{2/3} = \frac{Q * n}{1.49\, S^{1/2}} \qquad \text{(B.5.2)}$$

Therefore by using a trial and error procedure, the required channel dimensions can be determined.

B.5.3 - Weir Flow

A sharp-crested weir is an obstruction placed in an open channel so that the fluid backs up and falls through a notch in the weir face. There are many types of weirs that can be used. The general relationship for a rectangular weir is given by:

$$Q = \frac{2}{3} C_d \sqrt{2g}\, L\, h_w^{2/3} \qquad \text{(B.5.3)}$$

where C_d = discharge coefficient, L = width of weir crest (ft), and h_w = depth upstream of weir (ft). (C, which depends on the type and shape of the weir, may be found in various books on fluid mechanics or from weir manufacturers or by direct calibration.)

Refer to Chapter 6, Section 6.3.6 for additional information on shapes of weir openings.

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Appendix C - REGULATORY AGENCIES FOR DRAINAGE PROJECTS

Section	Description	Page
C.1	LOCAL REGULATORY AGENCIES	C-1
C.2	STATE REGULATORY AGENCIES	C-2
C.3	FEDERAL AGENCIES	C-3

Appendix C - REGULATORY AGENCIES FOR DRAINAGE PROJECTS

The purpose of this appendix is to provide the designer with a general list of regulatory agencies, Federal, State and local, which are commonly involved in drainage projects. This list is not necessarily exhaustive, but is presented to give a general overview of the primary regulatory agencies that may have jurisdiction over a proposed drainage project.

C.1 - LOCAL REGULATORY AGENCIES

The designer should specifically check with each Local Regulatory Agency to determine if there are any local drainage ordinances that will affect the design requirement of the proposed storm drainage project.

County Drainage Board: Each county in Indiana has a County Drainage Board which has the primary responsibility for the construction, reconstruction and maintenance of all legal drains, with the exception of areas where the Board has relinquished the responsibility to sanitary districts, conservancy districts, towns or cities or any legal entity responsible for flood control and drainage (IC 19-4-1-1 thru 19-4-1-5).

County Surveyor: Each county in Indiana has a County Surveyor who has the responsibility of investigating, evaluating and surveying all legal drains. The County Surveyor also has the duty of preparing reports, plans, and profiles required for proposed land drainage improvements. In addition the County Surveyor is responsible for the removal of any obstructions from legal drains and repairing any damage, and permitting private drains to be connected to the legal drain (IC 19-4-1-9, 19-4-1-10).

City or Town Engineer: Cities or towns may employ an engineer or contract with a consulting firm to review and approve all proposed plans and documents for drainage projects. For large cities this responsibility may also be shared in part with the Board of Public Works.

Conservancy District: A conservancy district may be established for flood control or prevention, or improving drainage (IC 13-3-3-2). The district will have the authority to approve all plans and documents for a proposed drainage project.

A listing of all County officials may be obtained by calling the HERPICC office at (800) 428-7639.

C.2 - STATE REGULATORY AGENCIES

Whenever a drainage project encompasses or crosses land owned by the state, the department which has jurisdiction over that land has to be notified for approval; i.e., State Highway Department, State Park Service, etc. The following is a list of some state agencies that have some direct control over drainage projects.

Department of Natural Resources: The state of Indiana has created the Natural Resources Commission (NRC) to develop a policy of protecting water resources and preventing or limiting of floods and flood damage. The commission is authorized and empowered to represent and act for and in behalf of the state of Indiana, subject to the approval of the governor, in all matters of flood control and water resources of the state, with any state or Federal agency; to cooperate with, obtain, approve and/or accept any flood control works from and through the Corps of Engineers of the United States Army; and to cooperate with and obtain, approve and/or accept any works or grant of any character or description from and through any agency of the United States relating to flood control and water resources and to administer the funds in connection therewith (IC 13-2-22-12).

Permission must be obtained from the NRC anytime a structure, deposit or obstruction is placed in a floodway (IC 13-2-22-13).

Indiana Department of Environmental Management: The Department has been created and charged to preserve, protect and enhance the quality of the state's environment and to develop and operate programs for the most beneficial use of the resources of the state. The Department is to evolve standards and develop regulations to preserve, protect and enhance the quality of the environment. All parties discharging or proposing to discharge or emit contaminants affecting quality must furnish technical reports and receive approval as directed by the Department (IC 13-7-1-1).

C.3 - FEDERAL AGENCIES

While there are many Federal agencies which may have input to proposed drainage projects, the primary ones for Indiana are the Army Corps of Engineers and the National Resources Conservation Science (NRCS).

