

# Stormwater Drainage Manual

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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

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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**

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## 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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## Chapter 2 - RAINFALL

### LIST OF PARAMETERS

$c, d, \alpha, \beta$	Regional coefficients for use in rainfall intensity-duration-frequency equations
$C_s$	Skewness coefficient
$\Delta D$	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)
$\Delta t$	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.

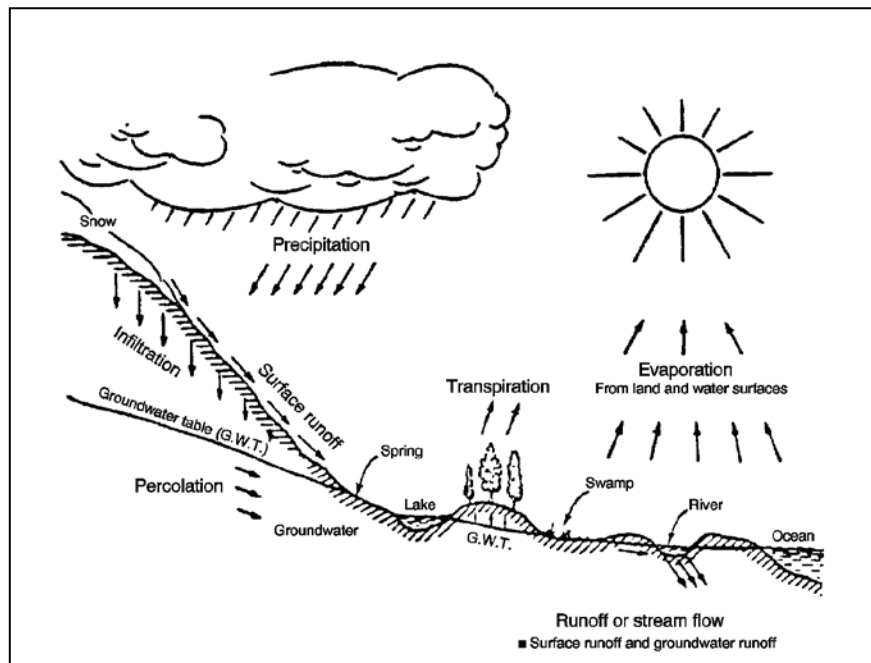


Figure 2.1.1  
The Water Cycle (Fair et al., 1971)

#### 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

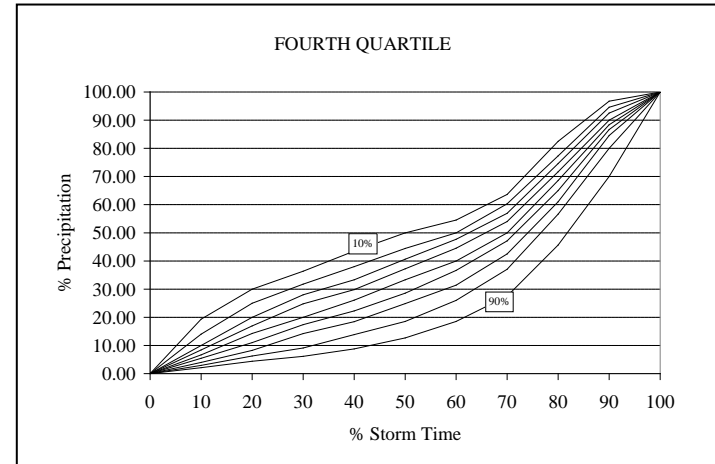
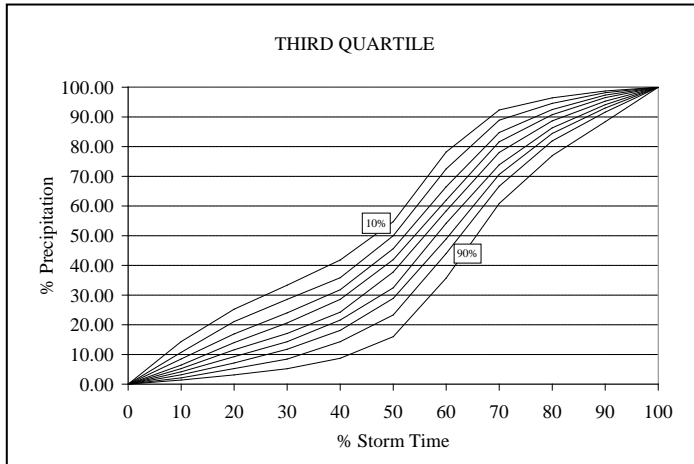
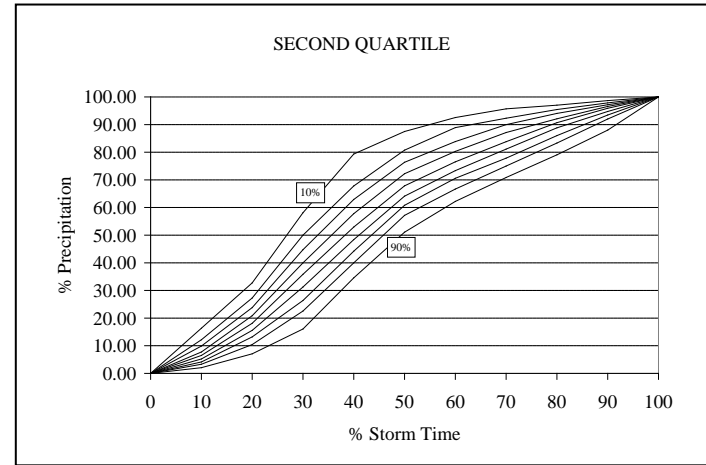
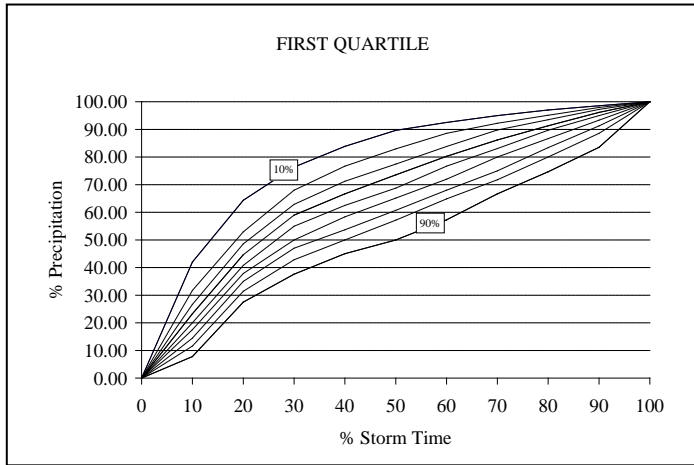
Table 2.1.8  
80% Huff Curve Ordinates (Purdue et al., 1992)

% 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)



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**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  $3.28 \times 80.83 / 100 = 2.65$  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  $2.46 \text{ in.} - 2.21 \text{ in.} = 0.25 \text{ in.}$  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

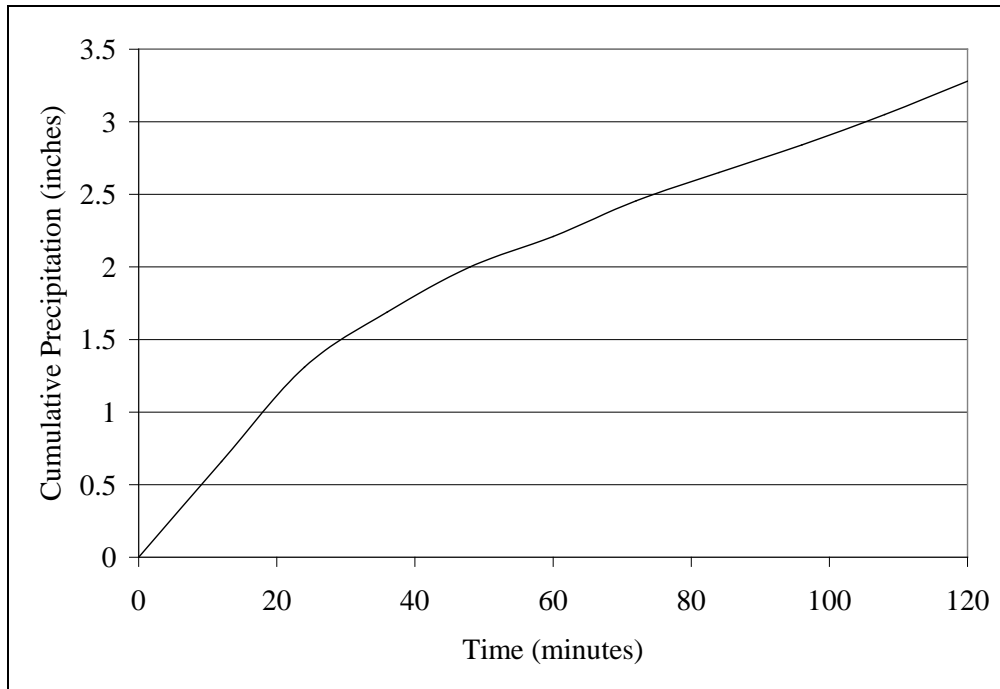


Figure 2.1.3  
Cumulative Rainfall as a Function of Time, Example 2.1.1

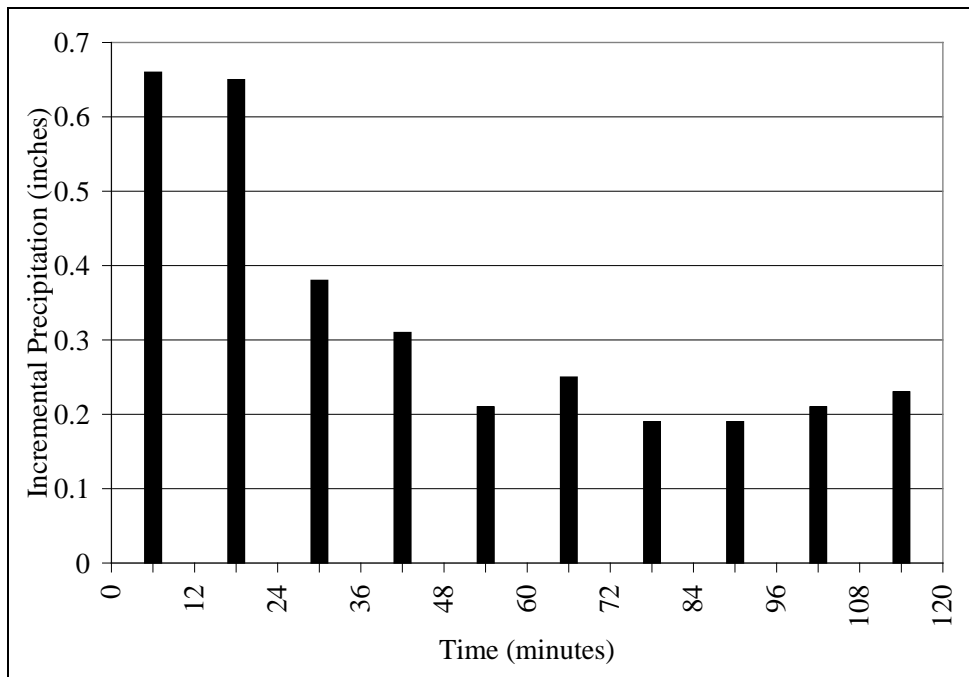


Figure 2.1.4  
Incremental Rainfall as a Function of Time, Figure 2.1.1

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.

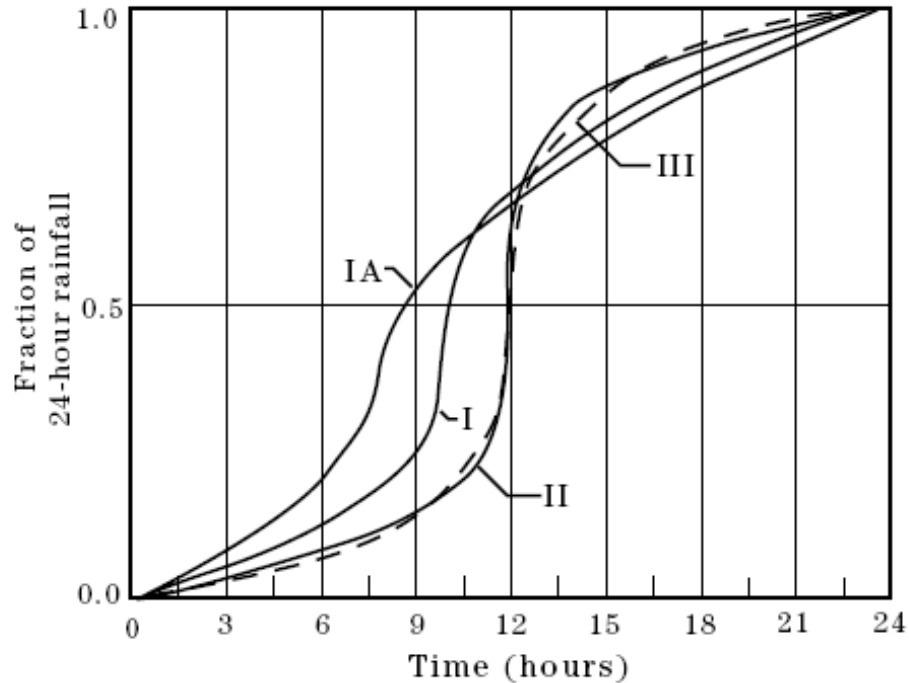


Figure 2.1.5  
SCS 24-hour Rainfall Distribution (NEH-4)

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., 1<sup>st</sup> Quartile, 2<sup>nd</sup> 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](http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_temporal.html) (Bonnin et al., 2006).

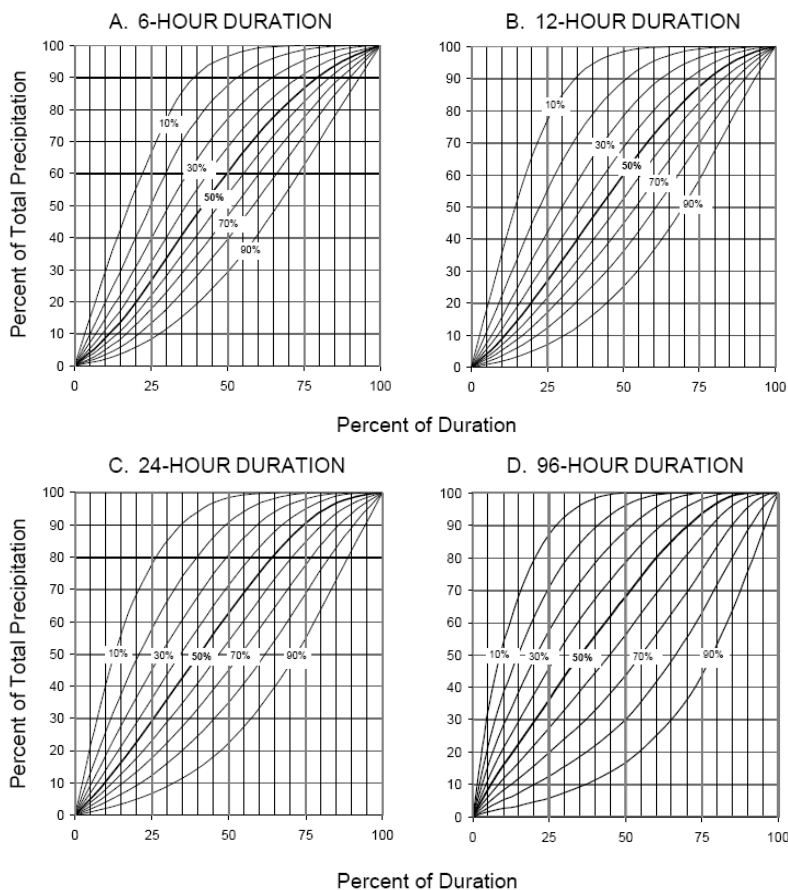


Figure 2.1.6  
Temporal Distribution for 6- through 96-Hour Storm Duration (Bonnin et al., 2006)



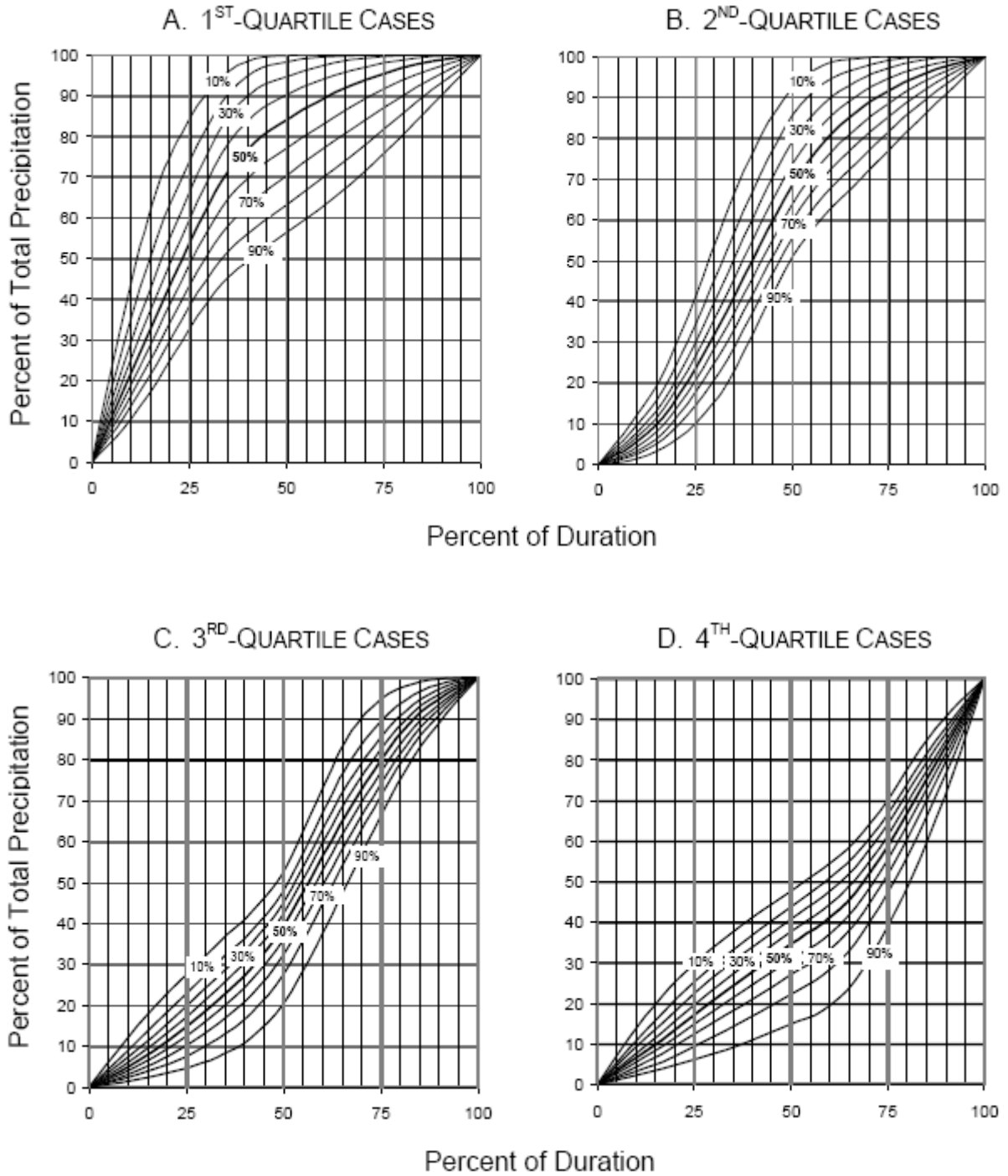


Figure 2.1.7  
 Temporal Distribution for the 6-Hour Storm Duration (Bonnin et al., 2006)

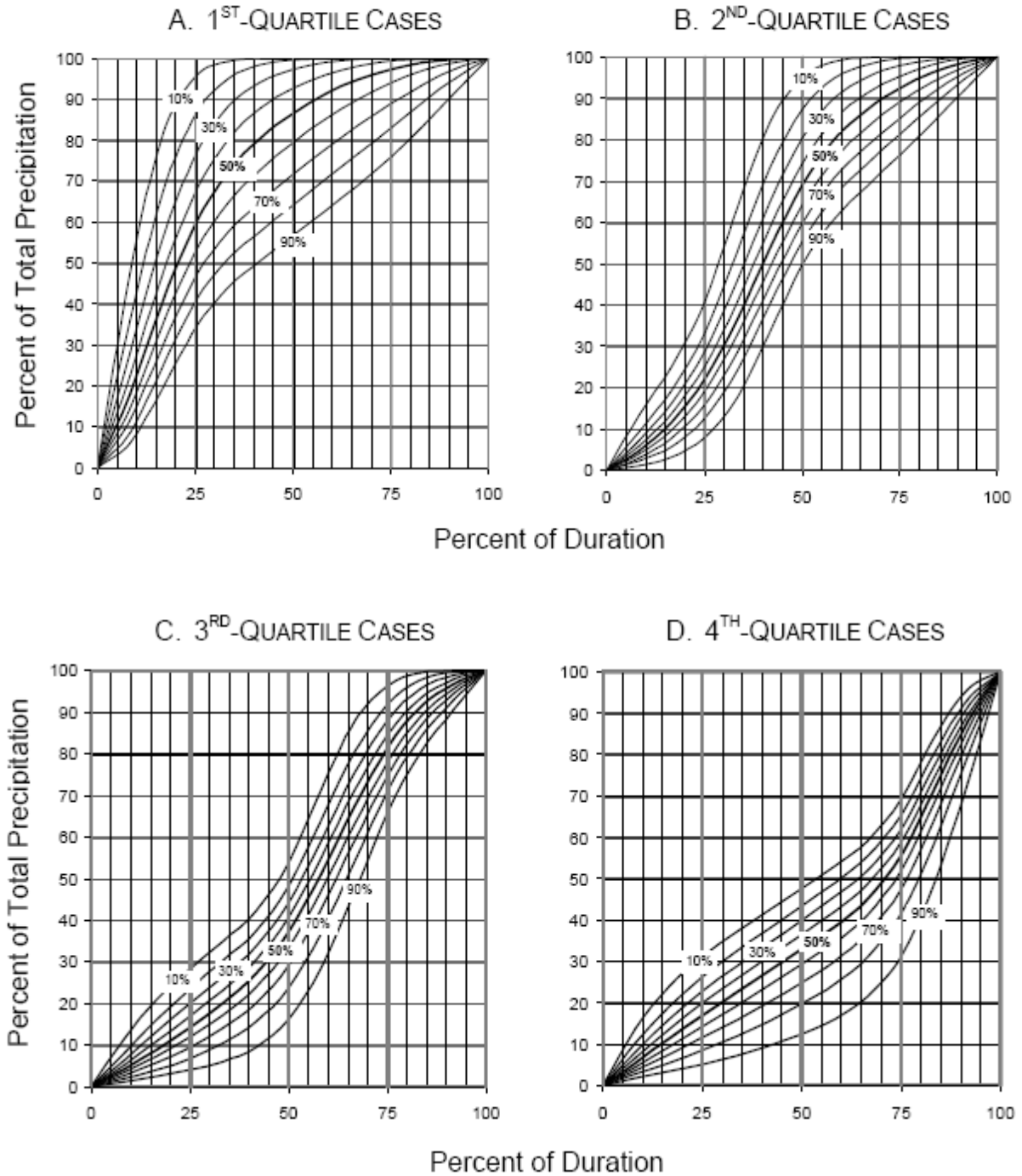


Figure 2.1.8  
 Temporal Distribution for the 12-Hour Storm Duration (Bonnin et al., 2006)

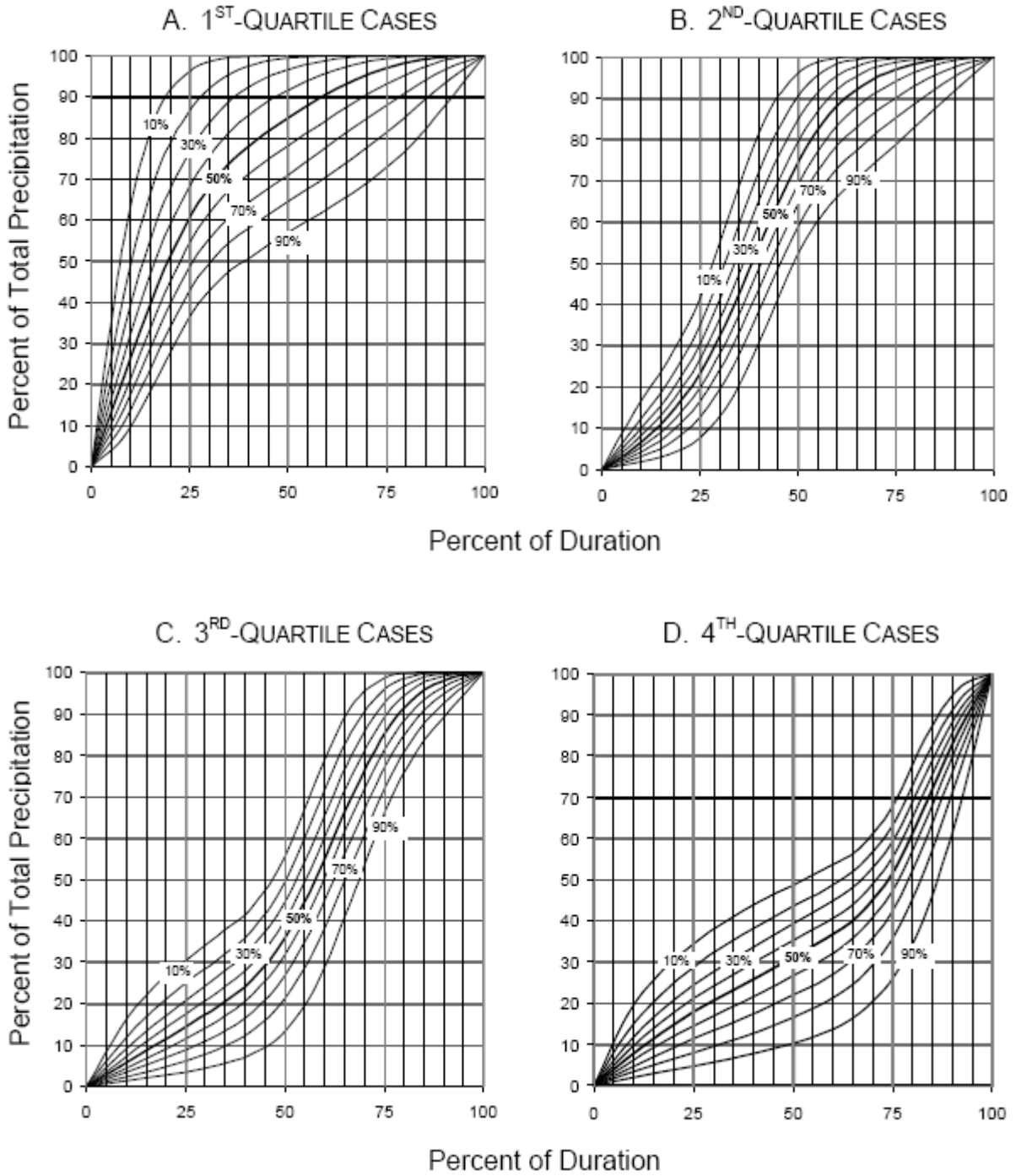


Figure 2.1.9  
Temporal Distribution for the 24-Hour Storm Duration (Bonnin et al., 2006)

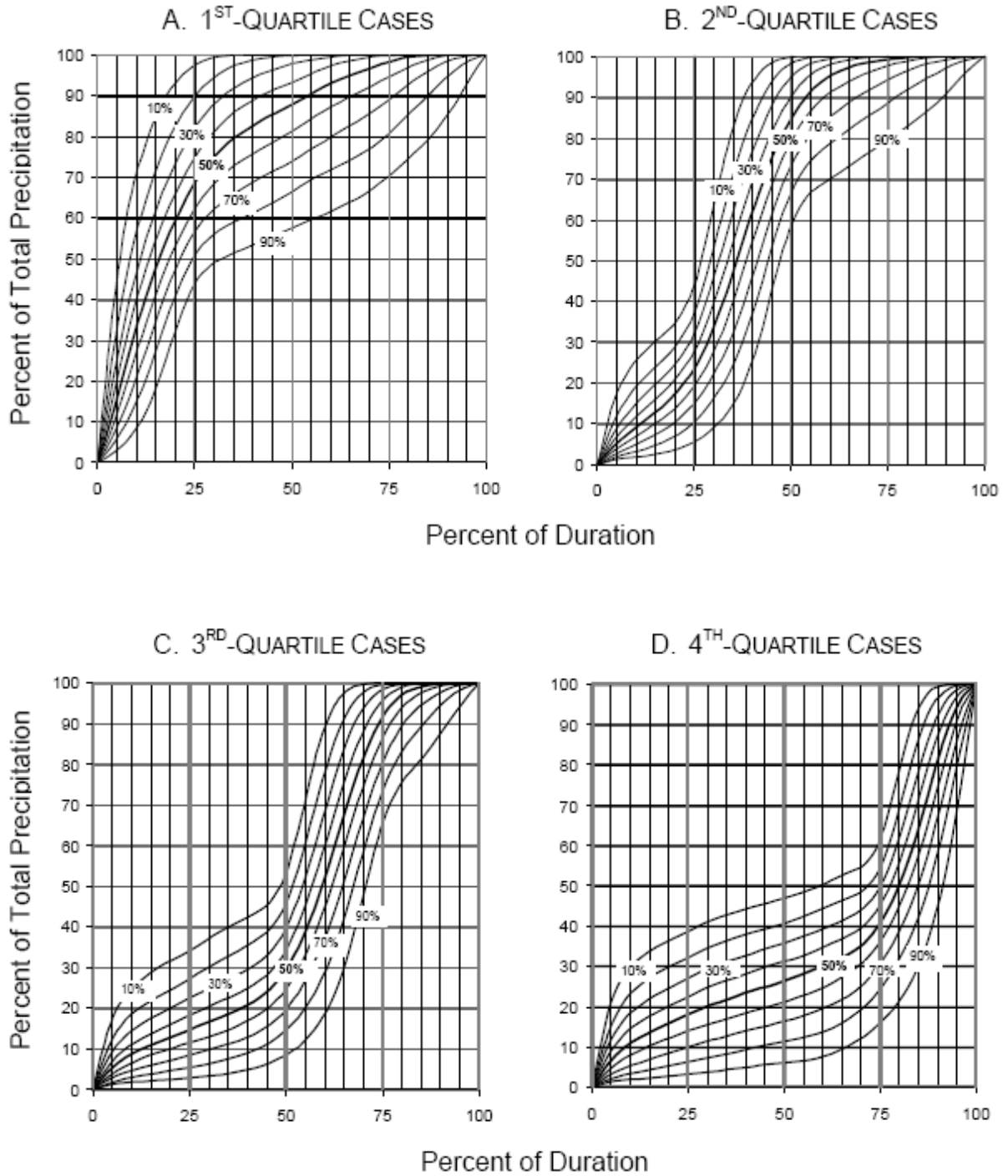


Figure 2.1.10  
 Temporal Distribution for the 96-Hour Storm Duration (Bonnin et al., 2006)

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**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.

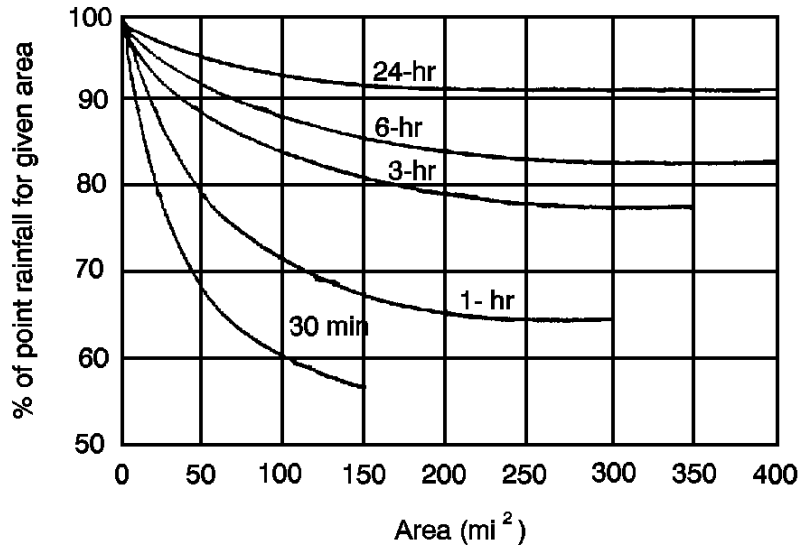


Figure 2.1.11  
Area-Depth Curves (after Hershfield, 1961)

---

#### 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  $2.00 \times 0.93 = 1.86$  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 known 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 remote 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 planimetry 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



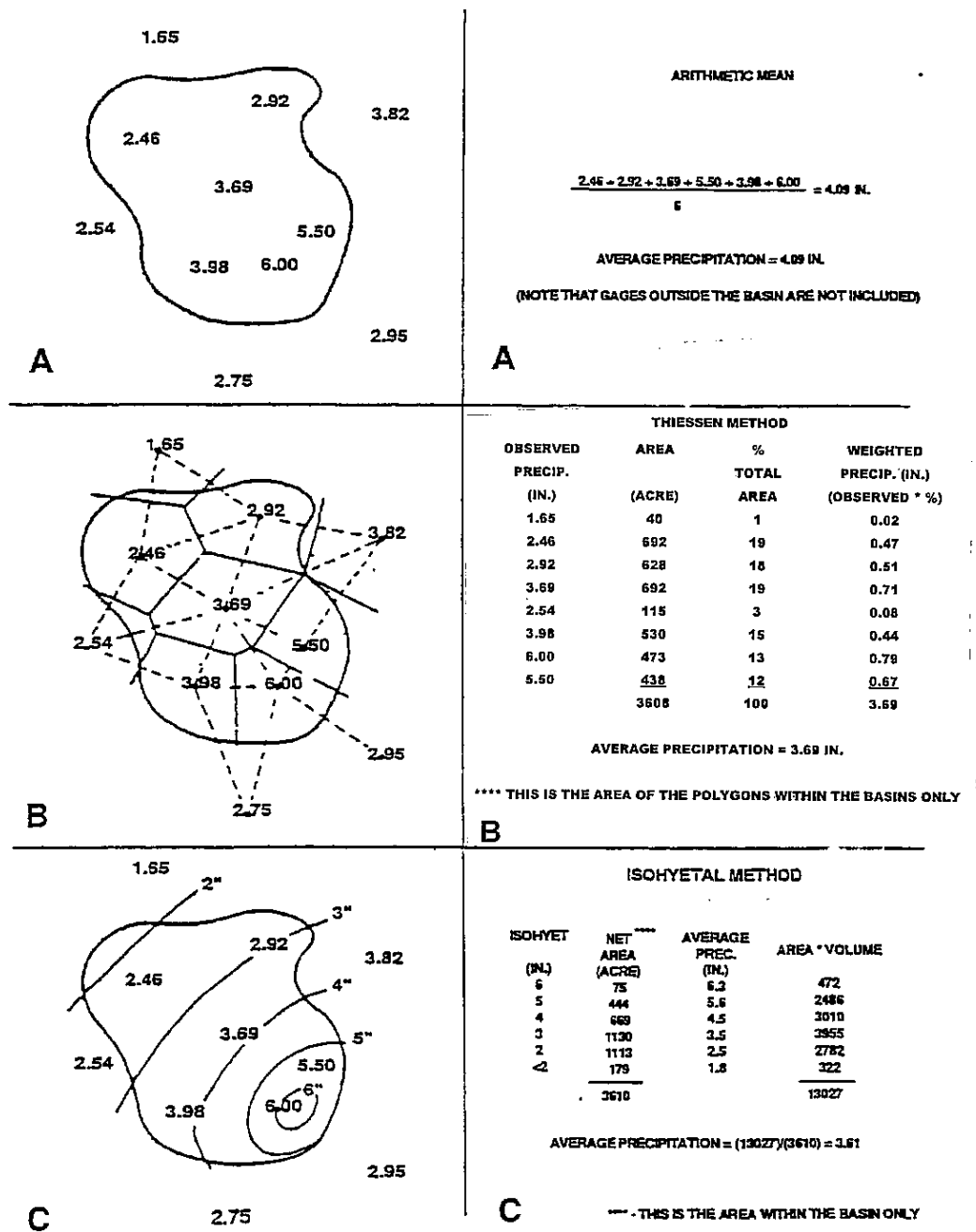


Figure 2.2.1  
Areal Averaging of Precipitation by (a) Arithmetic Method (b) Thiessen Method (c) Isohyetal Method (Linsey et al., 1975)

### 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:

$$T_r = \frac{n+1}{m} \quad (2.2.1)$$

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:

$$P = \frac{1}{T_r} = \frac{m}{1+n} \quad (2.2.2)$$

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),

$$p = \bar{p} + Ks \quad (2.2.3)$$

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:

$$\bar{p} = \frac{1}{n} \sum_{i=1}^n p_i \quad (2.2.4)$$

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:

$$s = \left[ \frac{1}{n-1} \sum_{i=1}^n (p_i - \bar{p})^2 \right]^{1/2} \quad (2.2.5)$$

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).

$$\log p = \bar{y} + K_t s_y \quad (2.2.6)$$

$$\text{where } \bar{y} = \frac{1}{n} \sum_{i=1}^n y_i \quad (2.2.7)$$

$$\text{where } y_i = \log ( p_i ) \quad (2.2.8)$$

$$s_y = \left( \frac{1}{n-1} \sum_{i=1}^n ( y_i - \bar{y} )^2 \right)^{\frac{1}{2}} \quad (2.2.9)$$

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).

$$C_s = \frac{n \sum_{i=1}^n ( y_i - \bar{y} )^3}{(n-1)(n-2) s_y^3} \quad (2.2.10)$$

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:

$$\bar{p} = \frac{1}{n} \sum_{i=1}^n p_i = \frac{1}{25}(3.220 + 1.830 + \dots + 1.063) = 1.347$$

$$s = \left( \frac{1}{n-1} \sum_{i=1}^n (p_i - \bar{p})^2 \right)^{\frac{1}{2}}$$

$$= \left( \frac{1}{24} ((3.220 - 1.347)^2 + (1.830 - 1.347)^2 + \dots + (1.063 - 1.347)^2) \right)^{\frac{1}{2}}$$

$$s = 0.434$$

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.

$$p_1^{10} = 1.347 + 1.575(0.434) = 2.031 \text{ inches}$$

$$p_1^{50} = 1.347 + 3.088(0.434) = 2.687 \text{ inches}$$

## 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:

$$J = 1 - (1 - P)^n = 1 - \left(1 - \frac{1}{T_r}\right)^n \quad (2.2.11)$$

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  $n = 5$

$$J = 1 - \left(1 - \frac{1}{T_r}\right)^n = 1 - \left(1 - \frac{1}{10}\right)^5 = 0.41$$

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,  $\Delta D$ , divided by an increment of time,  $\Delta t$ , as shown in Equation 2.2.12.

$$i = \frac{\Delta D}{\Delta t} \quad (2.2.12)$$

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:

$$i = \frac{c T_r^\alpha}{(t + d)^\beta} \quad (2.2.13)$$

where  $c$ ,  $d$ ,  $\alpha$ , and  $\beta$  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,  $c=2.1048$      $\alpha=0.1733$      $d=0.470$      $\beta=1.1289$

Referring to the IDF equation,

$$i = \frac{c T_r^\alpha}{(t + d)^\beta} = \frac{2.1048(10)^{0.1733}}{\left(\frac{15}{60} + 0.470\right)^{1.1289}} = 4.545 \text{ inches/hour}$$

Table 2.2.2  
Regional Coefficients for the IDF Equation (Eq. 2.2.13)  
(Purdue et al., 1992)

Station	c	$\alpha$	d	$\beta$
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



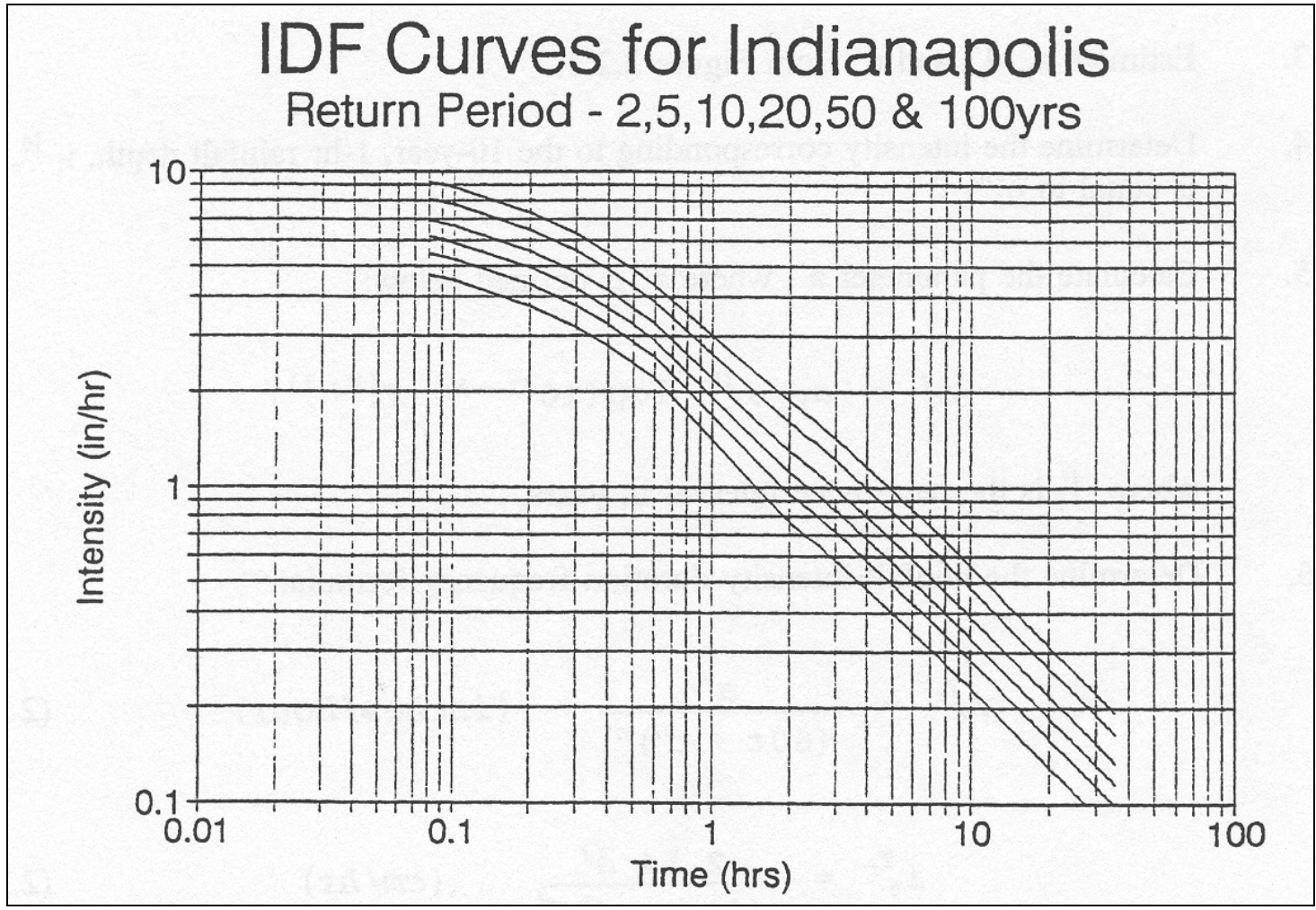


Figure 2.2.2  
Intensity-Duration-Frequency Relationship for Indianapolis by IDF Equation (adapted from Purdue et al., 1992)

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### LIST OF PARAMETERS

A	Area (acres)
A'	Area (ft <sup>2</sup> or m <sup>2</sup> )
A <sub>m</sub>	Area (mi <sup>2</sup> )
B	Coefficient for Izzard's t <sub>c</sub> 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 <sub>c</sub> formula
CN	Curve number
C <sub>m</sub>	Map skew
C <sub>s</sub>	Coefficient of skewness
C <sub>w</sub>	Weighted skew
C <sub>comp</sub>	Composite runoff coefficient
CN <sub>comp</sub>	Composite curve number
DA	Drainage area (mi <sup>2</sup> )
ΔD	Rainfall duration
F(t)	Total infiltration
G	WRC method parameter
H	WRC method parameter
i	Rainfall intensity (in/hr or cm/hr)
I <sub>a</sub>	Initial abstraction
K	Coefficient for t <sub>c</sub> equations
K <sub>n</sub>	Coefficient used in outlier detection equation
K <sub>T</sub>	Coefficient used in WRC method
L	Length (ft or m)
L'	Watershed lag
L <sub>m</sub>	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
v	min [t, NP]
P(t)	Accumulated rainfall (in)
PREC	Mean annual precipitation (inches)
q	Excess rainfall (in)
q <sub>p</sub>	Peak flow (cfs)
q <sub>t</sub>	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.

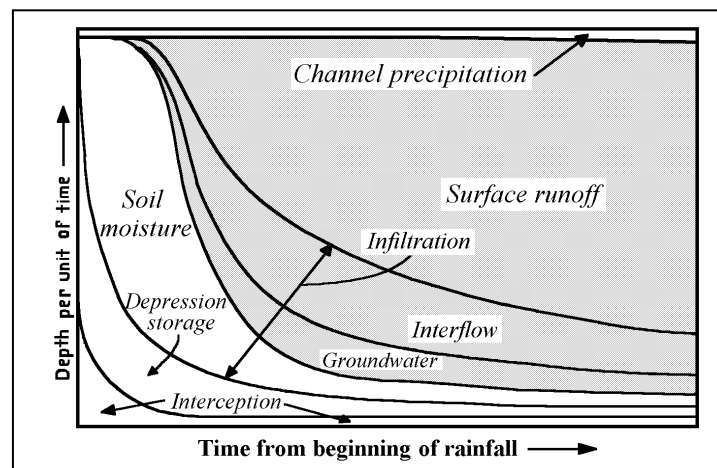


Figure 3.1.1  
Schematic Diagram of the Disposition of Storm Rainfall  
(after Linsley et al., 1975)

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:

$$Q = C i A \quad (3.2.1)$$

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,

$$Q = 0.02778 C i A \quad (3.2.2)$$

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)

<u>Vegetation and Topography</u>	<u>Open Sandy Loam</u>	<u>Soil Texture</u>	
		<u>Clay and Silt Loam</u>	<u>Tight Clay</u>
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)

<u>Description of Area</u>	<u>Runoff Coefficients</u>
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)

<u>Character of Surface</u>	<u>Runoff Coefficients</u>
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:

$$t_t = \frac{L}{v} \frac{1 \text{ minute}}{60 \text{ seconds}} = \frac{L A'}{Q} \frac{1 \text{ minute}}{60 \text{ seconds}} \quad (3.2.3)$$

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<sup>2</sup>, m<sup>2</sup>).

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)**

$$t_c = \frac{K(1.1 - C)L^{0.5}}{s^{1/3}}$$

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)

<u>Type of Surface</u>	<u>N</u>
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)

<u>Type of Surface</u>	<u><i>n</i></u>
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

<u>Surface</u>	<u><i>c'</i></u>
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:



$$C_{comp} = \frac{(C_1 A_1 + C_2 A_2 + \dots + C_n A_n)}{A_t} \quad (3.2.4)$$

where  $C_1, C_2, \dots, C_n$  are the runoff coefficients (taken from Tables 3.2.1 and 3.2.2) associated with component areas  $A_1, A_2, \dots, A_n$  and  $A_t = A_1 + A_2 + \dots + A_n$ .

**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.

<i>Basin</i>	<i>Area (acres)</i>	<i>Basin Travel Time (min)</i>	<i>Land Use</i>
Basin 1	2.4	10	Apartment Complex
Basin 2	13.7	37	Park
Basin 3	3.8	31	Light Industry

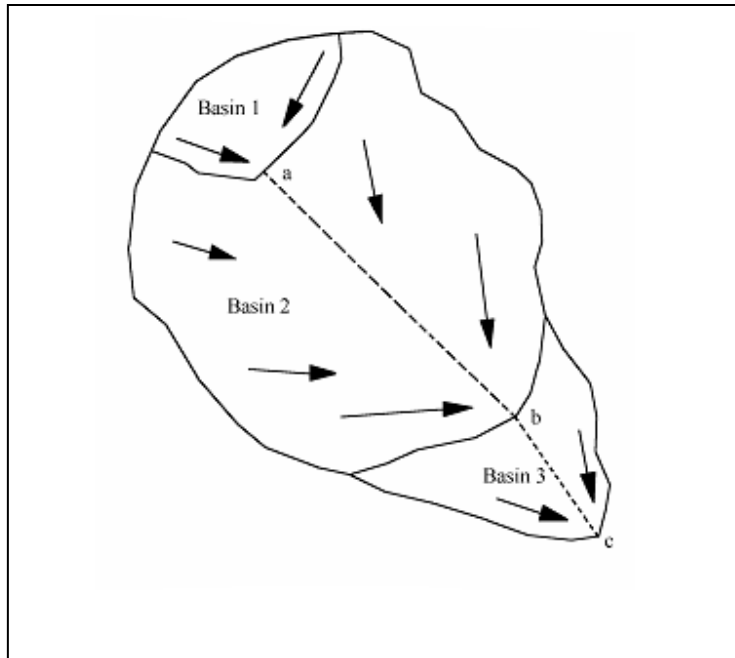


Figure 3.2.1  
Hypothetical Watershed for Example 3.2.1

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":  $C_{\text{comp}} = (0.60 \cdot 2.4) / 2.4 = 0.60$

Point "b":  $C_{\text{comp}} = (0.60 \cdot 2.4 + 0.15 \cdot 13.7) / (2.4 + 13.7) = 0.22$

Point "c":  $C_{\text{comp}} = (0.60 \cdot 2.4 + 0.15 \cdot 13.7 + 0.65 \cdot 3.8) / (2.4 + 13.7 + 3.8) = 0.30$

Step 3: Calculate the time of concentration for points "a", "b", and "c".

Point "a":  $t_c = 10 \text{ min}$

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  $600/3 \cdot 1/60 = 3.33$  minutes.

Basin 1 travel time + pipe flow time<sub>a-b</sub> =  $10 + 3.33 = 13.33$  minutes

Basin 2 travel time = 37 minutes  $t_c = 37 \text{ minutes}$

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<sub>b-c</sub> =  $37 + 0.5 = 37.5$  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 :  $c=1.7204$   $\alpha=0.1753$   $d=0.485$   $\beta=1.6806$ ,  
From Table 2.2.2 (South Bend for  $5 \text{ min} < t \leq 60 \text{ min}$ )

Point "a":

$$i = \frac{c T_r^\alpha}{(t + d)^\beta} = \frac{1.7204 \cdot (10)^{0.1753}}{\left(\frac{10}{60} + 0.485\right)^{1.6806}} = 5.29 \text{ inches/hr}$$

Point "b":

$$i = \frac{c T_r^\alpha}{(t + d)^\beta} = \frac{1.7204 * 10^{0.1753}}{\left(\frac{37}{60} + 0.485\right)^{1.6806}} = 2.19 \text{ inches/hr}$$

Point "c":

$$i = \frac{c T_r^\alpha}{(t + d)^\beta} = \frac{1.7204 * 10^{0.1753}}{\left(\frac{37.5}{60} + 0.485\right)^{1.6806}} = 2.16 \text{ inches/hr}$$

Step 5: Calculate the flowrates at points "a", "b", and "c":

Point "a":  $Q = C i A = 0.60 * 5.29 * 2.4 = 7.62 \text{ cfs}$

Point "b":  $Q = C i A = 0.22 * 2.19 * (2.4 + 13.7) = 7.76 \text{ cfs}$

Point "c":  $Q = C i A = 0.30 * 2.16 * (2.4 + 13.7 + 3.8) = 12.90 \text{ cfs}$

### 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:

$$R(t) = \frac{(P(t) - 0.2S)^2}{P(t) + 0.8S} \quad (3.3.1)$$

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.

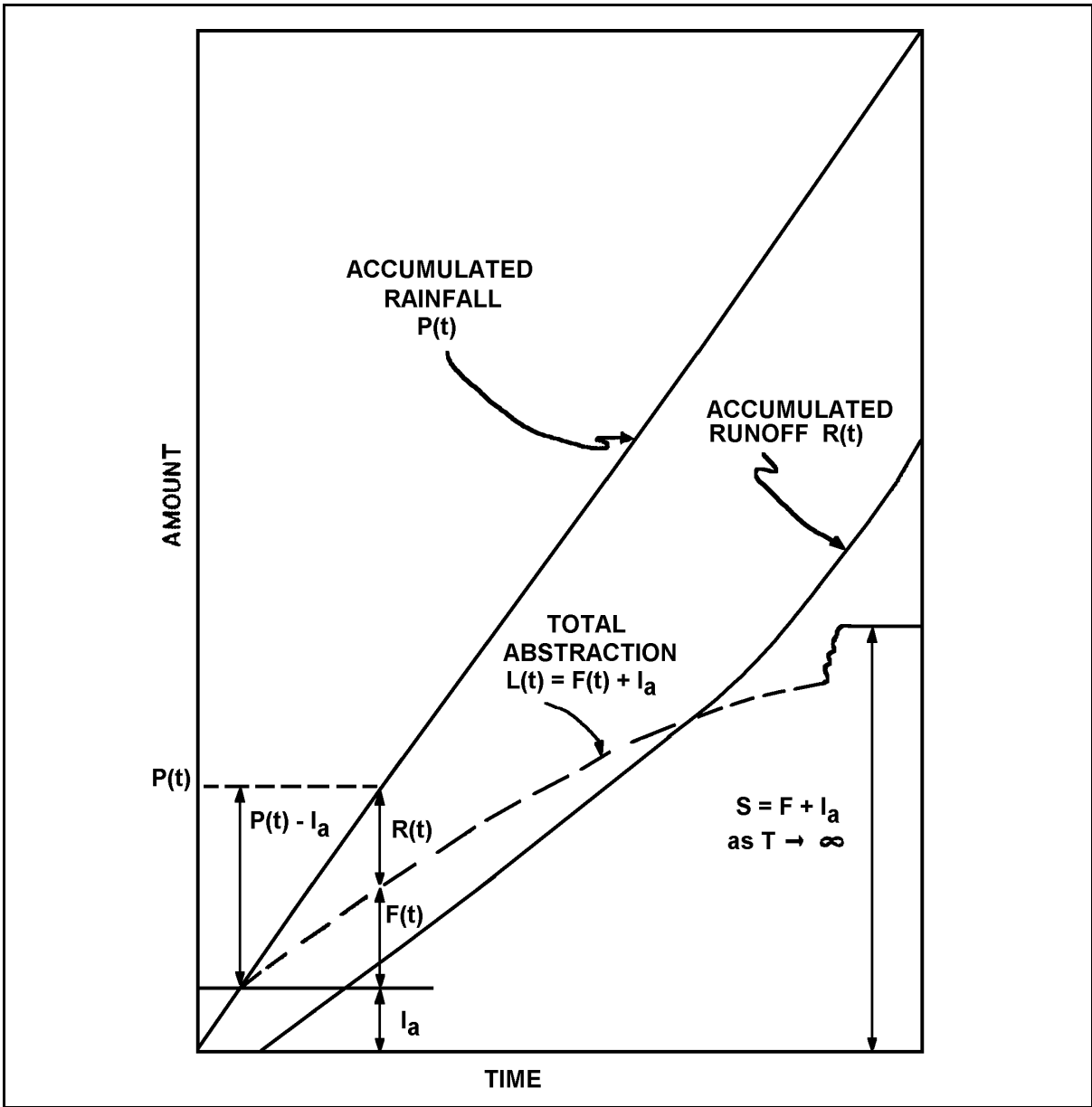


Figure 3.3.1  
Diagram of Accumulated Rainfall, Runoff and Infiltration (SCS; NEH-4)

### 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)

<u>Soil Group</u>	<u>Characteristics</u>
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:

$$S = \left( \frac{1000}{CN} \right) - 10 \quad (3.3.2)$$

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 alternative 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.

$$CN_{comp} = \frac{(CN_1 A_1 + CN_2 A_2 + \dots + CN_n A_n)}{A_t} \quad (3.3.3)$$

Where  $CN_1, CN_2, \dots, CN_n$  are the curve numbers associated with component areas  $A_1, A_2, \dots, A_n$  and  $A_t = A_1 + A_2 + \dots + A_n$ .

Table 3.3.2  
Hydrologic Soil Groups For Indiana Soils  
(Shown in parentheses after the soil classification)

Abscota (A)	Bloomfield (A)	Coesse (C/D)	Edenton (C)		Kurtz (C)
Ackerman (A/D)	Blount (C)	Cohoctah (B/D)	Edwards (B/D)	Hagerstown (C)	
Ade (A)	Bobtown (C)	Coloma (A)	Eel (B)	Haney (B)	
Adrian (A/D)	Bonnell (C)	Colyer (D)	Eldean (B)	Hanna (B)	Lafayette (B)
Alford (B)	Bonnie (C/D)	Comfrey (B/D)	Elkinsville (B)	Harpster (B/D)	La Hogue (B)
Algansee (B)	Bono (D)	Conotton (B)	Elliott (C)	Haskins (C)	Landes (B)
Algiers (C/D)	Booker (D)	Conover (C)	Elston (B)	Haubstadt (C)	Lash (B)
Alida (B)	Boonesboro (B)	Conrad (A/D)	Evansville (B/D)	Haymond (B)	Laramie (B)
Allison (B)	Boots (A/D)	Coolville (C)		Hennepin (B)	Lawrence (C)
Alvin (B)	Bourbon (B)	Corwin (B)		Henshaw (C)	Lenawee (B/D)
Ambraw (B/D)	Bowes (B)	Cory (C)	Fabius (B)	Hickory (C)	Lindside (C)
Andres (B)	Boyer (B)	Corydon (D)	Fairmount (D)	High Gap (C)	Linkville (C)
Aptakasic (B)	Brady (B)	Coupee (B)	Fairpoint (C)	Hillsdale (B)	Linwood (A/D)
Armiesburg (B)	Branch (B)	Craigmile (B/D)	Farmington (C)	Holton (C)	Lisbon (B)
Ashkum (B/D)	Brems (A)	Crane (B)	Faxon (B/D)	Homer (B)	Lobdell (B)
Atkins (D)	Brenton (B)	Crawleyville (B)	Fincastle (C)	Hononegah (A)	Lomax (B)
Aubbeenaubee (B)	Bromer (C)	Crider (B)	Flanagan (B)	Hoopeston (B)	Longlois (B)
	Bronson (B)	Crosby (C)	Foresman (B)	Hoosierville (C)	Losantville (C)
Ava (C)	Brookston (B/D)	Crosier (C)	Fox (B)	Hosmer (C)	Lucas (D)
Avonburg (D)	Bruno (A)	Cuba (B)	Frederick (B)	Houghton (A/D)	Lydick (B)
Ayr (B)	Bryce (D)	Cyclone (B/D)	Free (B/D)	Hoytville (C/D)	Lyles (B/D)
Ayrmount (B)	Burside (B)		Fulton (D)	Huntington (B)	
Ayrshire (C)				Huntsville (B)	
		Dana (B)			Mahalasville (B/D)
	Cadiz (B)	Darroch (B)	Genesee (B)		
Banlic (C)	Camden (B)	Darroch, bedrock substratum (C)	Gessie (B)	Iona (B)	Maplehill (C)
Barce (B)	Cana (C)		Gilboa (B)	Ipava (B)	Marker (B)
Barry (B/D)	Caneyville (C)		Gilford (B/D)	Iroquois (B/D)	Markham (C)
Bartle (D)	Carlisle (A/D)	Dearborn (B)		Iva (C)	Markland (C)
Battleground (B)	Carmel (C)	Del Rey (C)			Markton (C)
Baxter (B)	Carmi (B)	Deputy (C)	Gilford, stratified substratum (D)	Jasper (B)	Martinsville (B)
Beanblossom (B)	Casco (B)	Derinda (C)		Jennings (C)	Martisco (B/D)
Beasley (C)	Catlin (B)	Desker (A)		Johnsburg (D)	Maumee (A/D)
Beaucoup (B/D)	Celina (C)	Dickinson (B)	Gilpin (C)	Jules (B)	McGary (C)
Beckville (B)	Ceresco (B)	Digby (B)	Ginat (D)	Junius (C)	Medora (B)
Bedford (C)	Chagrín (B)	Door (B)	Glenhall (B)		Medway (B)
Beecher (C)	Chalmers (B/D)	Dowagiac (B)	Glynwood (C)		Mellott (B)
Belknap (C)	Chatterton (A)	Driftwood (C/D)	Gosport (C)		Mermill (B/D)
Belleville (B/D)	Cheektowaga (D)	Drummer (B/D)	Granby (A/D)	Kalamazoo (B)	Metamora (B)
Belmore (B)	Chelsea (A)	Du Page (B)	Gravelton (B/D)	Kendallville (B)	Metea (B)
Berks (C)	Chetwynd (B)	Dubois (C)	Grayford (B)	Kentland (A/D)	Miami (B)
Bethesda (C)	Cincinnati (C)	Dunning (D)	Griswold (B)	Kerston (A/D)	Miamian (C)
Bewleyville (B)	Clarence (D)		Grovecity (B)	Kibbie (B)	Middlebury (B)
Billett (B)	Cleremont (D)		Gudgel (C)	Kings (D)	Milford (B/D)
Birds (C/D)	Clyde (B/D)	Ebal (B)	Guthrie (D)	Kokomo (B/D)	Millbrook (B)
Birkbeck (B)	Cobbsford (D)	Eden (C)		Kosciusko (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)	Parr (B)	Rimer (C)	Stockland (B)	
Milton (C)	Pate (C)	Riverdale (A)	Stonelick (B)	
Monitor (C)	Patton (B/D)	Roby (C)	Stoy (C)	Wabash (D)
Montgomery (D)	Pekin (C)	Rockcastle (D)	Strawn (B)	Wakeland (C)
Montmorenci (B)	Pella (B/D)	Rockfield (B)	Strole (C)	Wallkill (C/D)
Morley (C)	Peoga (C)	Rockton (B)	Suman (B/D)	Warners (C/D)
Morocco (B)	Peotone (B/D)	Rodman (A)	Sumava (B)	Warsaw (B)
Moundhaven (A)	Petrolia (B/D)	Ross (B)	Sunbury (B)	Wasepi (B)
Muldavia (B)	Pewamo (C/D)	Rossmoyne (C)	Swanwick (D)	Washtenaw (C/D)
Mulvey (B)	Philo (B)	Ruark (B/D)	Switzerland (B)	Watseka (B)
Mundelein (B)	Piankeshaw (B)	Rush (B)	Swygert (C)	Waupecan (B)
Muren (B)	Pike (B)	Russell (B)	Sylvan (B)	Wauseon (B/D)
Muskego (A/D)	Pinevillage (B)	Ryker (B)	Symerton (B)	Wawasee (B)
Muskingum (C)	Pinhook (B/D)			Waynetown (C)
Mussey (B/D)	Piopolis (C/D)			Wea (B)
	Pipestone (B)	Sable (B/D)	Taftown (B)	Weikert (C/D)
	Plainfield (A)	Saranac (C/D)	Taggart (C)	Weinbach (C)
Napoleon (A/D)	Plano (B)		Tama (B)	Weisburg (C)
Nappanee (D)	Pope (B)	Saranac, gravelly substratum	Tawas (A/D)	Wellston (B)
Negley (B)	Princeton (A)	(C)	Tecumseh (B)	Westland (B/D)
Nesius (A)	Prochaska (A/D)		Tedrow (B)	Wheeling (B)
Newark (C)	Proctor (B)	Saugatuck (C)	Thackery (B)	Whitaker (C)
Newglarus (B)		Sawabash (B)	Throckmorton (B)	Whitson (D)
Newton (A/D)		Sciotoville (C)		Wilbur (B)
Nicholson (C)	Quinn (B/D)	Seafield (B)	Tice (B)	Wilhite (C/D)
Nineveh (B)		Sebewa (B/D)	Tilsit (C)	Willette (A/D)
Nolin (B)		Selfridge (B)	Tippecanoe (B)	Williamsport (C)
	Ragsdale (B/D)	Selma (B/D)	Toledo (D)	Williamstown (C)
	Ragsdale, overwash (B)	Seward (B)	Toronto (C)	
Oakville (A)		Shadeland (C)	Toto (B/D)	Wingate (B)
Ockley (B)	Rahm (C)	Shakamak (C)	Tracy (B)	Wirt (B)
Octagon (B)	Rainsville (B)	Shipshe (B)	Trappist (C)	Wolcott (B/D)
Odell (B)	Randolph (C)	Shoals (C)	Treaty (B/D)	Woodmere (B)
Oldenburg (B)	Rarden (C)	Sidell (B)	Trevlac (B)	Woolper (C)
Onarga (B)	Raub (C)	Simonin (B)	Troxel (B)	Wynn (B)
Ormas (B)	Rawson (B)	Sisson (B)	Tuscola (B)	
Orrville (C)	Reddick (B/D)	Skelton (B)	Tyner (A)	
Oshtemo (B)	Reesville (C)	Sleeth (C)		Xenia (B)
Otwell (C)	Rensselaer (B/D)	Sloan (B/D)		
Ouiateno (A)	Rensselaer, nonstratified substratum (C)	Sparta (A)	Uniontown (B)	Zaborosky (B)
Owosso (B)		Spinks (A)		Zadog (A/D)
		St. Charles (B)		Zanesville (C)
		St. Clair (D)	Varna (C)	Zipp (D)
Palms (A/D)	Richardsville (B)	Starks (C)	Vigo (D)	
Papineau (C)	Riddles (B)	Steff (C)	Vincennes (C/D)	
Parke (B)	Ridgeville (B)	Stendal (C)	Volinia (B)	

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
<i>Undeveloped Areas</i>				
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
<i>Fully developed urban areas (with established vegetation)</i>				
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
<i>Developing Urban Areas</i>				
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.

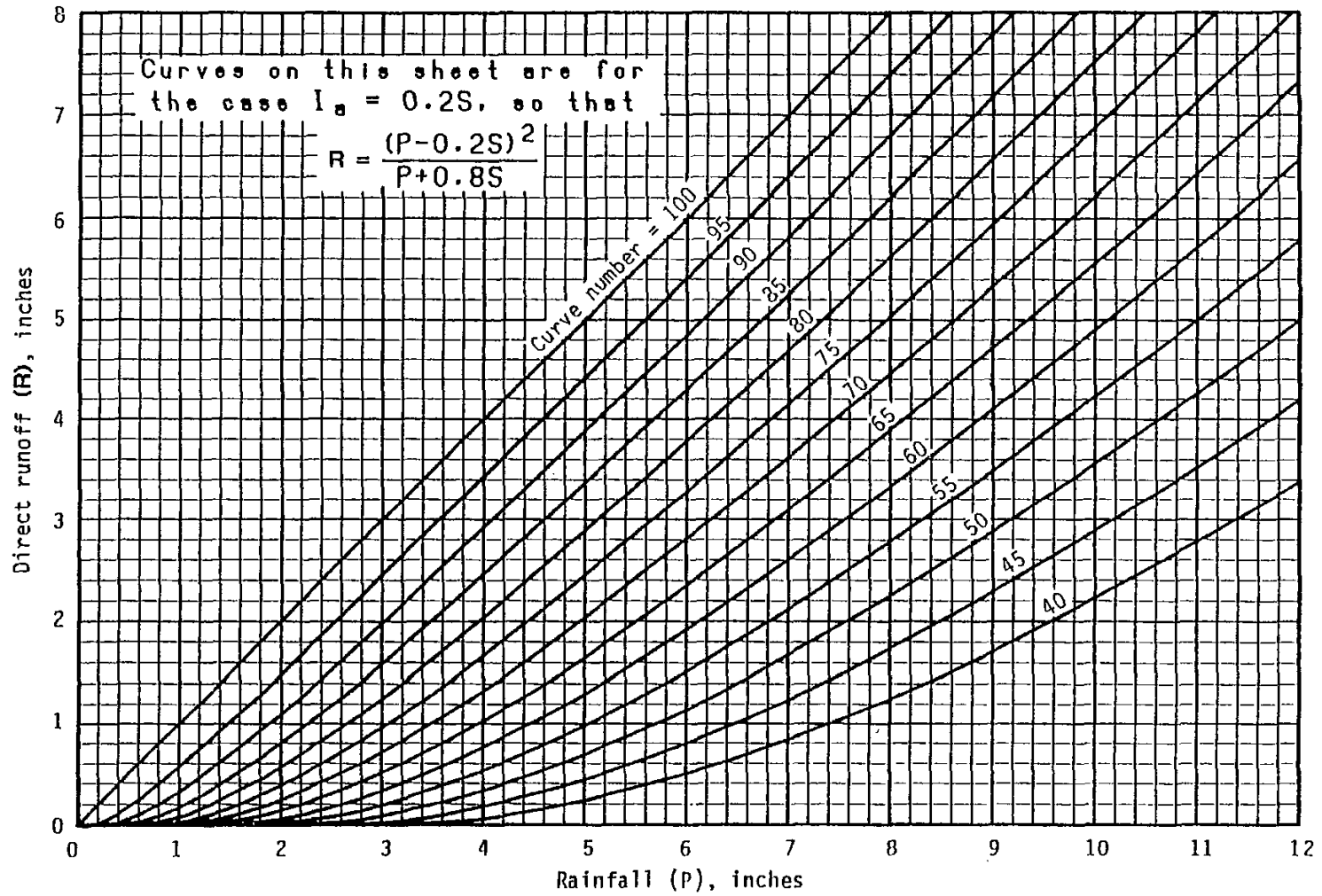


Figure 3.3.2  
Graphical Solution of Equation 3.3.1 (SCS, 1986)

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### 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:

<u>Subbasin</u>	<u>Soil Type</u>	<u>Practice</u>	<u>Condition</u>	<u>Area (acres)</u>
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.

<u>Subbasin</u>	<u>Soil Classification</u>	<u>CN</u>
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<sup>3</sup>).

$$1.096 \text{ inches} \cdot \frac{1 \text{ foot}}{12 \text{ inches}} \cdot 210 \text{ acres} \cdot \frac{43,560 \text{ ft}^2}{1 \text{ acre}} = 835,246 \text{ ft}^3$$

### 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

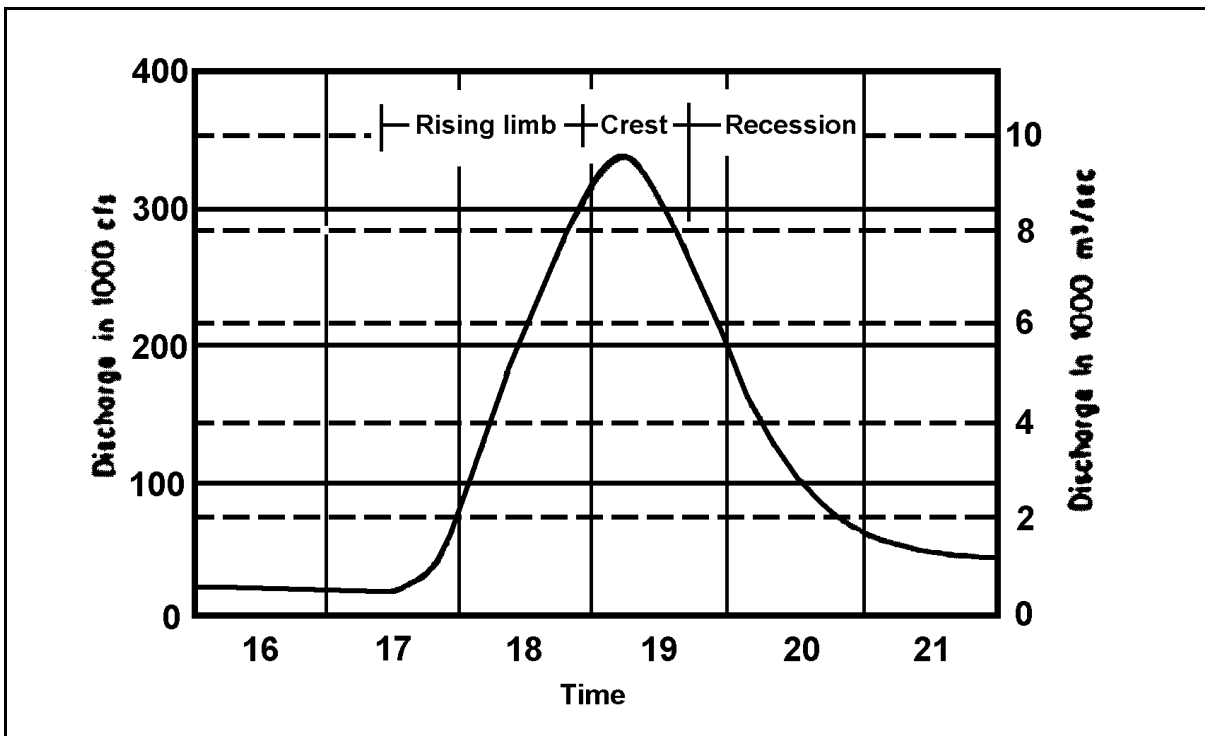


Figure 3.4.1  
Definition of Hydrograph Terms (Linsley et al., 1975)

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  $t_i$ .

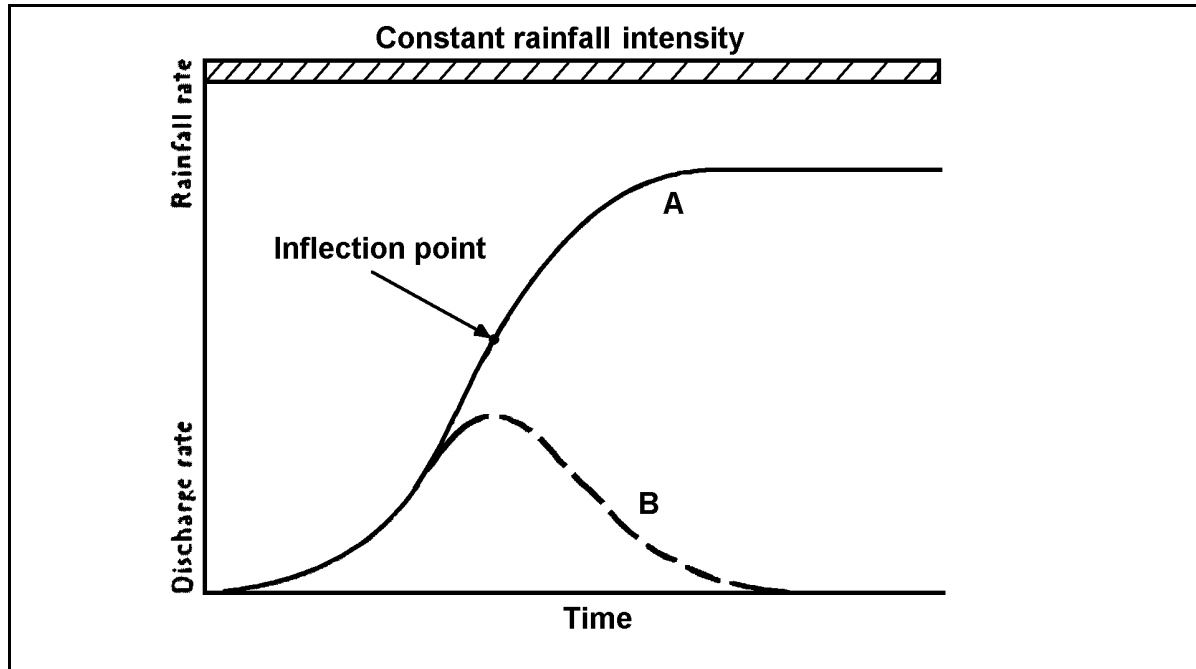


Figure 3.4.2  
Equilibrium Discharge Hydrograph (Viessman et al., 1989)

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  $1.67 t_p$ .

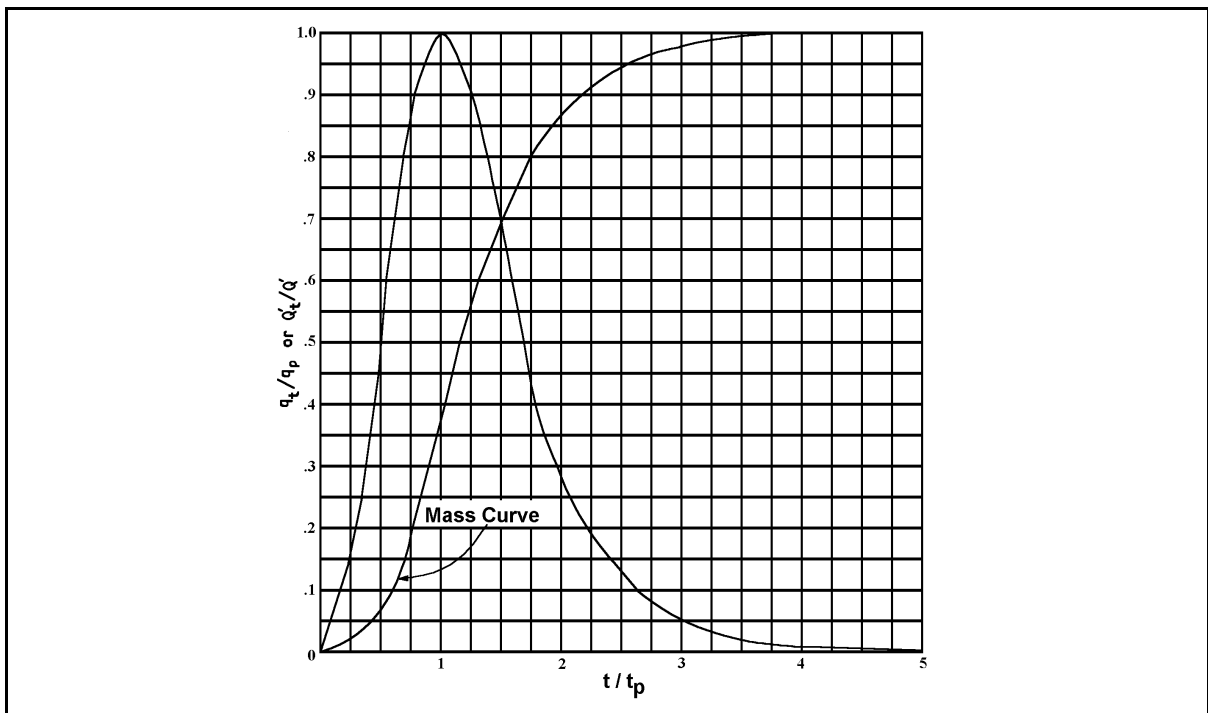


Figure 3.4.3  
Dimensionless Unit Hydrograph and Mass Curve (SCS; NEH-4)

Table 3.4.1  
Ratios for Dimensionless Unit Hydrograph  
and Mass Curve (SCS; NEH-4)

<u>Time Ratios</u>	<u>Discharge Ratios</u>	<u>Mass Curve Ratios</u>
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

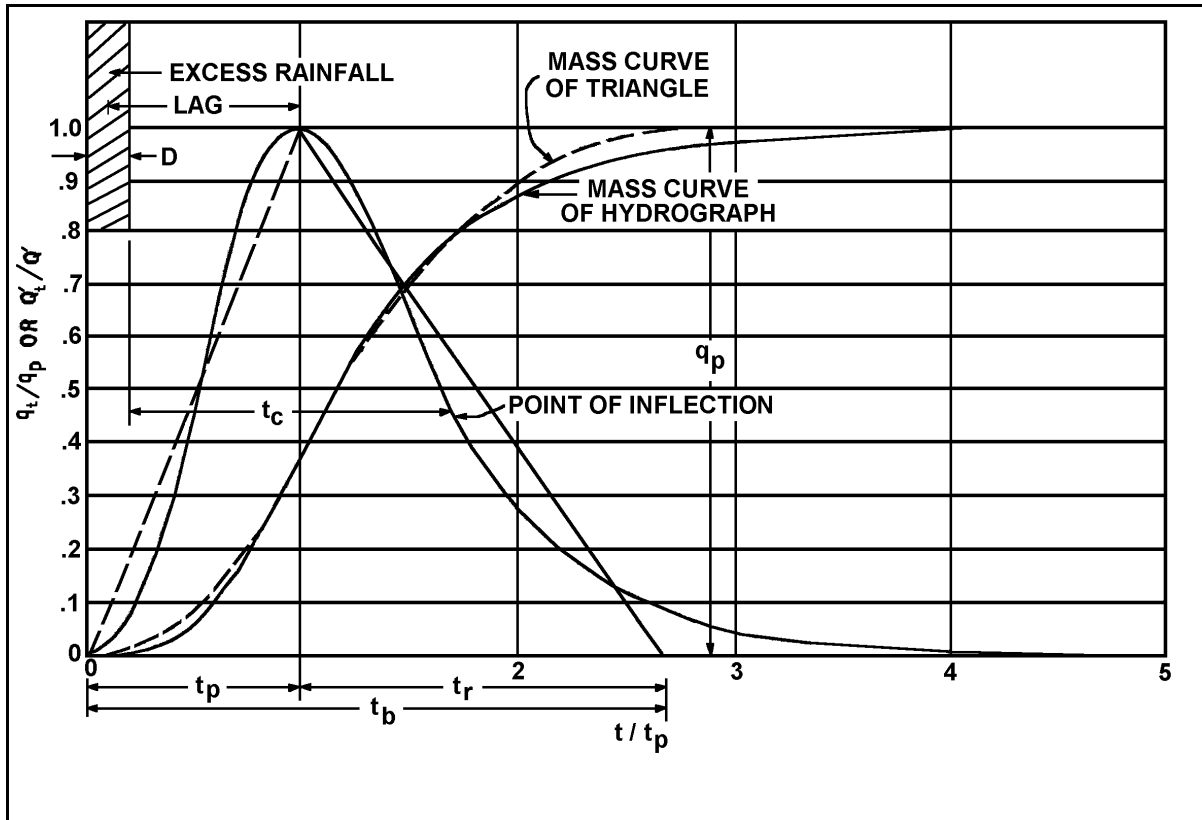


Figure 3.4.4  
Dimensionless Curvilinear Unit Hydrograph and  
Equivalent Triangular Hydrograph (SCS; NEH-4)

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:

$$V_t = Q' = \frac{t_p q_p}{2} + \frac{t_r q_p}{2} \quad (3.4.1)$$

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.

$$q_p = \frac{484 A_m Q'}{t_p} \quad (3.4.2)$$

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 ( $\Delta D$ ) and  $L'$  as shown in Equation 3.4.3.

$$t_p = \frac{\Delta D}{2} + L' = \frac{\Delta D}{2} + 0.6 t_c \quad (3.4.3)$$

$$\therefore q_p = \frac{484 A_m Q'}{t_p} = \frac{484 A_m Q'}{\frac{\Delta D}{2} + 0.6 t_c} \quad (3.4.4)$$

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  $\Delta D$  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  $\Delta D$  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.

$$t_t = \frac{L}{3600 v} \quad t_c = t_{t1} + t_{t2} + \dots t_m \quad (3.4.5)$$

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  $\Delta D$  is the rainfall duration.

$$q_p = \frac{484 A_m Q'}{t_p} \quad t_p = \frac{\Delta D}{2} + L' = \frac{\Delta D}{2} + 0.6 t_c$$

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.

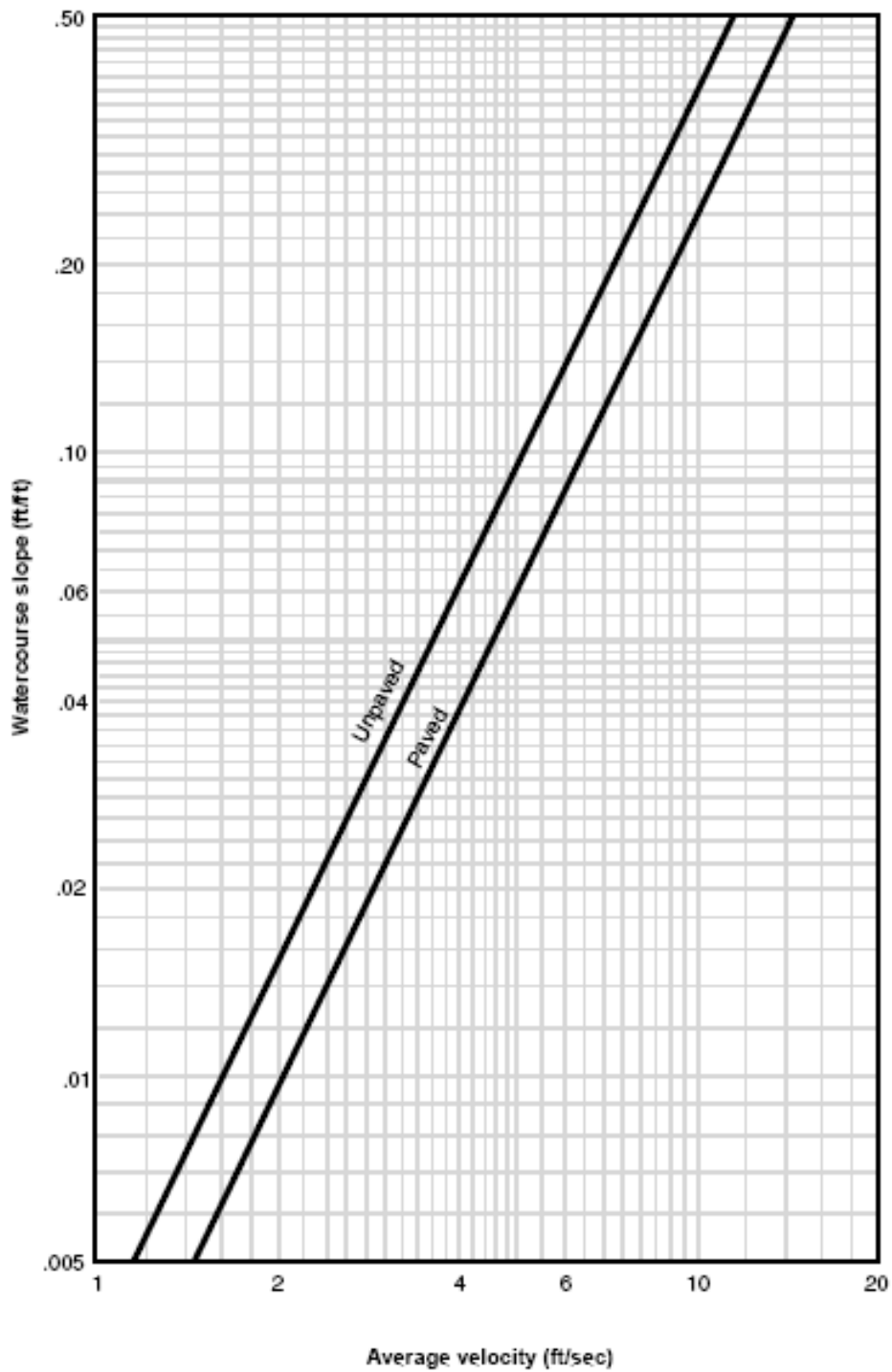


Figure 3.4.5  
Average Velocities for Estimating Travel Time (USDA, 1986)

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 ordinates of  $q_t$  and multiplying by  $\Delta D$ . This may be expressed as:

$$V_{UH} = \Delta D \sum_{i=1}^{\left(\frac{5t_p}{\Delta D}\right)} q_i \quad 3.4.6$$

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:

$$V = A_m * 645.33 \quad (3.4.7)$$

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  $\Delta D$  intervals. Tabulate the unit hydrograph at  $\Delta D$  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.



$$Q_t = \sum_{i=\mu}^{\nu} (u_{t-i+1})(q_i) \quad t = 1, \dots, NP + NU - 1 \quad (3.4.8)$$

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,  $\mu$  is  $\max [1, t+1-NU]$ , and  $\nu$  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  $\Delta D$  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<sup>2</sup>) 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

$$t_c = \frac{L}{v} = \frac{9000 \text{ ft}}{2 \text{ ft/sec}} = 4500 \text{ seconds} = 75 \text{ minutes}$$

Step 2: Determine the values of  $\Delta D$ ,  $t_p$  and  $q_p$ . From page 3-31:  
 $\Delta D = 0.133t_c = 0.133(75) = 9.98 \text{ minutes} \approx 10 \text{ minutes}$   
 $t_p = (\Delta D/2) + 0.6t_c = (10/2) + 0.6(75) = 50 \text{ min} \approx 0.83 \text{ hours}$

$$q_p = \frac{484 A_m Q'}{t_p} = \frac{484(0.47 \text{ mi}^2)(1 \text{ inch})}{0.83 \text{ hours}} = 274 \text{ cfs}$$

Step 3: Determine the coordinates of the dimensionless unit hydrograph using Table 3.4.1.  $\Delta D$  was computed to be 10 minutes.

Example Calculation:  $t = t/t_p * t_p = 0.8 * 50 = 40$                        $q = q/q_p * q_p = 0.930 * 274 = 254.82$

<u>Time Ratio</u>	<u>Time</u>	<u>Discharge Ratio</u>	<u>Discharge</u>
$t / t_p$	t (min.)	(q / $q_p$ )	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.

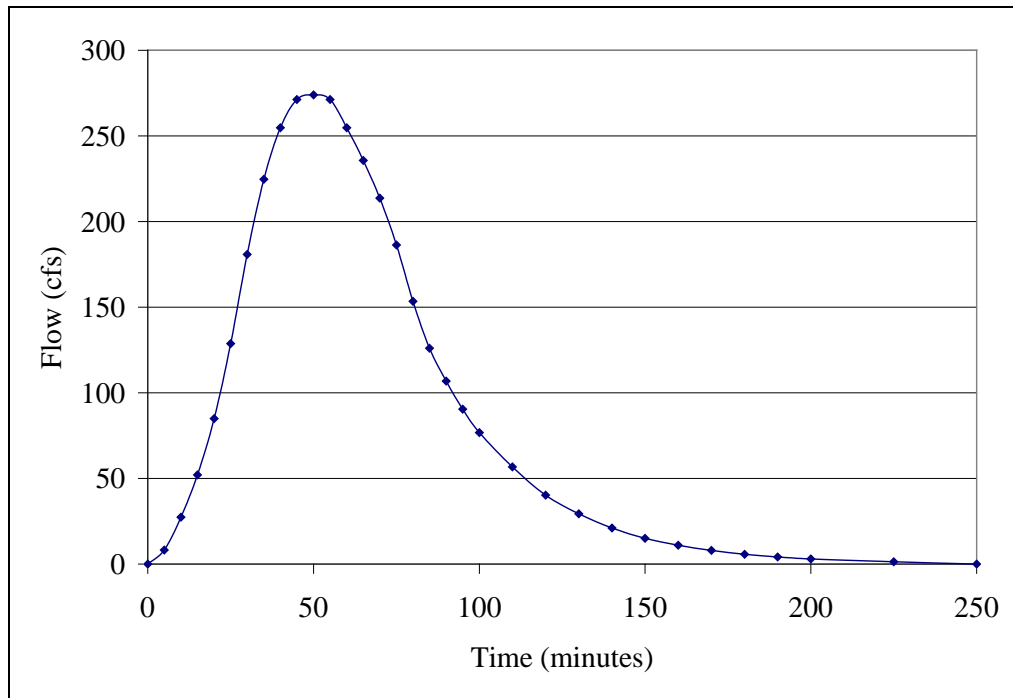


Figure 3.4.6  
Dimensionless Unit Hydrograph for Example 3.4.1

Step 4: Check to see that the volume of the unit hydrograph is equal to 1 inch. Sum the unit hydrograph ordinates.

$$(0+8.22+27.40+\dots+106.86+90.42+76.72)*5 + (56.72+40.28+\dots+4.11+3.01)*10 + (1.37+0)*25$$

$$= 18,088 \text{ cfs-min. } (*)$$

\*Note: Discharges for Time Ratio greater than 4.5 are considered zero.

$$18,088 \frac{ft^3 \text{ min}}{sec} * \frac{60 \text{ sec}}{min} * \frac{acre}{43,560 \text{ ft}^2} = 24.91 \text{ acre - feet}$$

$$1 \text{ inch} * \frac{1 \text{ ft}}{12 \text{ inches}} * 300 \text{ acres} = 25.00 \text{ acre - feet}$$

$$\frac{25.00 - 24.91}{24.91} * 100\% = 0.36\% \text{ difference}$$

Once the unit hydrograph has been established, the storm hydrograph is computed by using the Indiana LTAP Stormwater Drainage Manual - Revised December 2015 Chapter 3-36

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:

$$c = 1.2799 \quad \alpha = 0.1872 \quad d = 0.258 \quad \beta = 0.8252$$

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

$$* R(t) = \left( \frac{(P(t) - 0.2S)^2}{P(t) + 0.8S} \right) \text{ Where } S = \left( \frac{1000}{84} \right) - 10 = 1.90 \text{ inch, } R(t) = 0 \text{ if } P(t) < 0.2S$$

**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

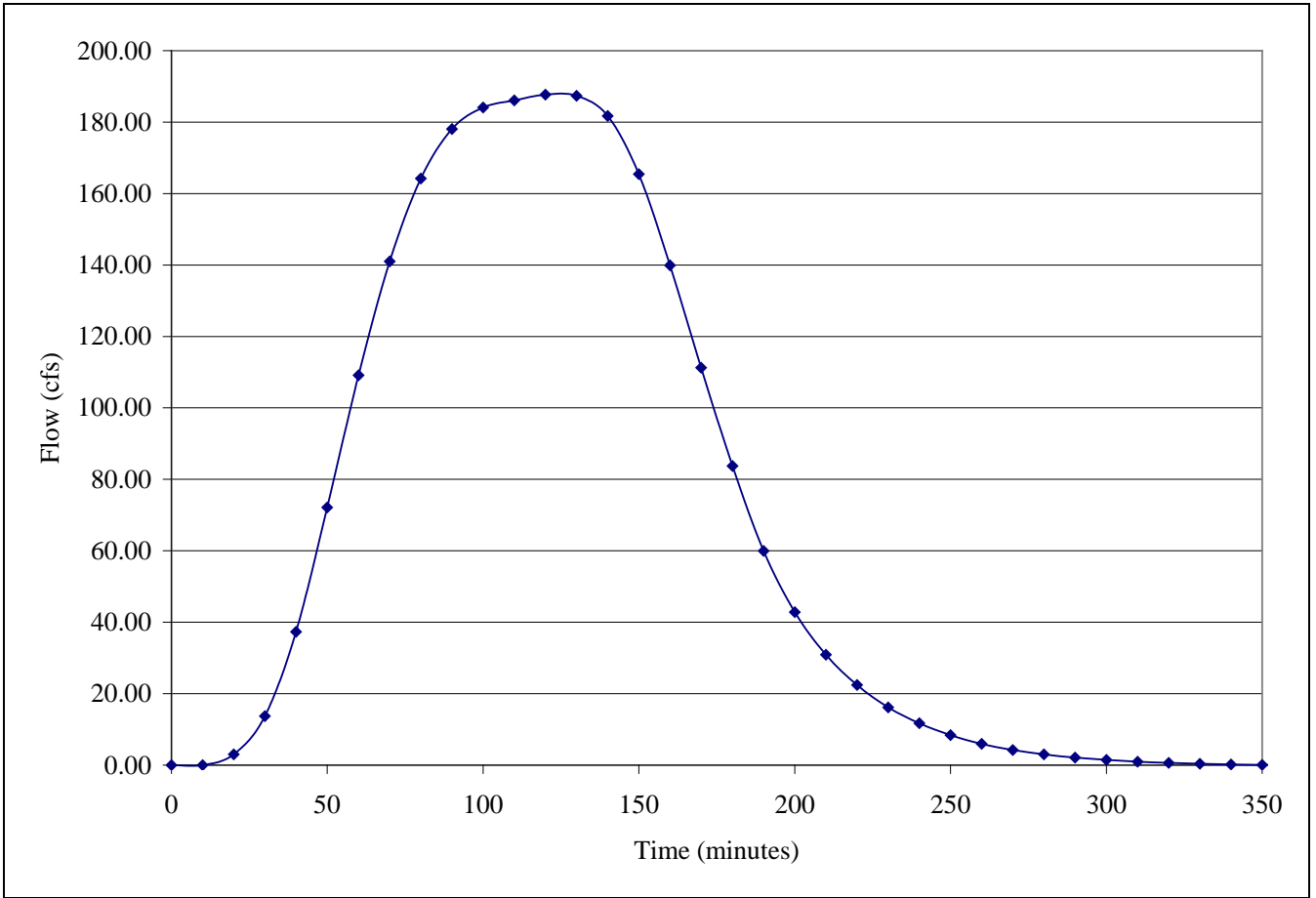


Figure 3.4.8  
Storm Hydrograph for Example 3.4.1



### 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  $x_i$ ,  $i=1, 2, \dots, n$ , 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:

$$G = -0.33 + 0.08/C_s \text{ if } |C_s| \leq 0.9$$

$$G = -0.52 + 0.30/C_s \text{ if } |C_s| > 0.9$$

$$H = 0.94 - 0.26/C_s \text{ if } |C_s| \leq 1.50$$

$$H = 0.55 \text{ if } |C_s| > 1.50$$

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.

$$C_w = W \cdot C_s + (1 - W) \cdot 0.303$$

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:

$$y_{T_r} = \bar{y} + K_T s_y$$

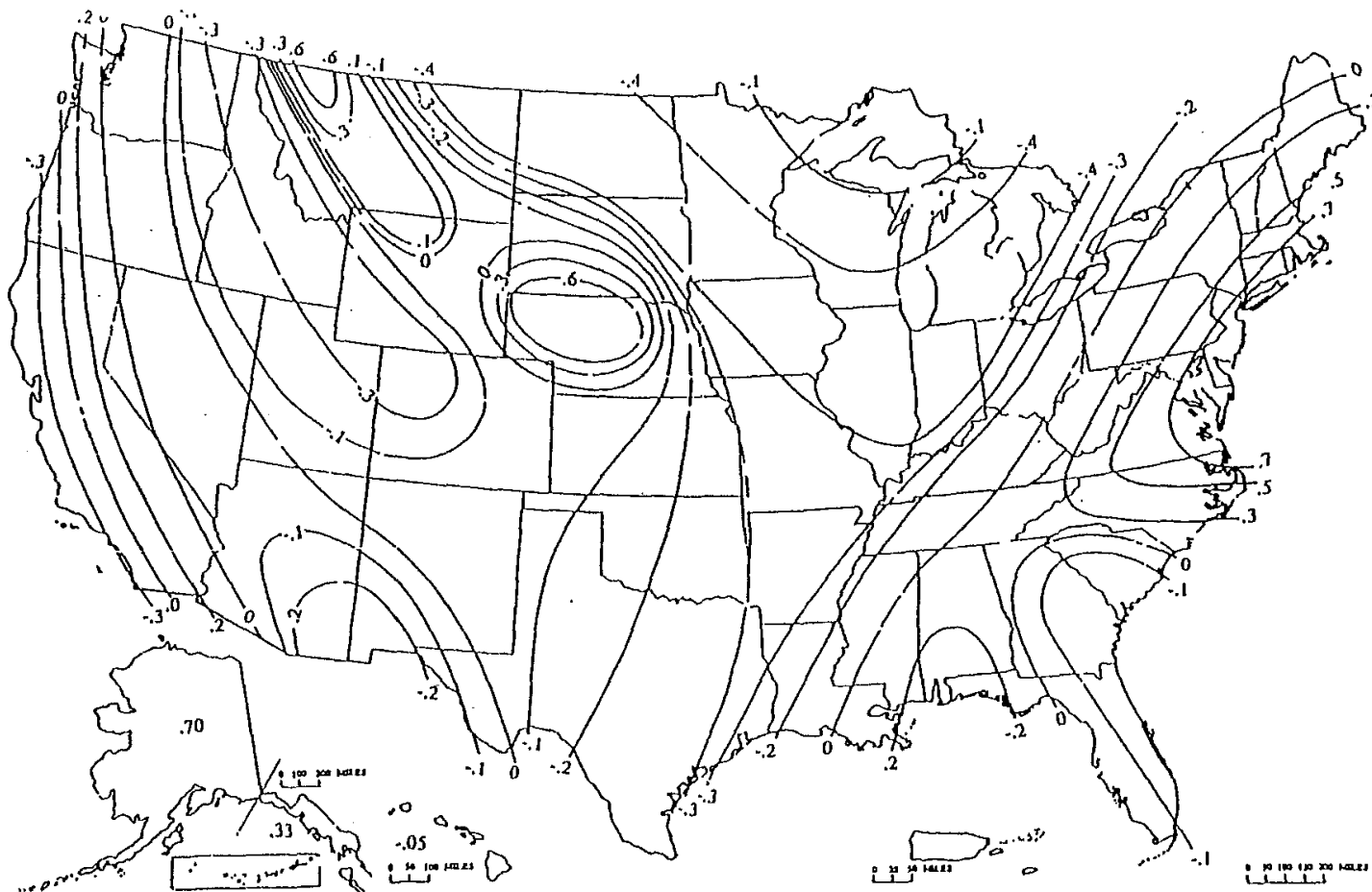
$$Q_{T_r} = 10^{y_{T_r}}$$

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 <sub>i</sub>	$x_i$ (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,  $K_n=2.768$

$$y_H = \bar{y} + K_n \cdot s_y = 3.6721 + 2.768 \cdot 0.2785 = 4.4430$$

$$Q_H = 10^{4.4430} = 27,732 \text{ cfs}$$

$$y_L = \bar{y} - K_n s_y = 3.6721 - 2.768 * 0.2785 = 2.9013$$

$$Q_L = 10^{2.9013} = 797 \text{ cfs}$$

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.  $C_s = -0.2872$ .

$$H = 0.94 - 0.26 / |C_s| = 0.94 - 0.26 / |-0.2872| = 0.865 \text{ (for } |C_s| \leq 1.50)$$

$$G = -0.33 + 0.08 / |C_s| = -0.33 + 0.08 / |-0.2872| = -0.307 \text{ (for } |C_s| \leq 0.9)$$

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.

$$C_w = W \cdot C_s + (1 - W) \cdot 0.303 = 0.712 \cdot (-0.2872) + (1 - 0.712) \cdot 0.303 = -0.117$$

**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:

<u>Return Period (<math>T_r</math>), years</u>	<u><math>K_T</math></u>
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.

$$y_{T_r} = \bar{y} + K_T s_y \quad Q_{T_r} = 10^{y_{T_r}}$$

<u><math>T_r</math></u>	<u><math>K_T</math></u>	<u><math>y_{T_r}</math></u>	<u><math>Q_{T_r}</math> (cfs)</u>
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.

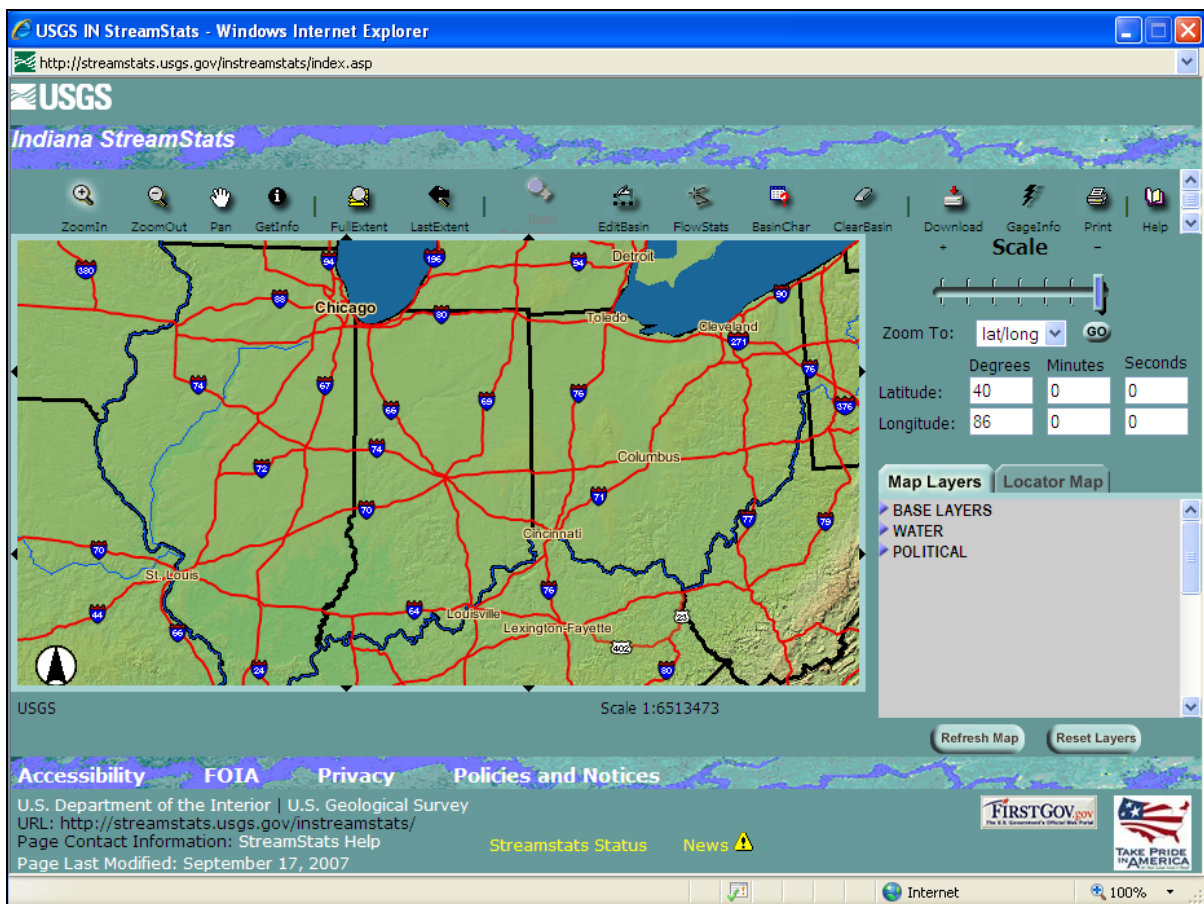


Figure 3.6.1  
Introductory Screen for the Indiana StreamStats Program

Step 2: Zoom in on the specified region until the “Basin Delineation” tool becomes available.