Corps of Engineers: The Corps will participate in urban flood-damage-reduction projects if certain criteria are met. Essentially, the drainage area must be greater than 1.5 square miles, or the flood discharge for a ten-year storm must be greater than 800 cfs. There are certain exceptions to these guidelines which are outlined in Regulation No. 1165-2-21. If these criteria are not met, all improvements are considered part of the storm sewer system and are the responsibility of local authorities (Regulation ER 1165-2-21).

There are three district offices which have jurisdiction in Indiana depending on location. The addresses for these are given below:

Chicago District
111 N. Canal St., Suite 600
Chicago, IL 60606
Phone: (312) 353-6400

Louisville District
P.O. Box 59
Attn: CEORLOR-F
Louisville, KY 40201
Phone: (502) 582-5607

Detroit District
P.O. Box 1027
Detroit, MI 48231
Phone: (313) 226-6813

There are many National Resources Conservation Service offices throughout the State of Indiana. The state has been divided into 5 areas with an Area office in each division. The following list contains the addresses for the State office in Indianapolis and the 4 Area offices. There are also field offices in each county which can be found by calling one of the Area offices or visitng the NRCS Indiana Webpage. (NRCS, July 1993).

State Office: U.S. Department of Agriculture
National Resources Conservation Service
6013 Lakeside Boulevard
Indianapolis, Indiana 46278-2933
Phone (317) 290-3200
Fax (317) 290-3225

Area Offices:

North West Area Office
1812 Troxel Drive, Suite B
Lafayette, IN 47909-7367
(765) 474-9992

North East Area Office
3718 New Vision Drive
Fort Wayne, IN 46845
(260) 484-5848

South West Area Office
2017 Hart Street
Vincennes, IN 47591-6329
(812) 882-8210 Ext. 8

South East Area Office
2600 North State Highway 7
North Vernon, IN 47265-9109
(812) 346-3411 Ext. 8

Appendix G

TR 55 Worksheet 3: Time of Concentration (T_c) or Travel Time (T_t)

Project: _____ Designed By: _____ Date: _____

Location: _____ Checked By: _____ Date: _____

Check one: ☐ Present ☐ Developed

Check one: ☐ T_c ☐ T_t ☐ through subarea _____

NOTES: Space for as many as two segments per flow type can be used for each worksheet. Include a map, schematic, or description of flow segments.

Sheet Flow (Applicable to T_c only)	Segment ID				
1. Surface description (Table 3-1)					
2. Manning's roughness coeff., n (Table 3-1)					
3. Flow length, L (total L ≤ 100 ft) ..... ft					
4. Two-year 24-hour rainfall, P_2 ..... in					
5. Land slope, s ..... ft/ft					
6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute T_t ..... hr		+		=	
Shallow Concetrated Flow	Segment ID				
7. Surface description (paved or unpaved)					
8. Flow length, L ..... ft					
9. Watercourse slope, s ..... ft/ft					
10. Average velocity, V (Figure 3-1) ..... ft/s					
11. $T_t = \frac{L}{3600\ V}$ Compute T_t ..... hr		+		=	
Channel Flow	Segment ID				
12. Cross sectional flow area, a ..... ft^2					
13. Wetted perimeter, P_w ..... ft					
14. Hydraulic radius, $r = \frac{a}{P_w}$ Compute r ..... ft					
15. Channel Slope, s ..... ft/ft					
16. Manning's Roughness Coeff., n					
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ..... ft/s					
18. Flow length, L ..... ft					
19. $T_t = \frac{L}{3600\ V}$ Compute T_t ..... hr		+		=	

20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19 ..... hr

Print Form | Reset Form | Save Form

Chapter 3: Time of concentration and travel time

Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. T_t is a component of time of concentration (T_c), which is the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system.

T_c influences the shape and peak of the runoff hydrograph. Urbanization usually decreases T_c, thereby increasing the peak discharge. But T_c can be increased as a result of (a) ponding behind small or inadequate drainage systems, including storm drain inlets and road culverts, or (b) reduction of land slope through grading.

Factors affecting time of concentration and travel time

Surface roughness

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

Channel shape and flow patterns

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

Slope

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

Computation of travel time and time of concentration

Water moves through 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.

Travel time (T_t) is the ratio of flow length to flow velocity:

$$T_t = \frac{L}{3600\ V} \qquad \text{[Eq. 3-1]}$$

where

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

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

$$T_c = T_{t_1} + T_{t_2} + \ldots T_{t_m} \qquad \text{[Eq. 3-2]}$$

where

T_c = time of concentration (hr) and
m = number of flow segments.