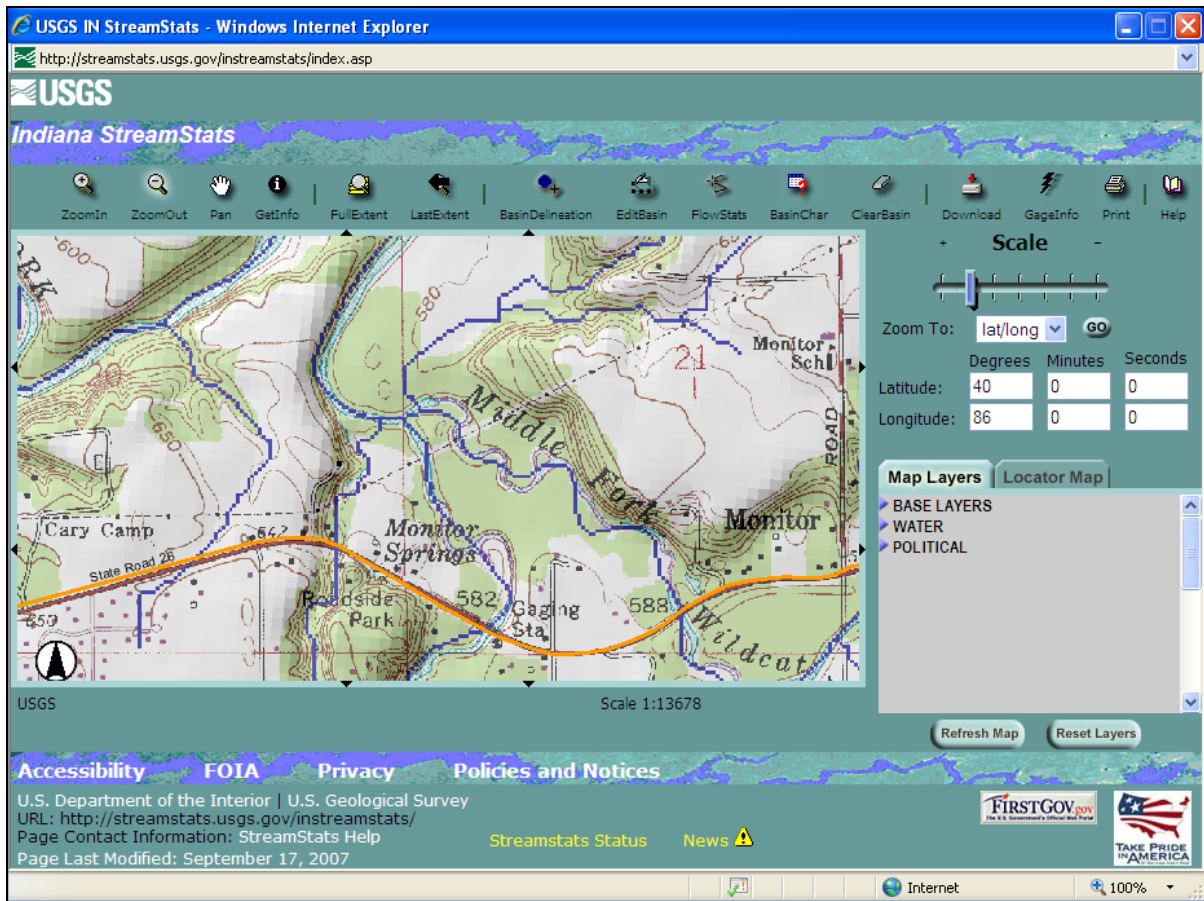


Figure 3.6.2  
Zoomed In Area of the South Fork Wildcat Creek at Lafayette, IN

**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.

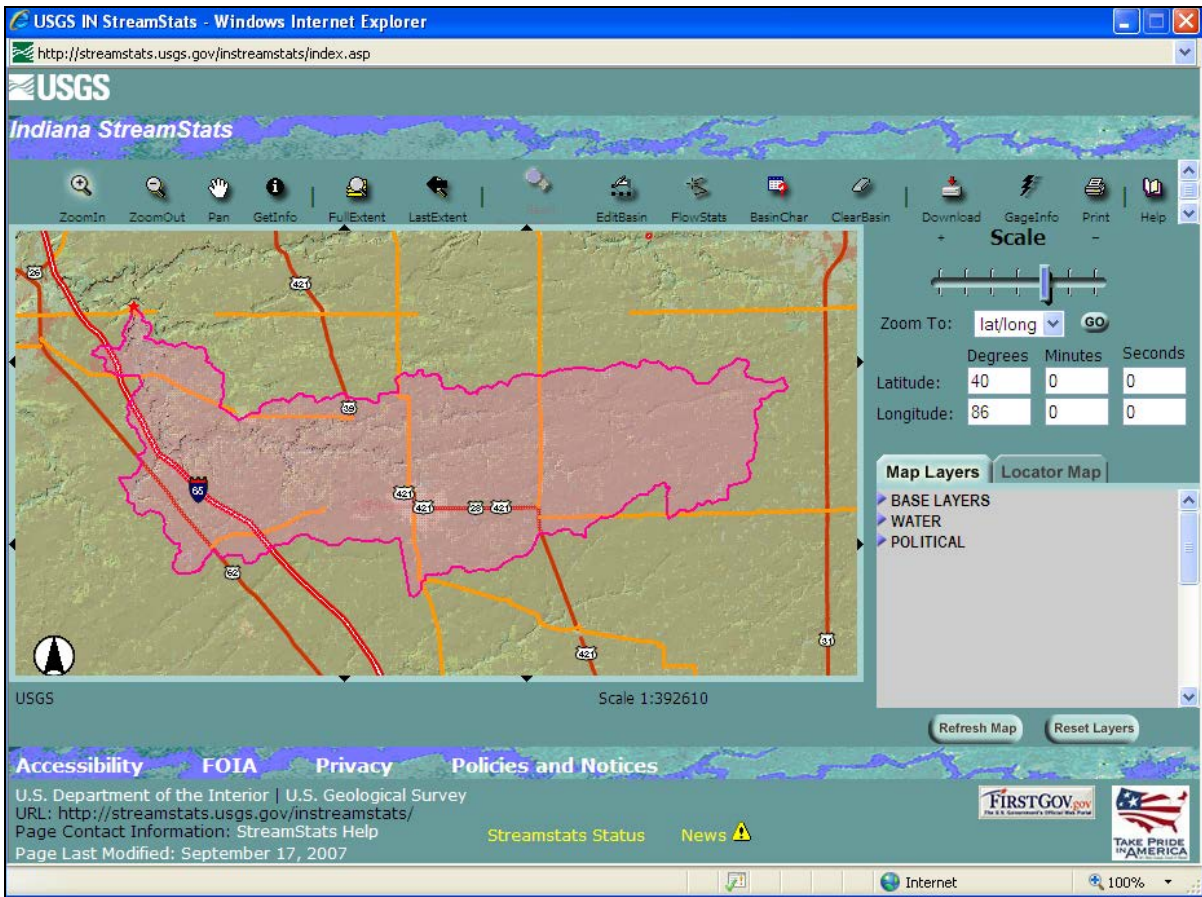


Figure 3.6.3  
Basin Delineation for South Fork Wildcat Creek at Lafayette, IN

**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.

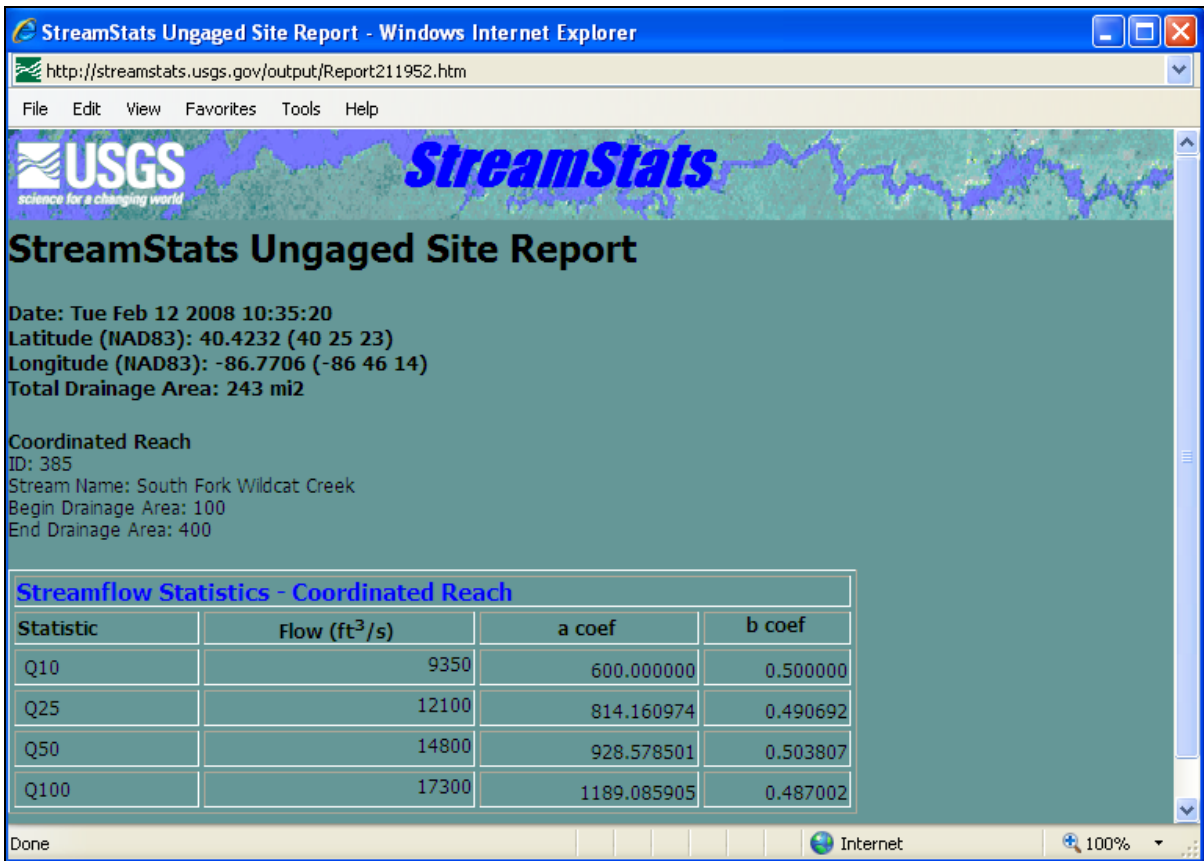


Figure 3.6.4  
 Peak Discharges for South Fork Wildcat Creek at Lafayette, IN

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.

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#### **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.

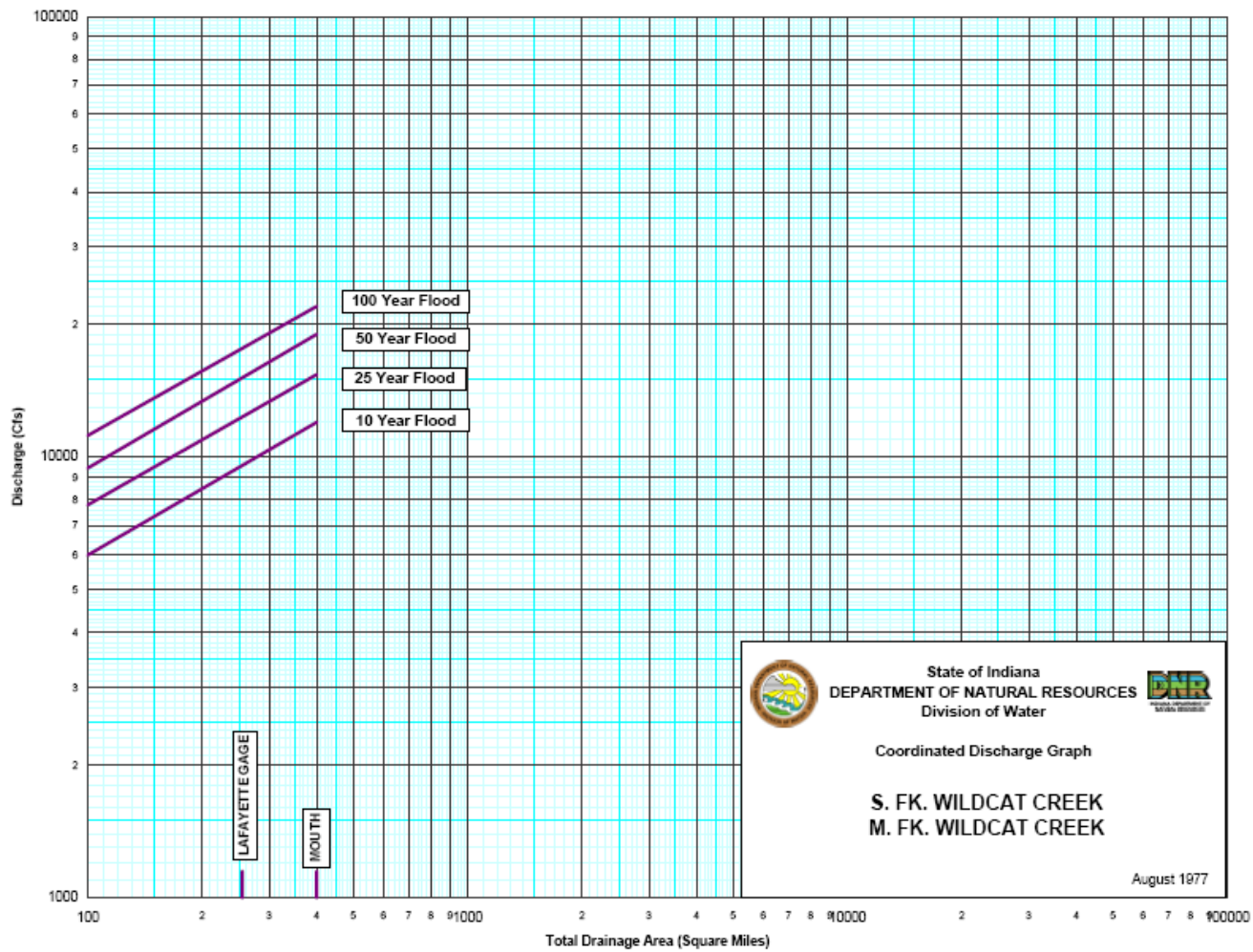


Figure 3.7.1  
Coordinated Discharge Curve, Example 3.7.1 (IDOT DNR, 1990)

### 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<sup>2</sup> 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](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 <sup>2</sup> or m <sup>2</sup> )
b	Bottom width of channel
B <sub>S</sub>	Surface width of the channel
d	Depth of flow (normal to flow) (ft or m)
d <sub>max</sub>	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 <sup>2</sup> or m/sec <sup>2</sup> )
γ	Specific weight of water (lb/ft <sup>3</sup> or N/m <sup>3</sup> )
h <sub>L</sub>	Head loss (ft or m)
K	Ratio of the tractive force
K'	Conveyance
K <sub>b</sub>	Maximum shear on the bottom (lb/ft <sup>2</sup> )
K <sub>s</sub>	Maximum shear on the sides (lb/ft <sup>2</sup> )
L	Length (ft or m)
n	Manning's roughness coefficient
n <sub>E</sub>	Equivalent roughness coefficient
n <sub>j</sub>	Roughness coefficient for a subsection
P	Pressure
P <sub>w</sub>	Wetted perimeter
φ	Angle of the channel side-slope
Q	Volumetric flowrate (ft <sup>3</sup> /sec or m <sup>3</sup> /sec)
R	Hydraulic radius
R <sub>O</sub>	Mean radius of the bend
S	Slope of the energy grade line (ft/ft or m/m)
S <sub>o</sub>	Channel bed slope (ft/ft or m/m)
S <sub>w</sub>	Water surface slope
T	Top width of channel
τ <sub>b</sub>	Tractive force of channel bed (lb/ft <sup>2</sup> or
τ <sub>o</sub>	Average bed shear force, tractive force (lb/ft <sup>2</sup> or
τ <sub>s</sub>	Tractive force of channel side (lb/ft <sup>2</sup> 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.

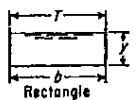
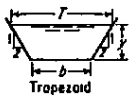
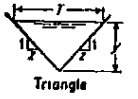
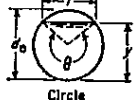
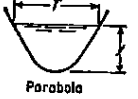
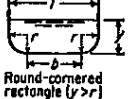
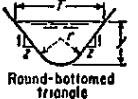
Steady flow occurs when the velocity at a point does not change with time -- that is  $\Delta V/\Delta t = 0$ . When the flow is unsteady,  $\Delta V/\Delta t \neq 0$ . Unsteady flows are difficult to analyze unless the change with time is sufficiently slow to permit a step type of analysis. Examples 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	$zy^2$	$2y\sqrt{1 + z^2}$	$\frac{zy}{2\sqrt{1 + z^2}}$	$2zy$	$\frac{1}{2}zy$	$\frac{\sqrt{2}}{2}zy^{1.5}$
 Circle	$\frac{1}{8}(\theta - \sin \theta)d_0^3$	$\frac{1}{2}\theta d_0$	$\frac{1}{4}\left(1 - \frac{\sin \theta}{\theta}\right)d_0$	$\frac{(\sin \frac{1}{2}\theta)d_0}{2\sqrt{y(d_0 - y)}}$ or $\frac{(\sin \frac{1}{2}\theta)d_0}{2\sqrt{y(d_0 - y)}}$	$\frac{1}{8}\left(\frac{\theta - \sin \theta}{\sin \frac{1}{2}\theta}\right)d_0$	$\frac{\sqrt{2}}{32}\frac{(\theta - \sin \theta)^{1.5}}{(\sin \frac{1}{2}\theta)^{0.5}}d_0^{2.5}$
 Parabola	$\frac{2}{3}Ty$	$T + \frac{8}{3}\frac{y^2}{T}$	$\frac{2T^2y}{3T^2 + 8y^2}$	$\frac{3}{2}\frac{A}{y}$	$\frac{2}{3}Ty$	$\frac{2}{3}\sqrt{6}Ty^{1.5}$
 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  $0 < z \leq 1$ , where  $z = 4y/T$ . When  $z > 1$ , use the exact expression  $P = (T/2)[\sqrt{1 + z^2} + 1/z \ln(z + \sqrt{1 + z^2})]$ .

### 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,

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma} + z_1 = \frac{V_2^2}{2g} + \frac{P_2}{\gamma} + z_2 + h_L \quad (4.1.1)$$

where  $h_L$  = head loss through reach (ft) and  $\gamma$  = specific weight of water (lbs/ft<sup>3</sup>).

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:

$$\frac{V_1^2}{2g} + z_1 + y_1 = \frac{V_2^2}{2g} + z_2 + y_2 + h_L \quad (4.1.2)$$

For small bed slopes ( $S_0$ ) the values of  $z_1$  and  $z_2$  are essentially equal, this gives Equation 4.1.3.

$$\frac{V_1^2}{2g} + y_1 = \frac{V_2^2}{2g} + y_2 + h_L \quad (4.1.3)$$

In Figure 4.1.1,  $S_w$  is defined as the water surface slope which for small slopes may be written as:

$$S_w = \frac{y_1 - y_2}{L} \quad (4.1.4)$$

The slope of the energy grade line is observed to be:

$$S = h_L/L \quad (4.1.5)$$

In uniform flow, the water surface slope ( $S_w$ ), the energy slope ( $S$ ) and the bed slope ( $S_0$ ) are all equal to one another.

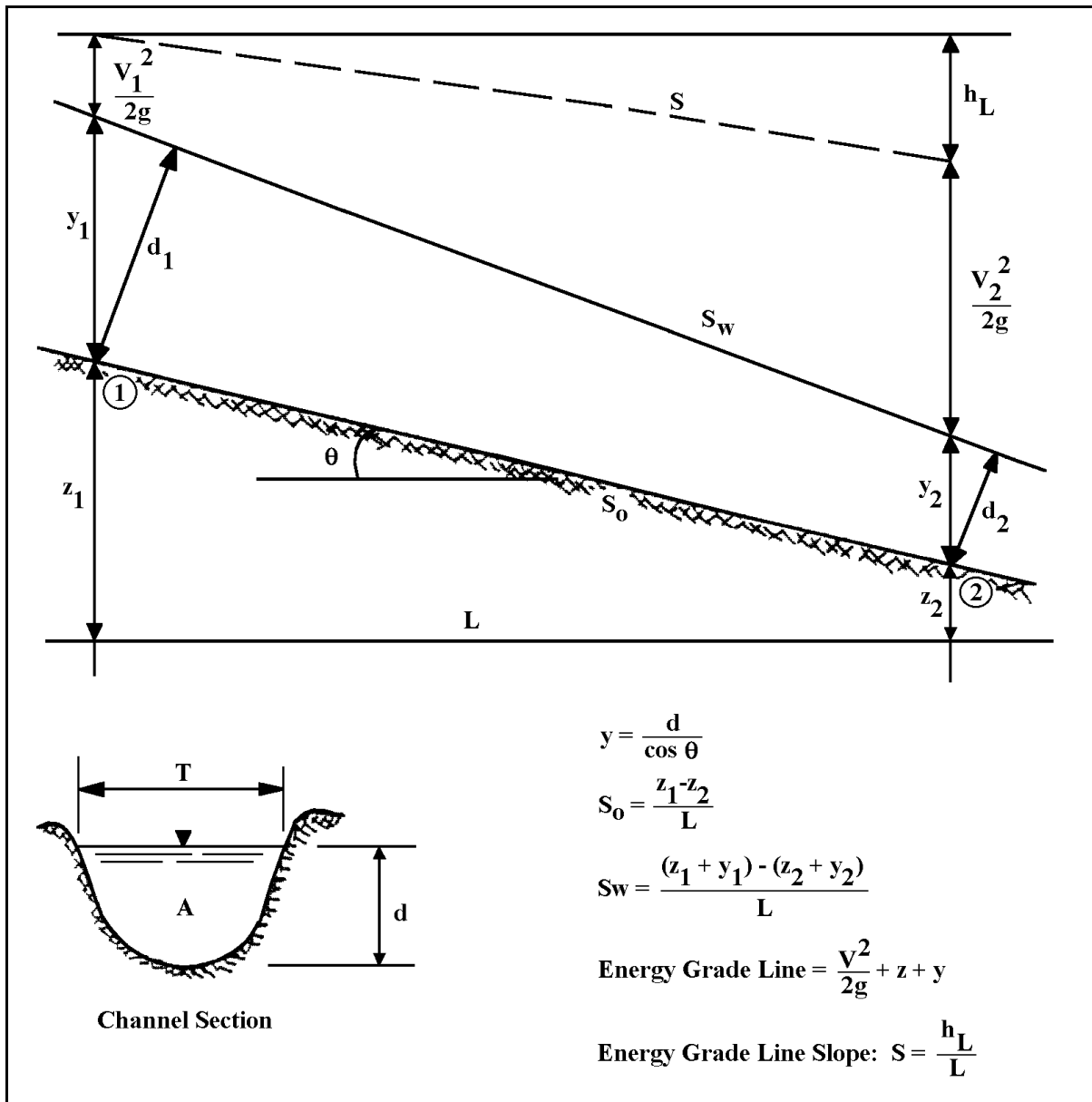


Figure 4.1.1  
Properties of Open Channel Flow

## 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,

$$Q = \frac{1.49}{n} (R)^{2/3} A(S)^{1/2} \quad (4.2.1)$$

where  $Q$  = volumetric flowrate (cfs),  $R$  = hydraulic radius (ft),  $A$  = cross-sectional area (ft<sup>2</sup>),  $S$  = 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 ( $S = S_0 = S_w$ ); 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)**

<b>Table 1. Manning's Roughness Coefficients, <i>n</i><sup>1</sup></b>			
<b>I. Closed Conduits</b>	Manning's <i>n</i> Range <sup>2</sup>	<b>IV. Highway Channels and Swales with Maintained Vegetation<sup>6</sup></b> (values shown here are for velocities of 2 and 6 f.p.s.):	Manning's <i>n</i> Range <sup>2</sup>
A. Concrete pipes-----	0.011-0.013	A. Depth of flow up to 0.7 feet:	
B. Corrugated-metal pipe or pipe-arch.		1. Bermuda grass, Kentucky bluegrass, Buffalo grass:	
1. 2 2/3 by 1/2-in. corrugation (riveted pipe): <sup>3</sup>		a. Mowed to 2-inches-----	0.045-0.07
a. Plain or fully coated-----	0.024	b. Length 4-6 inches-----	0.05-0.09
b. Paved invert (range values are for 25 and 50 percent of circumference paved):		2. Good stand, any grass:	
(1) Flow full depth-----	0.018-0.021	a. Length about 12-inches-----	0.09-0.18
(2) Flow 0.8 depth-----	0.016-0.021	b. Length about 24-inches-----	0.15-0.30
(3) Flow 0.6 depth-----	0.013-0.019	3. Fair stand, any grass:	
2. 6 by 2-in. corrugation (field bolted)-----	0.03	a. Length about 12-inches-----	0.08-0.14
C. Vitrified clay pipe-----	0.012-0.014	b. Length about 24-inches-----	0.13-0.25
D. Cast-iron pipe, uncoated-----	0.013	B. Depth of flow 0.7-1.5 feet:	
E. Steel pipe-----	0.009-0.011	1. Bermuda grass, Kentucky bluegrass, Buffalo grass:	
F. Brick-----	0.014-0.017	a. Mowed to 2-inches-----	0.035-0.05
G. Monolithic concrete:		b. Length 4-6 inches-----	0.04-0.06
1. Wood forms, rough-----	0.015-0.017	2. Good stand, any grass:	
2. Wood forms, smooth-----	0.012-0.014	a. Length about 12-inches-----	0.07-0.12
3. Steel forms-----	0.012-0.013	b. Length about 24-inches-----	0.10-0.20
H. Cemented rubble masonry walls:		3. Fair stand, any grass:	
1. Concrete floor and top-----	0.017-0.022	a. Length about 12-inches-----	0.06-0.10
2. Natural floor-----	0.019-0.025	b. Length about 24-inches-----	0.09-0.17
I. Laminated treated wood-----	0.015-0.017	<b>V. Street and Expressway Gutters:</b>	
J. Vitrified clay liner plates-----	0.015	A. Concrete gutter, troweled finish-----	0.012
<b>II. Open Channels, Lined<sup>4</sup> (straight alignment):<sup>5</sup></b>		B. Asphalt pavement:	
A. Concrete with surfaces as indicated:		1. Smooth texture-----	0.013
1. Formed, no finish-----	0.013-0.017	2. Rough texture-----	0.016
2. Trowel finish-----	0.012-0.014	C. Concrete Gutter with asphalt pavement:	
3. Float finish-----	0.013-0.015	1. Smooth-----	0.013
4. Float finish, some gravel on bottom-----	0.015-0.017	2. Rough-----	0.015
5. Gunite, good section-----	0.016-0.019	D. Concrete pavement:	
6. Gunite, wavy section-----	0.018-0.022	1. Float finish-----	0.014
B. Concrete, bottom float finished, sides as indicated:		2. Broom finish-----	0.016
1. Dressed stone in mortar-----	0.015-0.017	E. For gutters with small slope, where sediment may accumulate, increase above values of <i>n</i> by-----	0.002
2. Random stone in mortar-----	0.017-0.020		

Table 4.2.1 (cont.)  
 Values of Manning's Roughness Coefficient, *n*  
 (Federal Highway Administration, 1961)

B. Concrete, bottom float finished, sides as indicated:		E. For gutters with small slope, where sediment may accumulate, increase above values of <i>n</i> by-----	0.002
1. Dressed stone in mortar-----	0.015-0.017		
2. Random stone in mortar-----	0.017-0.020		
<b>II. Open Channels, Lined<sup>4</sup> (straight alignment):<sup>5</sup> (continued)</b>		<b>VI. Natural stream channels:<sup>8</sup></b>	
3. Cement rubble masonry-----	0.020-0.025	A. Minor streams <sup>9</sup> (surface width at flood stage less than 100 ft.):	
4. Cement rubble masonry, plastered-----	0.016-0.020	1. Fairly regular section:	
C. Gravel bottom, sides as indicated:		1. Some grass and seeds, little or no brush-----	0.030-0.035
1. Formed concrete-----	0.017-0.020	b. Dense growth of weeds, depth of flow materially greater than weed height-----	0.035-0.05
2. Random stone in mortar-----	0.020-0.033	c. Some weeds, light brush on banks-----	0.035-0.05
3. Dry rubble (riprap)-----	0.023-0.033	d. Some weeds, heavy brush on banks-----	0.05-0.07
D. Brick-----	0.014-0.017	e. Some weeds, dense willows on banks-----	0.06-0.08
E. Asphalt:		f. For trees within channel, with branches submerged at high stage, increase all above values by-----	0.01-0.02
1. Smooth-----	0.013	2. Irregular sections, with pools, slight channel meander; increase values given in 1 a-e about-----	0.01-0.02
2. Rough-----	0.016	3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
F. Wood, planed, clean-----	0.011-0.013	a. Bottom of gravel, cobbles, and few boulders-----	0.04-0.05
G. Concrete-lined excavated rock:		b. Bottom of cobbles, with large boulders-----	0.05-0.07
1. Good section-----	0.017-0.020	B. Flood plains (adjacent to natural streams)	
2. Irregular section-----	0.022-0.027	1. Pasture, no brush:	
<b>III. Open Channels, excavated<sup>4</sup> (straight alignment,<sup>5</sup> natural lining):</b>		a. Short grass-----	0.030-0.035
A. Earth, uniform section:		b. High grass-----	0.035-0.05
1. Clean, recently completed-----	0.016-0.018	2. Cultivated areas:	
2. Clean, after weathering-----	0.018-0.020	a. No crop-----	0.03-0.04
3. With short grass, few weeds-----	0.022-0.025	b. Mature row crops-----	0.035-0.045
4. In gravelly soil, uniform section, clean-----	0.022-0.025	c. Mature field crops-----	0.04-0.05
B. Earth, fairly uniform section:		3. Heavy weeds, scattered brush-----	0.05-0.07
1. No vegetation-----	0.022-0.025	4. Light brush and trees: <sup>10</sup>	
2. Grass, some weeds-----	0.025-0.030	a. Winter-----	0.05-0.06
3. Dense weeds or aquatic plants in deep channels-----	0.030-0.035	b. Summer-----	0.06-0.08
4. Sides clean, gravel bottom-----	0.025-0.030	5. Medium to dense brush: <sup>10</sup>	
5. Sides clean, cobble bottom-----	0.030-0.040	a. Winter-----	0.07-0.11
C. Dragline excavated or dredged:		b. Summer-----	0.10-0.16
1. No vegetation-----	0.028-0.033	6. Dense willows, summer, not bent over by current-----	0.15-0.20
2. Light brush on banks-----	0.035-0.050		
D. Rock:			
1. Based on design section-----	0.035		

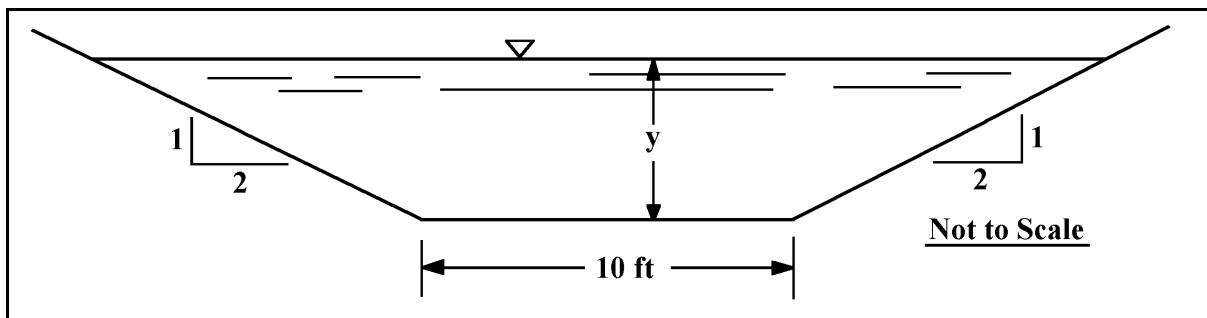


Table 4.2.1 (cont.)  
 Values of Manning's Roughness Coefficient, *n*  
 (Federal Highway Administration, 1961)

<p><b>III. Open Channels, excavated<sup>4</sup></b> (straight alignment,<sup>5</sup> natural lining): (cont)</p> <p>E. Channels no maintained, weeds and brush uncut: 0.08-0.12</p> <p>1. Dense weeds, high as flow depth----- 0.05-0.08</p> <p>2. Clean bottom, brush on sides----- 0.07-0.11</p> <p>3. Clean bottom, brush on sides, highest stage of flow----- 0.10-0.14</p> <p>4. Dense brush, high stage-----</p>	<p><b>VI. Natural stream channels:<sup>8</sup></b> (cont)</p> <p>8. Heavy stand of timber, a few down trees, little undergrowth:</p> <p style="padding-left: 20px;">a. Flood depth below branches----- 0.10-0.12</p> <p style="padding-left: 20px;">b. Flood depth reaches branches----- 0.12-0.16</p> <p>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 vegetation on banks. Values of <i>n</i> may be somewhat reduced. Follow recommendation in publication cited<sup>8</sup> if possible. The value of <i>n</i> for larger streams of most regular section, with no boulders or brush, may be in the range of----- 0.028-0.033</p>
<p><b>Footnotes to Table 1</b></p> <p><sup>1</sup> Estimates are by Bureau of Public roads unless otherwise noted.</p> <p><sup>2</sup> 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 <i>n</i>.</p> <p><sup>3</sup> 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.</p> <p><sup>4</sup> For important work and where accurate determination of water profiles is necessary, the designer is urged to consult the following references and to select <i>n</i> 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.</p> <p><sup>5</sup> With channel of an alignment other than straight, loss of head by resistance forces will be increased. A small increase in the value of <i>n</i> may be made, to allow for the additional loss of energy.</p> <p><sup>6</sup> 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</p> <p><sup>7</sup> 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.</p> <p><sup>8</sup> 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 <i>n</i> applicable to streams of any specific locality. Where this procedure is not followed, the table may be used as a guide. The values of <i>n</i> tabulated have been derived from data reported by C.E. Ramser (see footnote 4) and from other incomplete data.</p> <p><sup>9</sup> The tentative values of <i>n</i> 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 <i>n</i> 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.</p> <p><sup>10</sup> The presence of foliage on trees and brush under flood stage will materially increase the value of <i>n</i>. 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, <i>n</i> will increase with rising stage.</p>	

- (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

$$\begin{aligned} A &= by + zy^2 = (10)(5.4) + 2.5(5.4)^2 \\ A &= 54 + 72.9 = 126.9 \text{ ft}^2 \end{aligned}$$

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:

$$\begin{aligned} Q &= \frac{1.49}{(0.012)} (126.9)(3.25)^{2/3} (0.0008)^{1/2} \\ Q &= 977.84 \text{ cfs} \end{aligned}$$

### 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.

$$\frac{V^2}{2g} + y + z = \text{CONSTANT}$$

If the datum is taken at the channel bottom, the specific energy  $E$  is given by

$$E = y + \frac{V^2}{2g} \quad (4.2.3)$$

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:

$$E = y + \frac{Q^2}{2g(A)^2} \quad (4.2.4)$$

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.

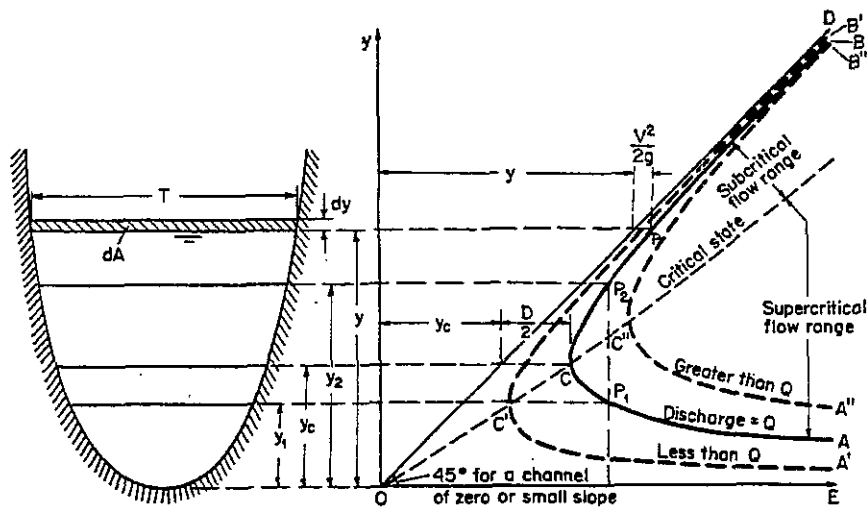


Figure 4.2.1  
Specific-Energy Curve (Chow, 1988)

#### 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

$$\frac{dE}{dy} = \frac{Q^2}{gA^3} \frac{dA}{dy} = 1 - \frac{V^2}{gA} \frac{dA}{dy} \quad (4.2.5)$$

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:

$$\frac{dE}{dy} = 1 - \frac{Q^2 T}{gA^3} = 1 - \frac{Q^2}{gA^2 D} = 1 - \frac{V^2}{gD} \quad (4.2.6)$$

At the critical state of flow, the specific energy is a minimum ( $dE/dy = 0$ ).

$$\frac{V^2}{2g} = \frac{D}{2} \quad (4.2.7)$$

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.

$$\frac{Q^2}{gA^2 D} = 1 = \frac{Q^2 T}{gA^3} \quad (4.2.8)$$

Solving for Q gives

$$Q = \left[ \frac{g A^3}{T} \right]^{1/2} = [g A^2 D]^{1/2} \quad (4.2.9)$$

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:

$$A = (10 + 2y)y \quad T = 10 + 4y$$

$$P_w = 10 + 2y\sqrt{5} \quad D = \frac{A}{T} = \frac{(10 + 2y)y}{(10 + 4y)}$$

$$R = A / P_w$$

From Equation 4.2.9

$$Q = 225 = [gA^2 D]^{1/2} = \frac{32.2^{1/2} [(10 + 2y)y]^{3/2}}{(10 + 4y)^{1/2}}$$

Solving by trial and error a critical depth,  $y_c$  is found to be 2.16 ft.

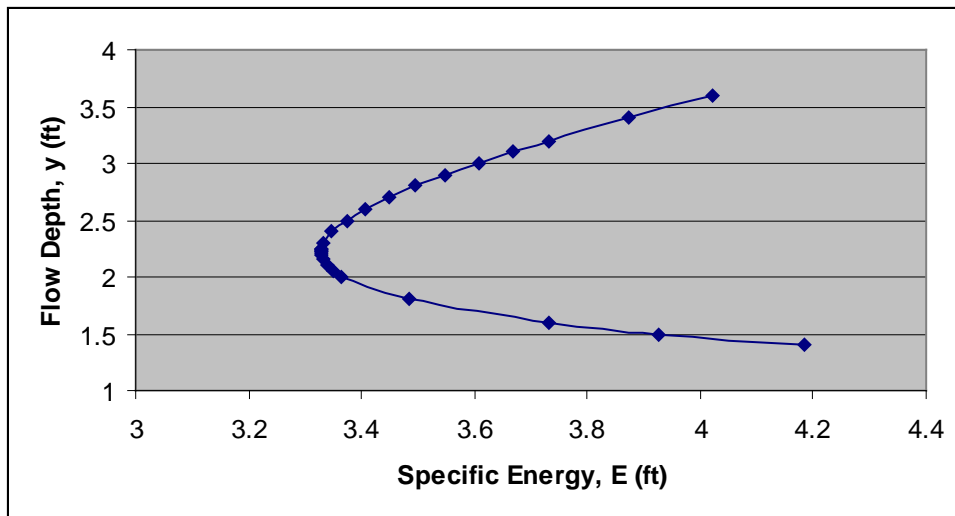
#### Example 4.2.4

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,  $b$ .

$$Q = \left[ \frac{g A^3}{T} \right]^{1/2} \Rightarrow 150 \text{ cfs} = \left[ \frac{32.2 * (8y)^3}{8} \right]^{1/2} \Rightarrow 10.92 = y^3 \Rightarrow y = 2.22 \text{ ft}$$

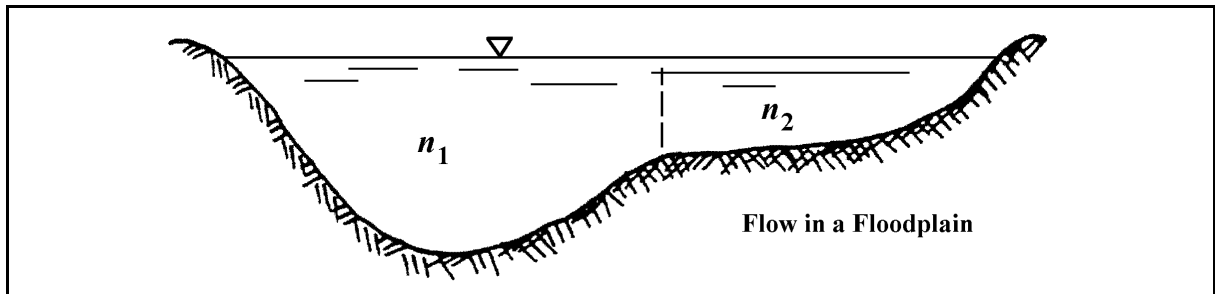
The minimum specific energy corresponding to this critical flow depth is

$$E_{\min} = y + \frac{Q^2}{2g(A)^2} = 2.22 + \frac{150^2}{2 * 32.2 * (8 * 2.22)^2} = 3.33 \text{ ft}$$

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:

$$\begin{aligned} Q &= Q_1 + Q_2 \\ &= K'_1 \sqrt{S} + K'_2 \sqrt{S} \\ &= (K'_1 + K'_2) \sqrt{S} \end{aligned} \tag{4.2.11}$$

$$\text{where } K'_i = \frac{1.49}{n_i} A_i R_i^{2/3} \tag{4.2.12}$$

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 floodplain 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.

$$K'_i = \frac{1.49}{n_i} A_i R_i^{2/3}$$

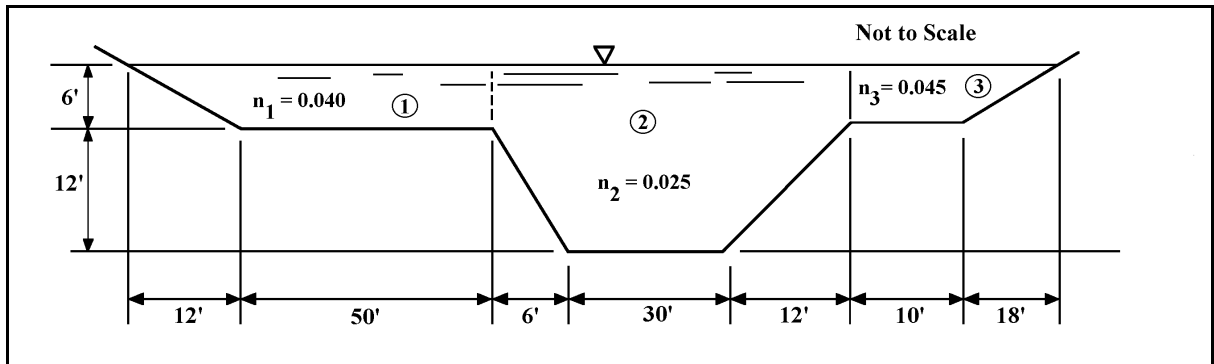
\* Section 1

$$A_1 = 50 \times 6 + 1/2(12 \times 6) = 336 \text{ ft}^2$$

$$P_{w1} = 13.42 \text{ ft} + 50 \text{ ft} = 63.42 \text{ ft}$$

$$R_1 = \frac{A_1}{P_{w1}} = \frac{336 \text{ ft}^2}{63.42 \text{ ft}} = 5.30 \text{ ft}$$

$$K'_1 = \frac{1.49}{0.040} (336)(5.30)^{2/3} = 38,046.62$$



\* Section 2

$$A_2 = 6(30 + 6 + 12) + 12(30) + 1/2(12)(12) + 1/2(6)(12) = 756 \text{ ft}^2$$

$$P_{w2} = 16.97 + 30 + 13.42 = 60.39 \text{ ft}$$

$$R_2 = \frac{A_2}{P_w 2} = \frac{756 \text{ ft}^2}{60.39 \text{ ft}} = 12.52 \text{ ft}$$

$$K'_2 = \frac{1.49}{0.025} (756)(12.52)^{2/3} = 242,942.94$$

\*Section 3

$$A_3 = (10)(6) + 1/2(18)(6) = 114 \text{ ft}^2$$

$$P_{w3} = 10 + 18.97 = 28.97 \text{ ft}$$

$$R_3 = \frac{A_3}{P_w 3} = \frac{114}{28.97} = 3.94 \text{ ft}$$

$$K'_3 = \frac{1.49}{0.045} (114)(3.94)^{2/3} = 9,416.21$$

The total flow therefore is equal to:

$$Q = (K'_1 K'_2 + K'_3) \sqrt{S} = (38046.58 + 242944.8 + 9416.21) \sqrt{0.0006} = 7,113.50 \text{ cfs}$$

### 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 $T$	Hydrau- lic depth $D$	Section factor $Z$
Trapezoid, half of a hexagon	$\sqrt{3} y^2$	$2 \sqrt{3} y$	$\frac{1}{2} y$	$\frac{4}{3} \sqrt{3} y$	$\frac{3}{4} y$	$\frac{3}{2} y^2.5$
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	$\frac{\pi}{2} y^2$	$\pi y$	$\frac{1}{2} y$	$2y$	$\frac{\pi}{4} y$	$\frac{\pi}{4} y^2.5$
Parabola, $T = 2 \sqrt{2} y$	$\frac{4}{3} \sqrt{2} y^2$	$\frac{8}{3} \sqrt{2} y$	$\frac{1}{2} y$	$2 \sqrt{2} y$	$\frac{3}{8} y$	$\frac{8}{9} \sqrt{3} y^2.5$
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.

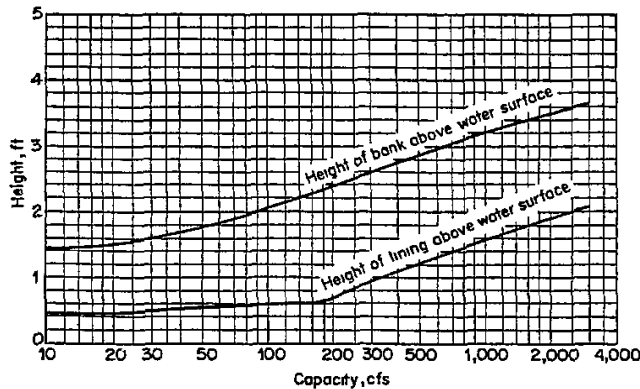


Figure 4.3.1  
Recommended Freeboard and Height of Bank for Lined Channels  
(U.S. Bureau of Reclamation, 1952)

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

$$\tau_o = \gamma RS \quad (4.3.2)$$

where  $\tau_o$  = the average bed shear force, tractive force (lb/ft<sup>2</sup>).

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:

$$K = \tau_s / \tau_b \tag{4.3.3}$$

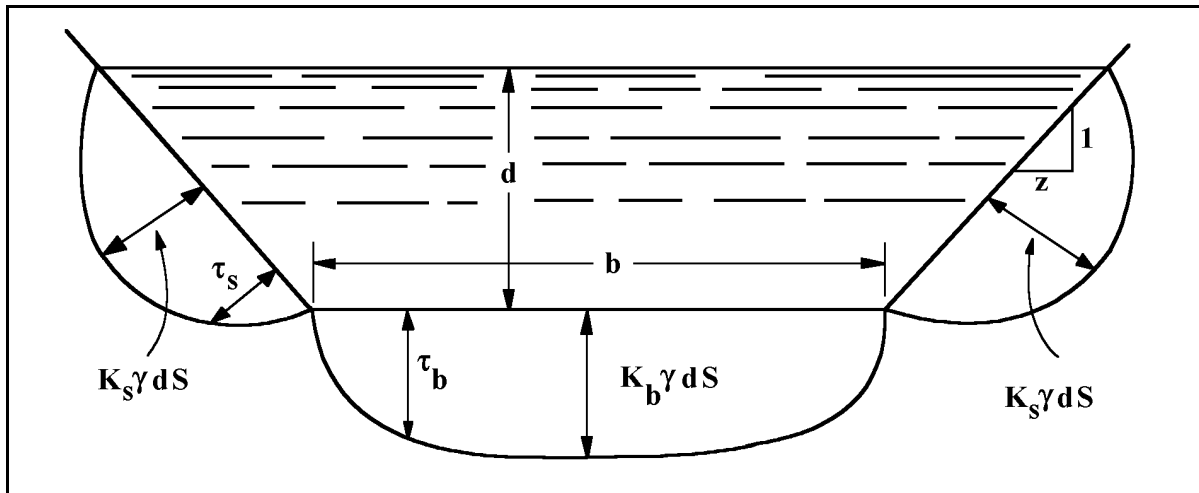


Figure 4.3.2  
Tractive Stress Distribution on Channel Bed

$$K = \left[ 1 - \frac{\sin^2 \phi}{\sin^2 \theta} \right]^{1/2} \tag{4.3.4}$$

where  $\phi$  = angle of the side-slope and  $\theta$  = 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  $\Delta X$ , 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.

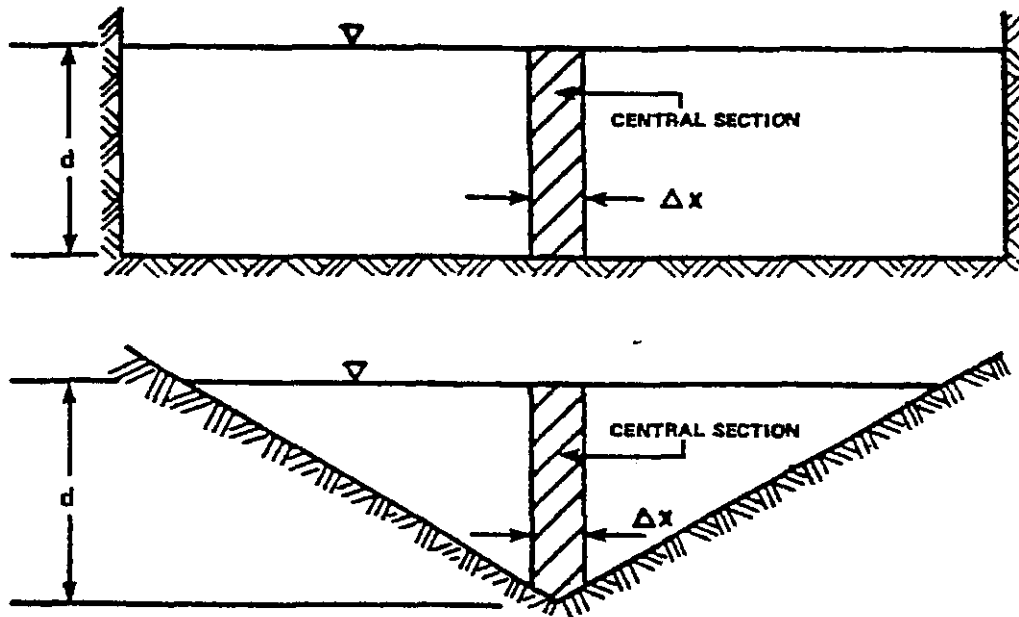


Figure 4.3.3  
Schematic Diagram of Channels of Different Shapes (Norman, 1975)

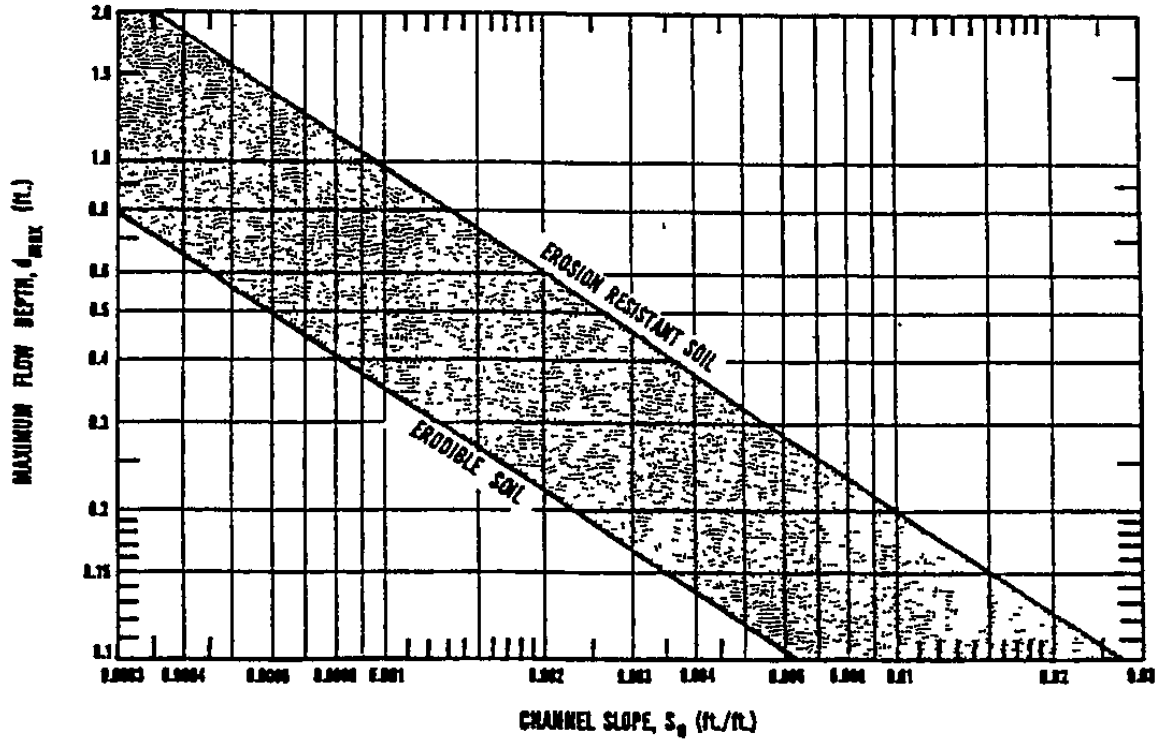


Figure 4.3.4  
Maximum Permissible Depth of Flow ( $d_{max}$ ) for Unlined Channels (Normann, 1975)

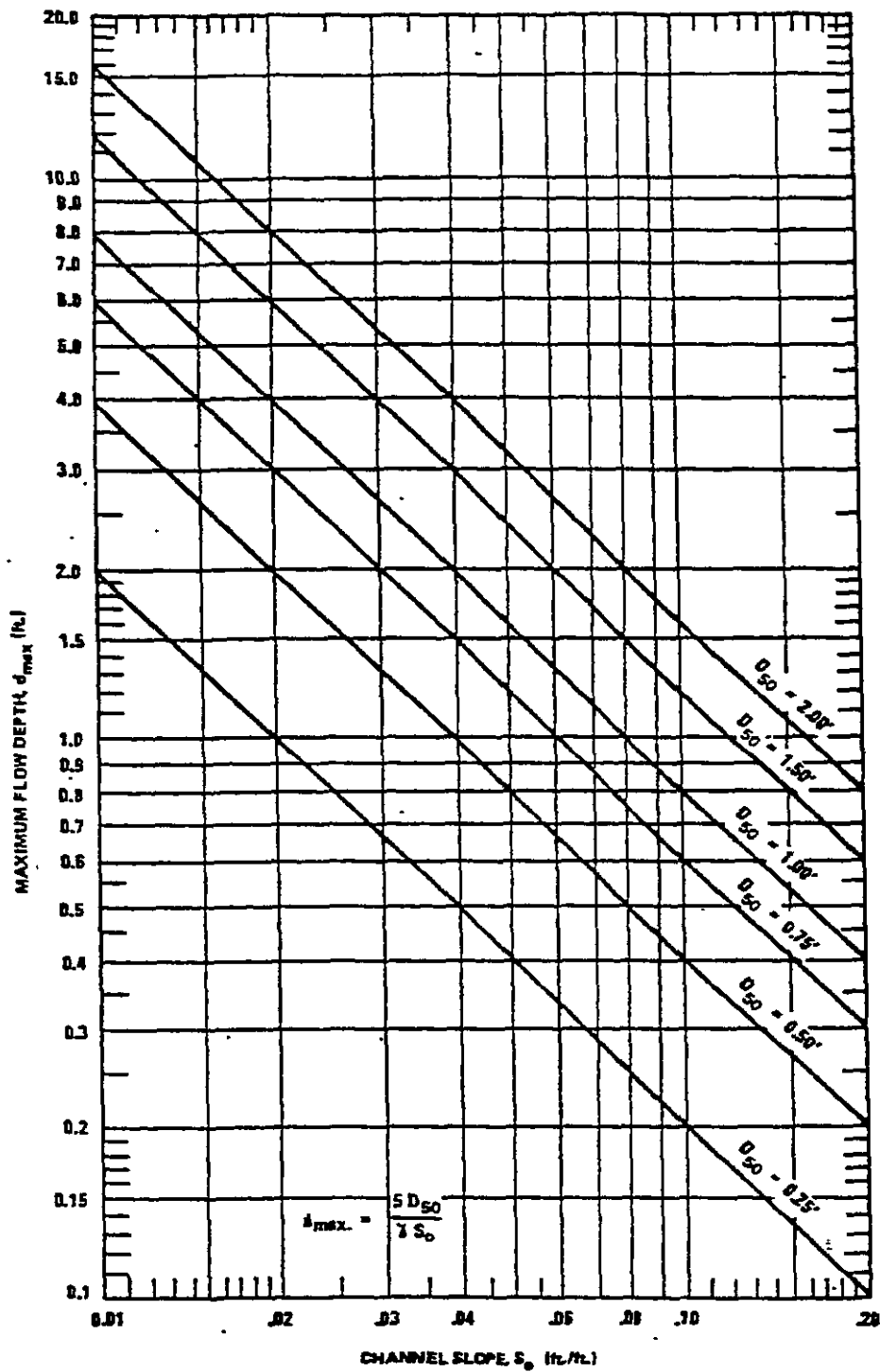


Figure 4.3.5

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 blue-stem, 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.)
C Moderate	Kudzu . . . . .	Dense growth, uncut
	Blue grama . . . . .	Good stand, uncut (av 13 in.)
	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.)
D Low	Centipede grass . . . . .	Very dense cover (av 6 in.)
	Kentucky bluegrass . . . . .	Good stand, headed (6 to 12 in.)
	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.)
E Very low	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
	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  $S_0$ . The curves which are presented in Figures 4.3.8 - 4.3.12 illustrate this relationship for the 5 types of vegetation retardance.

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### **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.



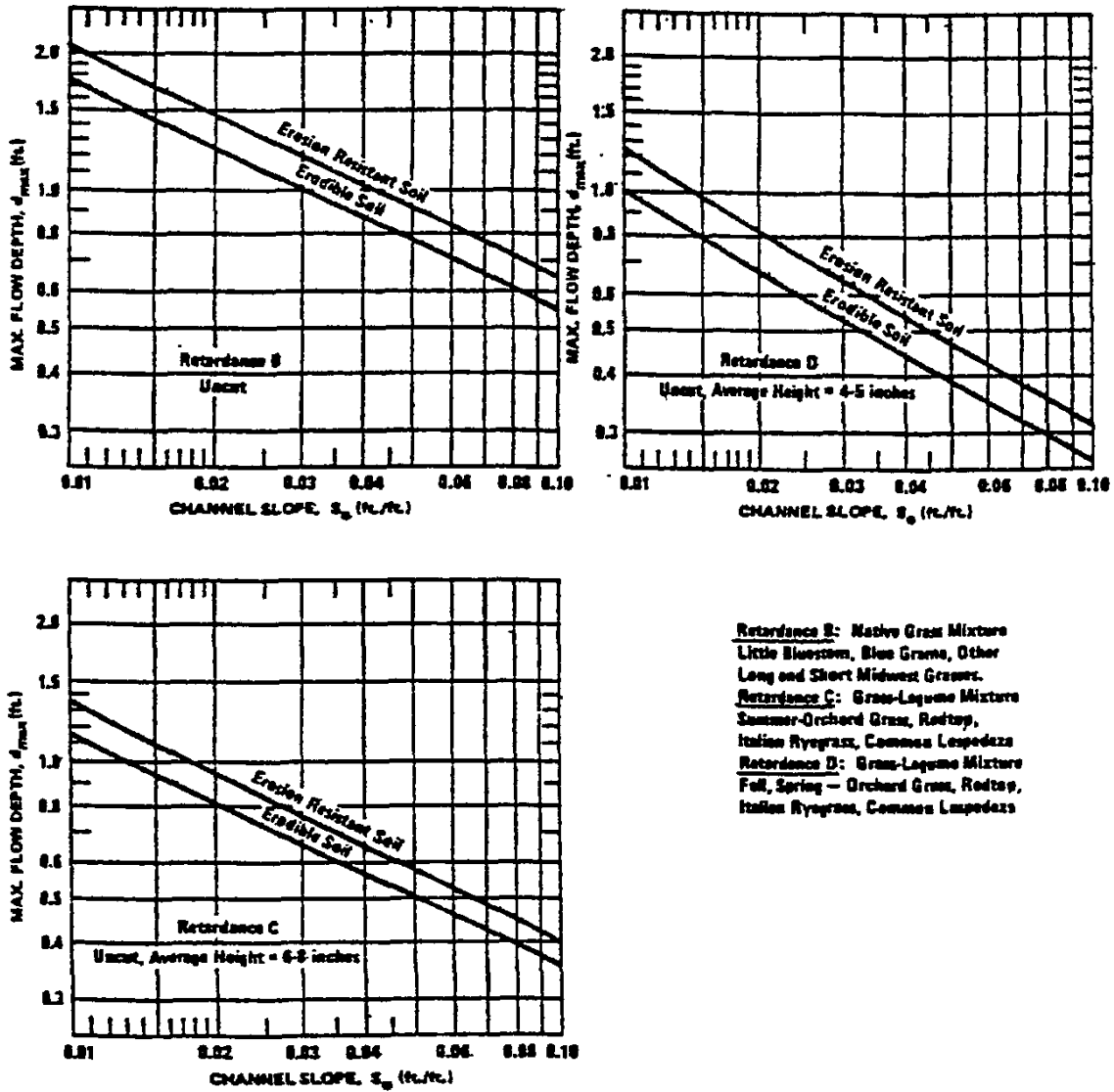


Figure 4.3.7  
Maximum Permissible Depth of Flow ( $d_{max}$ ) for Channels lined  
with Grass Mixtures, Good Standing, Uncut. (Normann, 1975)

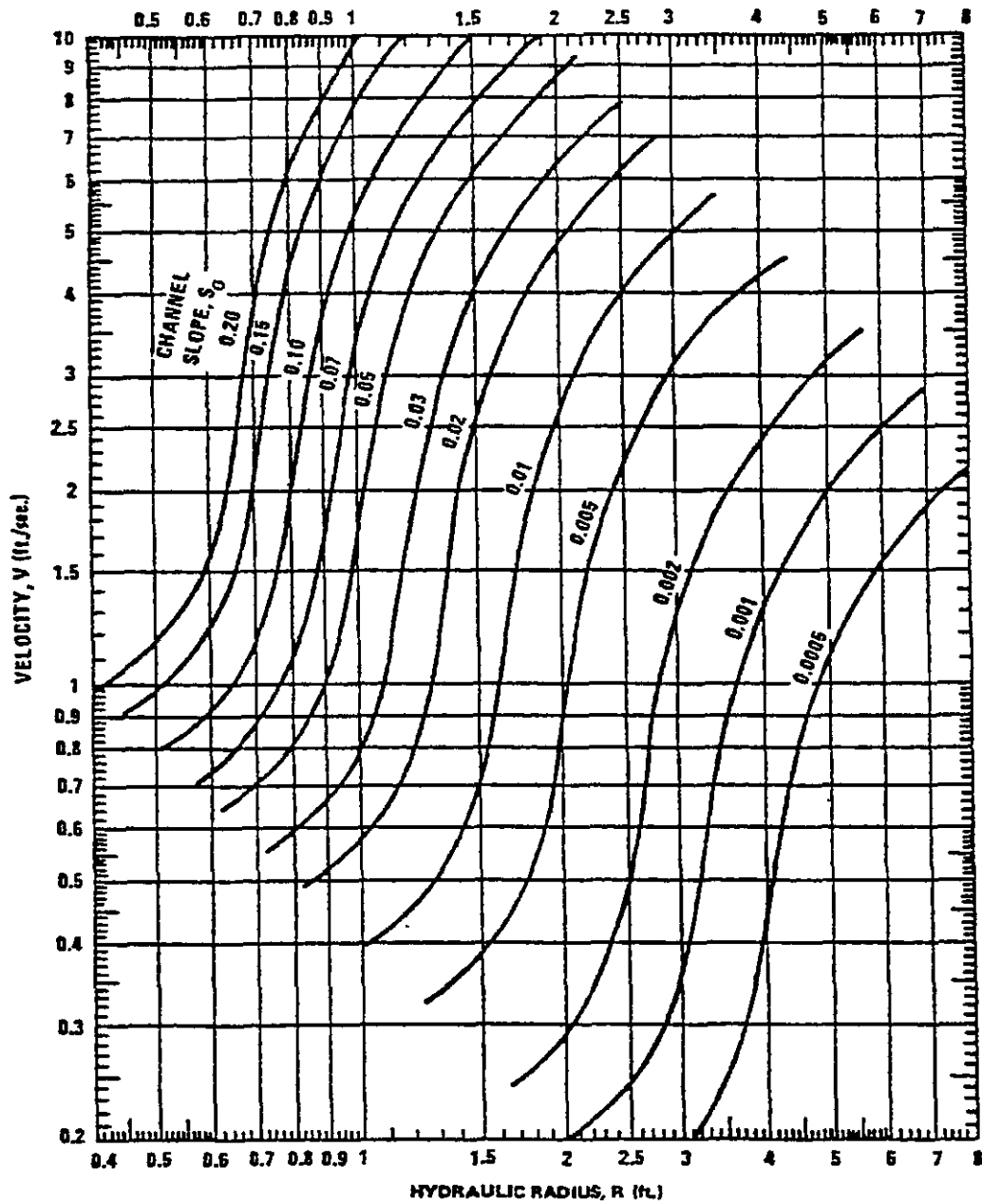


Figure 4.3.8  
 Flow Velocity for Channels Lined with Vegetation of Retardance A (Normann, 1975)



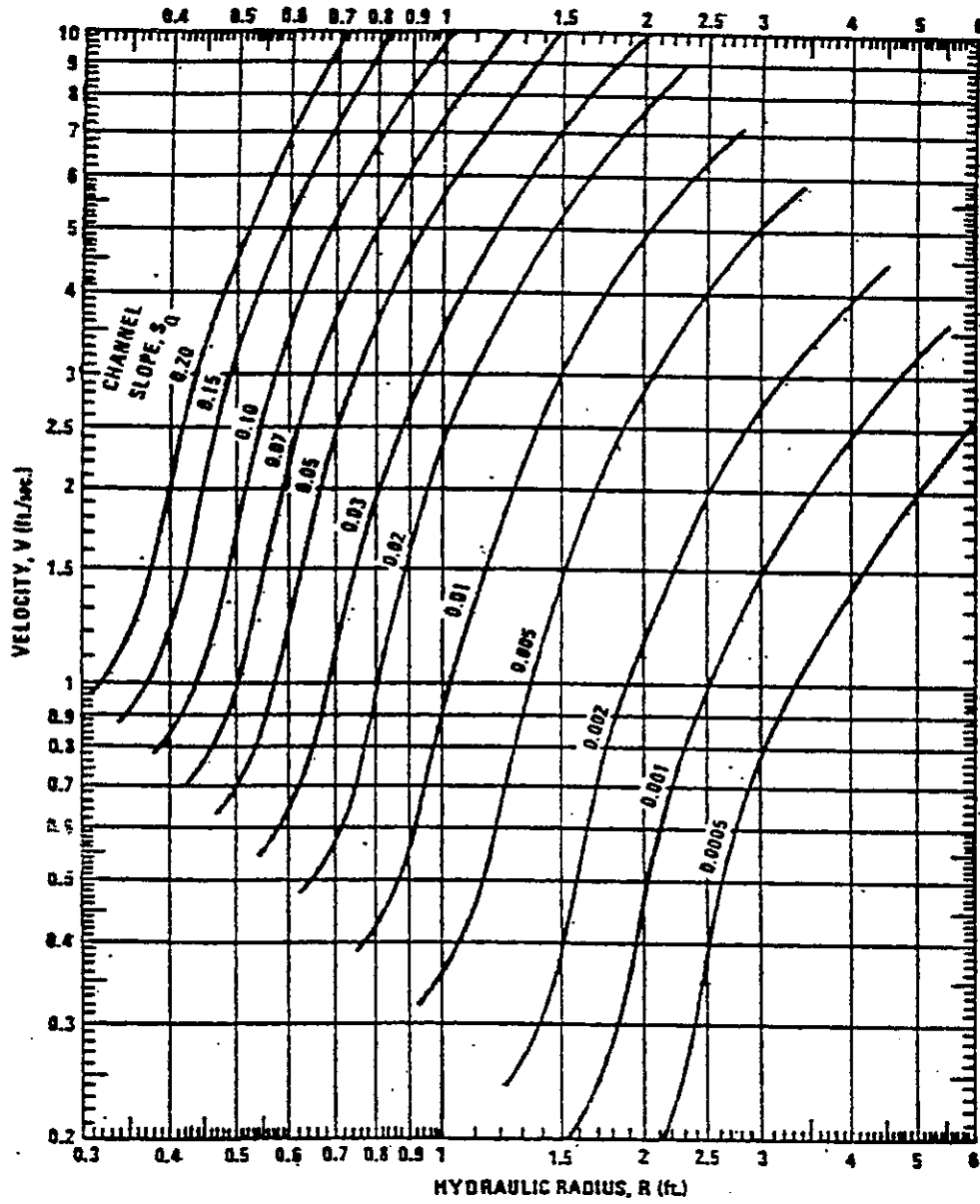


Figure 4.3.9  
Flow Velocity for Channels Lined with Vegetation of Retardance B (Normann, 1975)

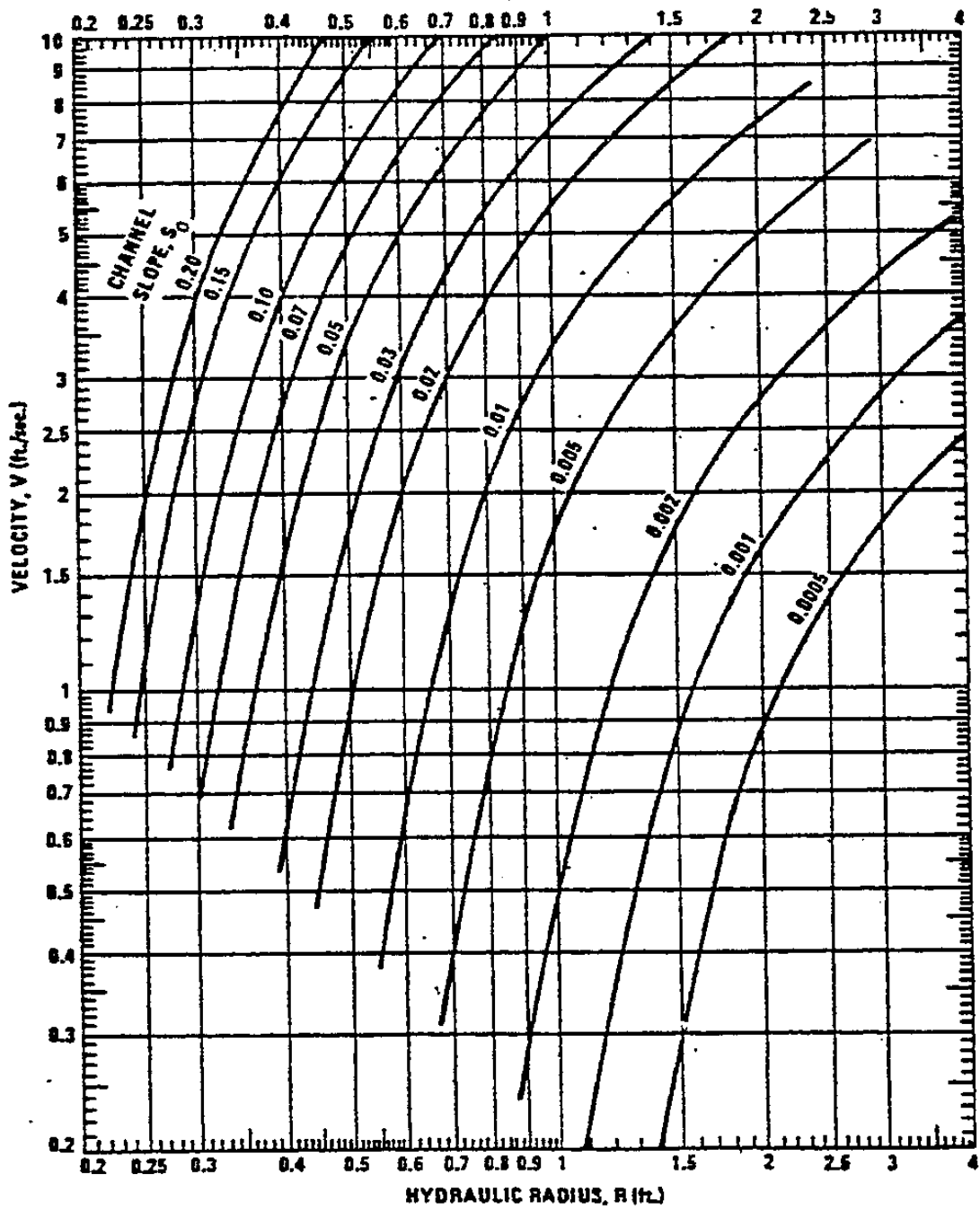


Figure 4.3.10  
 Flow Velocity for Channels Lined with Vegetation of Retardance C (Normann, 1975)

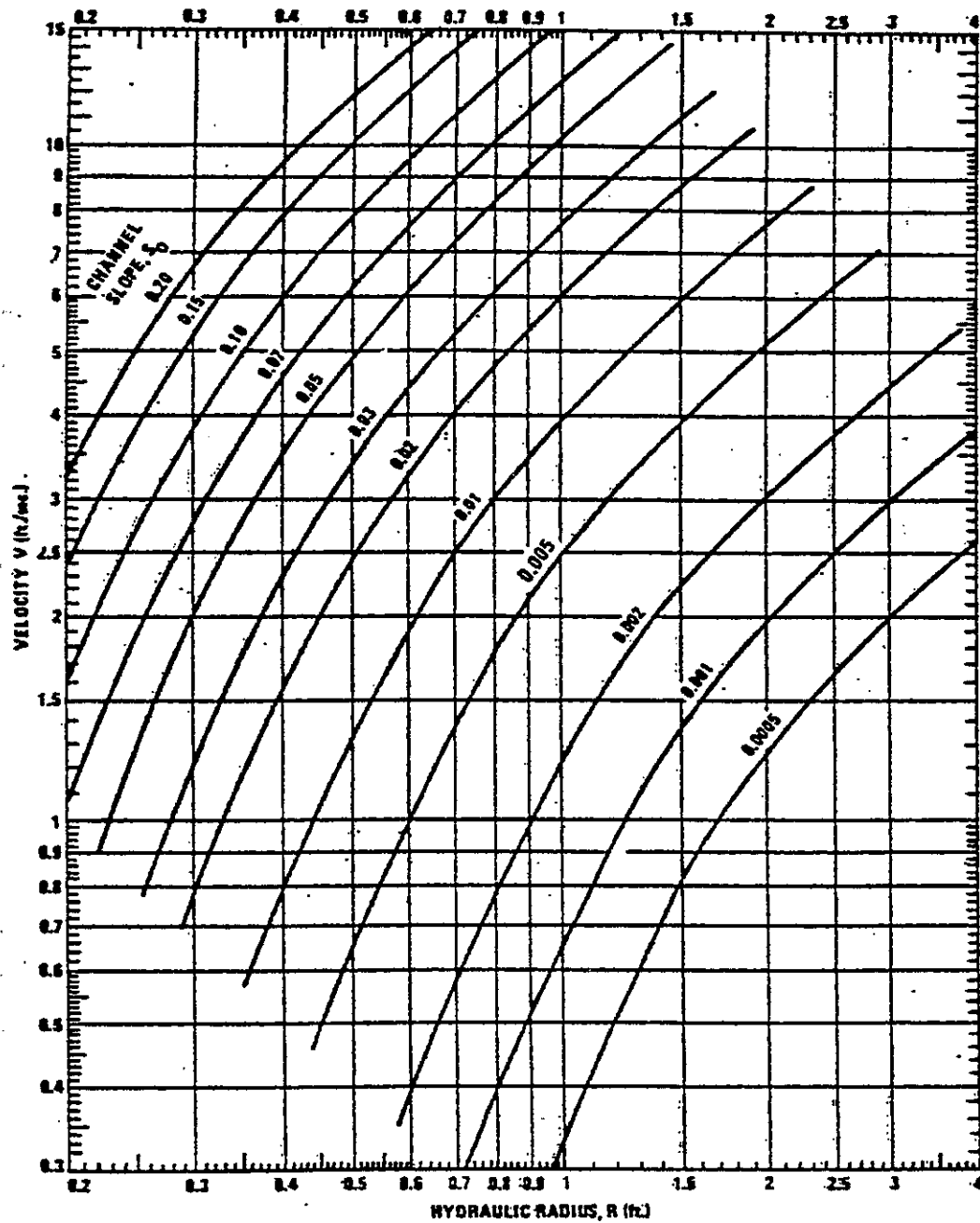


Figure 4.3.11  
 Flow Velocity for Channels Lined with Vegetation of Retardance  $D$  (Normann, 1975)

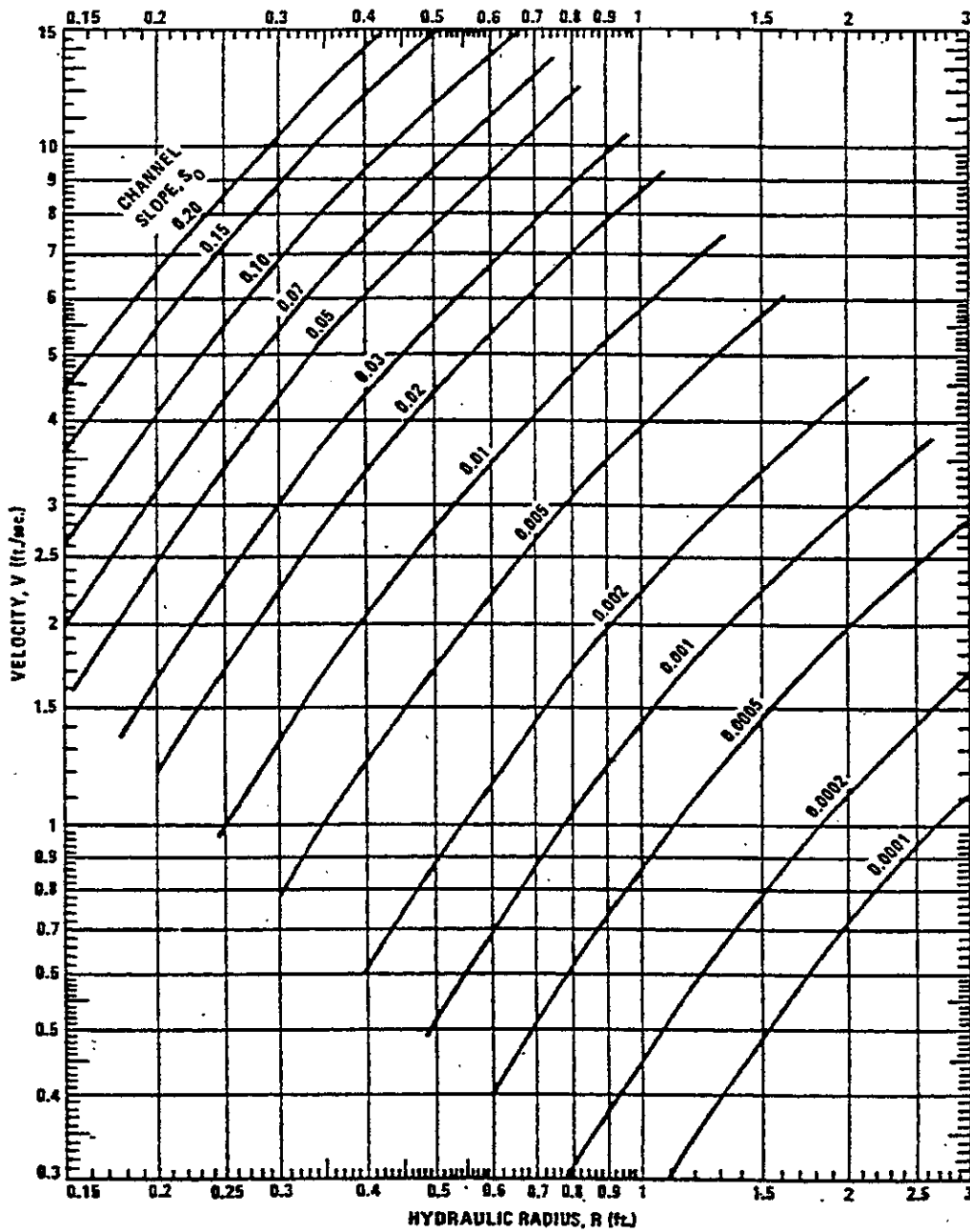
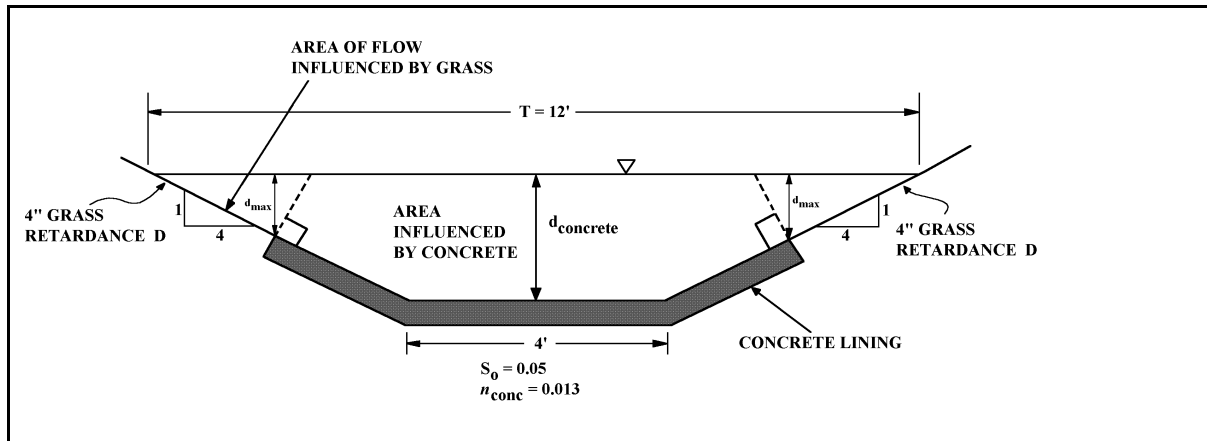


Figure 4.3.12  
Flow Velocity for Channels Lined with Vegetation of Retardance E (Normann, 1975)

**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.



$$d_{max} \text{ (grass mixture)} = 0.45 \text{ ft (Figure 4.3.7) (Take the average value)}$$

For  $T_{max} = 12 \text{ ft.}; d \text{ (concrete)} = 1.0 \text{ ft}$   
 $A_{total} = (4.0)(1.0) + 4(1.0)^2 = 8.0 \text{ ft}^2$   
 $P_{w \text{ total}} = 4.0 + 2(\sqrt{17})(1.0) = 12.25 \text{ ft}$

For shear boundary, take normal to side slopes at edge of concrete lining.

$$A_{grass} = (4d_{max} + 0.25d_{max})_{grass} * d_{max \text{ grass}} = (4.25)(0.45)^2 = 0.86 \text{ ft}^2$$

$$P_{w \text{ grass}} = 2(\sqrt{17}) d_{max} = 2(\sqrt{17})(.45) = 3.71 \text{ ft}$$

$$R_{grass} = (0.86/3.71) = 0.23 \text{ ft}$$

$$V_{grass} = 1.00 \text{ ft/sec (Figure 4.14)}$$

$$Q_{grass} = AV = (1.00)(0.86) = 0.86 \text{ cfs}$$

$$A_{concrete} = A_{total} - A_{grass} = 8.0 \text{ ft}^2 - 0.86 \text{ ft}^2 = 7.14 \text{ ft}^2$$

$$P_{w \text{ concrete}} = P_{w \text{ total}} - P_{w \text{ grass}} = 12.25 \text{ ft} - 3.71 = 8.54 \text{ ft}$$

$$R_{concrete} = 7.14 \text{ ft}^2 / 8.54 \text{ ft} = 0.84 \text{ ft}$$

$$Q_{concrete} = (1.49/n) AR^{2/3} S_o^{1/2} = (1.49/0.013) (7.14) (0.84)^{2/3} (0.05)^{1/2}$$

$$Q_{total} = 163 \text{ cfs}$$

$$\text{Outlet velocity} = (Q_{total}/A_{total}) = 163/8.0 = 20.4 \text{ ft/sec}$$

### 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

$$n = 0.0395 D_{50}^{1/6} \quad (4.3.5)$$

where  $D_{50}$  = mean stone size (feet).

- (b) For bare soil: Use

$$V = 22.81 R^{0.591} S_0^{0.286} \quad (4.3.6)$$

- (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**

**PROJECT:** \_\_\_\_\_ **DATE:** \_\_\_\_\_  
**STATION:** \_\_\_\_\_ **TO STATION:** \_\_\_\_\_ **DESIGNER:** \_\_\_\_\_  
**DRAINAGE AREA =** \_\_\_\_\_ **ACRES** **CHECKED BY:** \_\_\_\_\_  
**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)



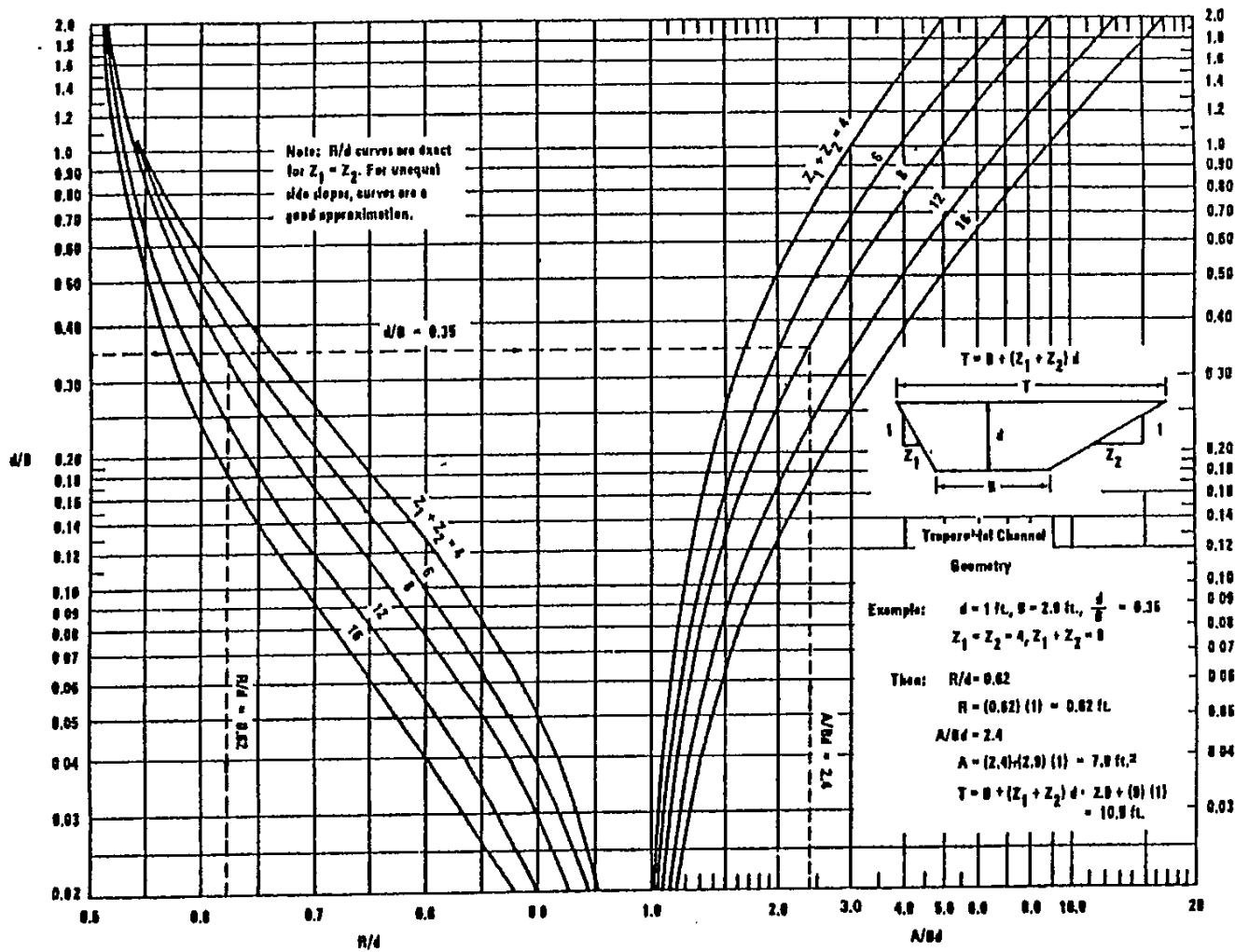


Figure 4.3.14  
 Nomograph for Design of Trapezoidal Channel Section (Normann, 1975)

#### 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  $B_S/R_O = 0.5$ ,  $K_3 = 1.18$ .

$$(D_{50})_{Bend} = 1.18(D_{50})_{Straight} = (1.18)(0.5) = 0.59 \text{ ft}$$

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

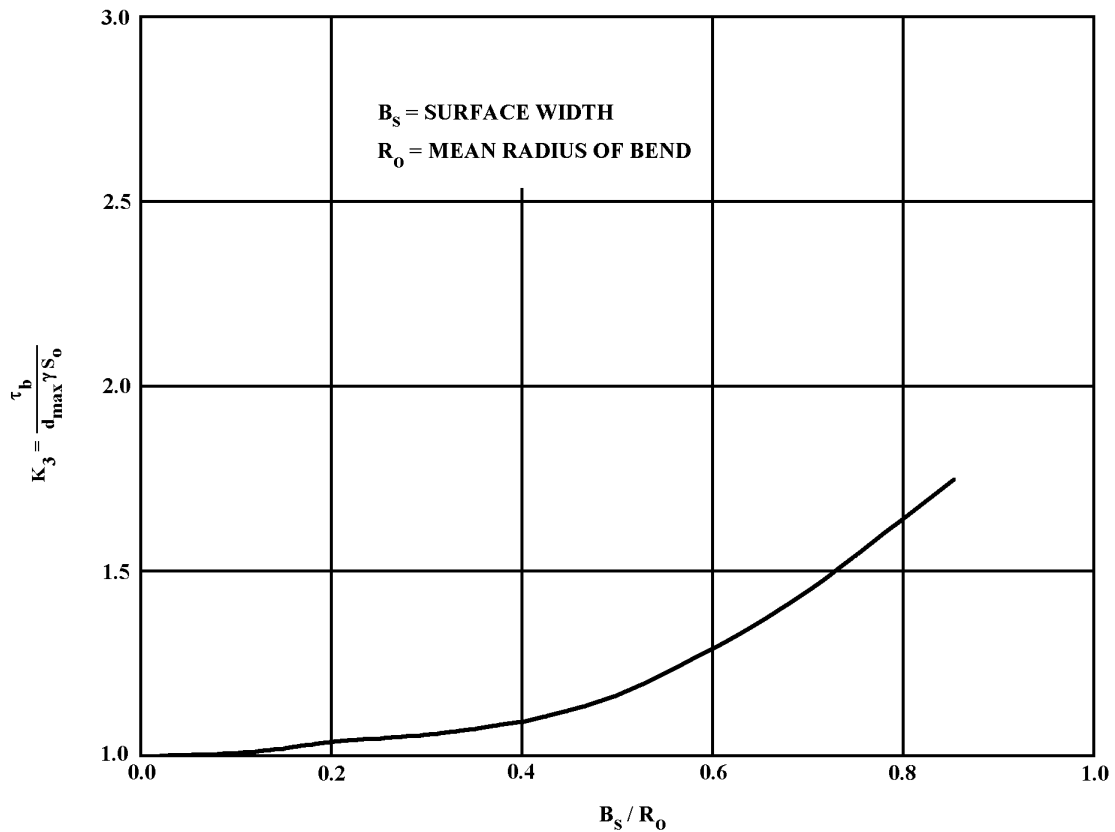


Figure 4.3.15  
 Ratio of Maximum Boundary Shear in Bends to Maximum  
 Bottom Shear in Straight Reaches (Anderson, 1973)  
 Note:  $B_s = T$

#### 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

$$H = \frac{V^2}{2g} + Z + d \cos \theta \quad (4.4.1)$$

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

$$\frac{dd}{dx} = \frac{S_o - S_f}{\cos \Theta + \frac{d(V^2/2g)}{dd}} \quad (4.4.2)$$

water surface slope  $dd/dx$ .

For small slopes  $\cos \Theta = 1$  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.

$$\frac{dy}{dx} = \frac{S_o - S_f}{1 + d(V^2/2g)dy} \quad (4.4.3)$$

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 $S_b = 0$	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 $0 < S_b < S_c$	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 $S_b = S_c > 0$	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 $S_b > S_c > 0$	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 $S_b < 0$	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.

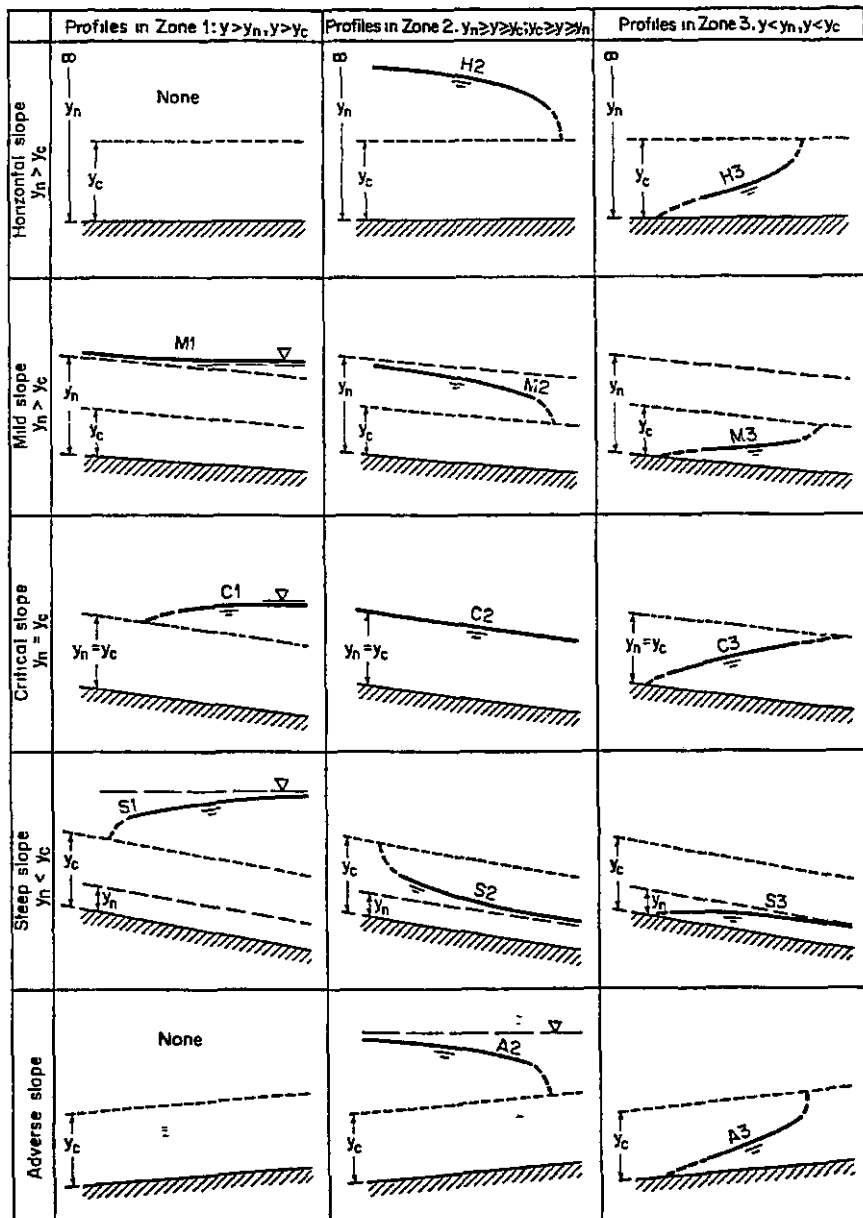


Figure 4.4.1  
Classification of Flow Profiles of Gradually Varied Flow (Chow, 1988)

#### 4.4.2 - Backwater Curve Calculations

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  $\Delta X$ , can be developed.

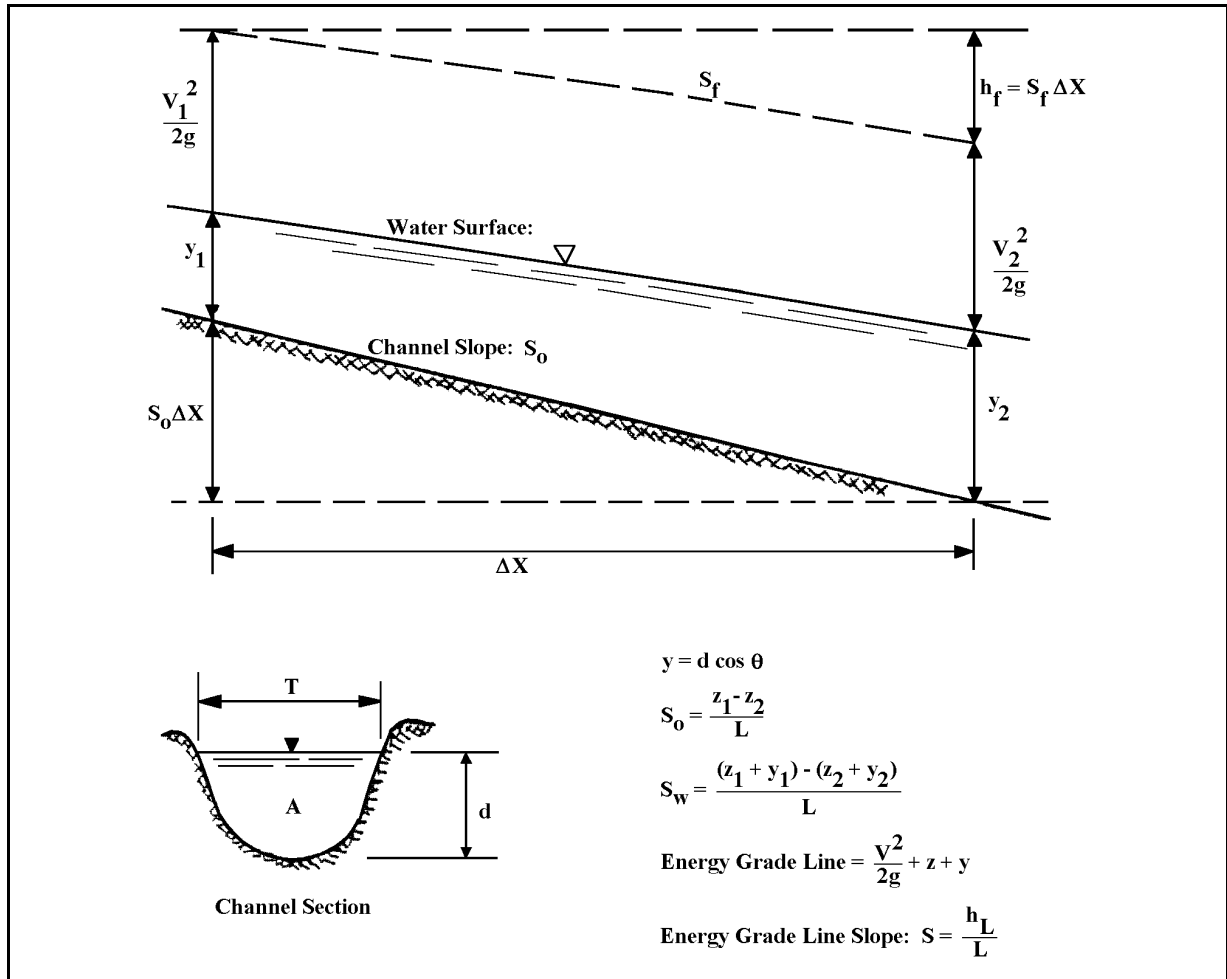


Figure 4.4.2  
A Channel Length Used for the Development of the Direct Step Method

$$S_o \Delta X + y_1 + \frac{V_1^2}{2g} = y_2 + \frac{V_2^2}{2g} + S_f \Delta X \quad (4.4.4)$$

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  $\Delta X$ .

$$\Delta X = \frac{E_2 - E_1}{S_o - S_f} = \frac{\Delta E}{S_o - S_f} \quad (4.4.5)$$

$S_o$  is found from channel characteristics and  $S_f$  is determined from the Mannings' Formula:

$$S_f = \frac{n^2 V^2}{2.22 R^{4/3}} \quad (4.4.6)$$

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 ( $\Delta E$ ) 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.



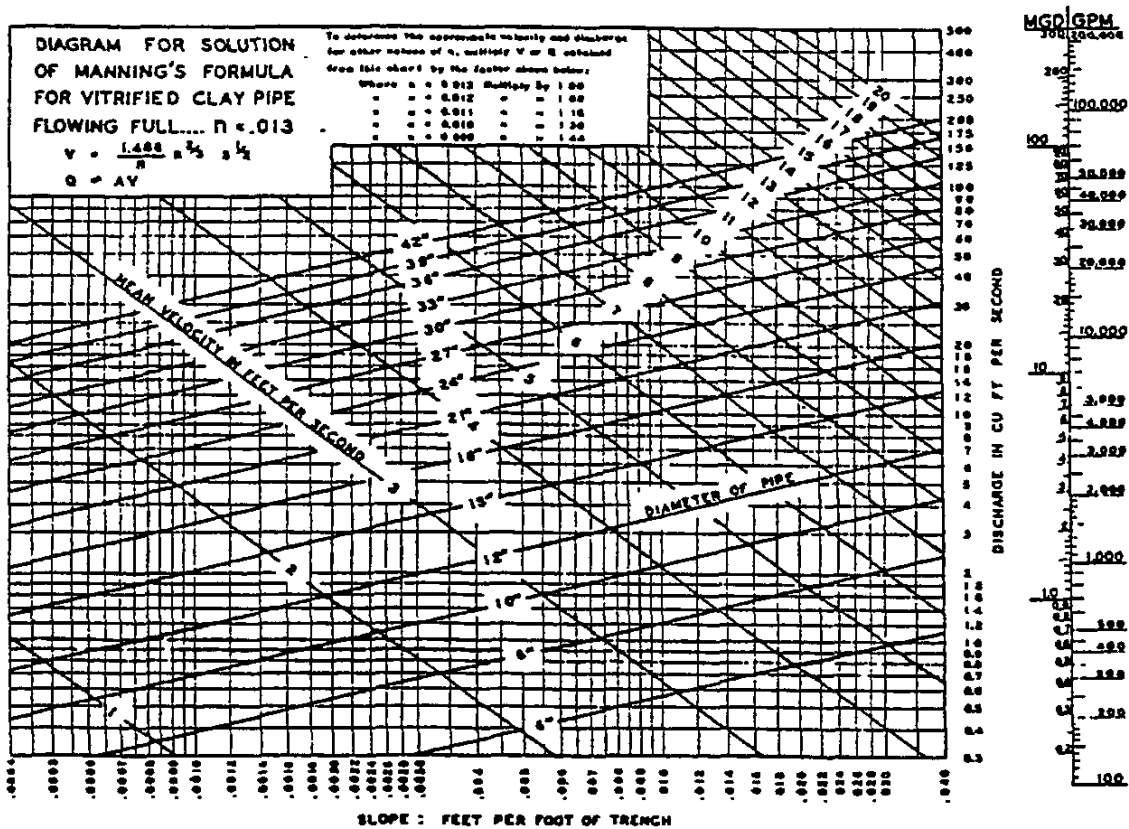


Figure 4.4.3  
 Graphical Solution of the Manning Equation for Circular Pipe Flowing Full

### 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  $\Delta X$  and the composite length  $X$  are found. For this example, the depth of 4.4 feet occurs 893 feet upstream.

<b>y</b> (ft)	<b>A</b> (ft <sup>2</sup> )	<b>R</b> (ft)	<b>R<sup>4/3</sup></b> (ft <sup>4/3</sup> )	<b>v</b> (ft/sec)	<b>v<sup>2</sup>/2g</b> (ft)	<b>E</b> (ft)	<b>ΔE</b> (ft)	<b>S<sub>f</sub></b> (ft/ft)	<b><math>\bar{S}_f</math></b> (ft/ft)	<b>S<sub>o</sub> - S<sub>f</sub></b> (ft/ft)	<b>ΔX</b> (ft)	<b>X</b> (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.

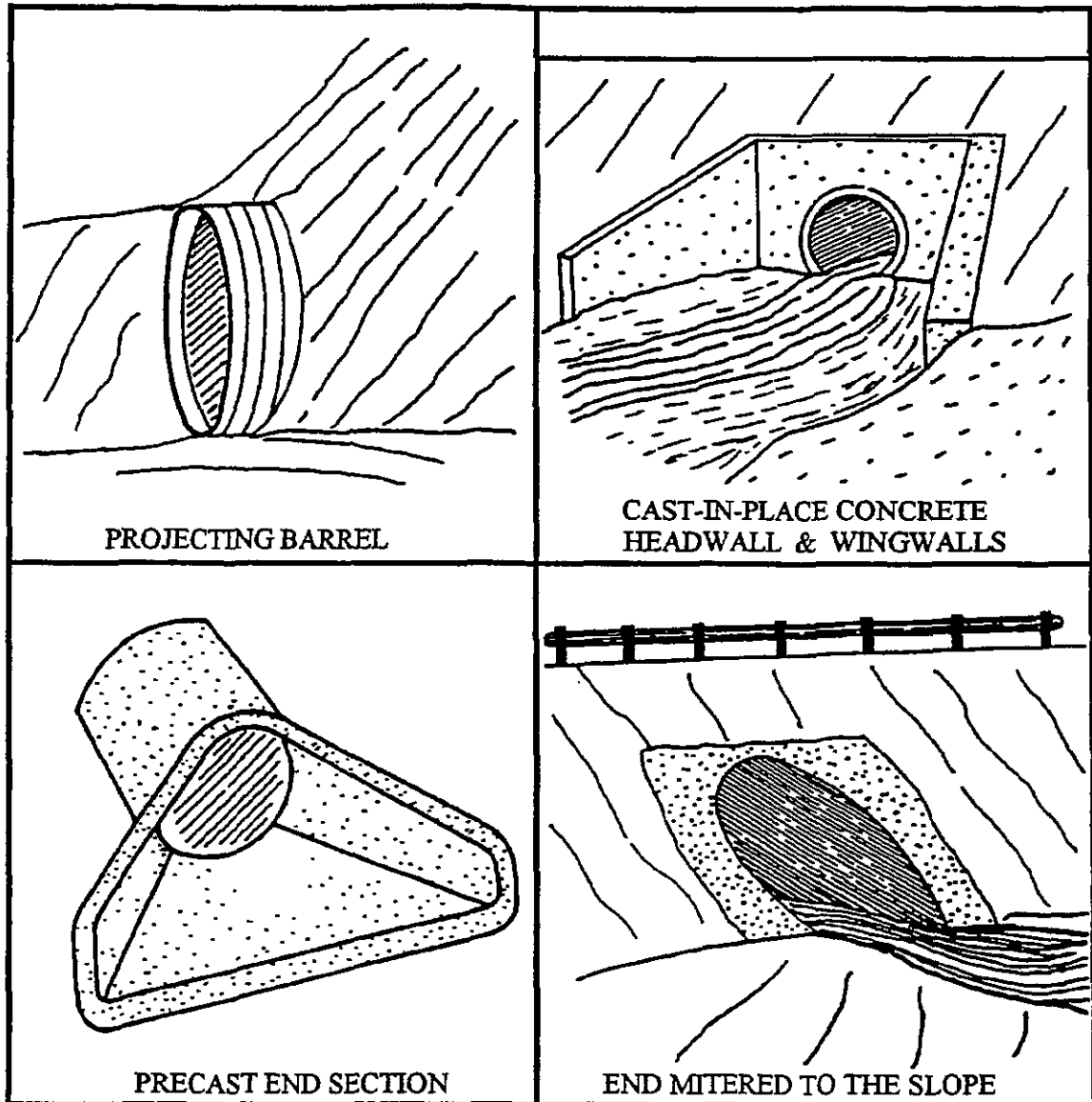


Figure 4.5.1  
Four Standard Inlet Types (schematic) (FHWA, 1985)

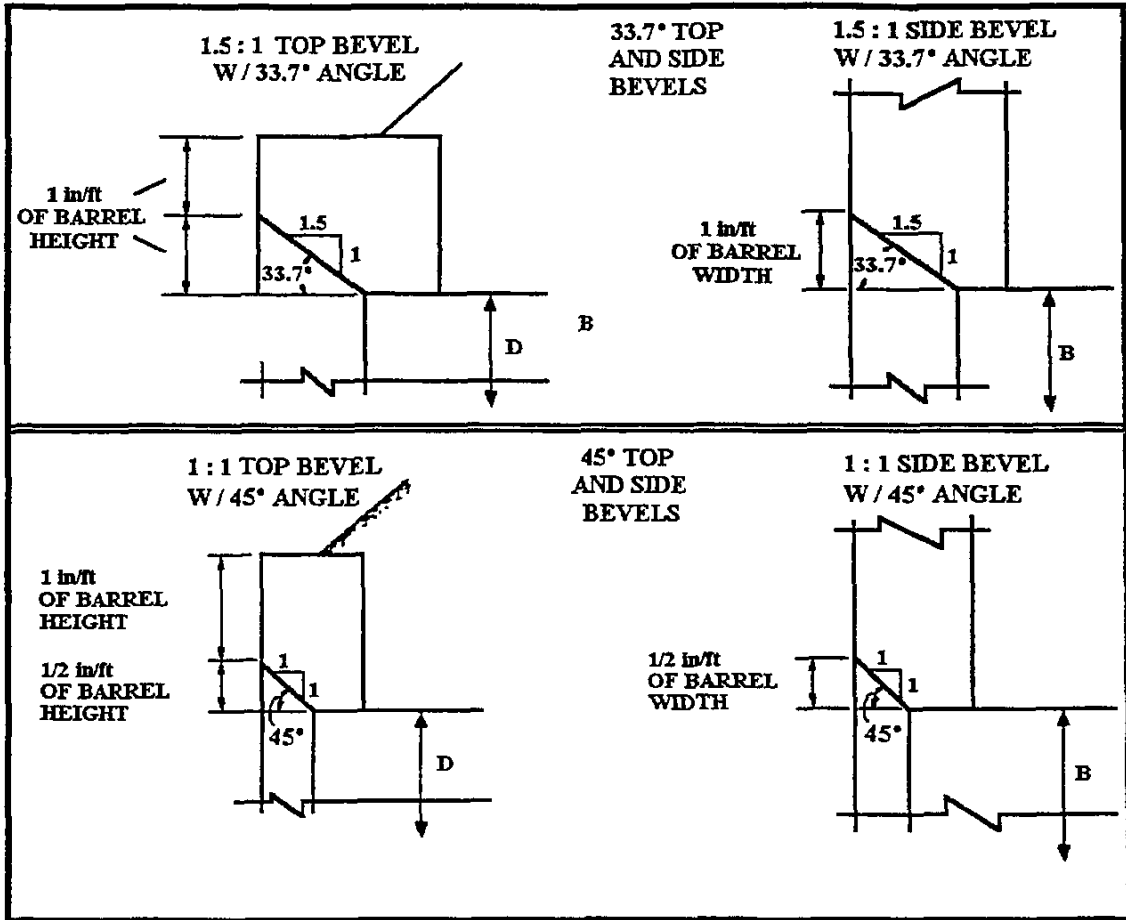


Figure 4.5.2  
Beveled Edge Inlets (FHWA, 1985)

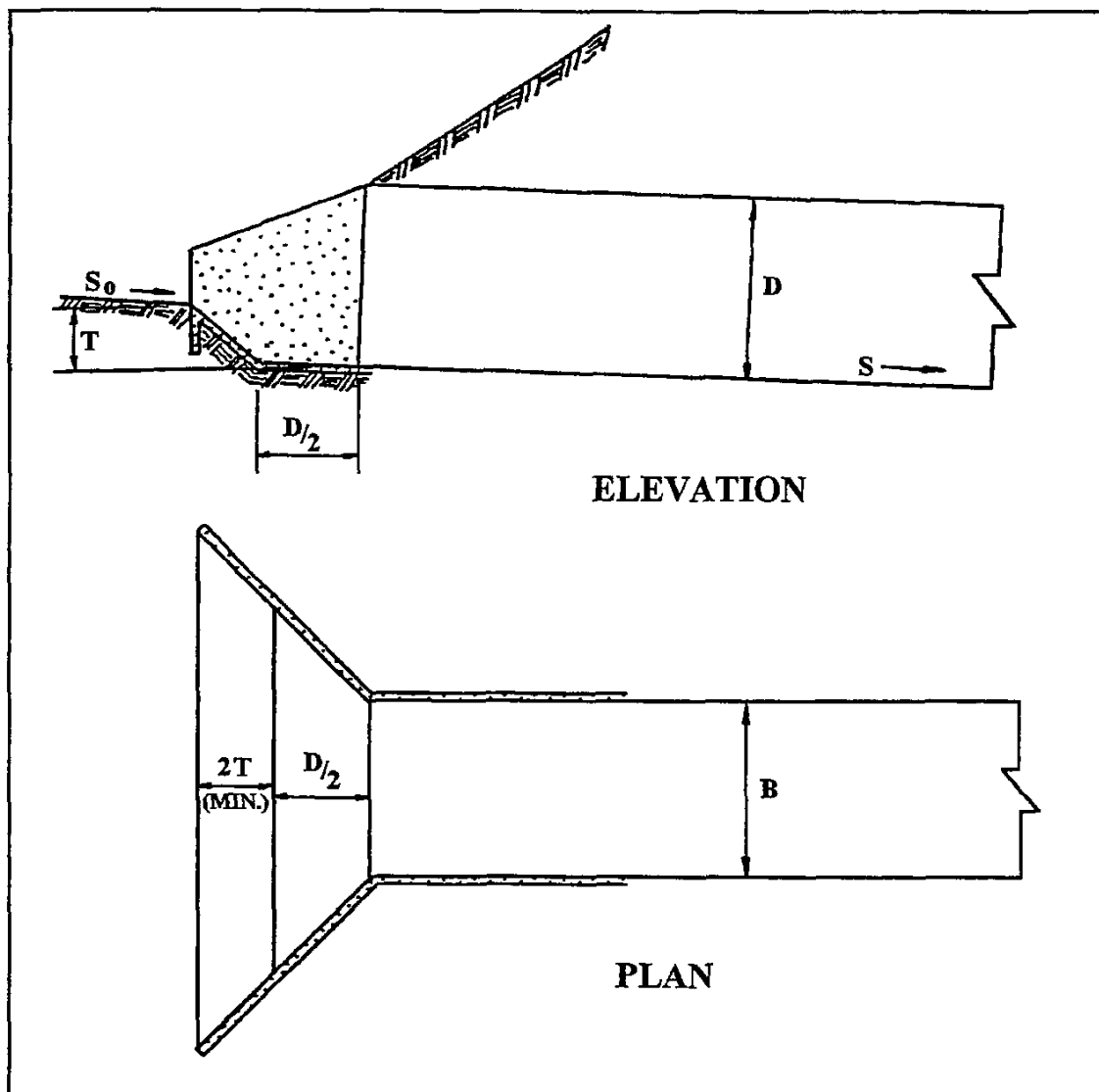


Figure 4.5.3  
Culvert with Depressed Apron and Wingwalls (FHWA, 1985)

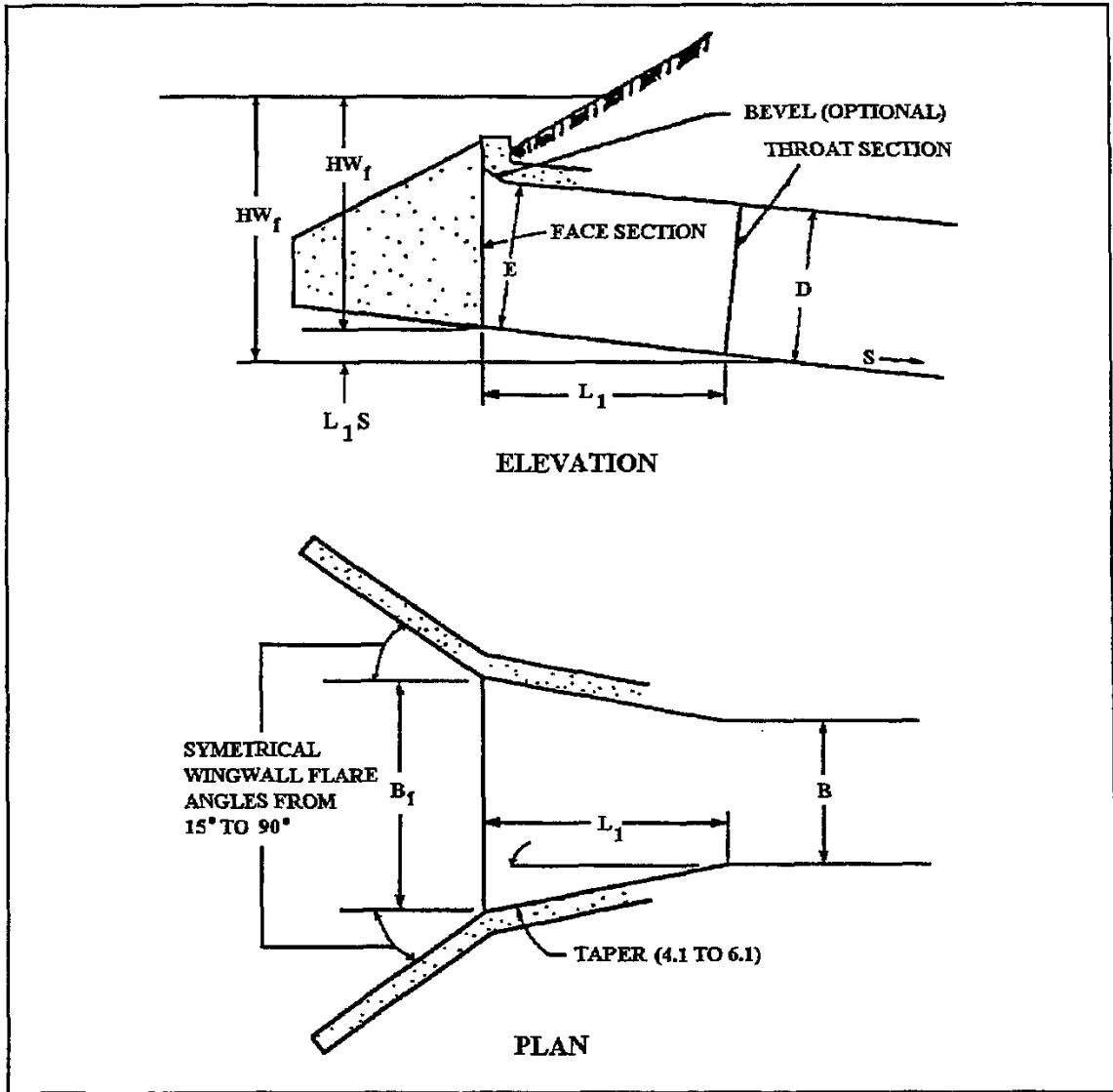


Figure 4.5.4  
Side-tapered Inlet (FHWA, 1985)

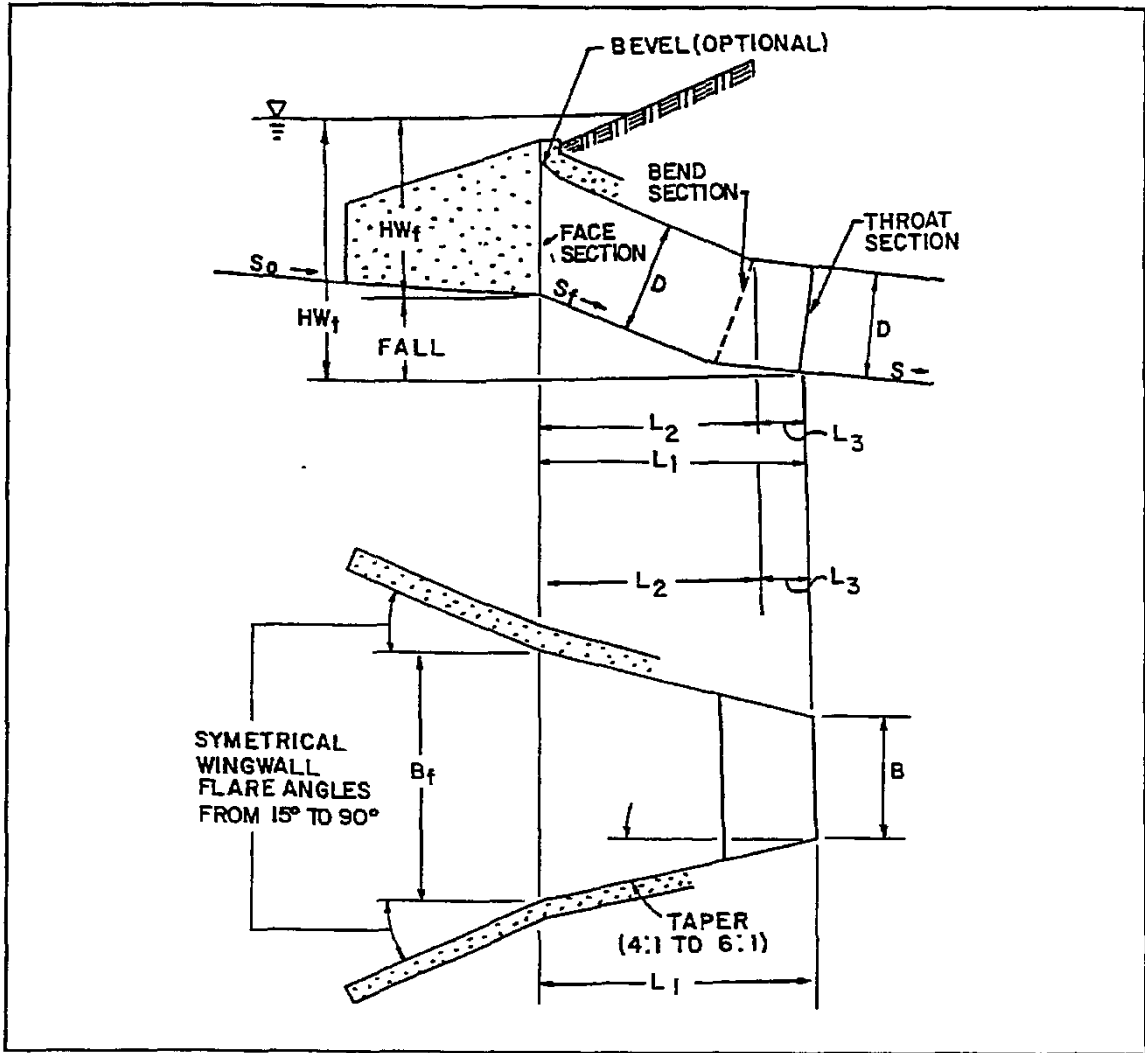


Figure 4.5.5  
Slope-tapered Inlet with Vertical Face (FHWA, 1985)

### 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 screenshot shows the 'Crossing Data - Example 4.5.1' window. It is divided into two main sections: 'Crossing Properties' and 'Culvert Properties'.