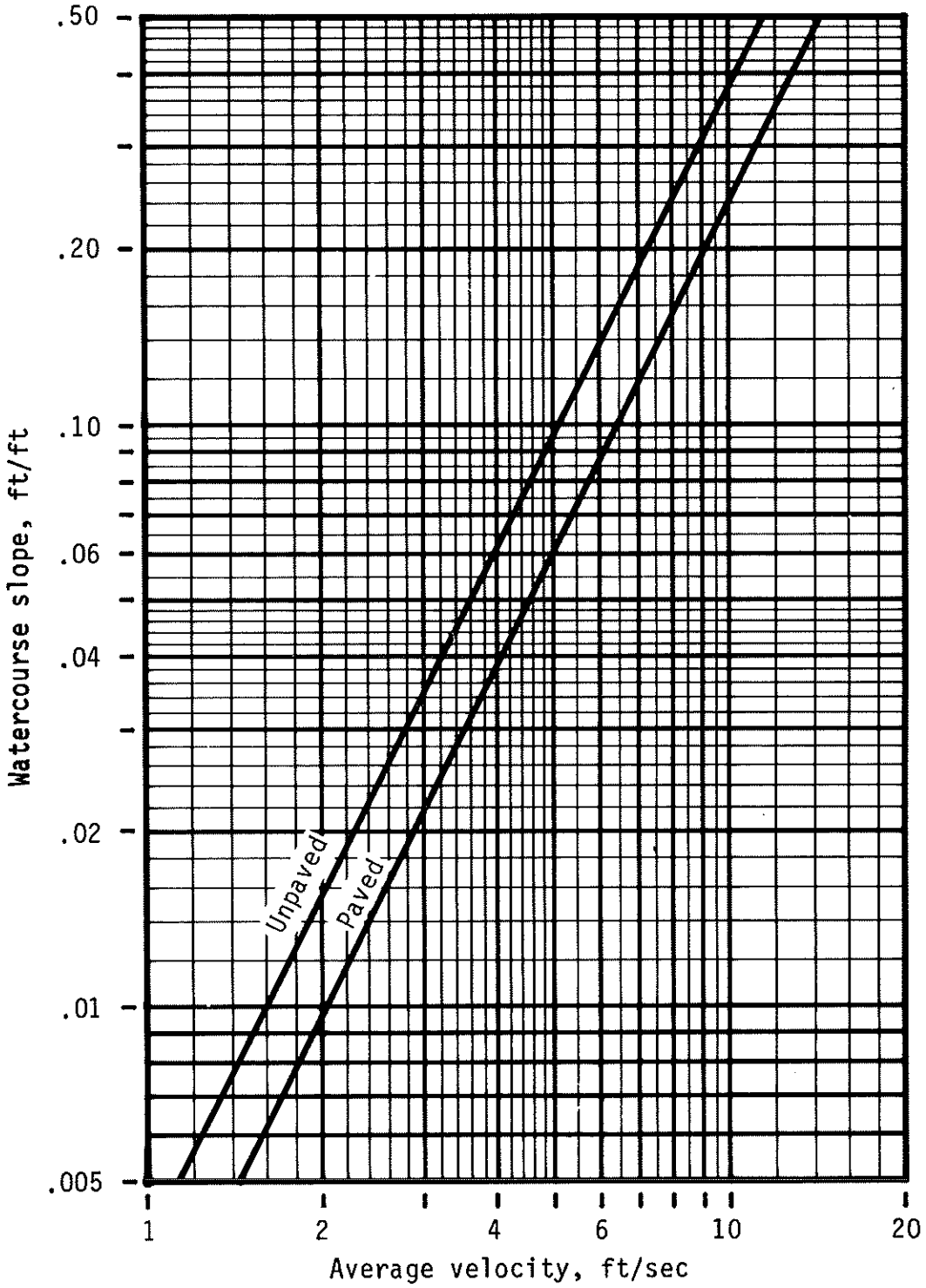


Figure 3-1.—Average velocities for estimating travel time for shallow concentrated flow.