**Crossing Properties:**

- Name: Example 4.5.1
- DISCHARGE DATA:** Minimum Flow: 0.00 cfs, Design Flow: 50.00 cfs, Maximum Flow: 100.00 cfs.
- TAILWATER DATA:** Channel Type: Irregular Channel.
- ROADWAY DATA:** Roadway Profile Shape: Irregular, Roadway Surface: Paved, Top Width: 48.00 ft.

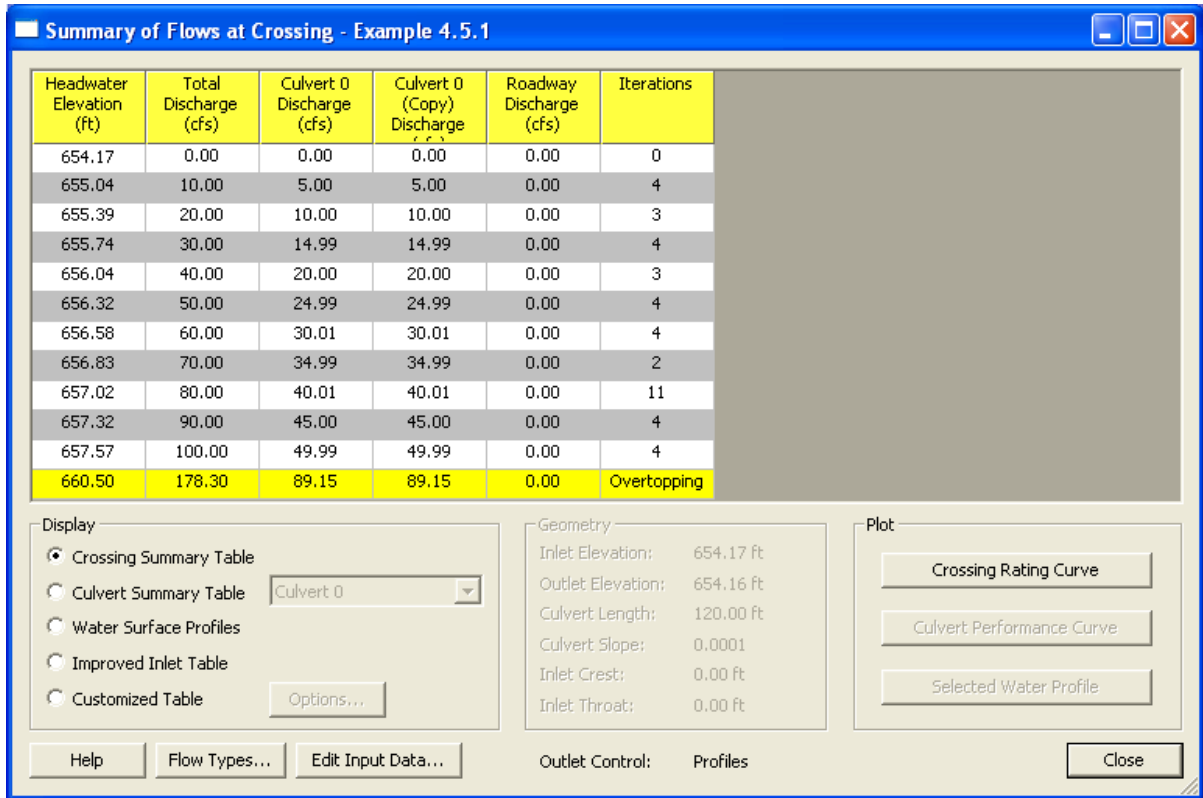
**Culvert Properties:**

- Culvert 0 (Copy)
- CULVERT DATA:** Name: Culvert 0, Shape: Pipe Arch, Material: Steel or Aluminum, Size: Define..., Span: 57.00 in, Rise: 38.00 in, Manning's n: 0.0240.
- INLET DATA:** Inlet Type: Conventional, Inlet Edge Condition: Projecting, Inlet Depression?: No.
- SITE DATA:** Site Data Input Option: Culvert Invert Data, Inlet Station: 0.00 ft, Inlet Elevation: 654.17 ft, Outlet Station: 120.00 ft.

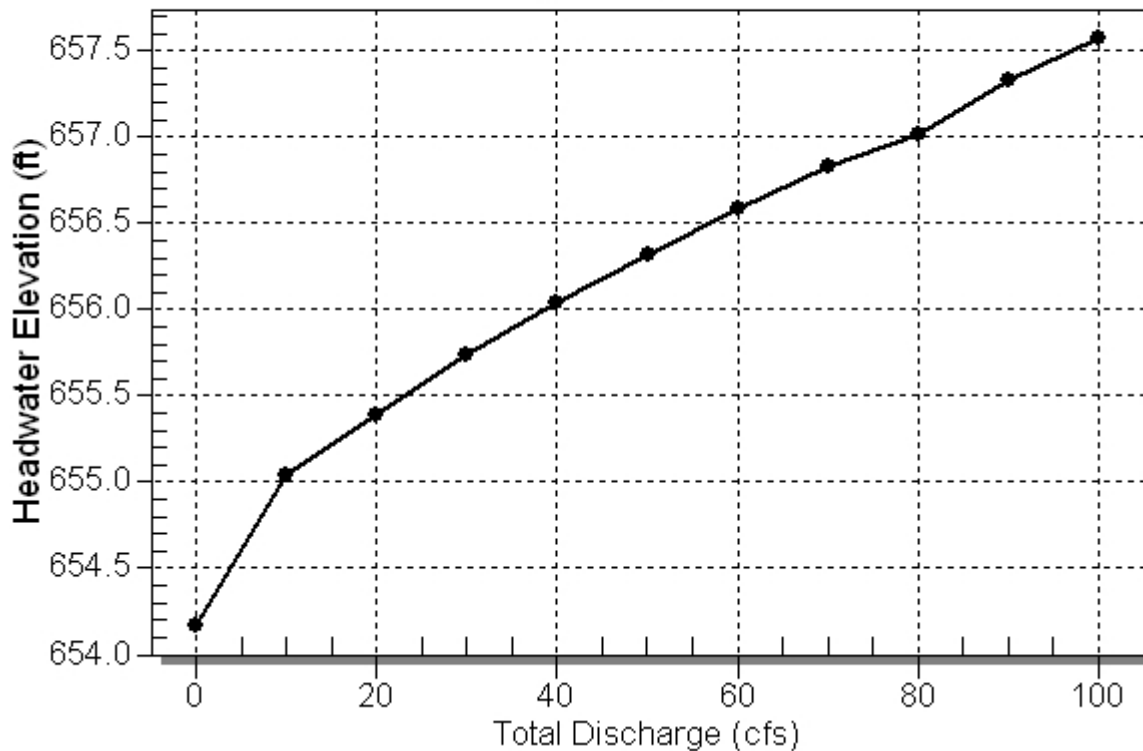
Buttons at the bottom include: Help, Analyze Crossing, OK, and Cancel.

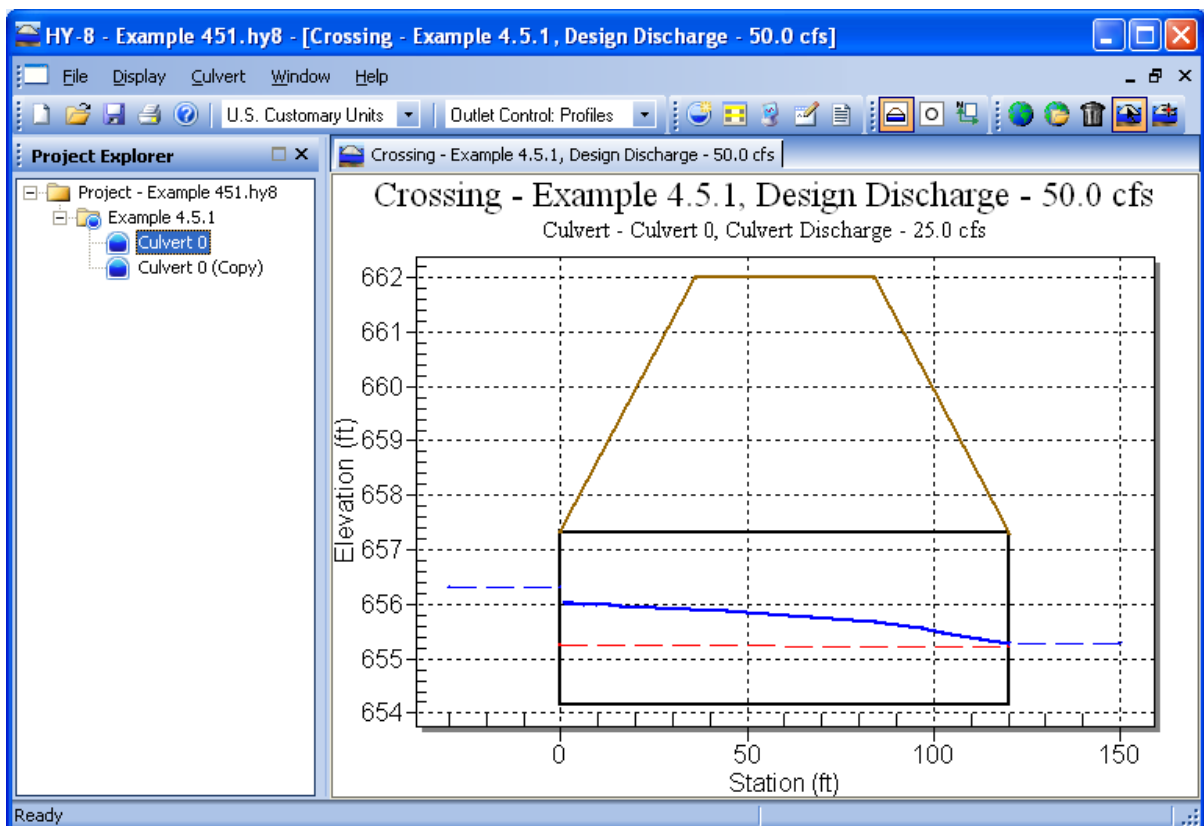
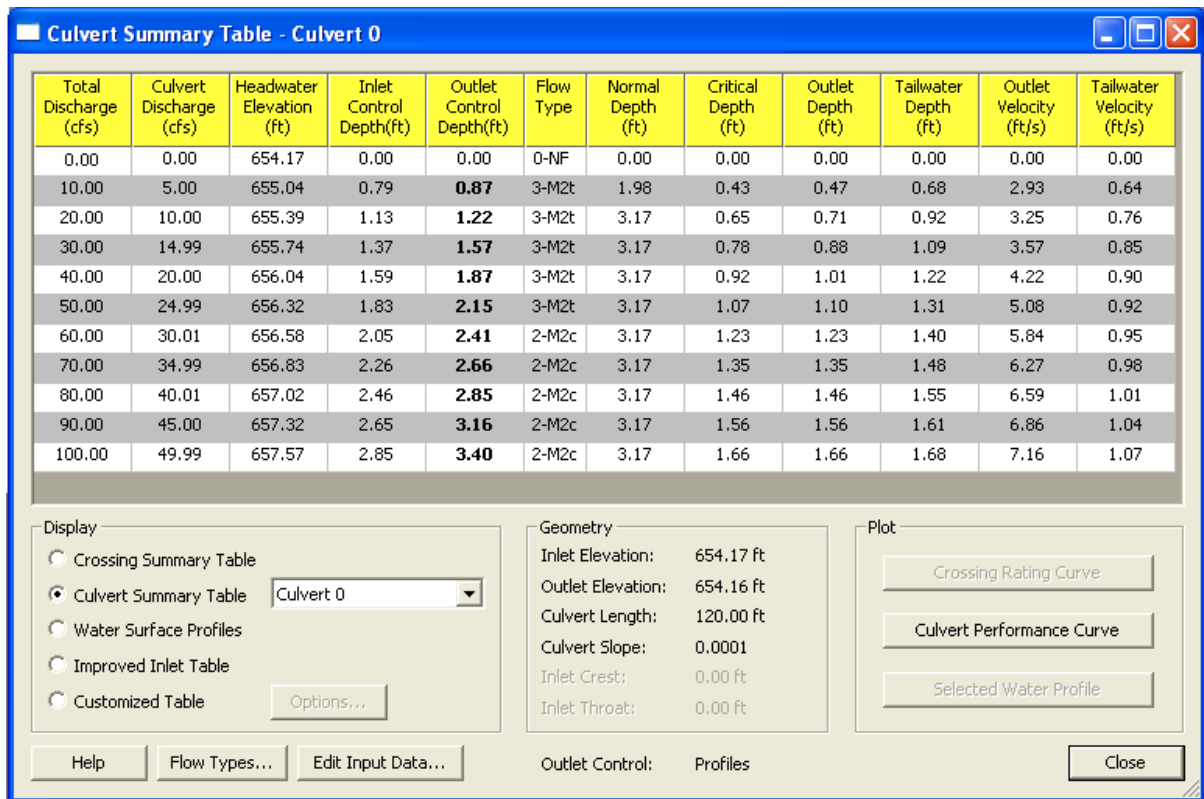
The following pages contain the HY-8 results that are presented in a report format as well as some graphs produced by the program. There is no overtopping for the culvert flows ranging between 0 and 50 cfs. The flow at which roadway overtopping occurs is at 178 cfs.





**Rating Curve Plot for Crossing: Example 4.5.1**  
**Total Rating Curve**  
**Crossing: Example 4.5.1**





**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**

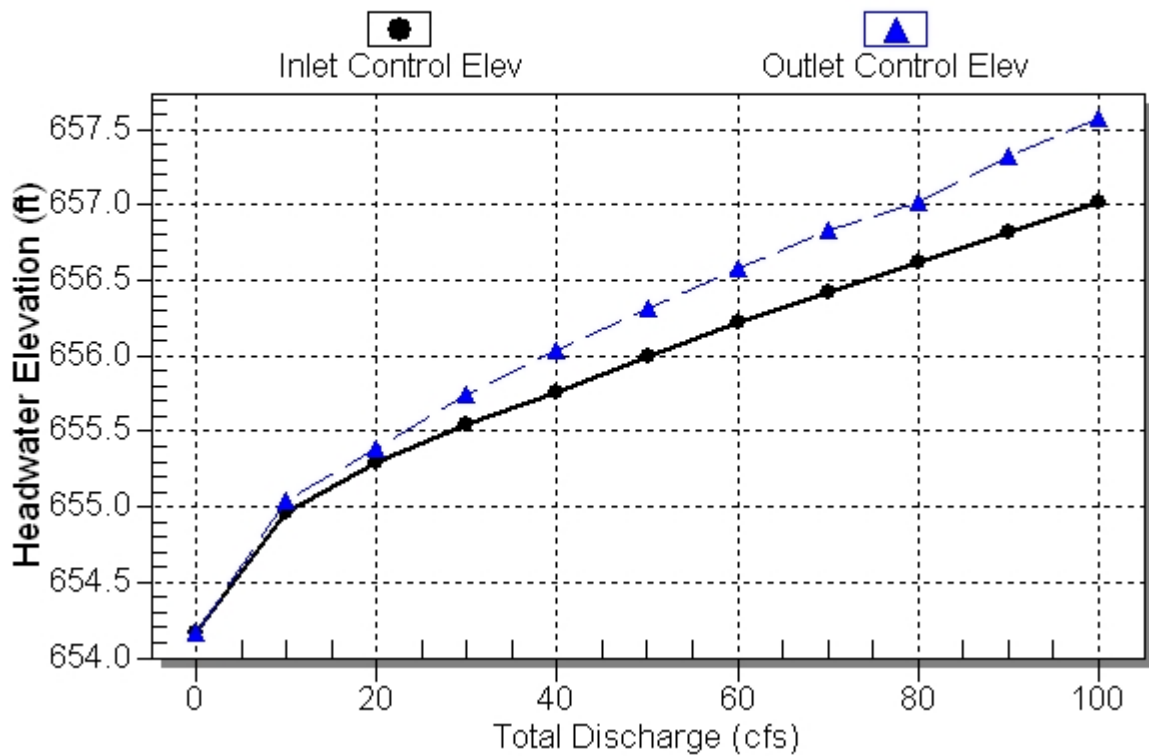
Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00	0.00	654.17	0.000	0.000	0-NF	0.000	0.000	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

\*\*\*\*\*  
 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**

**Performance Curve**

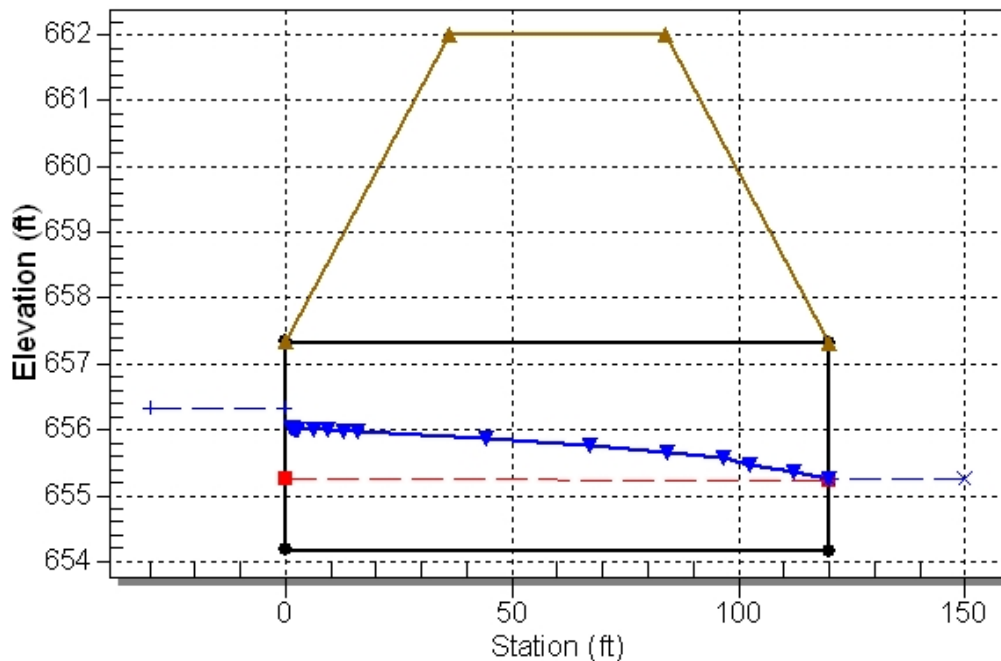
Culvert: Culvert 0



**Water Surface Profile Plot for Culvert: Culvert 0**

Crossing - Example 4.5.1, Design Discharge - 50.0 cfs

Culvert - Culvert 0, Culvert Discharge - 25.0 cfs



**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

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## Chapter 5 - FLOW IN GUTTERS AND INLETS

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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 <sup>2</sup> or m <sup>2</sup> )
$A_i$	Clear opening of grate inlet (ft <sup>2</sup> or m <sup>2</sup> )
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 <sup>2</sup> or m/sec <sup>2</sup> )
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 <sup>3</sup> /sec)
$Q_B$	Remaining flow in street between distance x and T (cfs or m <sup>3</sup> /sec)
$Q_c$	Carryover flow from upgrade inlet (cfs or m <sup>3</sup> /sec)
$Q_G$	Gutter flow (cfs or m <sup>3</sup> /sec)
$Q_i$	Capacity of inlet (cfs or m <sup>3</sup> /sec)
$Q_x$	Flow in street between gutter and a distance x from the gutter (cfs or m <sup>3</sup> /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  $S_T$ .

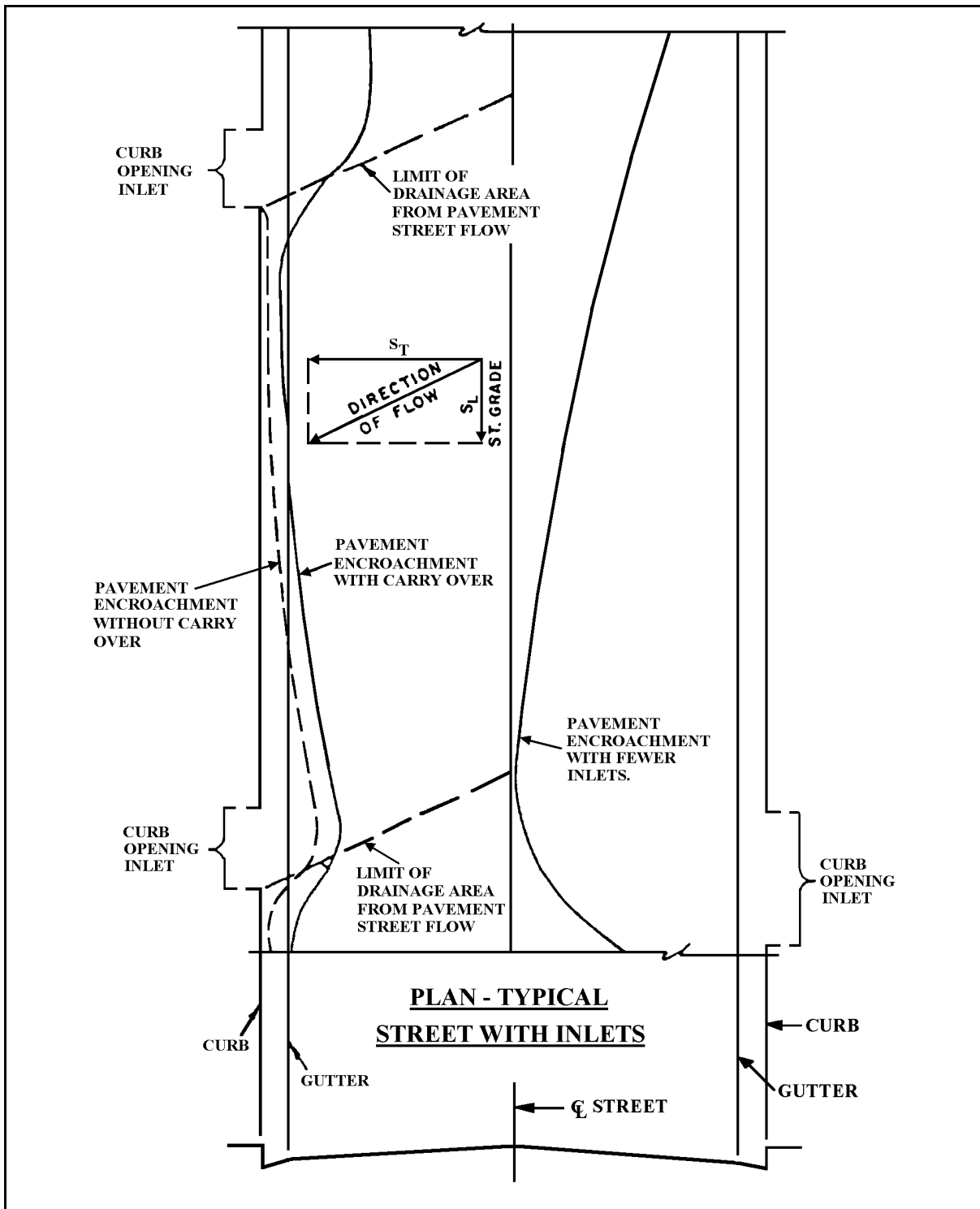


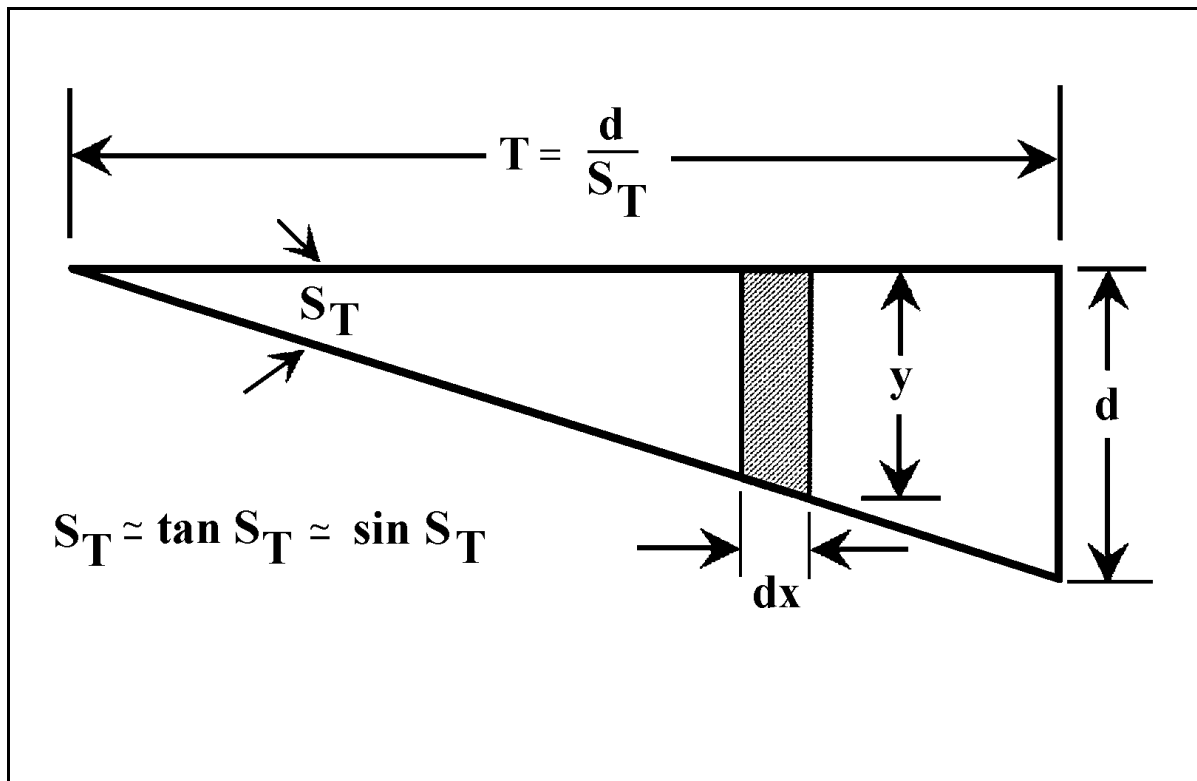
Figure 5.1.1  
 Diagram of Gutter and Pavement Flow Patterns (Wright, 1968)

The incremental flow rate,  $dQ_G$ , in the gutter is:

$$dQ_G = vy \, dx \quad (5.2.1)$$

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,  $y$ ,

$$v = \frac{1.49}{n} y^{2/3} S_L^{1/2} \quad (5.2.2)$$

where  $n$  is Manning's roughness coefficient (Table 4.2.1).

Substituting this relationship into Equation 5.2.1, the following equation is obtained.

$$dQ_G = \left[ \frac{1.49}{n} y^{5/3} S_L^{1/2} \right] dx \quad (5.2.3)$$

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:

$$Q_G = \frac{1.49}{n} \frac{S_L^{1/2}}{S_T} \frac{3}{8} d^{8/3} = \frac{0.56}{n} \frac{S_L^{1/2}}{S_T} d^{8/3} \quad (5.2.4)$$

The depth of flow against the gutter is given by Equation 5.2.5, while the width of the flow,  $T$ , is determined from:

$$T = \frac{d}{S_T}$$

$$d = \left[ 1.79 Q_G n \frac{S_T}{S_L^{1/2}} \right]^{3/8} \quad (5.2.5)$$

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.



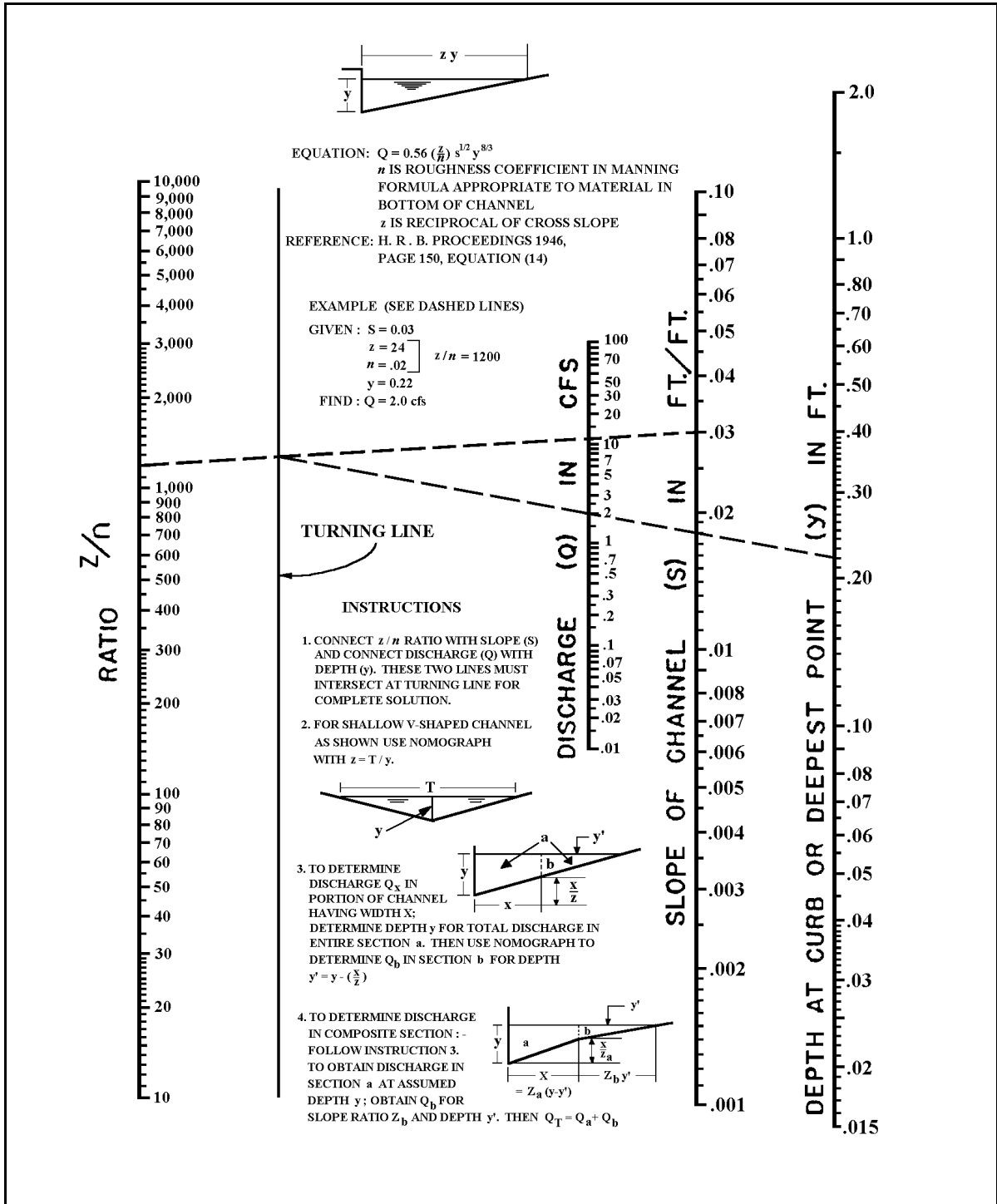


Figure 5.2.2  
 Nomograph Solution of Equations 5.2.4 and 5.2.5  
 (Bureau of Public Roads, 1968)

$$d' = d - x S_T \tag{5.2.6}$$

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:

$$Q_x = Q_G - Q_B \quad (5.2.7)$$

The information presented here is required to design inlets properly. Applications of these relationships are illustrated in Example 5.2.1.

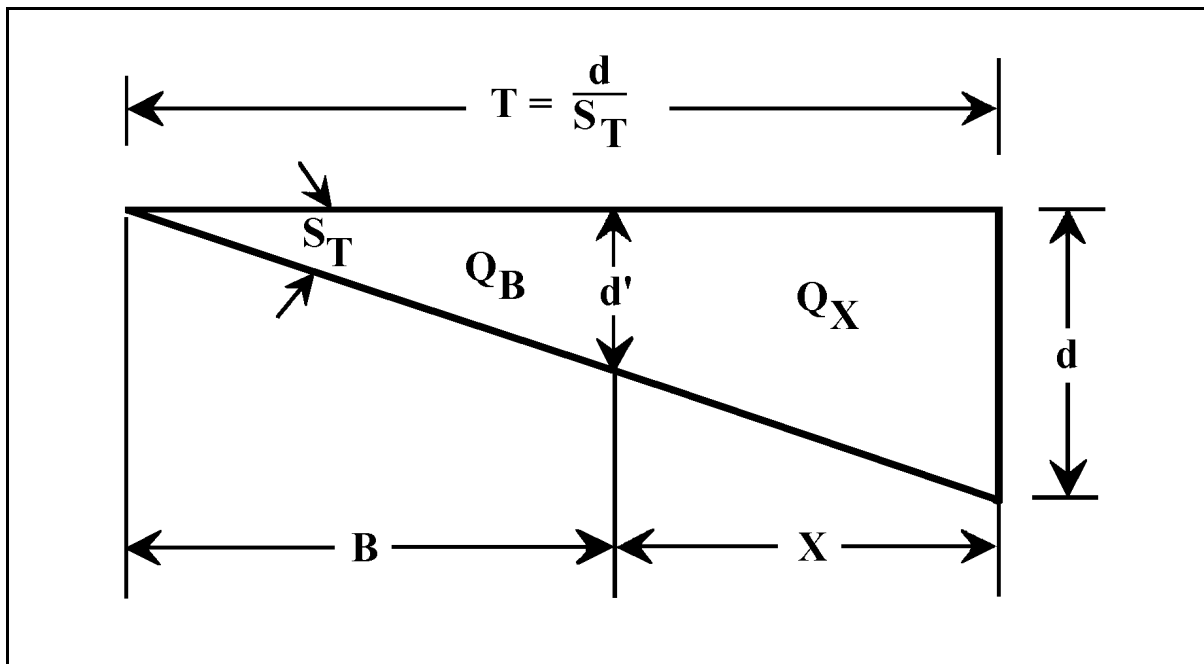


Figure 5.2.3  
Gutter Cross-section

---

**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

$$d = \left[ 1.79 Q_G n \frac{S_T}{S_L^{1/2}} \right]^{3/8} = \left[ (1.79)(3.0)(0.015) \frac{(0.02)}{(0.03)^{1/2}} \right]^{3/8} = 0.17 \text{ ft.}$$

Therefore the top width is:

$$T = \frac{d}{S_T} = \frac{0.17}{0.02 \text{ ft/ft}} = 8.44 \text{ ft.}$$

The depth of flow two feet from the gutter is calculated using Equation 5.2.6.

$$d' = d - xS_T = 0.17 - 2(0.02) = 0.13 \text{ ft.}$$

The flowrate in the street from 2 to 8.50 feet ( $Q_B$ ) is computed using Equation 5.2.4.

$$Q_b = \frac{0.56 S_L^{1/2}}{n S_T} d'^{8/3} = \frac{0.56 (0.03)^{1/2}}{0.015 \cdot 0.02} (0.13)^{8/3} = 1.40 \text{ cfs}$$

The flowrate in the street between the gutter and two feet from the gutter ( $Q_x$ ) is then computed as:

$$Q_x = Q_G - Q_B = 3 - 1.40 = 1.60 \text{ cfs}$$

## 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 all 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  $Q_G$ .

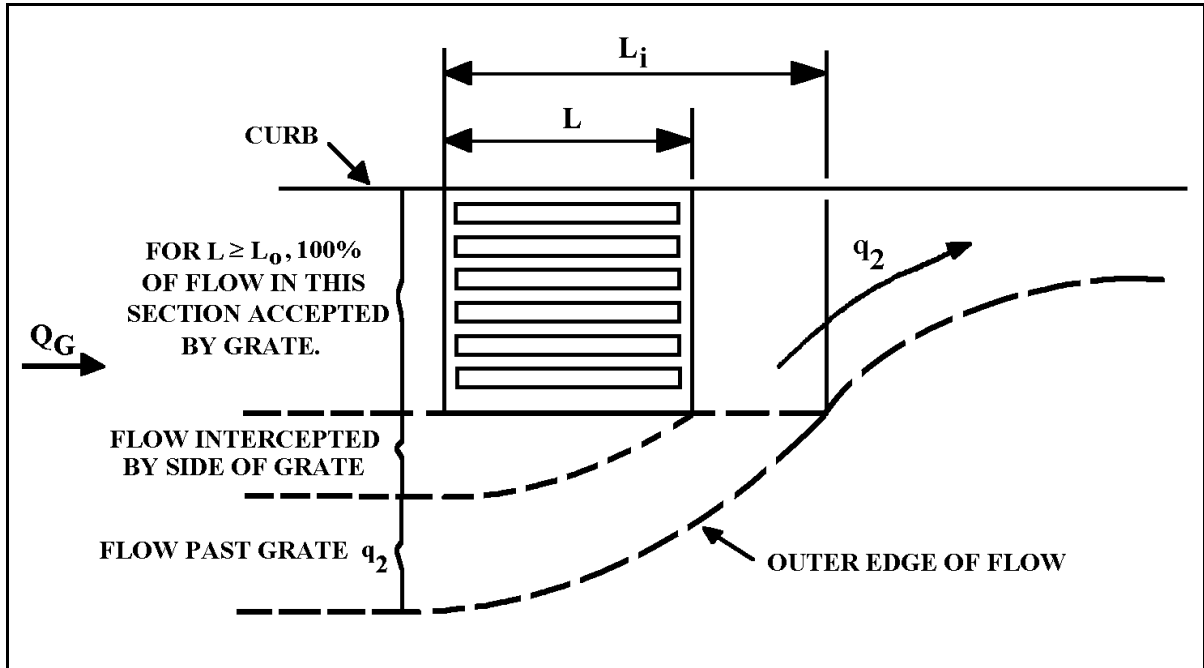


Figure 5.3.1  
Plan of Grated Inlet showing Flow Lines (Wright, 1968)

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

$$Q_i = K d^{5/3} \quad (5.3.1)$$

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,

$$Q_i = 3.0 P (d_w)^{1.5} \quad (d_w < 0.3 \text{ ft}) \quad (5.3.2)$$

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.

$$Q_i = 4.89 A_i (d_o)^{0.5} \quad (d_o > 0.4 \text{ ft}) \quad (5.3.3)$$

where  $A_i$  is the clear opening of grate ( $\text{ft}^2$ ) (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>

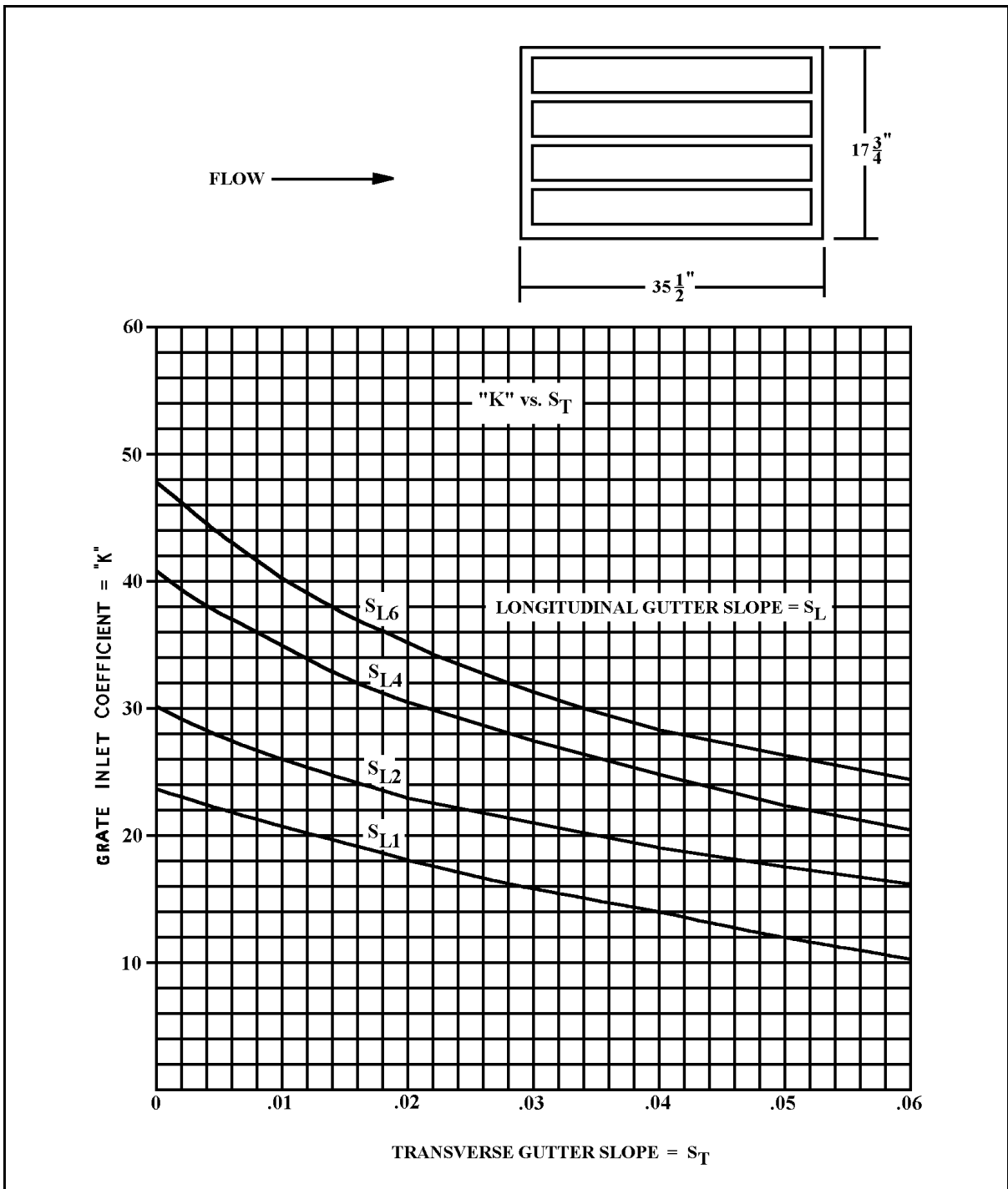


Figure 5.3.2  
 Typical Inlet Grate Capacity Chart (Neenah, 1987)  
 Note:  $S_{L6} = 6\%$ , etc.

---

**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.

$$d = \left[ 1.79(3.0)(0.014) \frac{0.02}{(0.04)^{1/2}} \right]^{3/8} = 0.16 \text{ ft.}$$

Therefore, by using Equation 5.3.1 the grate capacity is found as:

$$Q_i = K d^{5/3} = 30.3(0.16)^{5/3} = 1.43 \text{ cfs}$$

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.

$$Q_i = 3.0 P (d_w)^{1.5} \quad \therefore \quad d_w = \left[ \frac{Q_i}{3.0 P} \right]^{0.67}$$

$$P = 2.0 + 2.5 + 2.5 = 7.0 \text{ ft.}$$

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  $A = (2.5)(2.0)(0.5) = 2.50 \text{ ft}^2$  or using the factor of safety,  $A = 1.25 \text{ ft}^2$ .  
Therefore,

$$d_o = \left[ \frac{10.0}{(4.89)(1.25)} \right]^2 = 2.68 \text{ ft.} \geq 0.4 \text{ ft. (orifice flow)}$$

$$d_w = \left[ \frac{10.0}{(3.0)(3.5)} \right]^{0.67} = 0.96 \text{ ft. (weir flow)}$$

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.

### 5.3.2 - Curb Inlets

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,

$$L_c = 0.6 Q_G^{0.42} S_L^{0.3} \left( \frac{I}{n S_T} \right)^{0.6} \quad (5.3.4)$$

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,

$$E = 1 - \left( 1 - \frac{L}{L_c} \right)^{1.8} \quad (5.3.5)$$

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,

$$S_e = S_T + S_w E_o \quad (5.3.6)$$

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.