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 3-1 gives 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:

$$T_t = \frac{0.007\ (nL)^{0.8}}{(P_2)^{0.5}\ s^{0.4}} \qquad \text{[Eq. 3-3]}$$

Table 3-1.—Roughness coefficients (Manning's n) for sheet flow

Surface description	n[1]
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤20%	0.06
Residue cover >20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses[2]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods:[3]	
Light underbrush	0.40
Dense underbrush	0.80

[1]The n values are a composite of information compiled by Engman (1986).
[2]Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
[3]When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

where

T_t = travel time (hr),
n = Manning's roughness coefficient (table 3-1),
L = flow length (ft),
P_2 = 2-year, 24-hour rainfall (in), 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. Rainfall depth can be obtained from appendix 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 3-1, in which average velocity is a function of watercourse slope and type of channel. For slopes less than 0.005 ft/ft, use equations given in appendix F for figure 3-1. Tillage can affect the direction of shallow concentrated flow. Flow may not always be directly down the watershed slope if tillage runs across the slope.

After determining average velocity in figure 3-1, use equation 3-1 to estimate travel time for the shallow concentrated flow segment.

Open channels

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

$$V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n} \qquad \text{[Eq. 3-4]}$$

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 textbooks such as Chow (1959) or Linsley et al. (1982). After average velocity is computed using equation 3-4, T_t for the channel segment can be estimated using equation 3-1.

Reservoirs or lakes

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

Limitations

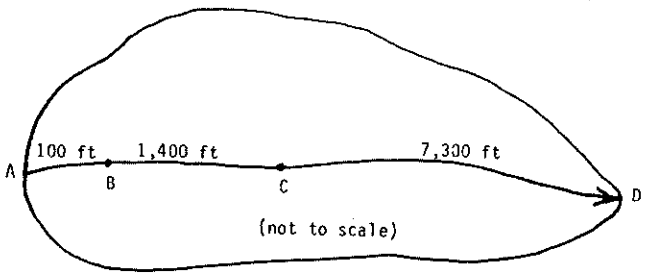
- Manning's kinematic solution should not be used for sheet flow longer than 300 feet. Equation 3-3 was developed for use with the four standard rainfall intensity-duration relationships.
- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate T_c. Storm 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_c used in TR-55 is 0.1 hour.
- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. The procedures in TR-55 can be used to determine the peak flow upstream of the culvert. Detailed storage routing procedures should be used to determine the outflow through the culvert.

Example 3-1

The sketch below shows a watershed in Dyer County, northwestern Tennessee. The problem is to compute T_c at the outlet of the watershed (point D). The 2-year 24-hour rainfall depth is 3.6 inches. All three types of flow occur from the hydraulically most distant point (A) to the point of interest (D). To compute T_c, first determine T_t for each segment from the following information:

Segment AB: Sheet flow; dense grass; slope (s) = 0.01 ft/ft; and length (L) = 100 ft.
Segment BC: Shallow concentrated flow; unpaved; s = 0.01 ft/ft; and L = 1400 ft.
Segment CD: Channel flow; Manning's n = .05; flow area (a) = 27 ft^2; wetted perimeter (p_w) = 28.2 ft; s = 0.005 ft/ft; and L = 7300 ft.

See figure 3-2 for the computations made on worksheet 3.



Worksheet 3: Time of concentration (T_c) or travel time (T_t)

Project Heavenly Acres By DW Date 10/6/85

Location Dyer County, Tennessee Checked RW Date 10/8/85

Circle one: Present (Developed)

Circle one: (T_c) T_t through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)		Segment ID	AB		
1. Surface description (table 3-1)			DENSE GRASS		
2. Manning's roughness coeff., n (table 3-1)			0.24		
3. Flow length, L (total L ≤ 300 ft)	ft		100		
4. Two-yr 24-hr rainfall, P_2	in		3.6		
5. Land slope, s	ft/ft		0.01		
6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$	Compute T_t	hr	0.30	+	= 0.30
Shallow concentrated flow		Segment ID	BC		
7. Surface description (paved or unpaved)			Unpaved		
8. Flow length, L	ft		1400		
9. Watercourse slope, s	ft/ft		0.01		
10. Average velocity, V (figure 3-1)	ft/s		1.6		
11. $T_t = \frac{L}{3600\ V}$	Compute T_t	hr	0.24	+	= 0.24
Channel flow		Segment ID	CD		
12. Cross sectional flow area, a	ft^2		27		
13. Wetted perimeter, p_w	ft		28.2		
14. Hydraulic radius, $r = \frac{a}{p_w}$	Compute r	ft	0.957		
15. Channel slope, s	ft/ft		0.005		
16. Manning's roughness coeff., n			0.05		
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$	Compute V	ft/s	2.05		
18. Flow length, L	ft		7300		
19. $T_t = \frac{L}{3600\ V}$	Compute T_t	hr	0.99	+	= 0.99
20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19)		hr			1.53

Figure 3-2.—Worksheet 3 for example 3-1.