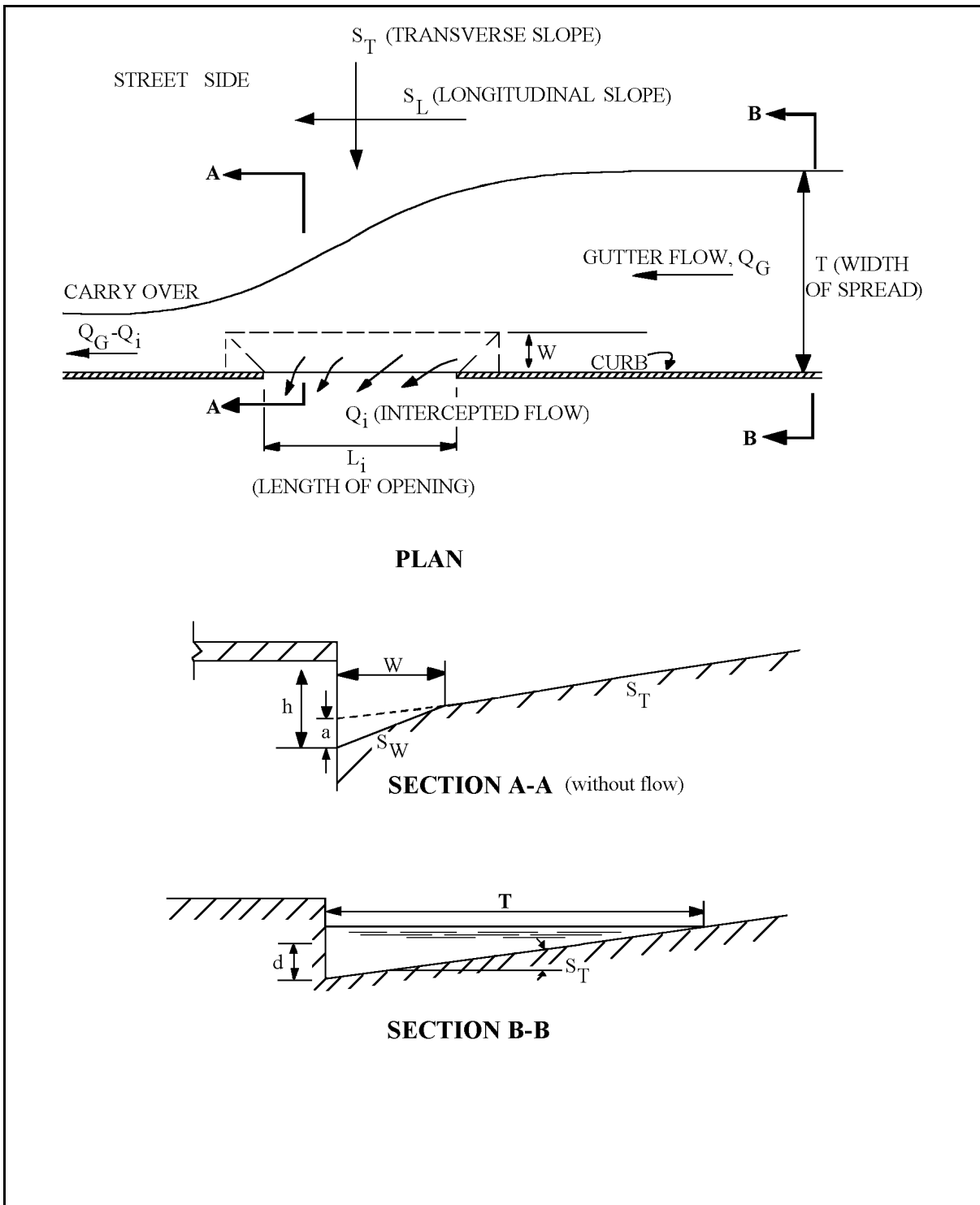


Figure 5.3.3  
Plan View and Notations Used for Curb Inlets (Bauer and Woo, 1964)

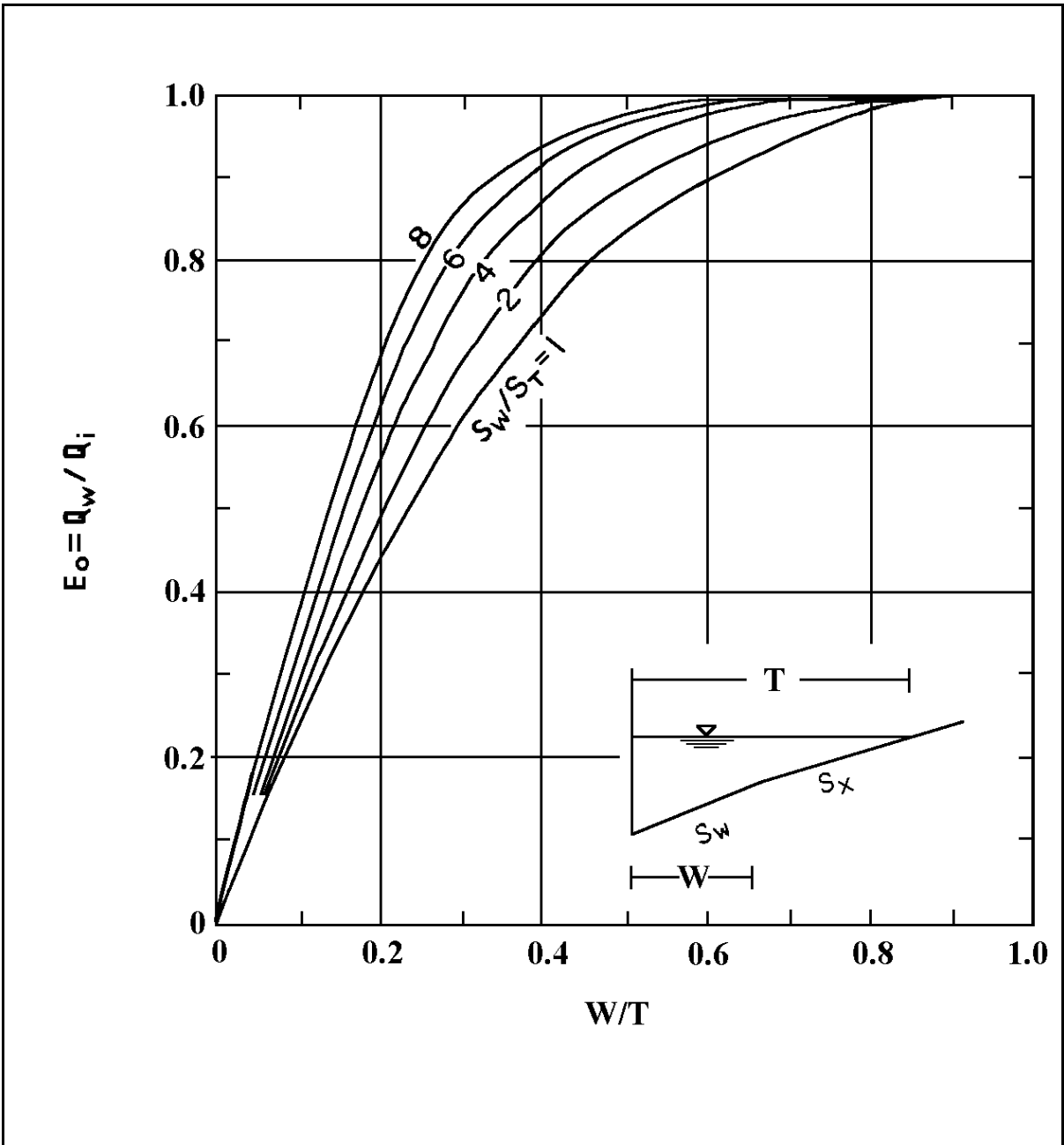


Figure 5.3.4  
Ratio of Frontal Flow to Total Gutter Flow (U.S. DOT, 1969)

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,

$$Q_i = C_w (L + 1.8W) d^{1.5} \quad \left( d \leq h + \frac{a}{12} \right) \quad (5.3.7)$$

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.

$$Q_i = C_w L d^{1.5} \quad (d \leq h) \quad (5.3.8)$$

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,

$$Q_i = 0.67 A_g \left[ 2g \left( d - \frac{h}{2} \right) \right]^{1/2} \quad (d \geq 1.4 h) \quad (5.3.9)$$

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:

$$d = [1.79(2.0)(0.016) \frac{(0.02)}{\sqrt{0.03}}]^{3/8} = 0.15 \text{ ft.}$$

The gutter flow spread is:

$$T = \frac{d}{S_T} = \frac{0.15}{0.02} = 7.6 \text{ ft.}$$

The slope of the depressed area is:

$$S_w = \frac{a}{12W} = \frac{2}{24} = 0.083$$

$$\text{The ratio of } \frac{S_w}{S_T} = \frac{0.083}{0.02} = 4.17$$

$$\text{The ratio of } \frac{W}{T} = \frac{2}{7.6} = 0.26$$

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:

$$L_C = 0.6 Q^{0.42} S_L^{0.3} \left[ \frac{I}{n S_e} \right]^{0.6} = (0.6)(2^{0.42})(0.03^{0.3}) \left[ \frac{I}{(0.016)(0.081)} \right]^{0.6}$$

$$L_C = 15.14 \text{ ft.}$$

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:

<i>Length (ft)</i>	<i>E<sub>o</sub></i>	<i>Q<sub>i</sub> (cfs)</i>
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.

$$\begin{array}{ll} S_T = 0.05 & L = 5 \text{ feet} \\ h = 5'' = 0.42' & T = 8 \text{ feet} \end{array}$$

Calculate: the intercepted flow for an *undeepressed* curb opening.

$$d = T S_T = (8)(0.05) = 0.4 \text{ ft.} = 4.8 \text{ inches}$$

Since  $d < h$ , the flow is weir type and  $Q_i$  is computed from Equation 5.3.8.

$$Q_i = C_w L d^{1.5} = (2.3)(5.0)(0.4)^{1.5} = 2.91 \text{ cfs}$$

For a *depressed* curb opening with  $a = 2''$ ,  $W = 2$  feet (all other information remaining the same).

$$h + \frac{a}{12} = 0.42 + \frac{2}{12} = 0.59 \text{ ft.} > (1.4)d \quad \therefore \text{use Equation 5.3.7}$$

$$Q_i = 2.3(L + 1.8W)d^{1.5} = 2.3(5.0 + 1.8(2))(0.4)^{1.5} = 5.0 \text{ cfs}$$

The depressed curb opening inlet is about 72% more efficient than the undeepressed 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,

$$Q_i = 0.8 L W_s (2gd)^{0.5} \quad (d \geq 0.4 \text{ ft}) \quad (5.3.10)$$

where  $g$  is the acceleration due to gravity and  $W_s$  is the width of slot (feet); for

$$W_s = 1.75", Q_i = 0.94 Ld^{0.5}.$$

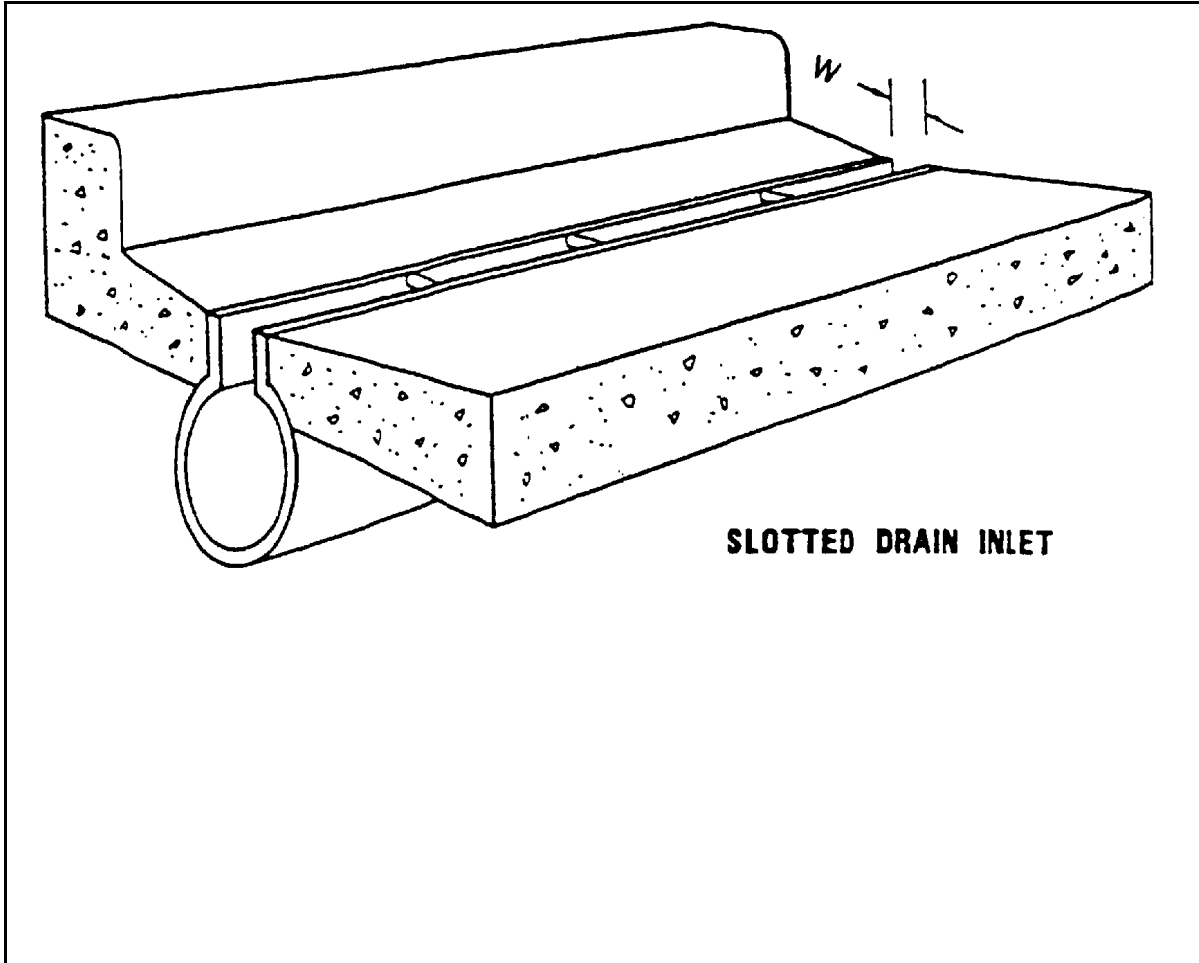


Figure 5.3.5  
Typical Slotted Drain Inlet (U.S. DOT, 1984)

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.

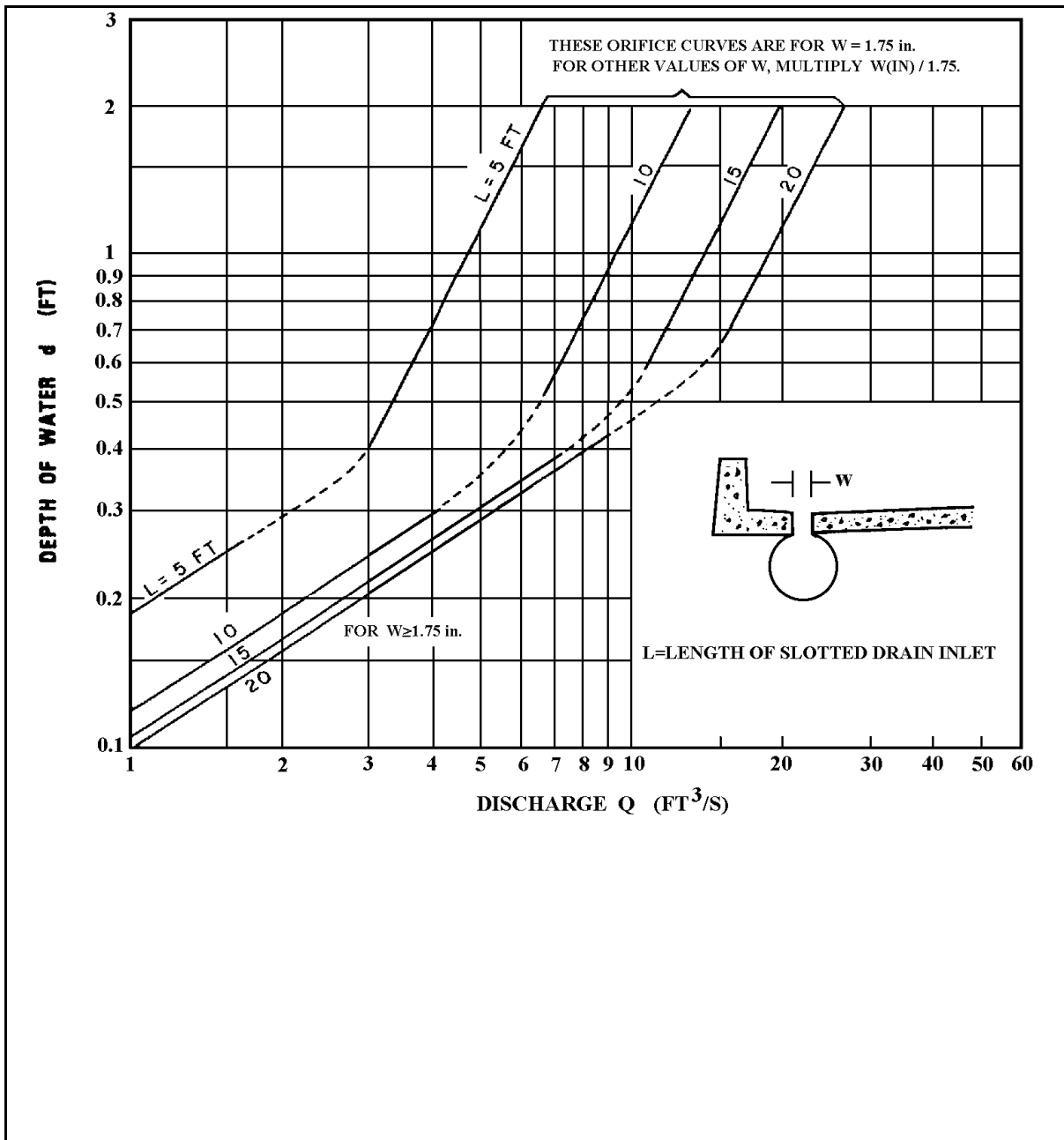


Figure 5.3.6  
Slotted Drain Inlet Capacity in Sump Locations, (U.S. DOT, 1984)

---

**Example 5.3.5** (Slotted Drain Inlet - Sump Condition)

Given: a slotted inlet with the following information:

$$\begin{array}{ll} W_s = 1.75 \text{ in.} & S_T = 0.05 \\ Q = 5 \text{ cfs} & S_L = 0.02 \end{array}$$

Calculate: the length of slotted drain inlet to limit depth at curb to 0.3 feet and 0.6 feet.

- 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.
- For  $d = 0.60$  feet, we have orifice flow and we can either use Figure 5.3.6 or Equation 5.3.10.

$$Q_i = 0.94 LD^{0.5}, \text{ for } W_s = 1.75 \quad \therefore$$

$$L = \frac{Q_i}{(0.94)(d)^{0.5}} = \frac{5 \text{ (cfs)}}{(0.94)(0.6)^{0.5}} = 6.9 \cong 7 \text{ ft.}$$

## 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,

$$Q_G = CiA + Q_c \quad (5.4.1)$$

where  $Q_G$  is the flowrate in gutter at the inlet location (cfs),  $Q_c$  is the carryover from upgrade  
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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,

$$Q_i = Q_G - Q_b \quad (5.4.2)$$

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,

$$L = \frac{43,560 (Q_i + Q_b - Q_c)}{Ciw} \quad (5.4.3)$$

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.



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<sub>10</sub>). The hydraulic gradeline shall not exceed the top of the inlet or manhole for the 50-year peak discharge (Q<sub>50</sub>). (INDOT, 1993)

Table 5.4.1  
Allowable Water Spread (INDOT, 1993)

TYPE OF FACILITY	ALLOWABLE SPREAD "T"
Freeways	Edge of travel lane for Q <sub>50</sub>
Multilane Non-Freeways	Spread acceptable across one entire travel lane for Q <sub>10</sub>
All Two-Lane Freeways	4 feet onto travel lane for Q <sub>10</sub>
Bridge Decks	4 feet onto travel lane for Q <sub>10</sub>
Ramps	8 feet of roadway must remain clear of water during Q <sub>10</sub>

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<sub>L</sub> 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.

$$\overline{S}_L = \frac{LG}{2D_H} \quad (5.4.4)$$

where G is the tangent grade and D<sub>H</sub> is the distance from high point to tangent point.

If Equation 5.4.4 is substituted into Equation 5.2.4 (to calculate Q<sub>G</sub>) and the result is substituted into Equation 5.4.3 (with Q<sub>C</sub> = 0), then Equation 5.4.5 gives the spacing for the first inlet on a crest vertical curve.

$$L = \left[ \frac{24,394}{C_i w n} \frac{d^{8/3}}{S_T} \right]^2 \frac{G}{2D_H} \quad (L < D_H) \quad (5.4.5)$$

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.



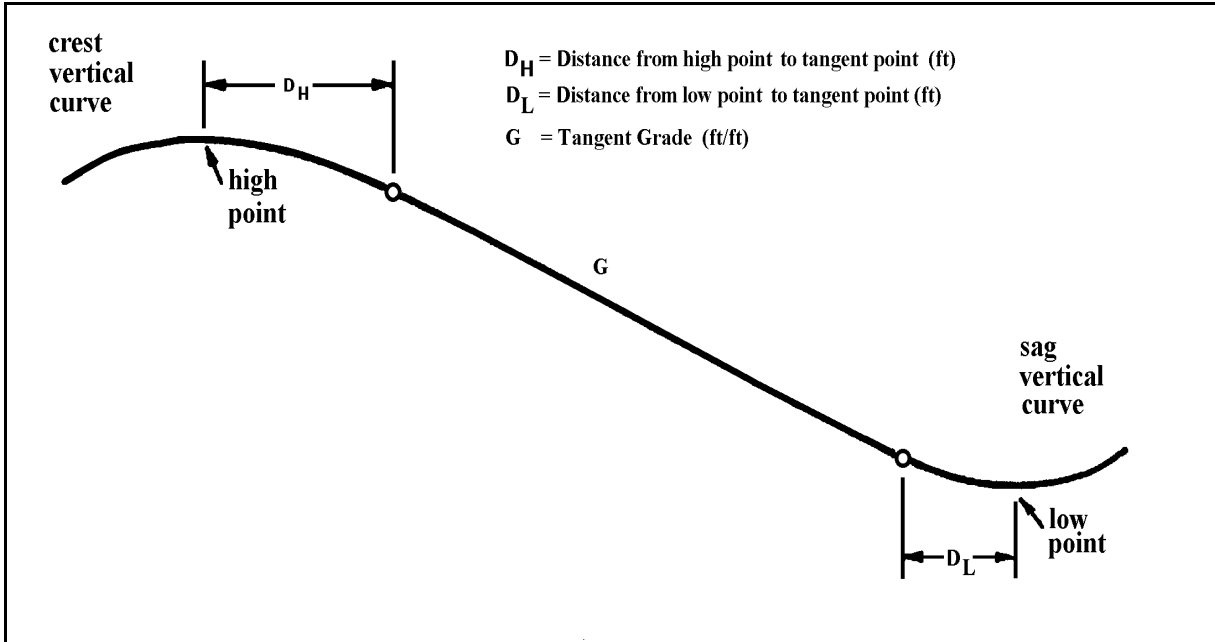


Figure 5.4.2  
Grade Profile for a Hypothetical Roadway

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.

$$\overline{S}_L = \frac{LG}{4D_L} \quad (5.4.6)$$

$$L = \left[ \frac{24,394}{wCin} \frac{d^{8/3}}{S_T} \right]^2 \frac{G}{4D_L} \quad (L < D_L) \quad (5.4.7)$$

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:

$$d = S_T T = (0.03)(4) = 0.12 \text{ feet}$$

From Table 4.2,  $n = 0.013$

The maximum gutter flowrate is determined using Equation 5.2.4.

$$Q_G = \frac{0.56 (0.01)^{1/2}}{(0.013)(0.03)} (0.12)^{8/3} = 0.50 \text{ cfs}$$

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:

$$Q_i = K(d)^{5/3} = 16(0.12)^{5/3} = 0.47 \text{ cfs}$$

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.

$$i = \frac{c T_r^\alpha}{(t + d)^\beta} \therefore i_5^{10} = \frac{2.1048 (10)^{0.1733}}{\left(\frac{5}{60} + 0.47\right)^{1.1289}} = 6.12 \text{ inches/hr}$$

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.

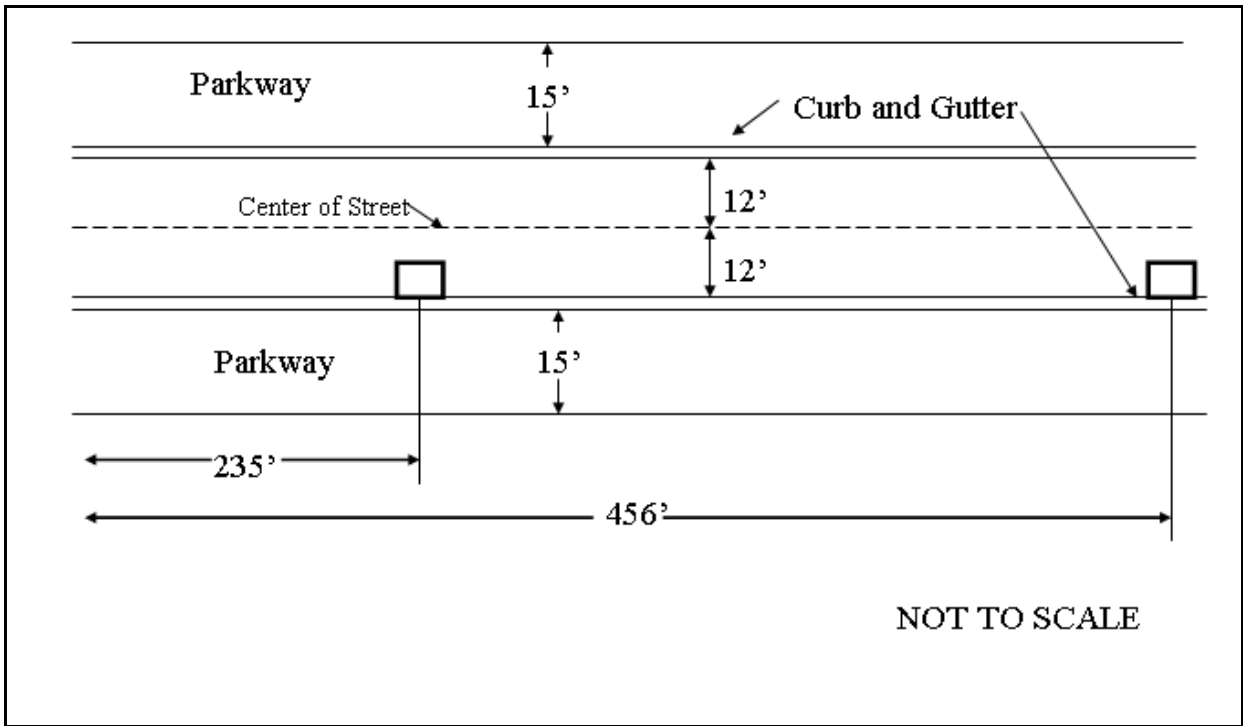


Figure 5.4.3  
 Depiction of Street Described in Example 5.4.1

COMPUTATION SHEET FOR INLET SPACING

Inlet Location			Roadway and Gutter Data							Inlet Data		Contributing Area Data			Inlet Spacing				
Inlet Number	Station	Road or Street	Inlet Type and Condition	Trans. Slope S <sub>T</sub> (%)	Longit. Slope S <sub>L</sub> (%)	Manning's Roughness Coefficient <i>n</i>	Depth of Flow <i>d</i> (ft)	Top Width <i>T</i> (ft)	Flow Rate <i>Q</i> (cfs)	Depression		Inlet Capac. <i>Q<sub>i</sub></i> (cfs)	Width <i>w</i> (ft)	Composite Runoff Coefficient <i>C</i>	Rainfall Intensity (in/hr)	Upgrade Carryover <i>Q<sub>C</sub></i> = <i>Q<sub>G</sub></i> - <i>Q<sub>i</sub></i> (cfs)	Inlet Bypass <i>Q<sub>b</sub></i> (cfs)	<i>Q<sub>i</sub></i> + <i>Q<sub>b</sub></i> - <i>Q<sub>C</sub></i> (cfs)	Inlet Spacing <i>L</i> (ft)
										Width <i>W</i> (ft)	Depth <i>a</i> (in)								
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 <sup>2</sup> or m <sup>2</sup> )
A <sub>D</sub>	Contributing developed area
A <sub>o</sub>	Area of the orifice (ft <sup>2</sup> or m <sup>2</sup> )
A <sub>T</sub>	Total contributing area (acres or hectares)
A <sub>U</sub>	Contributing undeveloped area
C <sub>d</sub>	Coefficient of discharge
C <sub>D</sub>	Developed runoff coefficient
C <sub>U</sub>	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 <sup>2</sup> or 9.81 m/sec <sup>2</sup> )
h <sub>L</sub>	Head loss or the gradient over a flow distance L (ft or m)
h <sub>o</sub>	Height of water above the orifice (ft or m)
h <sub>p</sub>	Height of water above the pipe opening (ft or m)
h <sub>w</sub>	Height of water above the weir opening (ft or m)
i <sub>d</sub>	Rainfall intensity at time t <sub>d</sub> (in/hr or cm/hr)
i <sub>U</sub>	Undeveloped rainfall intensity (in/hr or cm/hr)
I(t)	Inflow at time t (cfs or m <sup>3</sup> /sec)
K	Coefficient of permeability
K <sub>e</sub>	Entrance loss coefficient
K <sub>o</sub>	Outlet loss coefficient
L	Length (ft or m)
n	Manning's roughness coefficient
O(t)	Outflow or runoff (cfs or m <sup>3</sup> /sec) or (cfs/acre or m <sup>3</sup> /sec / hectare)
P(t)	Accumulated rainfall (inches)
Q	Infiltration flows (cfs or m <sup>3</sup> /sec)
Q <sub>U</sub>	Peak undeveloped flow (cfs or m <sup>3</sup> /sec)
q <sub>p</sub>	Peak flow (cfs or m <sup>3</sup> /sec)
Q <sub>I</sub> (t)	Magnitude of the inflow hydrograph at time t
Q <sub>O</sub> (t)	Magnitude of the outflow hydrograph at time t
R(t)	Direct surface runoff (inches or cm)
S	Ultimate abstraction
S(t <sub>d</sub> )	Required storage (acre-ft)
ΔS	Change in storage (ft <sup>3</sup> or m <sup>3</sup> )
S <sub>max</sub>	Required detention storage
t'	Time at which inflow is equal to outflow
t <sub>b</sub>	Time of base
t <sub>c</sub>	Time of concentration
t <sub>d</sub>	Storm duration

## LIST OF PARAMETERS (cont'd)

$t_p$	Time to peak
$t_t$	Time of travel (hrs)
$\Delta t$	Time interval (sec)
$v$	Velocity (ft/sec or m/sec)
$V$	Volume (ft <sup>3</sup> or m <sup>3</sup> )
$V_I(t_d)$	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:

$$I(t) - O(t) = \frac{\Delta S}{\Delta t} \quad (6.2.1)$$

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  $\Delta S$  is the change in storage ( $\text{ft}^3$ ) in time interval  $(\Delta t)(\text{sec})$ .

Equation 6.2.1 may also be written as:

$$(I_1 + I_2) \frac{\Delta t}{2} - (O_1 + O_2) \frac{\Delta t}{2} = S_2 - S_1 \quad (6.2.2)$$

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  $S_{max}$ .

$$S(t_d) = (C_D i_d A_D - C_U i_U A_U) \frac{t_d}{12} \quad (6.2.3)$$

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.

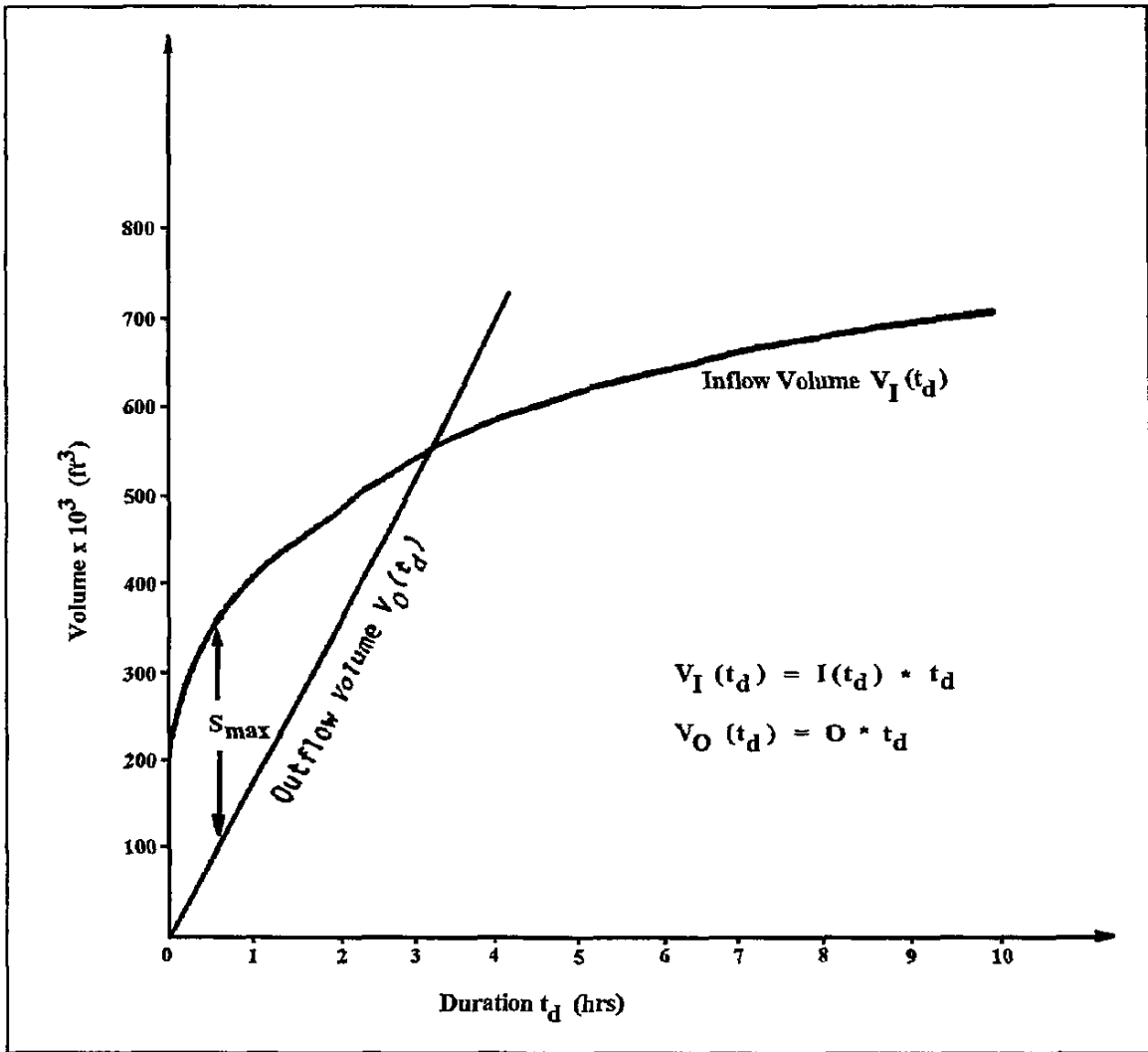


Figure 6.2.1  
Graphical Representation of Storage Volumes as Determined by the Rational Method



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,

$$S(t_d) = \frac{t_d}{12} [I(t_d) - O(t)] \quad (\text{acre-ft})$$

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 Coefficient ( $C_U$ ):	
Undeveloped Runoff Rate (O):	cfs
Developed Runoff Coefficient ( $C_D$ )	

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 $(I(t_d) - O)(t_d/12)$ (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

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### 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.

$$t_c = 0.83 \left[ \frac{(150)(0.40)}{\sqrt{0.01}} \right]^{0.467} ; \quad t_c = 16.46 \text{ minutes}$$

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.

$$c = 1.7204 \qquad \alpha = 0.1753 \qquad d = 0.485 \qquad \beta = 1.6806$$

$$i = \frac{c T_r^\alpha}{(t + d)^\beta} = \frac{1.7204 (3)^{0.1753}}{\left( \frac{16.46}{60} + 0.485 \right)^{1.6806}} = 3.313 \text{ inches/hour}$$

$$Q_U = (0.15)(3.31)(10) = 4.97 \text{ cfs}$$

Step 3: Compute the developed runoff coefficient. From Table 3.1, a composite runoff coefficient can be determined.

	Acres	Type of Cover	Runoff Coefficient	$C_n A_n$
	2	Parks	0.17	0.34
	5	Single Family Homes	0.40	2.00
	3	Business	0.60	1.80
<b>TOTAL</b>	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  $t \leq 1$  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:

$$i = \frac{c T_r^\alpha}{(t+d)^\beta} = \frac{1.2799(100)^{0.1872}}{(t+0.258)^{0.8252}}$$

<b>t (hours)</b>	<b>i (inches/hr)</b>	<b>t (hours)</b>	<b>i (inches/hr)</b>
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 Coefficient ( $C_U$ ):	0.15
Undeveloped Runoff Rate ( $O=C_U i_U A_U$ ):	4.97 cfs
Developed Runoff Coefficient ( $C_D$ )	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 $(I(t_d)-O)(t_d/12)$ (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
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	*	*

← **Peak Storage**

\* Since  $I(tD) - O < \text{zero}$ , 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,

$$S(t_d) = \frac{\Delta t}{12} \sum_{t=\Delta t}^{t'} [Q_i(t) - Q_o(t)] \quad (6.2.5)$$

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  $t$ ,  $\Delta t$  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).

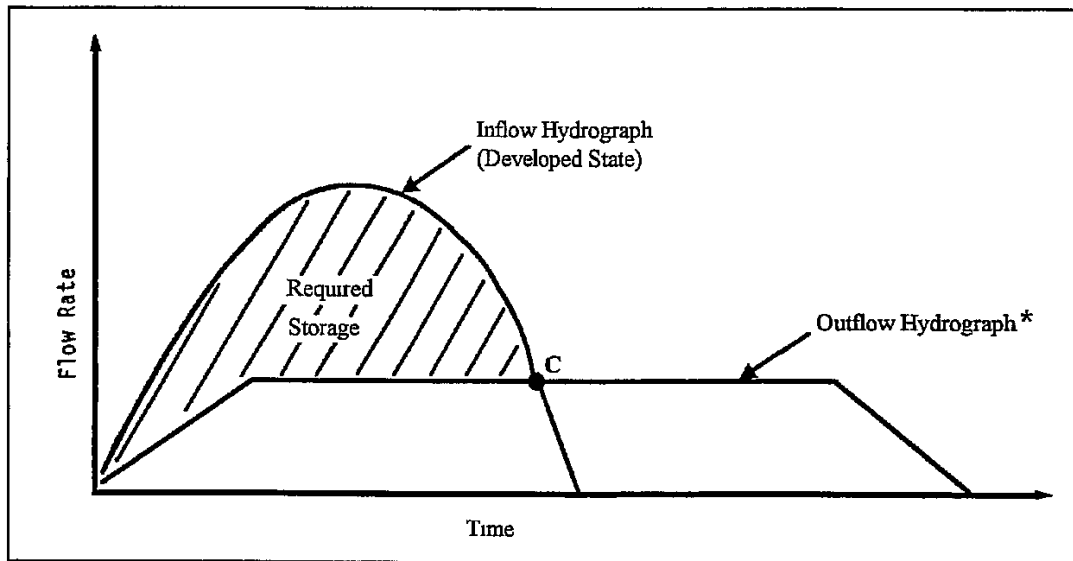


Figure 6.2.4  
Inflow and Outflow Hydrographs for a Hypothetical Detention Pond

\* 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  $\Delta D$ ,  $q_p$ , 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  $\Delta D$  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  $\Delta D$  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

75 acres Meadow (good condition)  
 45 acres Woods (good cover)  
 30 acres Pasture (good condition)

Travel Path of 5625 ft. over unpaved land with a 0.6% slope

#### Developed

50 acres Residential (1/2 acre lots)  
 10 acres Gravel road  
 40 acres Parking lots  
 50 acres Lawns (fair condition)  
 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.



<i>UNDEVELOPED</i>			
	CN	AREA	CN <sub>n</sub> x A <sub>n</sub>
Meadow (good condition)	71	75	5325
Woods (good cover)	70	45	3150
Pasture (good condition)	74	30	2220
Total		150	10,695

<i>DEVELOPED</i>			
	CN	AREA	CN <sub>n</sub> x A <sub>n</sub>
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:

$$t_c = \frac{L}{3600V} = \frac{5625}{3600(1.25)} = 1.25 \text{ hrs.}$$

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.

$$t_t = \frac{L}{3600V} = \frac{3060}{3600(2.0)} = 0.43 \text{ hrs.}$$

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.

$$V = \frac{Q}{A} = \frac{1.49}{n} R^{2/3} S^{1/2} = \frac{1.49}{0.013} \left[ \frac{2}{4} \right]^{2/3} [0.03]^{1/2} = 12.5 \text{ ft/sec.}$$

$$t_t = \frac{L}{3600V} = \frac{9000}{(3600)(12.5)} = 0.20 \text{ hrs.}$$

$$t_c = 0.20 \text{ hrs} + 0.43 \text{ hrs} = 0.63 \text{ hrs.}$$

**Step 3:** Calculate  $\Delta D$ ,  $t_p$  and  $q_p$  ( $A = 150 \text{ acres} = 0.234 \text{ mi}^2$ )

Undeveloped:  $\Delta D = (0.133)t_c = (0.133)(1.25) = 0.17 \text{ hrs (10 minutes)}$

$$t_p = \frac{\Delta D}{2} + 0.6t_c = \frac{0.17}{2} + 0.6(1.25) = 0.84 \text{ hrs.}$$

$$q_p = \frac{484AQ}{t_p} = \frac{484(0.234)(1)}{0.84} = 134.83 \text{ cfs}$$

Developed:  $\Delta D = (0.133)(0.63) = 0.08 \text{ hrs (5 minutes)}$

$$t_p = \frac{0.08}{2} + 0.6(0.63) = 0.42 \text{ hrs.}$$

$$q_p = \frac{(484)(0.234)(1)}{0.42} = 269.66 \text{ cfs}$$

**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,  $t_b = 2.67 t_p$ .

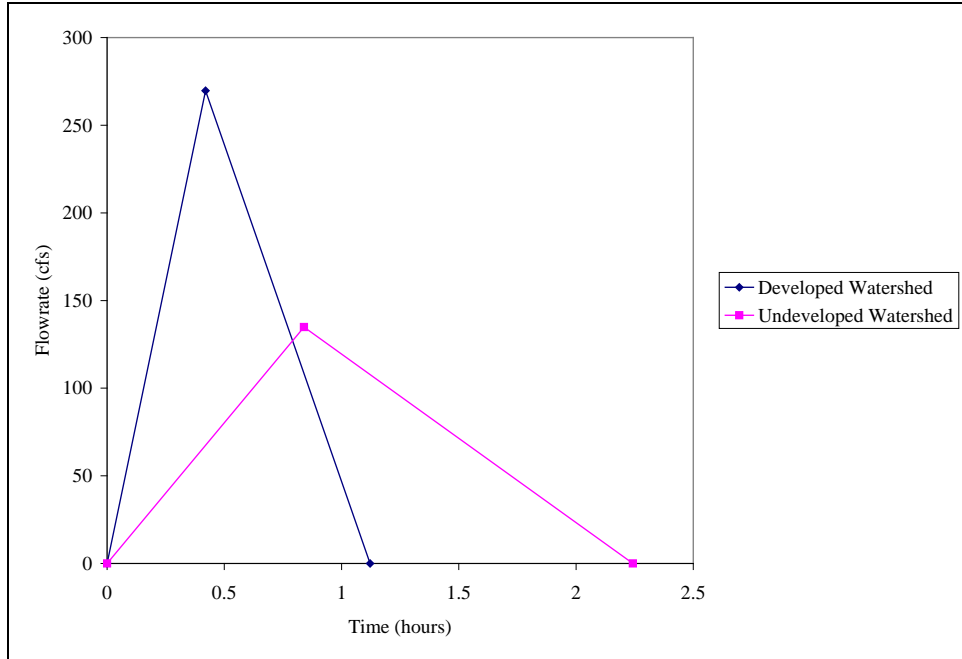


Figure 6.2.5  
Triangular Unit Hydrographs for Example 6.2.2

**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

$$i_1^{10} = \frac{2.1048(10)^{0.1733}}{(1 + 0.470)^{1.1289}} = 2.03 \text{ inches}$$

$$i_1^{100} = \frac{1.5899(100)^{0.2271}}{(1 + 0.725)^{0.8797}} = 2.80 \text{ inches}$$

Cum. % of Storm Time	Time (min.)	Percent of Total Storm Rainfall			Increment of Rainfall (in.)	
		Cumulative	Incremental		Undeveloped Total=2.03"	Developed Total=2.80"
			(10 min)	(5 min)		
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.

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

$$S = \frac{1000}{71.3} - 10 = 4.03 \text{ inches} \quad (\text{undeveloped})$$

$$S = \frac{1000}{85.1} - 10 = 1.75 \text{ inches} \quad (\text{developed})$$

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<sup>3</sup> gives d = 6.5 ft.

Time (min)	Unit Hydrograph (cfs)	Excess Precipitation (in)							Storm Hydrograph (cfs)
		0.00	0.01	0.02	0.05	0.07	0.07	0.07	
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
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

← Peak Flow

Figure 6.2.6  
Computation of Storm Hydrograph for Undeveloped Site for Example 6.2.2



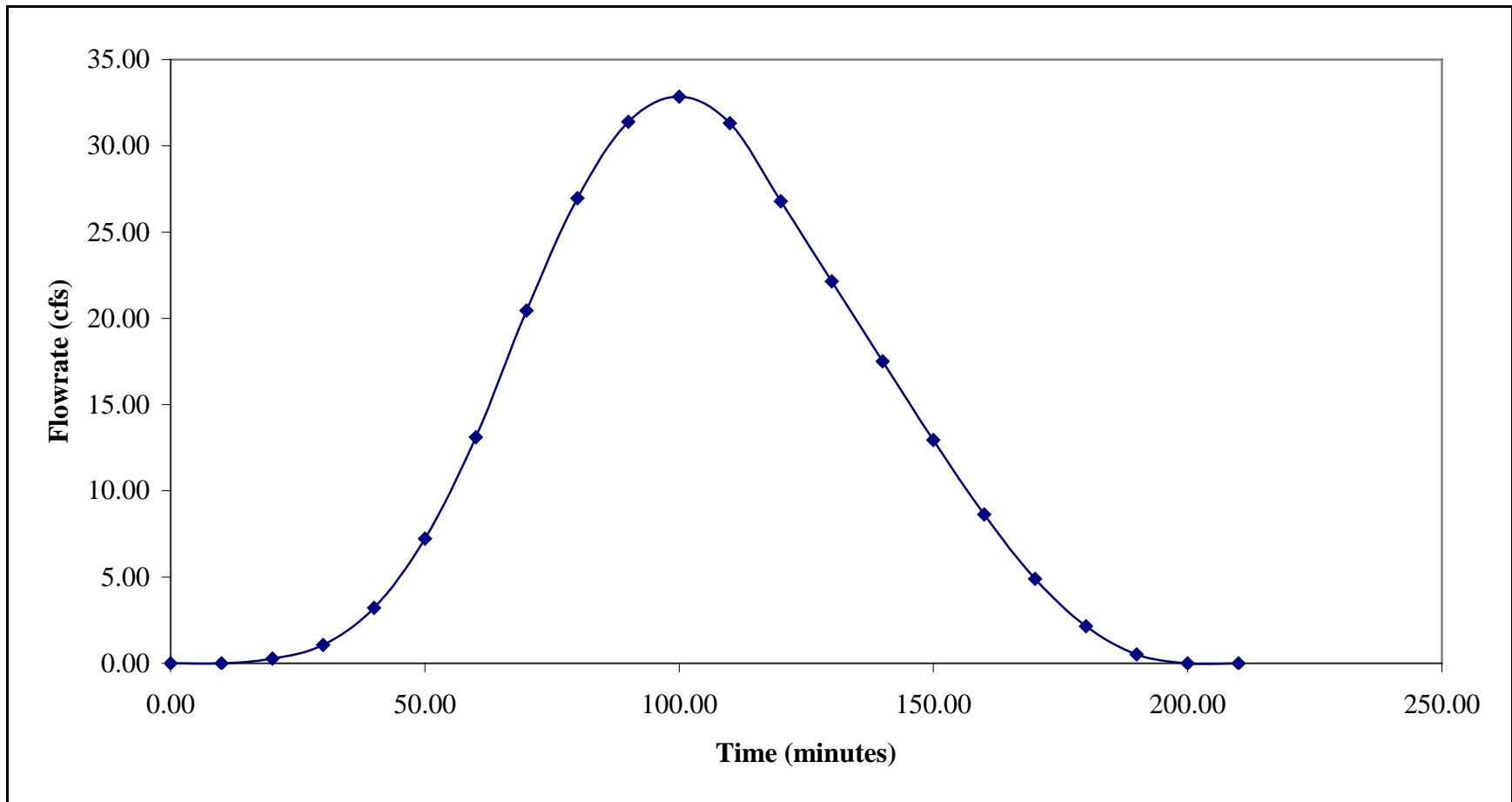


Figure 6.2.7  
Storm Hydrograph for Undeveloped Site for Example 6.2.2

Time (min)	Unit	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	Hydrograph (cfs)	
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

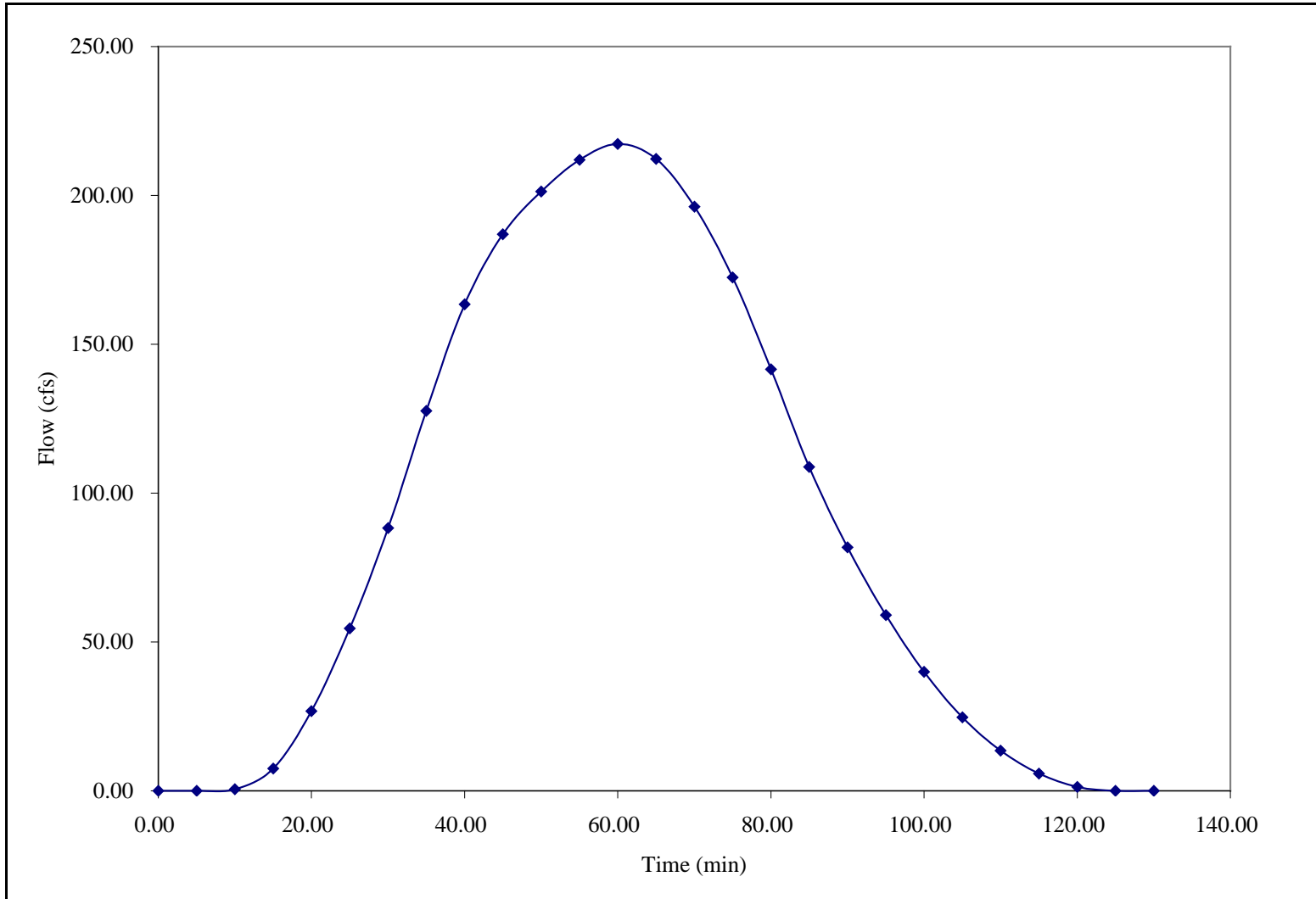


Figure 6.2.9  
Storm Hydrograph for Developed Site for Example 6.2.2

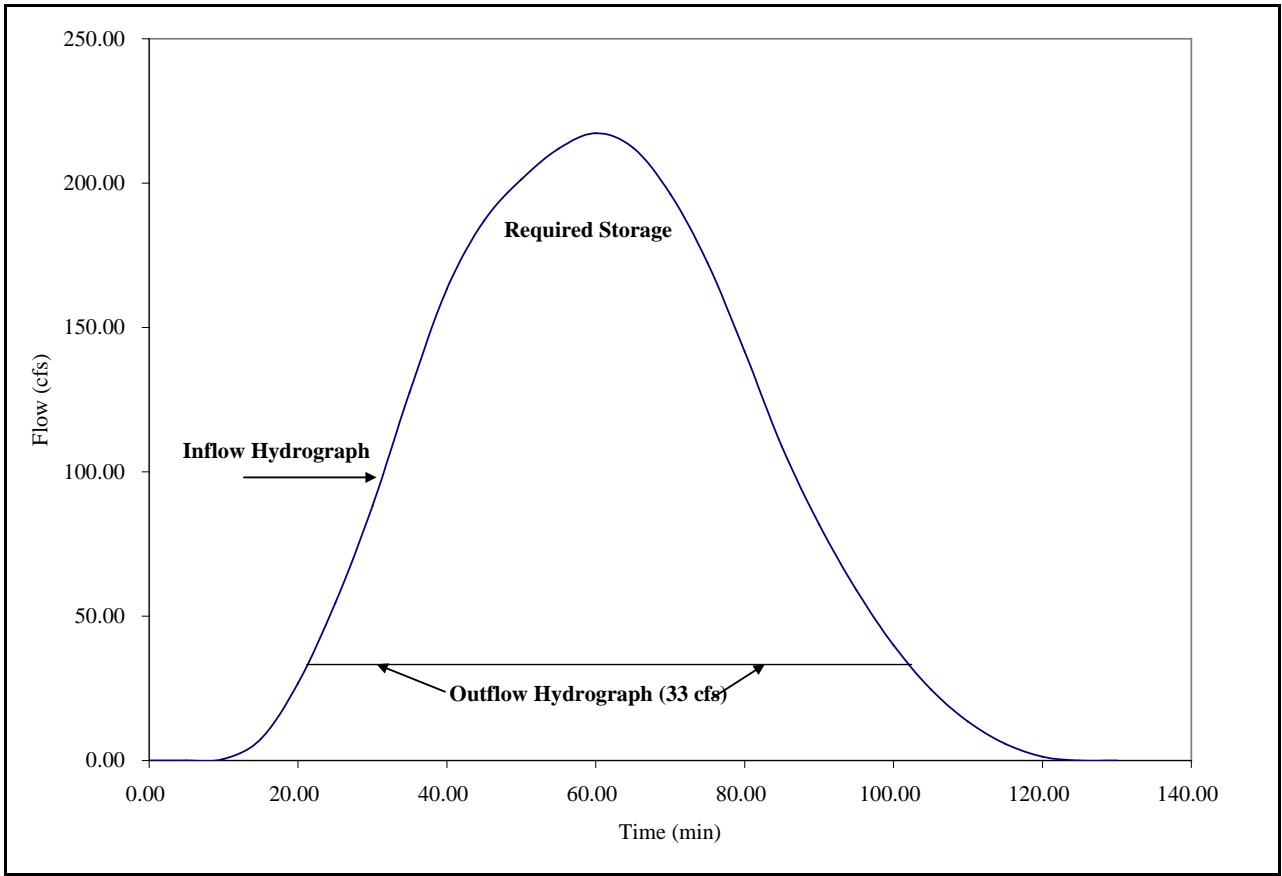


Figure 6.2.10  
Storage Detention Graphic for Example 6.2.2

t (minutes)	Q <sub>I</sub> (cfs)	Q <sub>O</sub> (cfs)	Q <sub>I</sub> -Q <sub>O</sub> (cfs)	s =(Q <sub>I</sub> -Q <sub>O</sub> )* Δt (ft <sup>3</sup> )	
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 <sup>3</sup>
				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 $5.0 \times 10^{-1}$	Very Permeable
Sand, Fine Sand	$5.0 \times 10^{-1} - 5.0 \times 10^{-3}$	Medium Permeability
Silty Sand, Dirty Sand	$5.0 \times 10^{-3} - 5.0 \times 10^{-5}$	Low Permeability
Silt	$5.0 \times 10^{-5} - 5.0 \times 10^{-7}$	Very Low Permeability
Clay	less than $5.0 \times 10^{-7}$	Practically Impervious

$$Q' = A'Kh_L \quad (6.3.1)$$

where  $Q'$  is the flow rate (cfs),  $A'$  is the cross-sectional area of soil through which the water flows ( $\text{ft}^2$ ),  $K$  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.

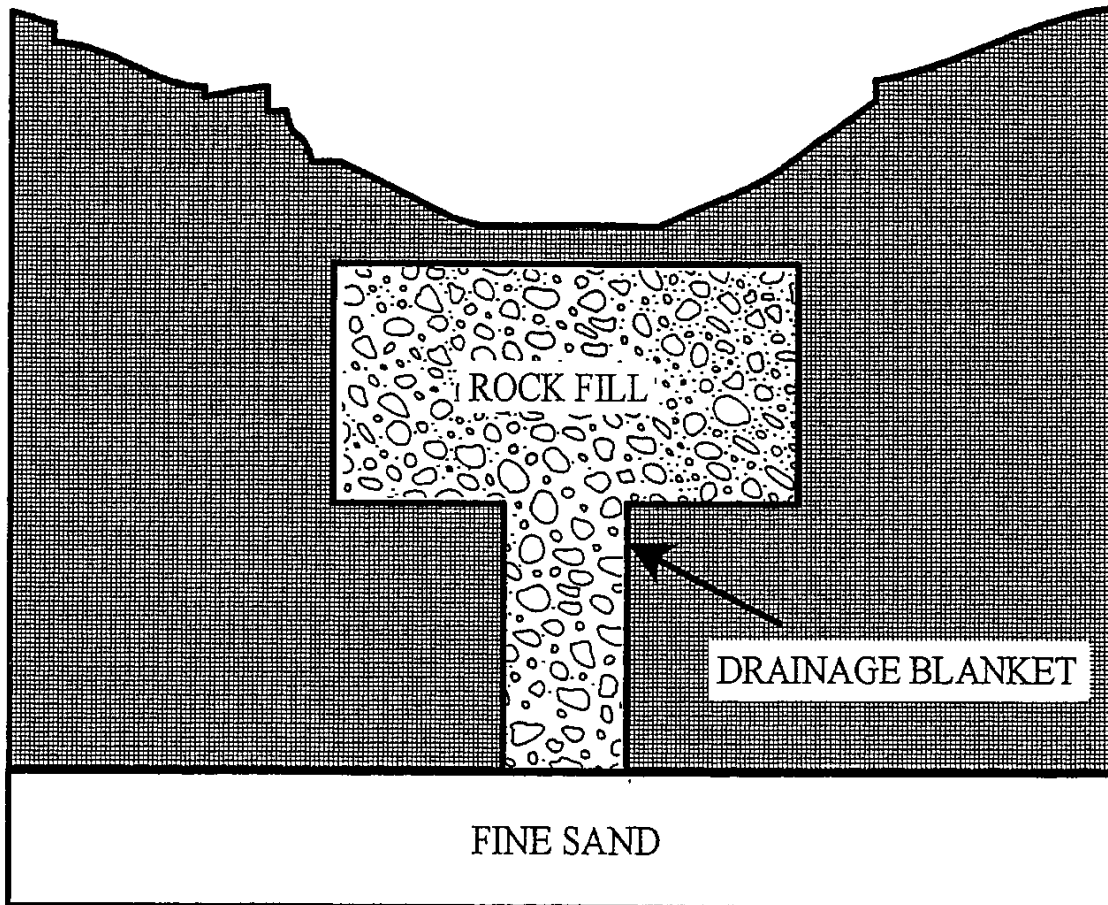


Figure 6.3.1  
Design of On-Site Discharge Ditch (Karaca, 1980)  
(The pervious drainage blanket shown above can be constructed of  
"Bidim" (by Monsanto) or equivalent).

### 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.

$$Q = C_d A_o \sqrt{2gh_o} \quad (6.3.2)$$

where  $C_d$  is the discharge coefficient,  $A_o$  is the area of the orifice ( $\text{ft}^2$ ),  $g$  is the acceleration due to gravity ( $32.2 \text{ ft/sec}^2$ ), 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.

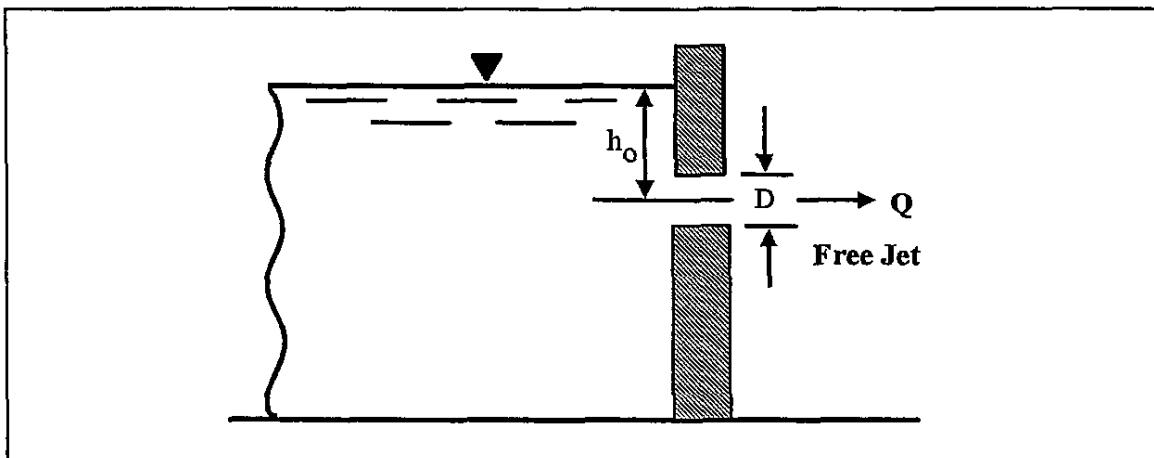
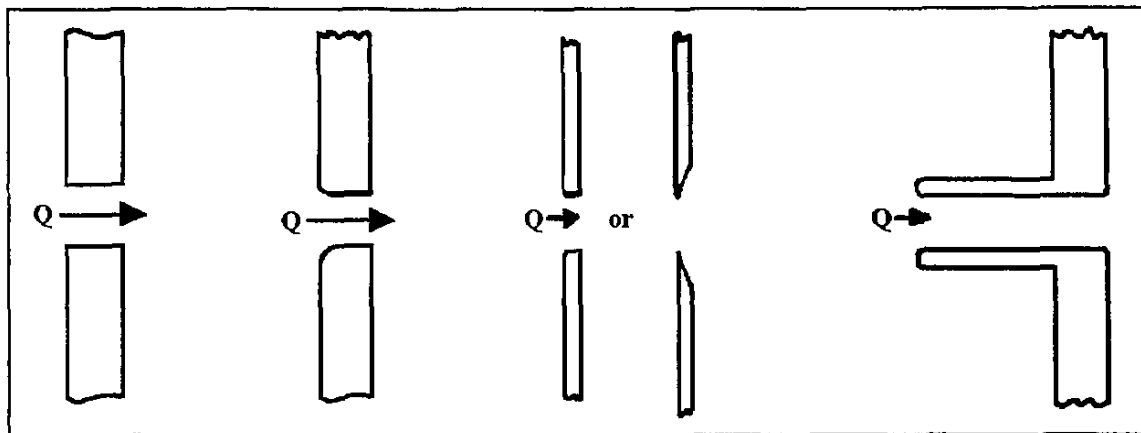


Figure 6.3.2  
Typical Orifice



Square-edged  
 $C_d = 0.79-0.82$

Round-edged  
 $C_d = 0.92-0.98$

Sharp-edged  
 $C_d = 0.58-0.64$

Projecting Sharp-edge  
 $C_d = 0.50$

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 through a rectangular weir is

$$Q = \frac{2}{3} C_d \sqrt{2g} L h_w^{3/2} \quad (6.3.3)$$

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 ( $32.2 \text{ ft/sec}^2$ ).

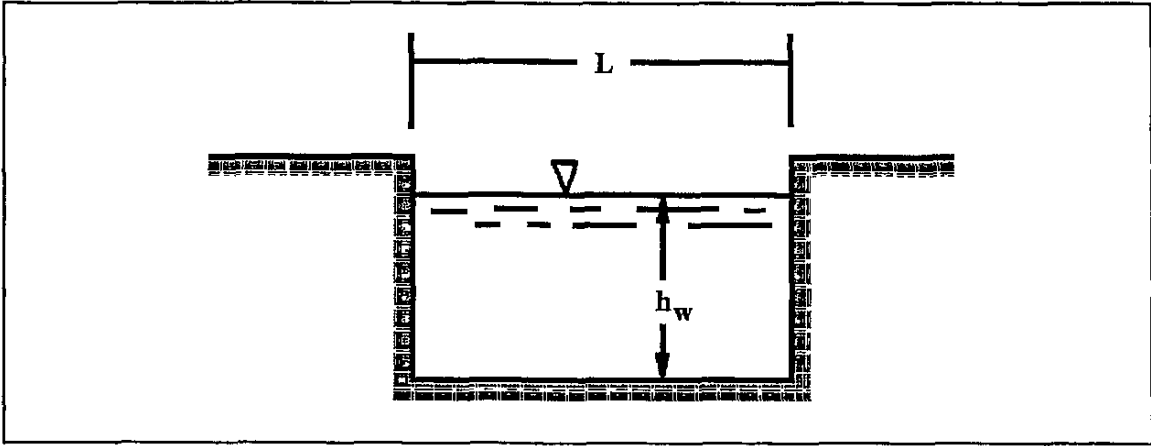


Figure 6.3.4  
Typical Rectangular Weir

For a triangular weir shown in Figure 6.3.5, Equation 6.3.4 is used to calculate the flowrate,

$$Q = \frac{8}{15} C_d \sqrt{2g} \tan \theta h_w^{5/2} \quad (6.3.4)$$

where  $\theta$  is  $\frac{1}{2}$  the total weir angle and  $C_d$ ,  $h_w$ , and  $g$  are the same as in Equation 6.3.3.

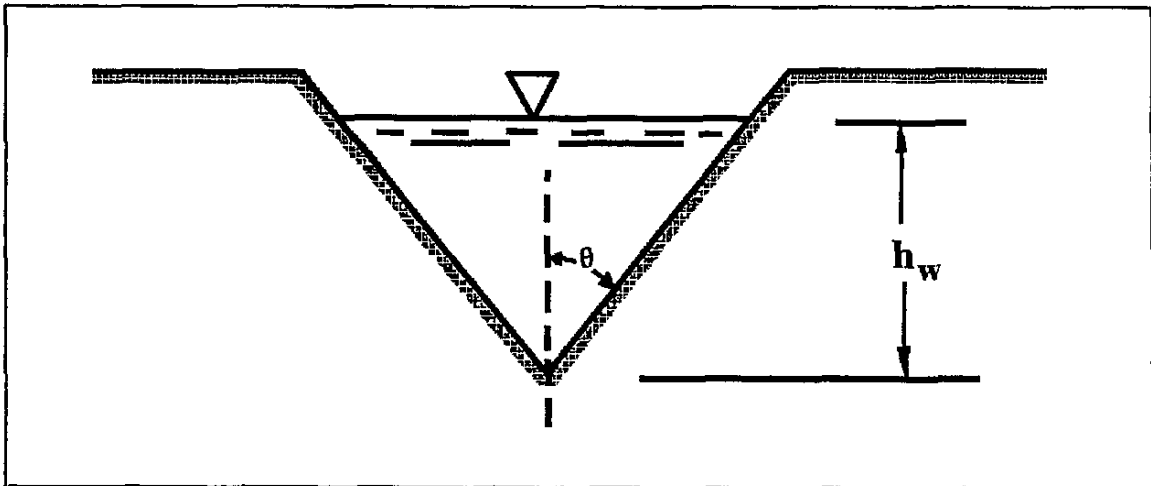


Figure 6.3.5  
Typical Triangular Weir

**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,

$$Q = A_p \left[ \frac{h_p}{\frac{K_e + K_o}{2g} + \frac{2.87n^2 L}{D^{4/3}}} \right]^{1/2} \quad (6.3.5)$$

where Q is the flowrate in cfs, A<sub>p</sub> is the area of the pipe (ft<sup>2</sup>), K<sub>e</sub> is the entrance loss coefficient (given in Table 6.3.3), K<sub>o</sub> 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<sub>p</sub> 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<sub>e</sub> (entrance losses)  
(Bureau of Reclamation, 1987)

<i>Entrance Condition</i>	<i>K<sub>e</sub> range</i>	<i>Average</i>
Square-edged inlets installed flush with vertical headwalls	0.43 to 0.70	0.50
Rounded Inlets installed flush with vertical headwalls, r/D ≤ 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:

$$i = \frac{c T_r^\alpha}{(t + d)^\beta} = \frac{1.9533(3)^{0.1747}}{\left(\frac{27}{60} + 0.522\right)^{1.6408}} = 2.479 \text{ inches/hour}$$

Therefore the allowable outflow rate is:

$$O(t) = C_U i_U A_U = (0.15)(2.479)(650 \times 650) \frac{1}{43,560} = 3.61 \text{ cfs}$$

Note that 0.15 is the maximum allowable runoff coefficient.

Calculate the developed runoff coefficient:

Type of Cover	Area Calculations	Area (ft <sup>2</sup> )	Area (acres)	C	C <sub>n</sub> A <sub>n</sub>
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

$$C_D = \frac{\sum C_n A_n}{A_T} = \frac{5.93}{9.7} = 0.61$$



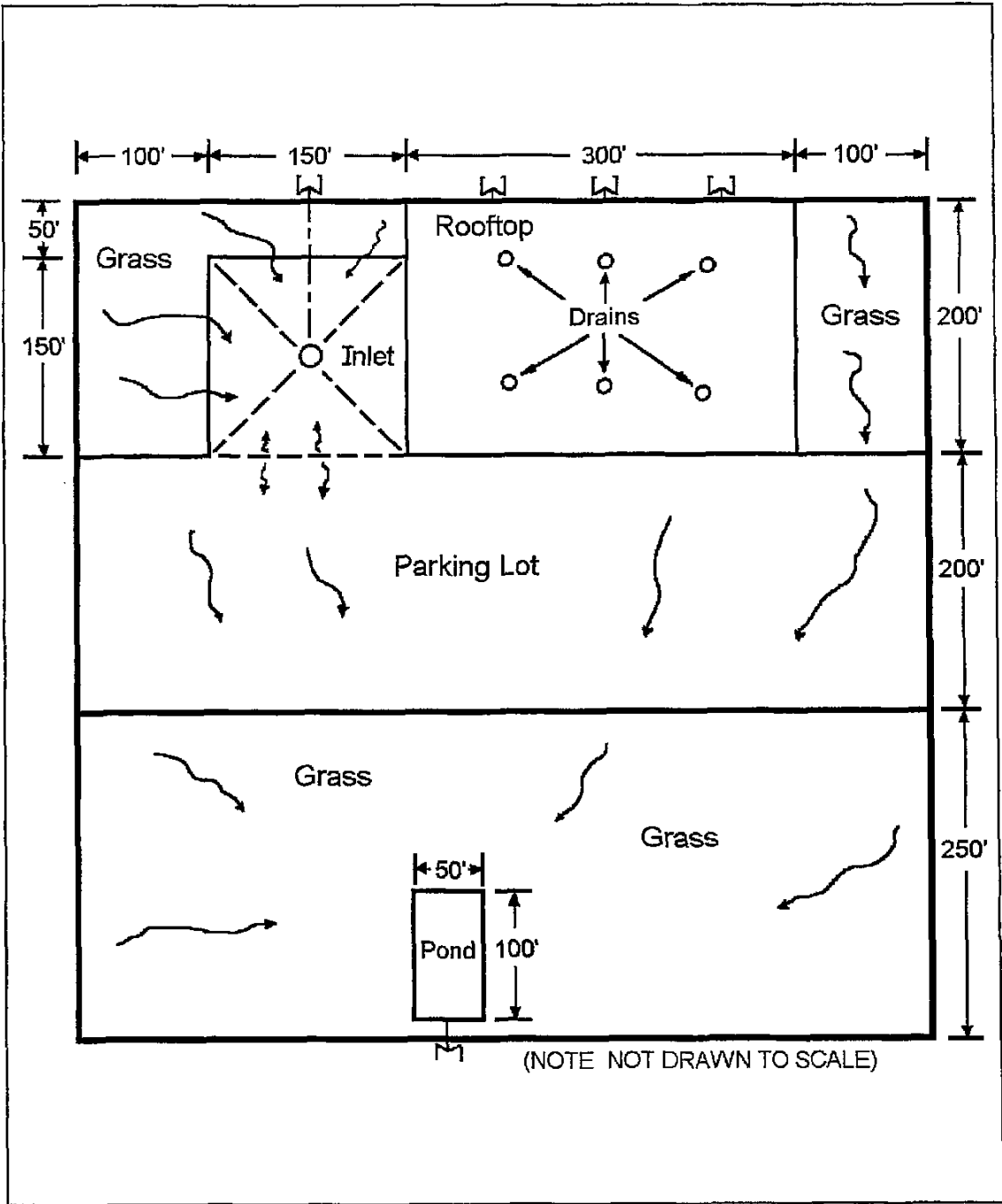


Figure 6.3.6  
Hypothetical Watershed for Example 6.3.1 (After MWRDGC)

**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 Coefficient ( $C_U$ ):	0.15
Undeveloped Runoff Rate ( $O=C_U i_U A_U$ ):	3.61 cfs
Developed Runoff Coefficient ( $C_D$ )	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 $(I(t_d)-O)(t_d/12)$ (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
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	*	*

← **Peak Storage**

\* Since  $I(tD) - O < \text{zero}$ , there is no storage needed

Figure 6.3.7  
 Detention Storage Calculations for the Entire Basin Using the Rational Method  
 Indiana LTAP Stormwater Drainage Manual - Revised December 2015 Chapter 6-38

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.

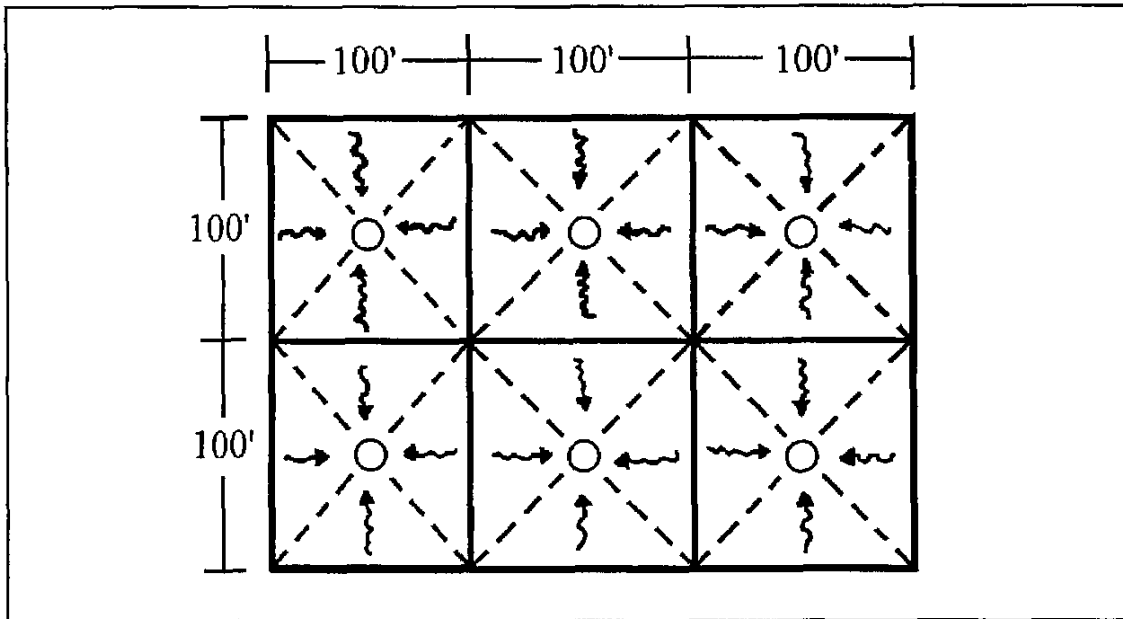


Figure 6.3.8  
Roof Drain Configuration

**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 Coefficient ( $C_U$ ):	---
Undeveloped Runoff Rate (O):	1.30 cfs
Developed Runoff Coefficient ( $C_D$ )	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 $(I(t_d)-O)(t_d/12)$ (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
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	*	*

← **Peak Storage**

\* Since  $I(tD) - O < \text{zero}$ , there is no storage needed

Figure 6.3.9  
 Detention Storage Calculations for the Roof Top Using the Rational Method  
 Indiana LTAP Stormwater Drainage Manual - Revised December 2015      Chapter 6-40

The volume of each cell is

$$V = \frac{1}{3}Ad \therefore \text{solving for } d: \quad d = \frac{3V}{A}$$

Since there are six "cells", the maximum depth required for each is:

$$d = \frac{3(0.08 \text{ acre} - ft)}{1.38 \text{ acres}} = 0.17 \text{ ft}$$

In summary, there are six drains needed. Each one must be able to handle a two-inch head and release

$$\frac{1.30 \text{ cfs}}{6} = 0.217 \text{ cfs} = 97.38 \text{ gpm}$$

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 <sup>2</sup> )	Area (acres)	C	C <sub>n</sub> A <sub>n</sub>
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

$$d = \frac{3V}{A} = \frac{3(0.05 \text{ acre} - ft)}{0.52 \text{ acres}} = 0.29 \text{ ft.}$$

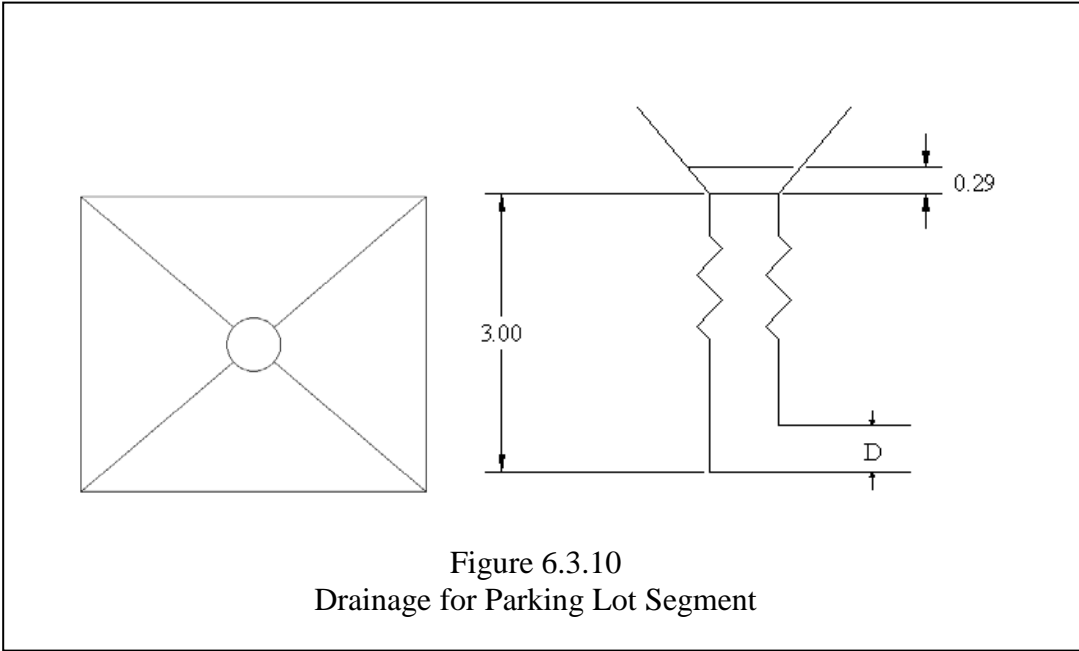


Figure 6.3.10  
Drainage for Parking Lot Segment

**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 Coefficient ( $C_U$ ):	---
Undeveloped Runoff Rate (O):	0.40 cfs
Developed Runoff Coefficient ( $C_D$ )	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 $(I(t_d) - O)(t_d/12)$ (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
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	*	*

← **Peak Storage**

\* Since  $I(tD) - O < \text{zero}$ , 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  $n = 0.013$ )

$$0.5 = A_p \left[ \frac{h_p}{\frac{K_e + K_o}{2g} + \frac{2.87n^2L}{D^{4/3}}} \right]^{1/2} = \frac{\pi D^2}{4} \left[ \frac{3.29 - \frac{D}{2}}{\frac{1.43}{64.4} + \frac{2.87(0.013)^2(100)}{D^{4/3}}} \right]^{1/2}$$

$$D = 0.61 \text{ feet} = 7.37 \text{ inches}$$

### 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 <sup>2</sup> )	Area (acres)	C	C <sub>n</sub> A <sub>n</sub>
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<sup>2</sup>, 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:

$$V = \left[ \frac{(100 + 6(4.75)) \times (50 + 6(4.75)) + (100)(50)}{2} \right] 4.75 = 0.82 \text{ ac} - \text{ft}$$



**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 Coefficient ( $C_U$ ):	---
Undeveloped Runoff Rate (O):	1.90 cfs
Developed Runoff Coefficient ( $C_D$ )	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 $(I(t_d)-O)(t_d/12)$ (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
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	*	*

← **Peak Storage**

\* Since  $I(tD) - O < \text{zero}$ , there is no storage needed

Figure 6.3.12  
Detention Storage Calculations for the Detention Pond Using the Rational Method

Conclusion:

<i>Type of Cover</i>	<i>Storage (acre-feet)</i>	<i>Release Rate (cfs)</i>
Pond	0.30	1.9
Rooftop	0.08	1.3
Parking Lot	0.05	0.4
Total	0.43	3.6

## 6.4 – COMPUTER PROGRAM

```

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
C   PARAMETER (ND=16)
C   CHARACTER LABEL*80
C   DIMENSION T(ND),RAIN(ND)
C
C   ARRAY T(ND) CONTAINS STORM DURATIONS IN HOURS
C   AS RECOMMENDED BY MSDGC.
C   DATA T/0.17, 0.33, 0.50, 0.67, 0.83, 1.0, 1.5, 2.0, 3.0, 4.0,
C   & 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
C   DATA RAIN/7.6, 5.5, 4.4, 3.7, 3.2, 2.8, 2.1, 1.7, 1.2, 1.0,
C   & 0.84, 0.73, 0.65, 0.58, 0.53, 0.49/
C
C   TR IS THE RETURN PERIOD
C   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
C   1 READ(5,100,END=99) LABEL
C   100 FORMAT(A80)
C   READ(5,101) TU,RUND,CU,AU,AD,CD
C   101 FORMAT(6F10.0)
C   WRITE(6,200) LABEL
C   200 FORMAT('1',//,1X,A80,/)
C   WRITE(6,201) TU,RUND,CU,AU,AD,CD
C   201 FORMAT(' UNDEVELOPED RETURN PERIOD (YEARS) = ',
C   &6X,F10.0,/, ' UNDEVELOPED RAINFALL INTENSITY (INCH/HOUR) = ',
C   &F10.3,/, ' UNDEVELOPED RUNOFF COEFFICIENT = ',12X,F10.3,/,
C   &' UNDEVELOPED AREA (ACRES) = ',18X,F10.3,/,
C   &' DEVELOPED AREA (ACRES) = ',20X,F10.3,/,
C   &' DEVELOPED RUNOFF COEFFICIENT = ',14X,F10.3,/)
C   CALCULATE ALLOWABLE OUTFLOW USING THE RATIONAL METHOD
C   OUT=CU*AU*RUND
C   WRITE(6,202) OUT
C   202 FORMAT(' ALLOWABLE OUTFLOW RATE (CFS) = ',14X,F10.3,/)
C   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

## CHAPTER 6 - REFERENCES

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## Chapter 7 - STORM SEWER SYSTEM DESIGN

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## 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

<u>Description</u>	<u>Reference</u>
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:

$$Q = CiA \quad (7.2.1)$$

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.



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.

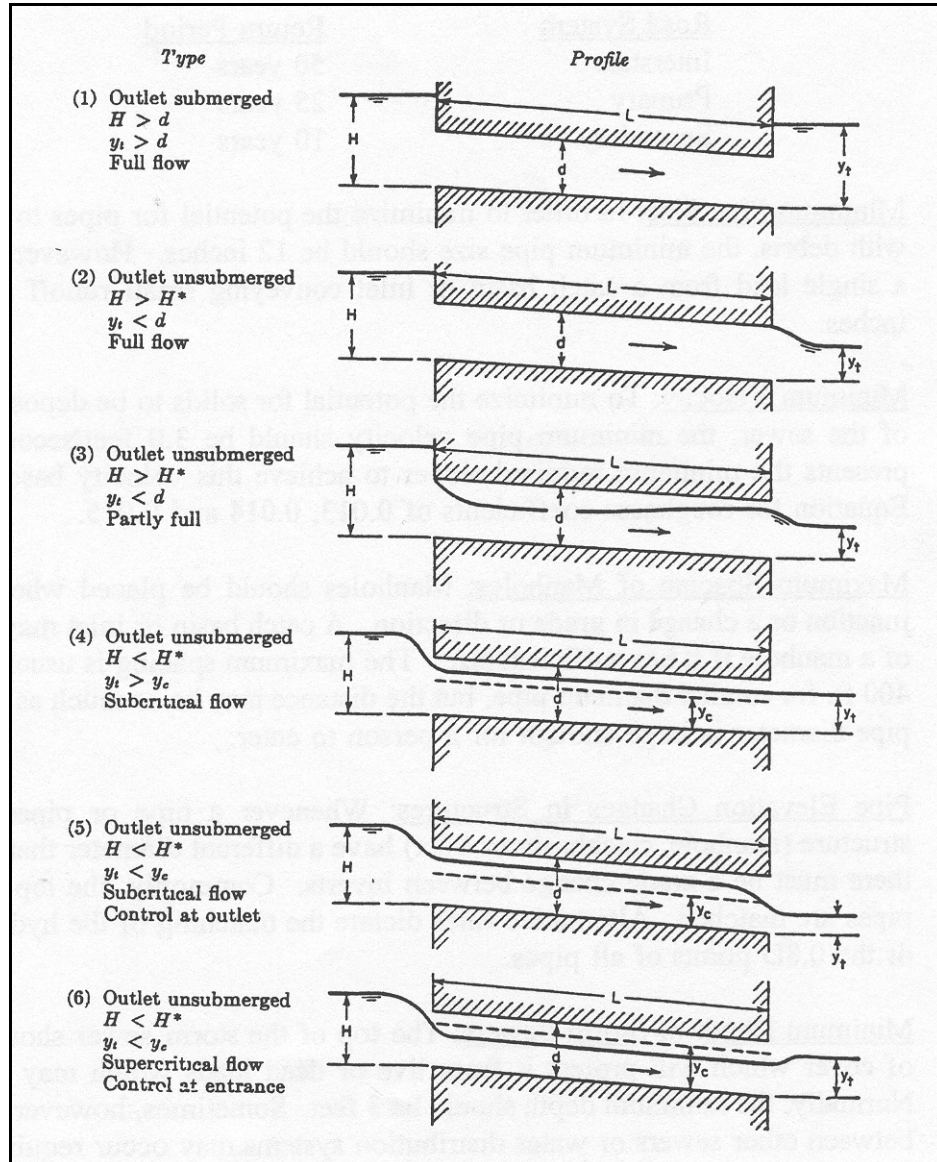


Figure 7.3.1  
Types of Culvert Flow (Chow, 1988)



## 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.

<u>Road System</u>	<u>Return Period</u>
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)		
	<i>n</i> = 0.013	<i>n</i> = 0.014	<i>n</i> = 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	Acrylonitrile - 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

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### 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:

$$i = \frac{c T_r^\alpha}{(t + d)^\beta} = \frac{2.0030(10)^{0.1655}}{(t + 0.516)^{1.4643}}$$

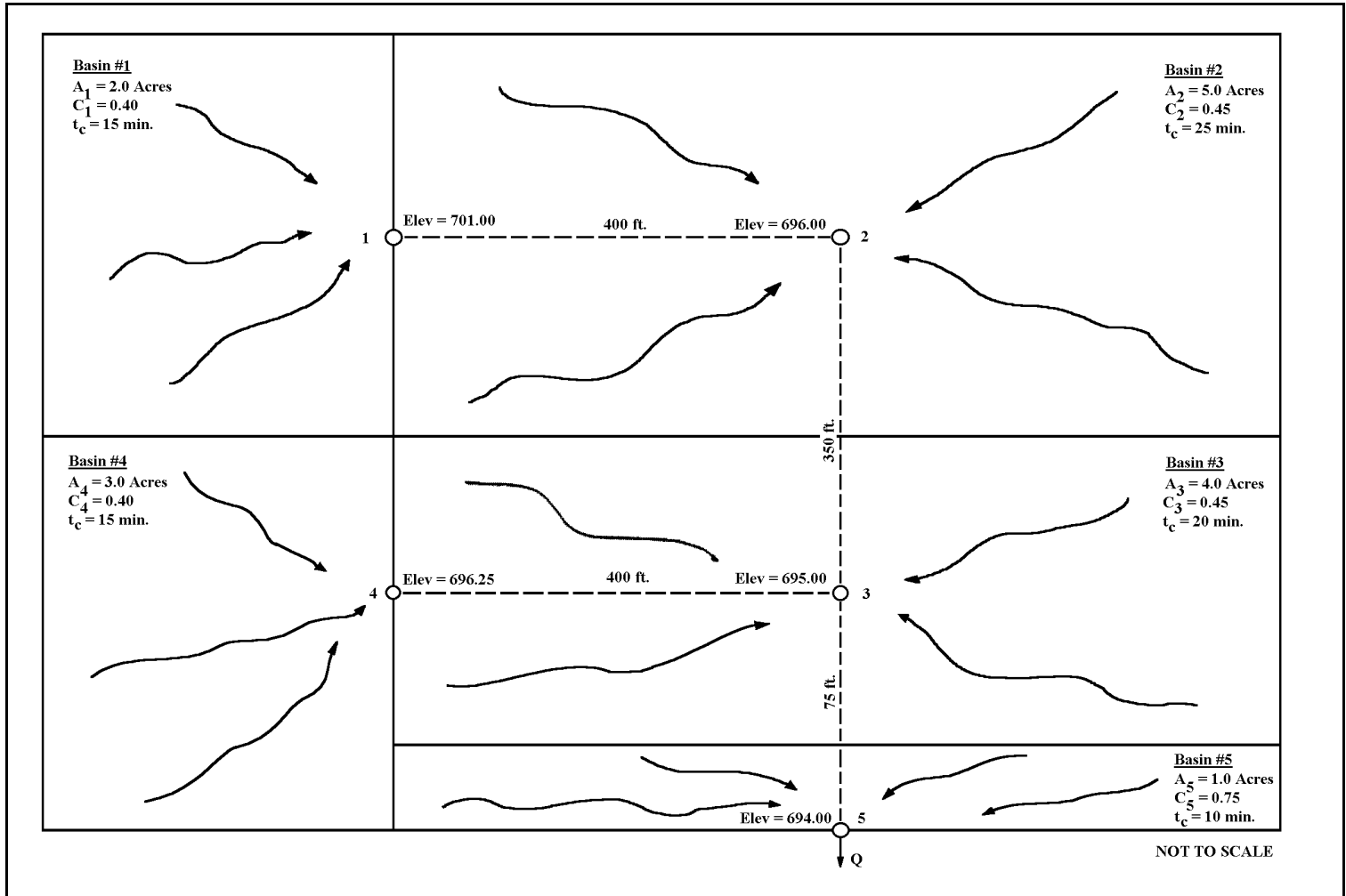
For  $t > 1$  hour:

$$i = \frac{c T_r^\alpha}{(t + d)^\beta} = \frac{1.4381(10)^{0.1878}}{(t + 0.525)^{0.8616}}$$

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 Manhole	Length (ft)	C <sub>f</sub>	A <sub>f</sub> (acres)	C <sub>f</sub> A <sub>f</sub>	ΣC <sub>f</sub> A <sub>f</sub>	t <sub>f</sub> (min)	t <sub>man</sub> (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	<b>3.47</b>	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	<b>9.90</b>	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	<b>5.20</b>	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	<b>18.89</b>	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	<b>21.15</b>	---	---	---	---	---	---	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	$C_j A_j$ Product of area and Runoff Coefficient for subbasin #4	From Figure 7.6.1
8	1.2	$\Sigma C_j A_j$ 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 $Q = i \Sigma C_j A_j$
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	$V = \text{Flow}/\text{Area}$
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

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### 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.



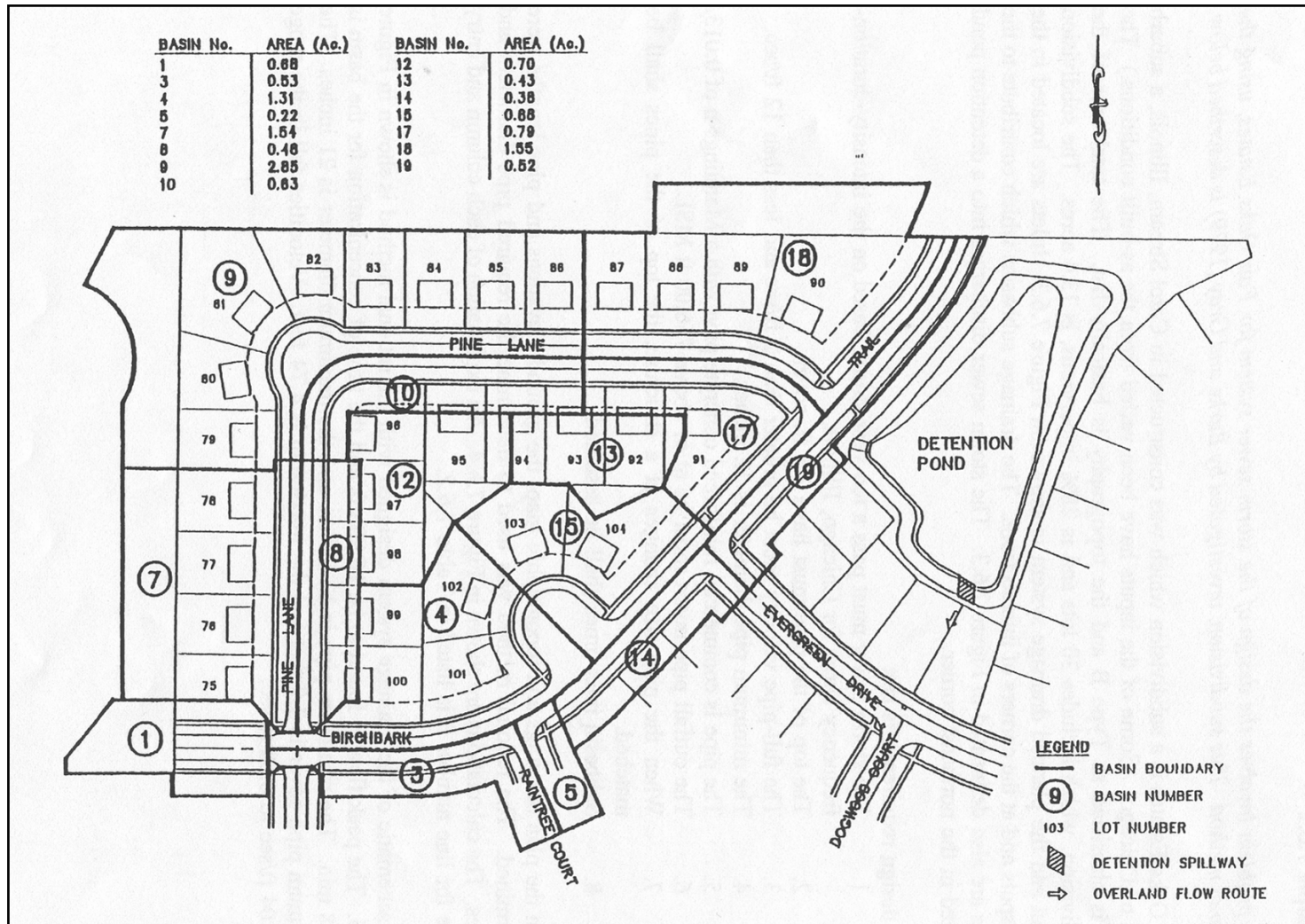


Figure 7.6.3  
Fair Oaks Estates Hydrologic System

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 <sub>j</sub>	A <sub>j</sub> (Acres)	C <sub>j</sub> A <sub>j</sub>	ΣC <sub>j</sub> A <sub>j</sub>	t <sub>j</sub> (min)	t <sub>cum</sub> (min)	i [inches/hour]	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 Cor. Upstream	Pipe Cor. 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	$C_j A_j$ Product of area and Runoff Coefficient for subbasin ( $0.44 \times 0.53 = 0.23$ )
8	0.41	$\Sigma C_j A_j$ 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 $Q = i \Sigma C_j A_j$ , $Q = 3.94 (0.41) = 1.61$ 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.)

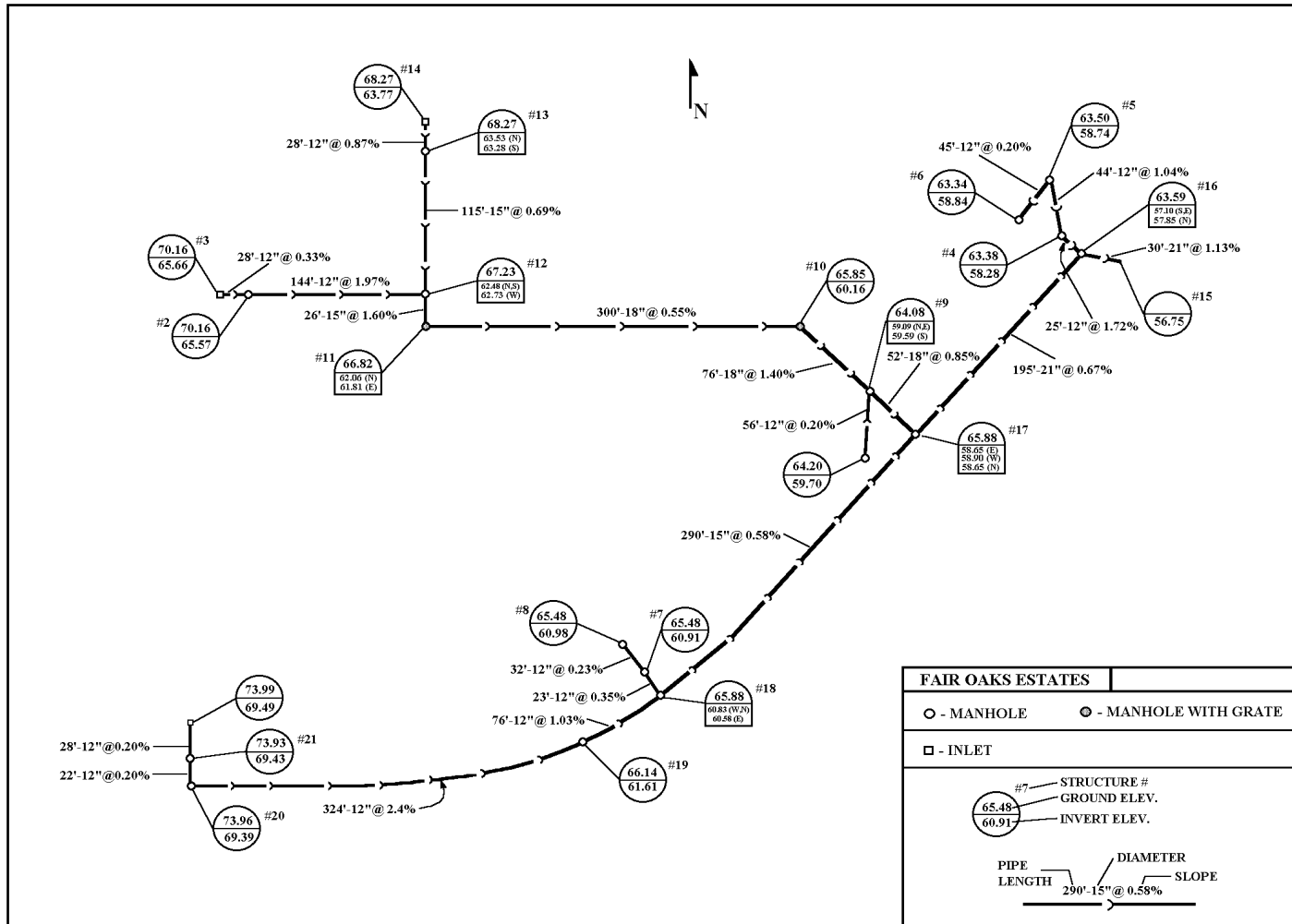


Figure 7.6.5  
 Layout of Storm Sewer Network System Designed by the Rational Method, Example 7.6.2

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 <sub>r</sub> * (cfs)	D (inches)	S (%)	V (ft/sec)	Q <sub>p</sub> * (cfs)	Q <sub>p</sub> / Q <sub>r</sub>
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<sub>r</sub> = Runoff from rational method  
 Q<sub>p</sub> = Capacity of the pipe flowing full

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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](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<sup>2</sup>). 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<sup>2</sup>, 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

<b>Runoff Curve Number</b>						
Project: <u>Ross Ditch- EXAMPLE 8.1.1</u>		By: _____		Date: <u>7/12/2006</u>		
Location: <u>Tippecanoe County</u>		Checked: _____		Date: <u>7/12/2006</u>		
Circle One: <span style="border: 1px solid black; border-radius: 50%; padding: 2px;">EXISTING</span> PROPOSED		Description: <u>SUBBASIN 1 (SA1)</u>				
Soil Name and Hydrologic Group  (Appendix A)	Cover Description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	Curve Number			Area <input type="checkbox"/> acres <input type="checkbox"/> sq. mi. <input checked="" type="checkbox"/> %	Product of Curve Number and Area
		Table 2-2	Fig. 2-3	Fig. 2-4		
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
<b>Totals =</b>					100.0%	84.23
<p>CN (weighted) = <math>\frac{\text{total product}}{\text{total area}} = \frac{84.23}{1} = \underline{84.23}</math></p> <p style="text-align: right;">Use CN = <span style="border: 1px solid black; padding: 5px; font-weight: bold; font-size: 1.2em;">84</span></p>						

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<sub>c</sub> Calculations for Example 8.1.1

<b>Time of Concentration (T<sub>c</sub>) or Travel Time (T<sub>t</sub>)</b>					
Project:	<u>Ross Ditch- EXAMPLE 8.1.1</u>	By:	<u>                    </u>	Date:	<u>                    </u>
Subarea:	<u>Subbasin 1</u>	Checked:	<u>                    </u>	Date:	<u>                    </u>
Location:	<u>Tippecanoe County</u>				
File:	<u>                                    </u>				
Circle One:	<span style="border: 1px solid black; border-radius: 50%; padding: 2px 10px;">EXISTING</span>	PROPOSED	Description: <u>                                    </u>		
<b><u>SHEET FLOW</u></b>					
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, P <sub>2</sub> (in)	3.00				
Land slope, s (ft/ft)	0.005				
T <sub>t</sub> = (0.007(nL) <sup>0.8</sup> )/(P <sub>2</sub> <sup>0.5</sup> s <sup>0.4</sup> ) (hr)	0.32	+			= <span style="border: 1px solid black; padding: 2px 10px;">0.32</span> hr
<b><u>SHALLOW CONCENTRATED FLOW</u></b>					
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 <sub>t</sub> = L / 3600 V (hr)	0.79	+		+	
				+	
					= <span style="border: 1px solid black; padding: 2px 10px;">0.79</span> hr
<b><u>CHANNEL FLOW</u></b>					
Segment ID	III				
Cross-sectional flow area, a (ft <sup>2</sup> )					
Wetted perimeter, P <sub>w</sub> (ft)					
Hydraulic radius, r = a/P <sub>w</sub> (ft)					
Channel slope, s (ft/ft)					
Manning's roughness coeff., n					
V = (1.49 r <sup>0.667</sup> s <sup>0.5</sup> ) / n (ft/s)					
Flow length, L (ft)					
T <sub>t</sub> = L / 3600 V (hr)		+			= <span style="border: 1px solid black; padding: 2px 10px;">          </span> hr
<b>Watershed or subarea T<sub>c</sub> or T<sub>t</sub></b>					= <span style="border: 1px solid black; border-radius: 50%; padding: 2px 10px;">1.11</span> 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.

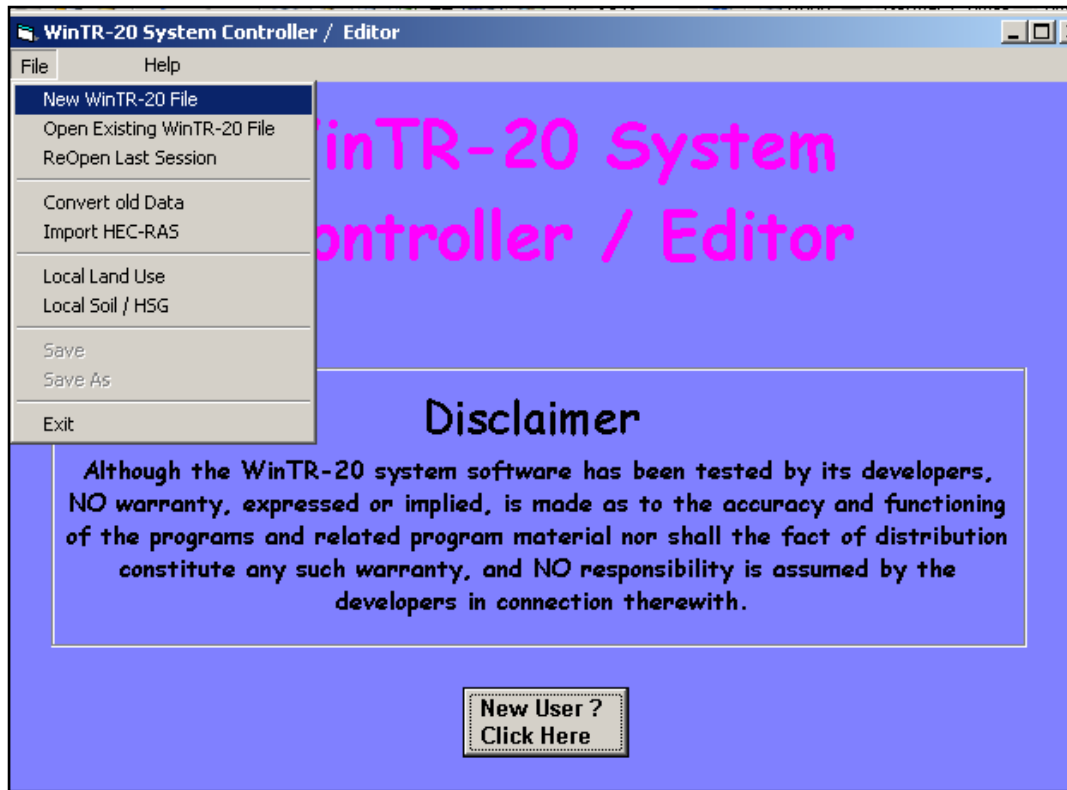


Figure 8.1.1 (a)  
New File Creation in Win TR-20

- 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.



**WinTR-20 Identifier:**

Input Units Code:  English  Metric

Output Units Code:  English  Metric

Minimum Hydrograph Value:  cfs

Watershed Description:

EXAMPLE 8.1.1-ROSS DITCH WATERSHED-TIPPECANOE COUNTY

EXISTING CONDITIONS-SUBBASIN 1

No Changes (Close)    Accept Changes (Close)

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.

**WinTR-20 System  
Controller / Editor**

**Click Data Section Name to Enter Data**  
(X indicates some data entered for section)

- Alternate Analysis:
- Dimensionless Unit Hydrograph:
- Duration Increment:
- Global Output:
- Input Hydrograph:
- Rainfall Distribution:
- Stream Cross Section:
- Stream Reach:
- Storm Analysis:
- Structure Rating:
- Sub-Area:
- Time Analysis:
- X WinTR-20: Version 1.0
- Verification:

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).

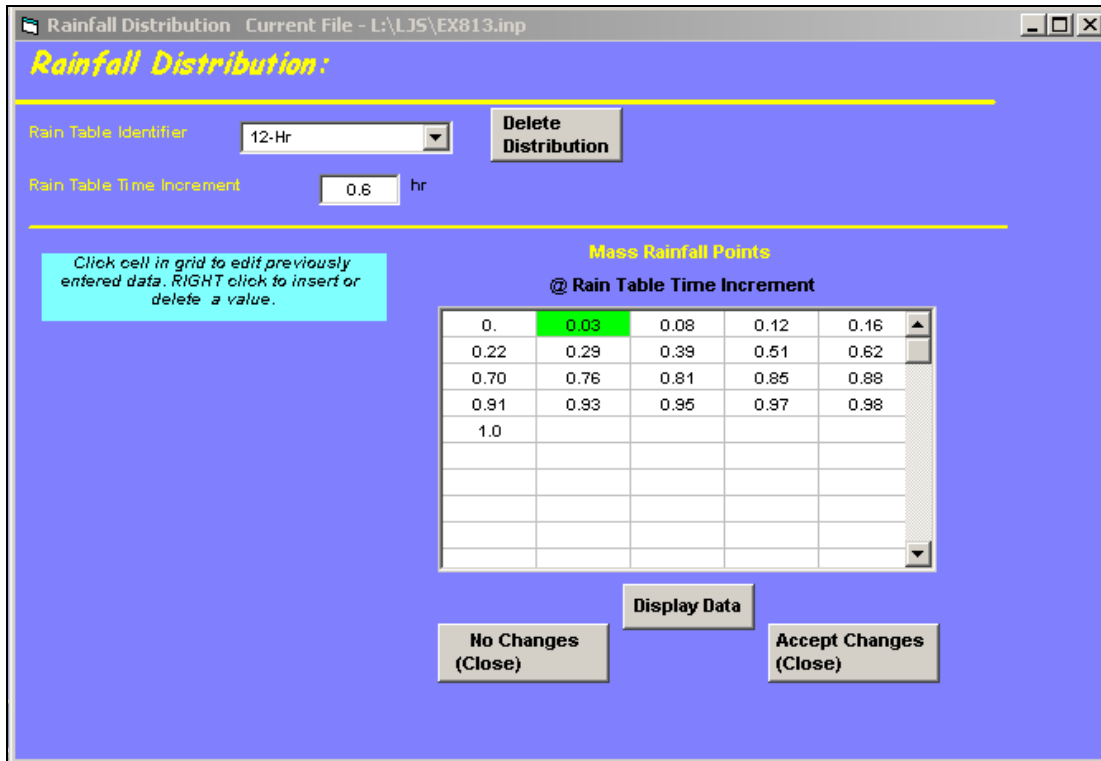


Figure 8.1.1 (f)  
Huff 2<sup>nd</sup> Quartile Rainfall Distribution for 12-Hour Storm Event

- 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.

Storm Analysis Current File - L:\LJ5\EX811.inp

### Storm Analysis:

Repeat the following for each Storm Identifier and Rain Gage Identifier combination

Storm Identifier: 100YR,12HR

Rain Gage Identifier:

Gage Starting Time: 0.0 hr

Gage Rainfall: 5.48 in

Gage Rain Table Identifier:

Gage Antecedent Runoff Condition:  1  2  3

2-Yr 24-Hr Rainfall:

Storm Id	Rain Gage Id	Start	Rain	Rain Table Id	ARC	2-Yr
100YR,12HR		0.0	5.48	12-Hr	2	

Click row in grid to edit previously entered data. RIGHT click to delete row.

No Changes (Close) Accept Changes (Close)

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.

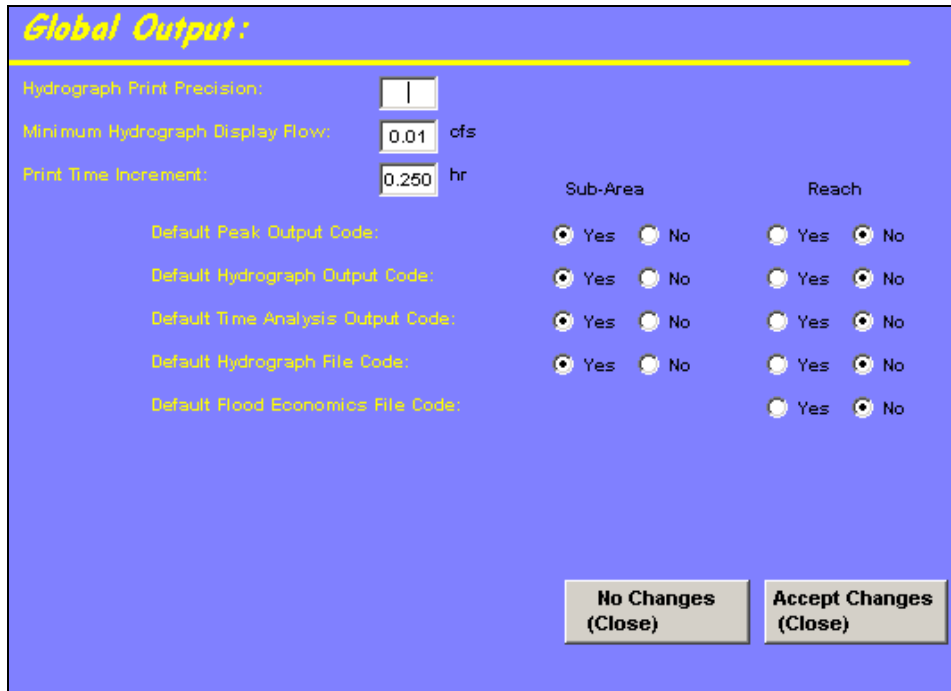


Figure 8.1.1 (h)  
Output Control for Example 8.1.1

- 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.

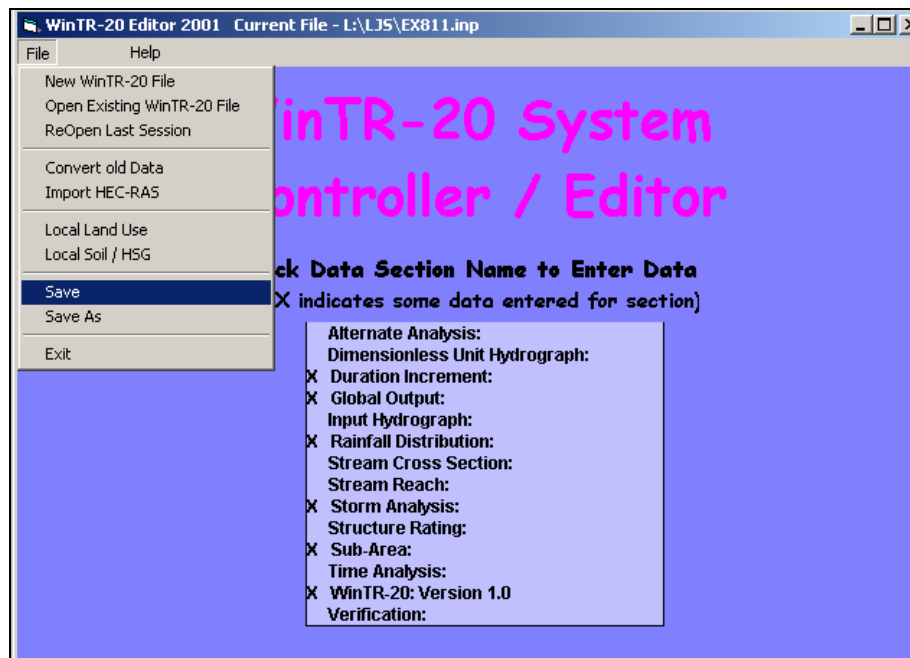


Figure 8.1.1 (i)  
Completion of Required Input Data

- 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.

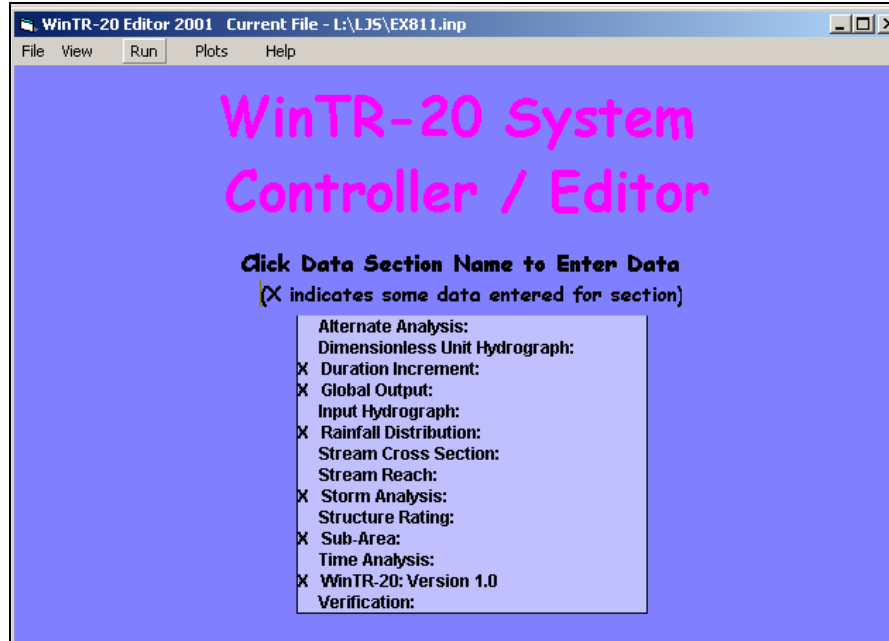


Figure 8.1.1 (j)  
The Run Tab Appears When Model is Saved

## 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)	----- Elevation (ft)	----- Peak Flow Time (hr)	----- Rate (cfs)	----- 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

<i>Stage (feet)</i>	<i>Discharge (cfs)</i>	<i>Storage (acre-ft)</i>
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).

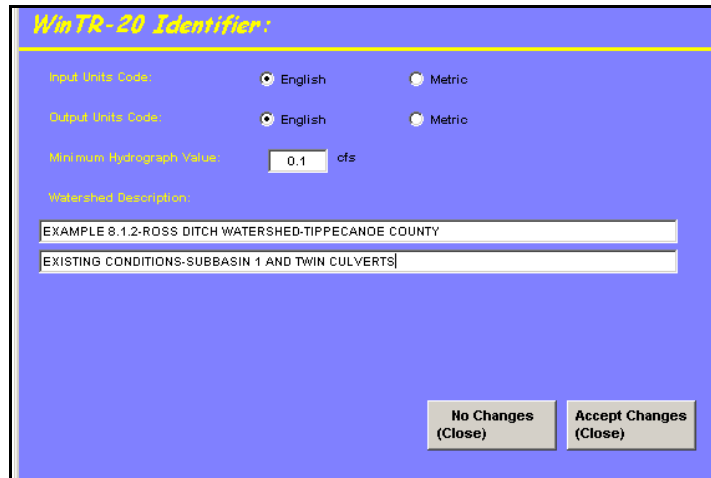


Figure 8.1.3 (a)  
Change Description From Previous Example

- 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.

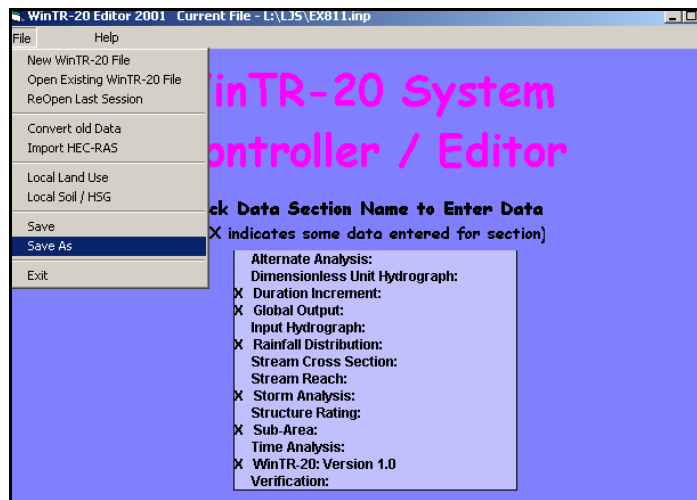


Figure 8.1.3 (b)  
Save and Rename as Example 8.1.2

- 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.



**Structure Rating:**

Structure Identifier:

Structure Starting Elevation:  ft

---

Repeat the following for each structure rating point for the Structure Identifier

Structure Elevation:  ft

Structure Discharge:  cfs

Structure Storage:  ac ft

Click row in grid to edit previously entered data.  
RIGHT click to delete row.

Elevation	Discharge	Storage
654.17	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.
656.59	50.	243.7

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.

**Stream Reach:**

Stream Reach Identifier:

Stream Receiving Reach Identifier:

Reach Cross Section Identifier:

Reach Structure Identifier:

Reach Channel Length:  ft

Reach Valley Length:  ft

Constant Base Flow:  csm

Reach Peak Output Code:  Yes  No  Blank

Reach Hydrograph Output Code:  Yes  No  Blank

Reach Time Analysis Code:  Yes  No  Blank

Reach Hydrograph File Code:  Yes  No  Blank

Reach Flood Economics File Code:  Yes  No  Blank

---

Split Flow Reach Identifier:

Split Flow Cross Section Identifier:

Starting Split Flow:  cfs

Ending Split Flow:  cfs

Split Flow Drainage Area:

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

<i>LOCATION</i>	<i>PEAK DISCHARGE (cfs)</i>	<i>RUNOFF AMOUNT (inches)</i>	<i>WATER SURFACE ELEVATION (feet)</i>	<i>TIME TO PEAK (hrs)</i>
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

STORH 100YR,12HR

Area or Drainage Rain Gage Runoff ----- Peak Flow -----
Reach Area ID or Amount Elevation Time Rate Rate
Identifier (sq mi) Location (in) (ft) (hr) (cfs) (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

<i>Drainage Area</i>	<i>Area (%)</i>	<i>Land Use (Hydrologic Group)</i>	<i>CN</i>	<i>Area * CN</i>
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



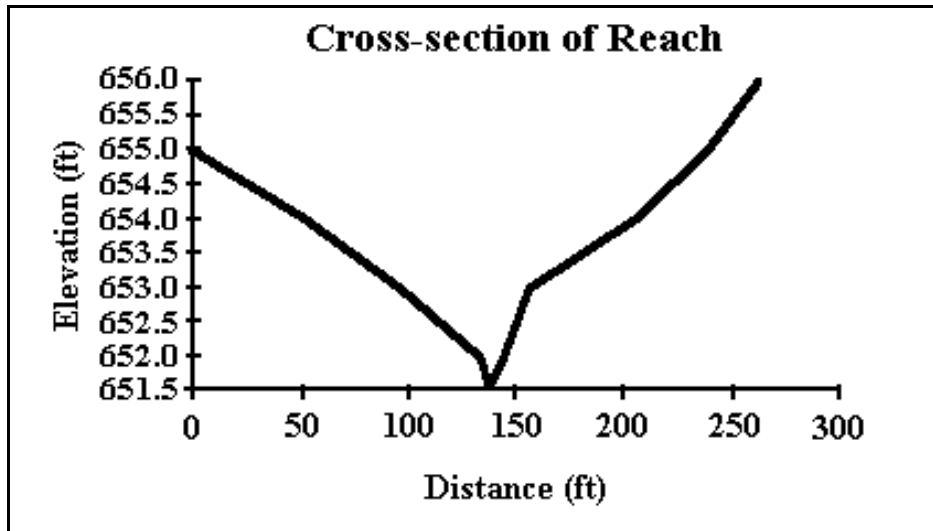


Figure 8.1.5  
Cross-section of Reach through Subbasin 2

### 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.

**Stream Cross Section** Current File - L:\LJ5\EX813.inp

*Stream Cross Section:*

Cross Section Identifier: Xsec 1 **Delete Cross Section**

Bankfull Elevation: 653.0 ft

Low Ground Elevation: 651.6 ft

Stream Identifier: [ ]

Stream Station: [ ] ft

*Repeat following variables for each cross section rating point in the Cross Section Identifier*

Cross Section Elevation: 651.6 ft

Cross Section Discharge: 0. cfs

Cross Section End Area: 0. sq ft

Cross Section Top Width: 0. ft

Cross Section Friction Slope: 0.0007

Elevation	Discharge	End Area	Top Width	Friction Slope
651.6	0.	0.	0.	0.0007
652.5	10.	407.	40.	0.0007
652.7	20.	712.	45.	0.0007
652.9	40.	1040.	50.	0.0007
653.3	80.	1812.	52.	0.001
653.5	100.	2233.	75.	0.001
653.6	150.	2837.	100.	0.001
653.9	200.	3944.	150.	0.001
654.0	250.	4188.	160.	0.001

*Click row in grid to edit previously entered data. RIGHT click to delete row.*

**Display Data**

**No Changes (Close)** **Accept Changes (Close)**

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.

**Stream Reach** Current File - L:\LJ5\EX813.inp

*Stream Reach:*

Stream Reach Identifier: Reach 2 **Delete Reach**

Stream Receiving Reach Identifier: OUTLET

Reach Cross Section Identifier: Xsec 1

Reach Structure Identifier: [ ]

Reach Channel Length: 1500.0 ft

Reach Valley Length: [ ] ft

Constant Base Flow: [ ] csm

Reach Peak Output Code:  Yes  No  Blank

Reach Hydrograph Output Code:  Yes  No  Blank

Reach Time Analysis Code:  Yes  No  Blank

Reach Hydrograph File Code:  Yes  No  Blank

Reach Flood Economics File Code:  Yes  No  Blank

Split Flow Reach Identifier: [ ]

Split Flow Cross Section Identifier: [ ]

Starting Split Flow: [ ] cfs

Ending Split Flow: [ ] cfs

Split Flow Drainage Area: [ ]

**No Changes (Close)** **Accept Changes (Close)**

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.

STORH 100YR, 12HR							
Area or Reach Identifier	Drainage Area (sq mi)	Rain Gage ID or Location	Runoff Amount (in)	----- Elevation (ft)	Peak Time (hr)	Flow Rate (cfs)	----- 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

<i>Area</i>	<i>Area (mi<sup>2</sup>)</i>	<i>Time of Peak (hours)</i>	<i>Peak Discharge</i>
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 Corps 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 Snyder 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)

<i>METHOD</i>	<i>DESCRIPTION</i>
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  $S_2$ .

$$\frac{S_2}{\Delta t} + \frac{O_2}{2} = \left( \frac{S_1}{\Delta t} + \frac{O_1}{2} \right) - O_1 + \frac{I_1 + I_2}{2} \quad (8.2.1)$$

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.

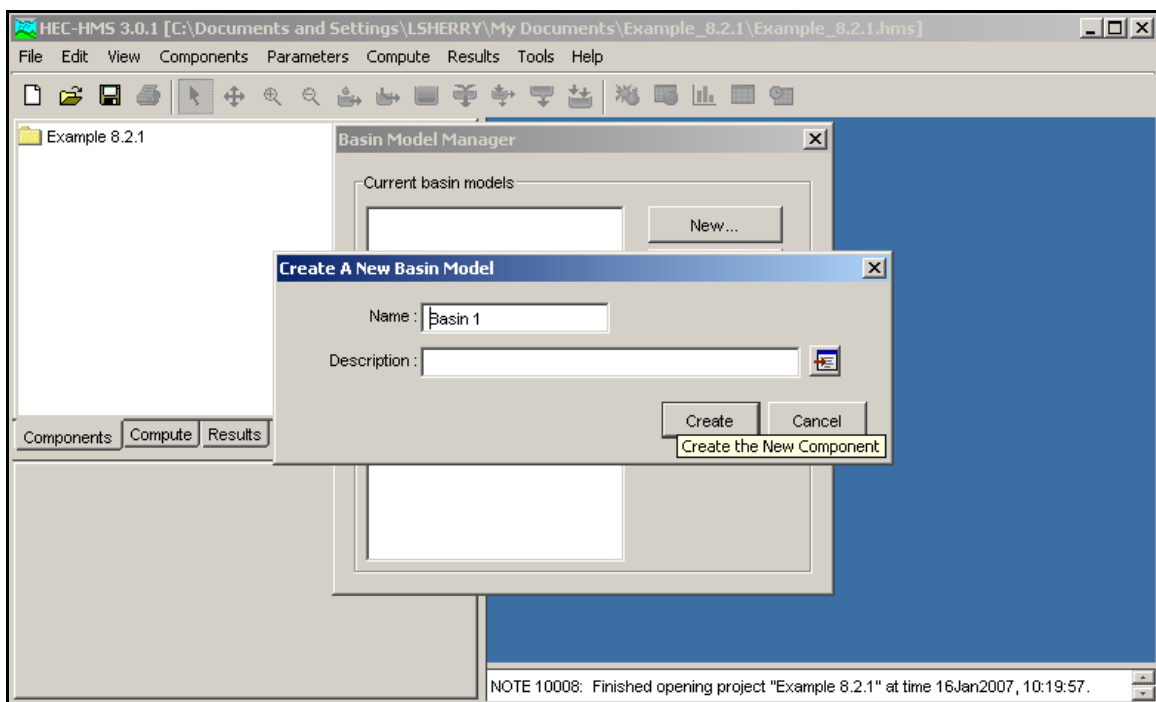


Figure 8.2.1 (a)  
Creation of New Basin Model

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.

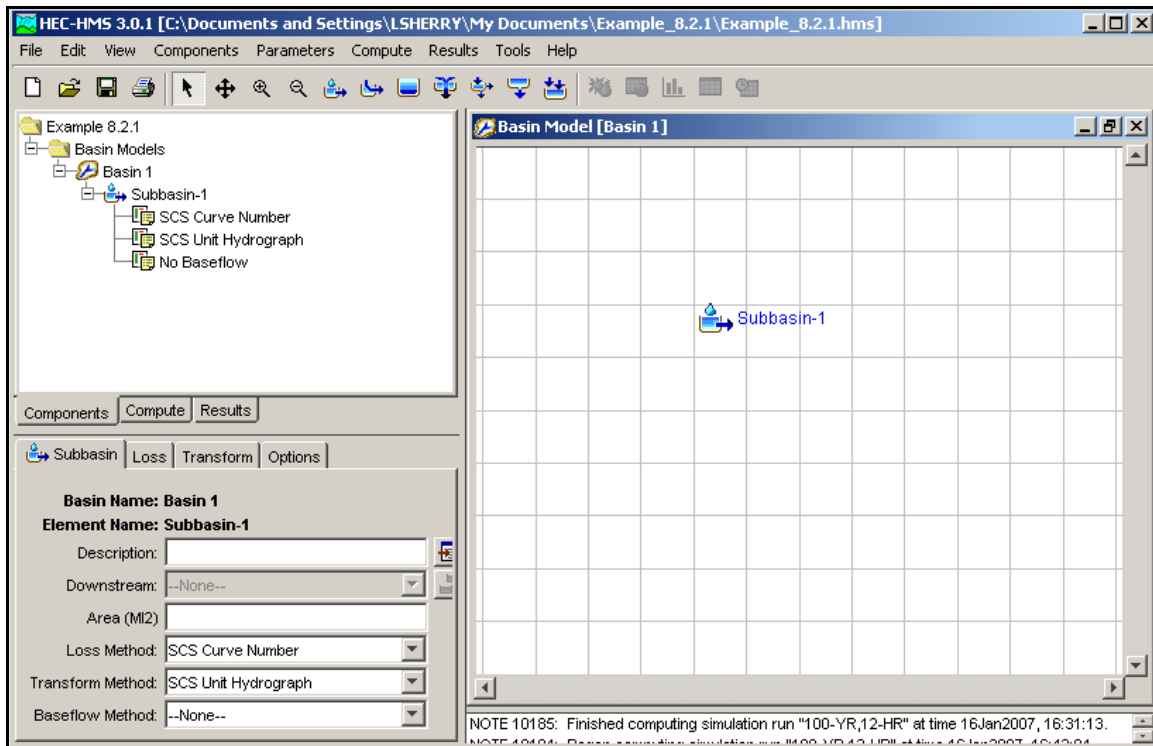


Figure 8.2.1 (b)  
Adding Subbasin 1 to the Basin Model

## Precipitation

Example 8.2.1 specifies that Subbasin 1 be analyzed for the 100-year, 12-hour storm event for Lafayette. The Huff 2<sup>nd</sup> 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 2<sup>nd</sup> Quartile Distribution for 12-hour Storm Event

To enter the Huff 2<sup>nd</sup> 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.

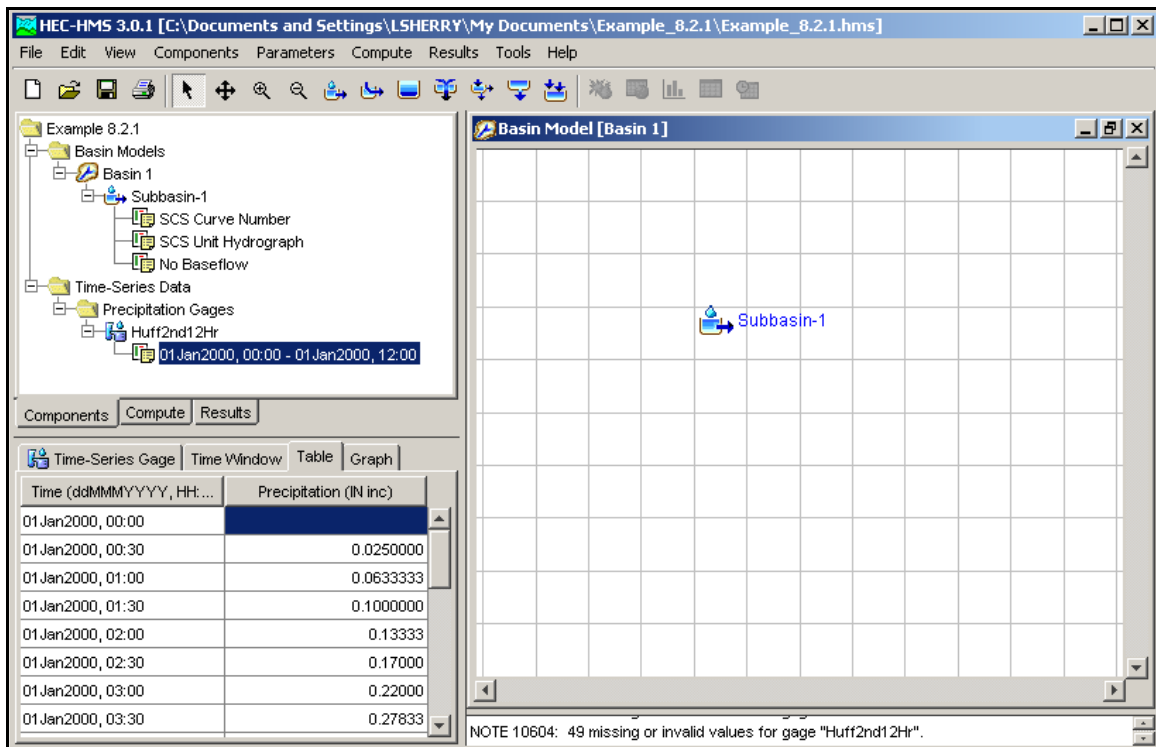


Figure 8.2.1 (c)  
Entering the 2<sup>nd</sup> Quartile Huff Distribution for a 12-Hour Storm Event

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.

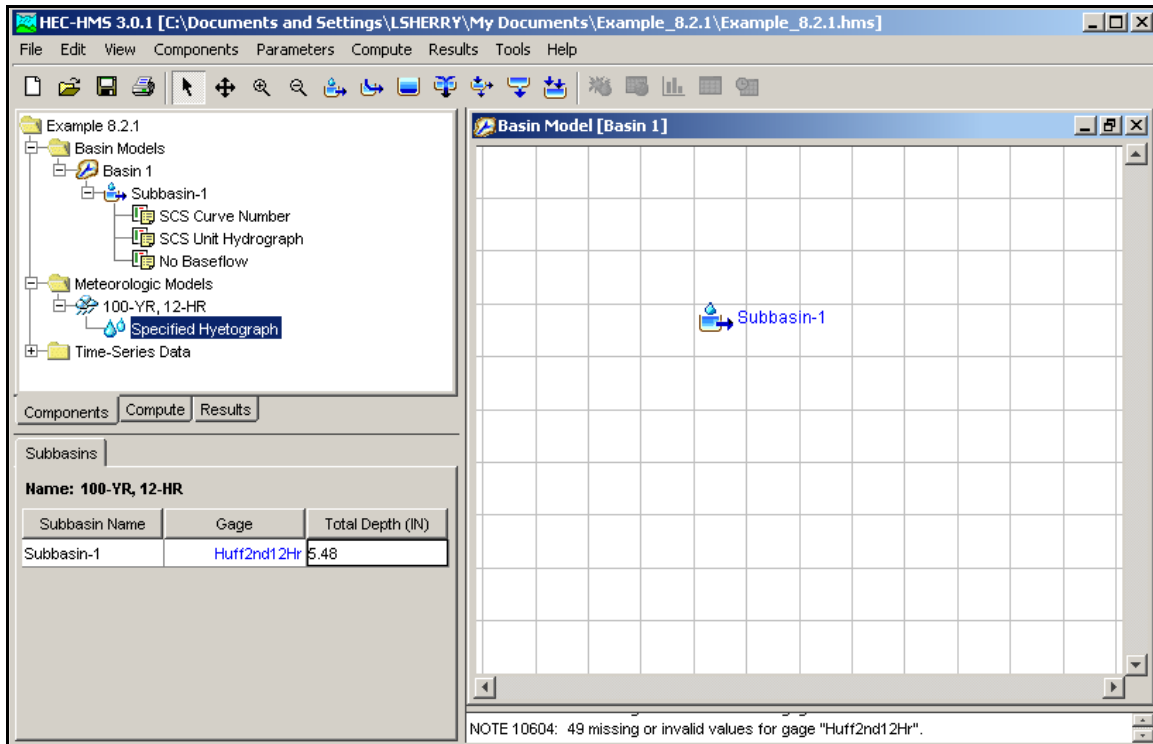


Figure 8.2.1 (d)  
Assigning the Precipitation Gage to Subbasin 1

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.



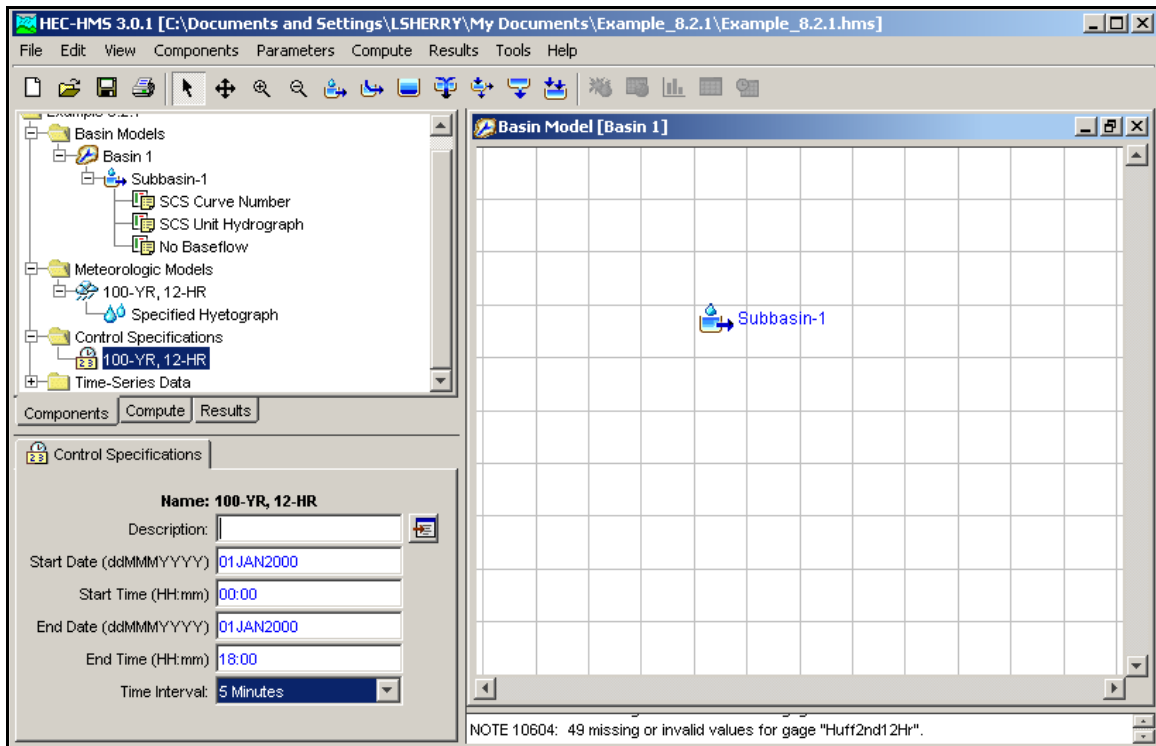


Figure 8.2.1 (e)  
Entering the Control Specifications

## 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.

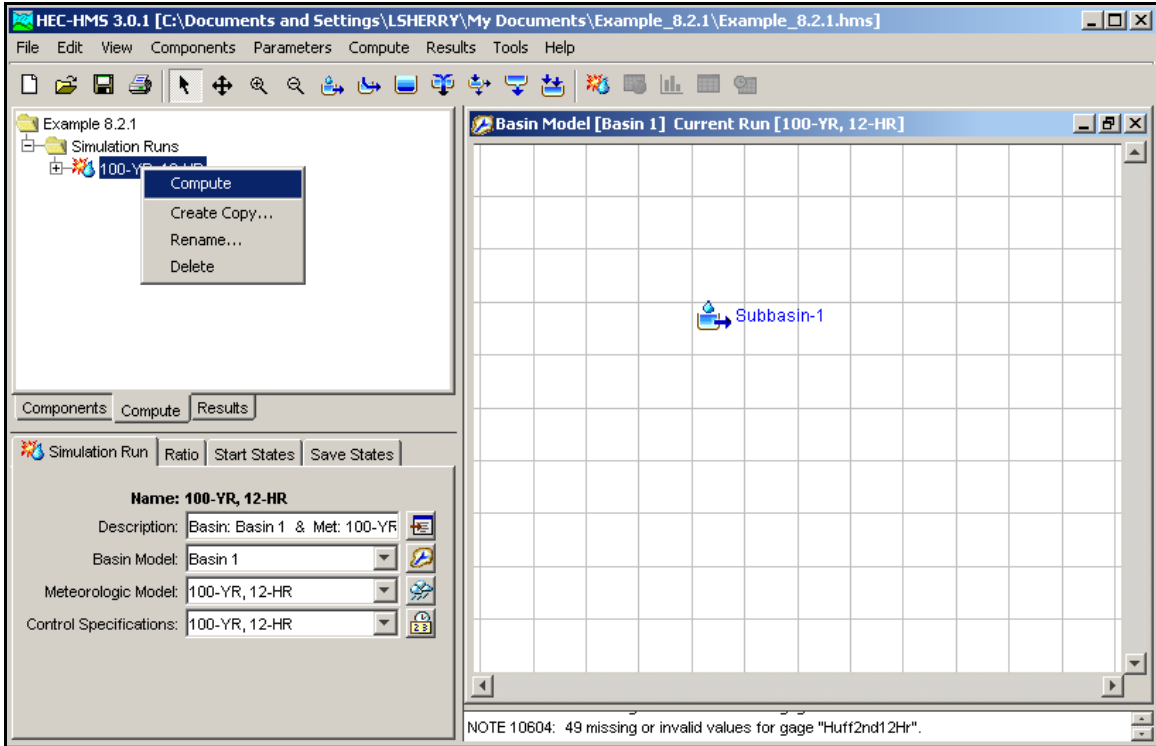


Figure 8.2.1 (f)  
Performing the Model Simulation Run

### 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.

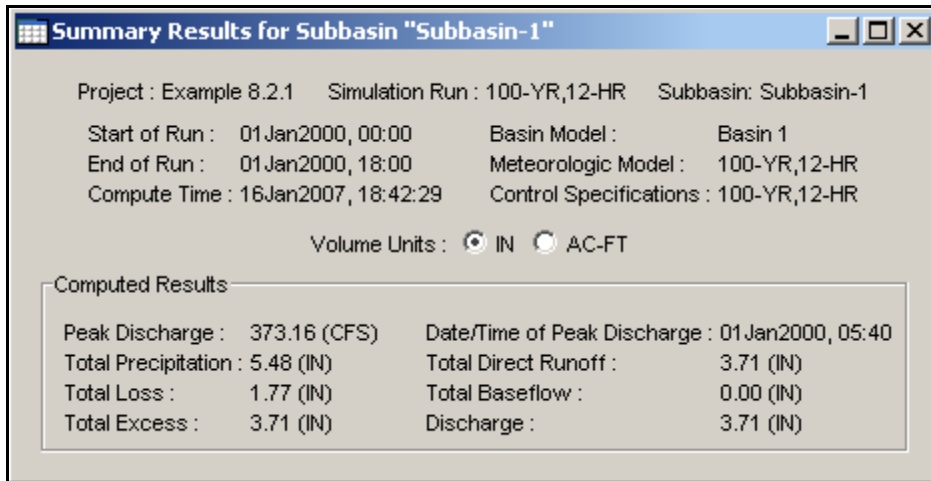


Figure 8.2.2

## 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.

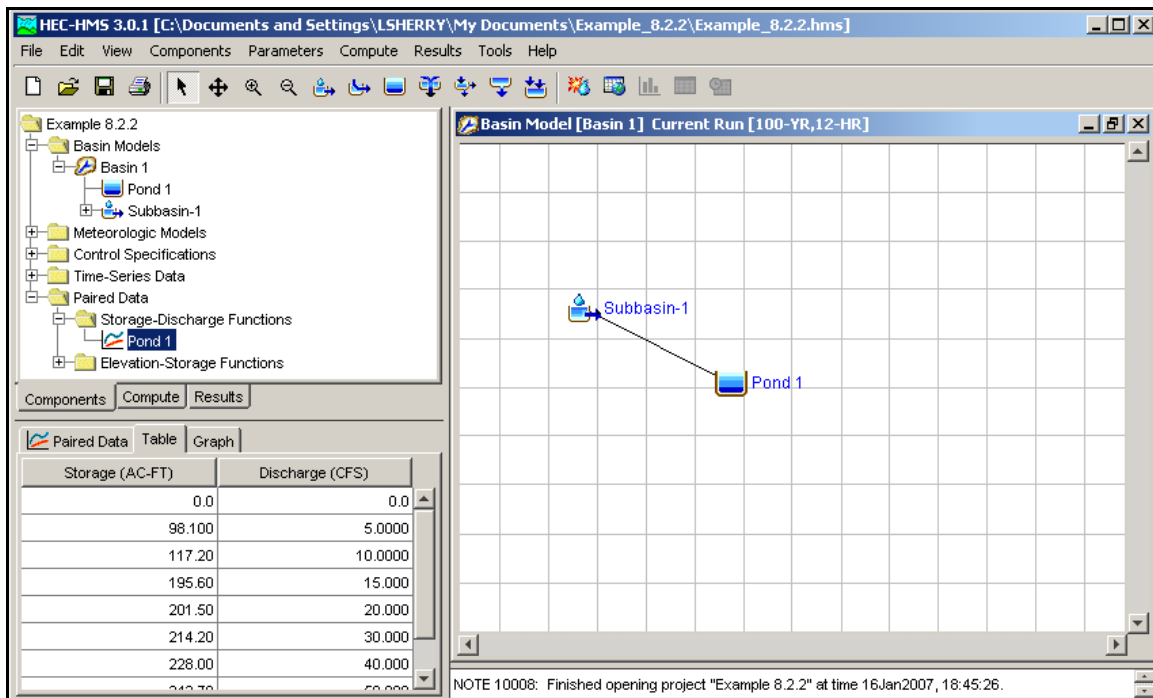


Figure 8.2.3  
Entering Stage-Storage-Discharge for Pond 1

#### 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.

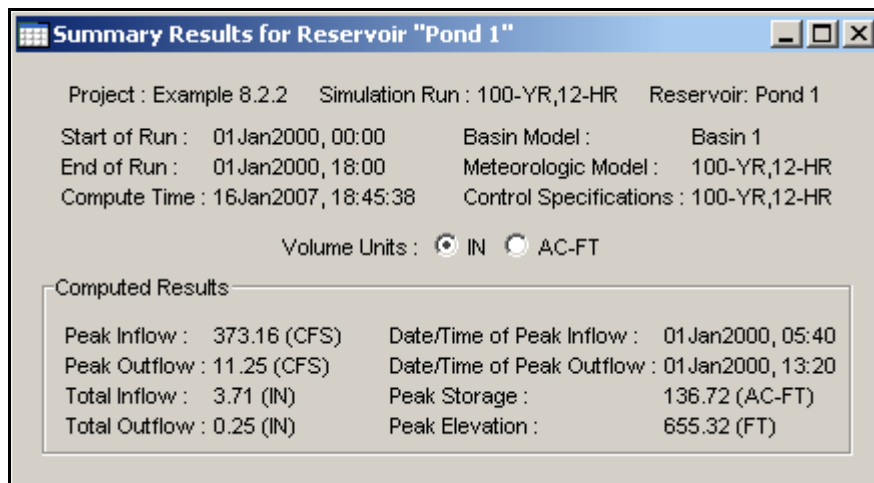


Figure 8.2.4  
HEC-HMS Output for Example 8.2.2

### 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.

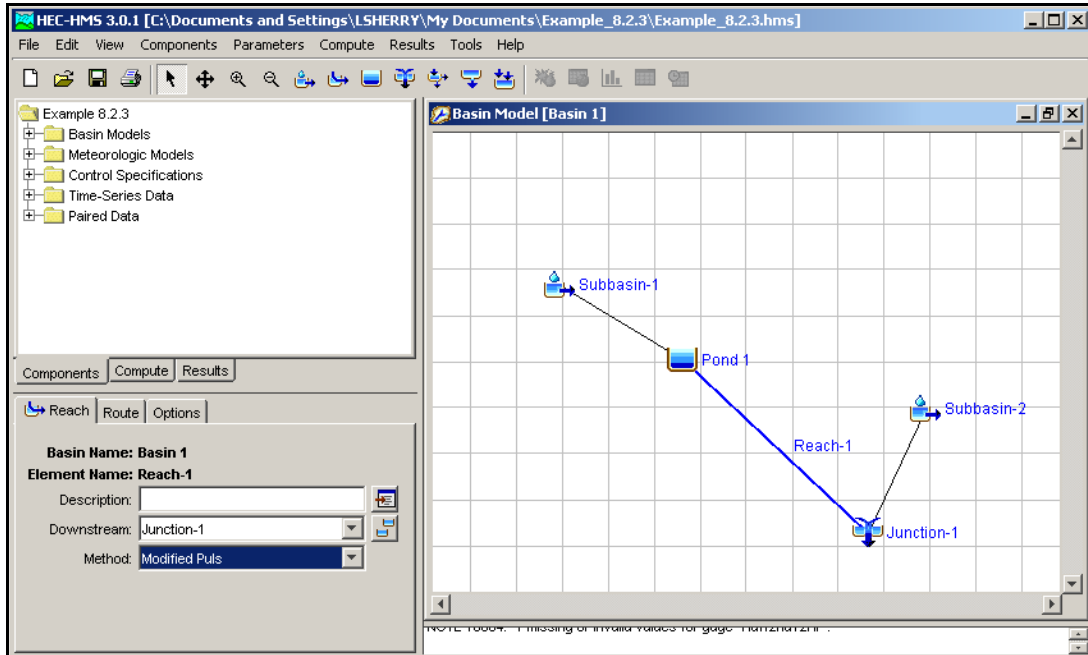


Figure 8.2.5  
Basin Schematic for Example 8.2.3

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.

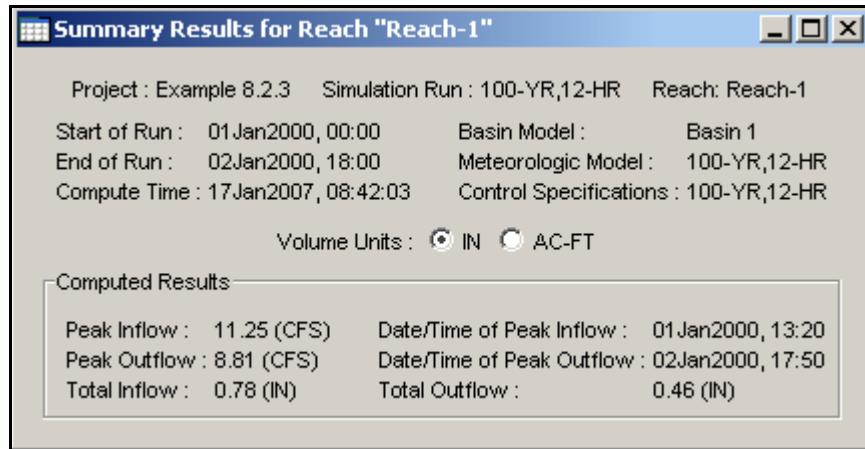


Figure 8.2.6 (a)  
Summary Output for Reach 1

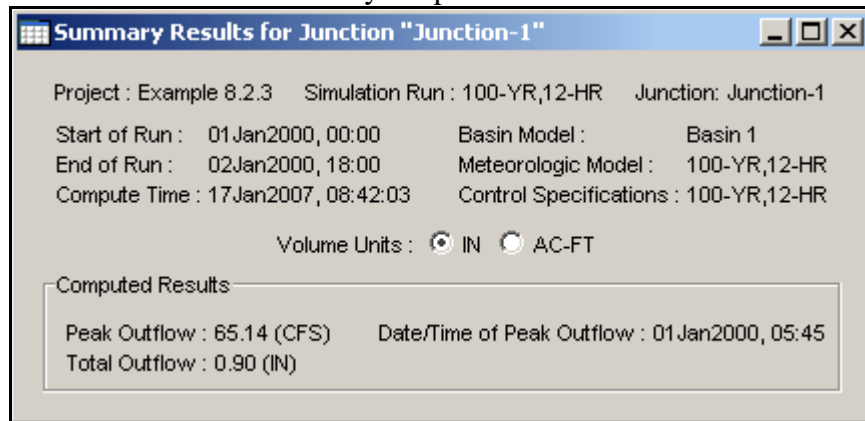
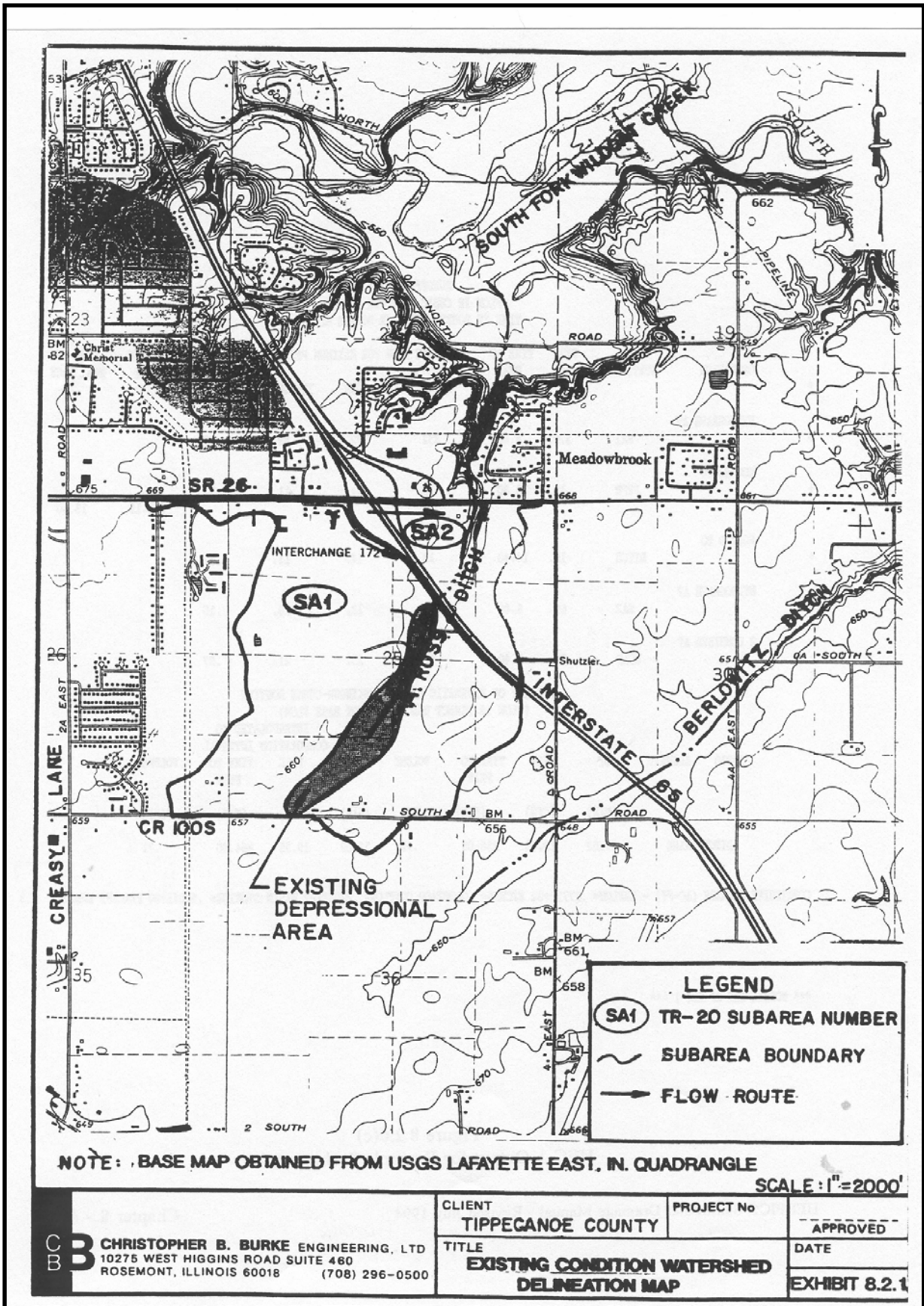


Figure 8.2.6 (b)  
Summary Output for Junction 1

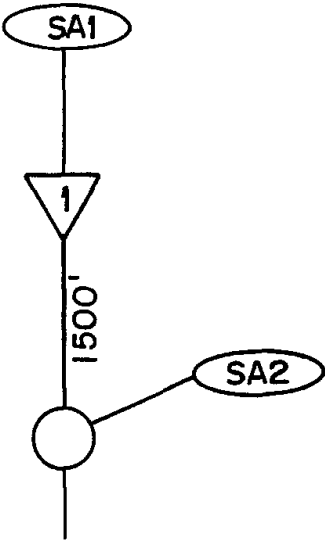
## 5. Summary/Explanation

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.





A. ROSS DITCH  
 EXISTING CONDITION  
 TR-20 MODEL  
 FLOW SCHEMATIC



LEGEND:

- (SA1)** TR-20 SUBAREA
- COMBINE FLOWS
- 1500'** REACH ROUTING
- ▽** RESERVOIR ROUTING

	CLIENT <b>TIPPECANOE COUNTY</b>		PROJECT No _____	-- APPROVED -- _____
	TITLE <b>FLOW SCHEMATIC OF          EXISTING CONDITIONS</b>			DATE _____
	10275 WEST HIGGINS ROAD SUITE 460 ROSEMONT, ILLINOIS 60018 (708) 296-0500			<b>EXHIBIT 8.2.2</b>

## 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](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  $(24 \times 0.2697) = 6.47$  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).

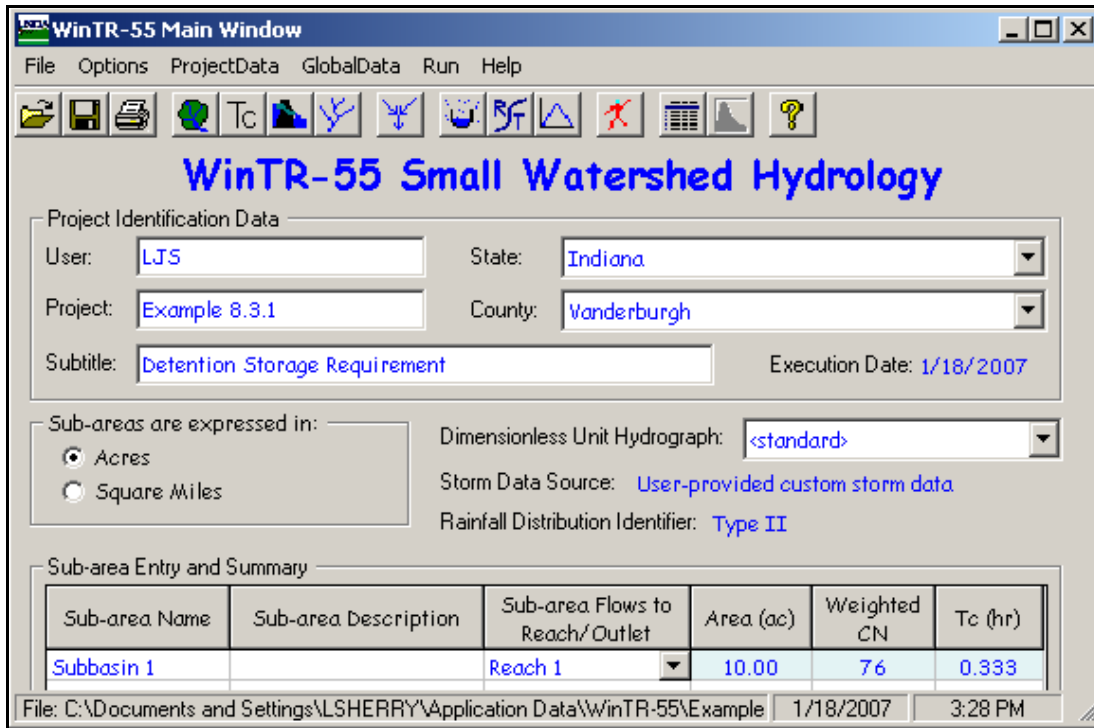


Figure 8.3.1 (a)  
Entry of Project Description and Subbasin Characteristics

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.

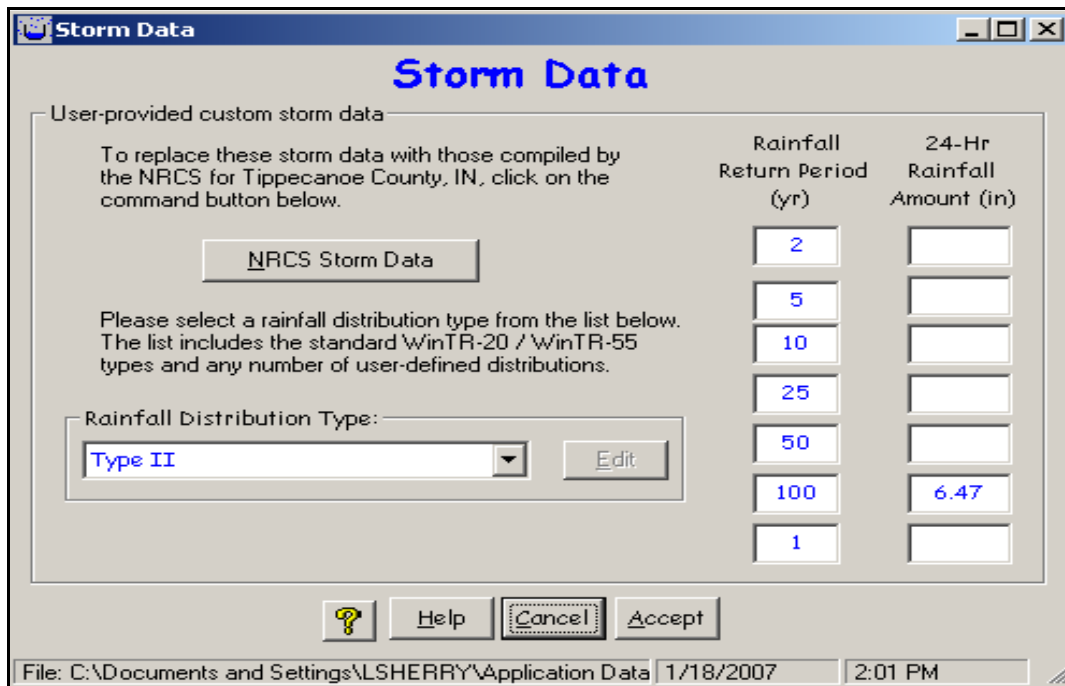
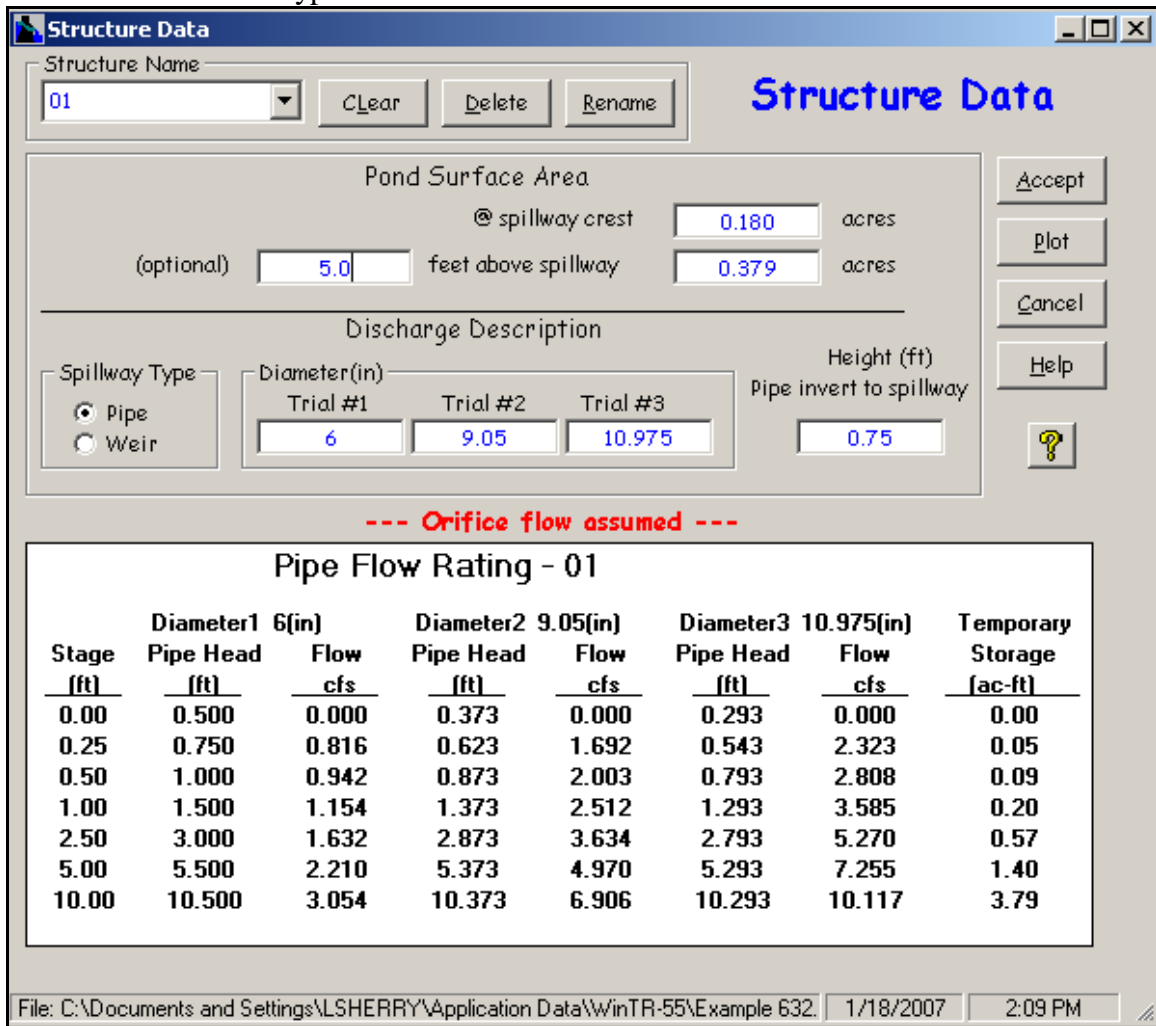


Figure 8.3.1 (b)

## SCS Type II Distribution with 100-Year 24-Hour Rainfall



**Structure Data**

Structure Name:

---

Pond Surface Area

@ spillway crest  acres

(optional)  feet above spillway  acres

---

Discharge Description

Spillway Type:  Pipe  Weir

Diameter(in): Trial #1  Trial #2  Trial #3

Height (ft) Pipe invert to spillway

**--- Orifice flow assumed ---**

Stage (ft)	Diameter1 6(in)		Diameter2 9.05(in)		Diameter3 10.975(in)		Temporary Storage (ac-ft)
	Pipe Head (ft)	Flow cfs	Pipe Head (ft)	Flow cfs	Pipe Head (ft)	Flow cfs	
0.00	0.500	0.000	0.373	0.000	0.293	0.000	0.00
0.25	0.750	0.816	0.623	1.692	0.543	2.323	0.05
0.50	1.000	0.942	0.873	2.003	0.793	2.808	0.09
1.00	1.500	1.154	1.373	2.512	1.293	3.585	0.20
2.50	3.000	1.632	2.873	3.634	2.793	5.270	0.57
5.00	5.500	2.210	5.373	4.970	5.293	7.255	1.40
10.00	10.500	3.054	10.373	6.906	10.293	10.117	3.79

File: C:\Documents and Settings\LSHERRY\Application Data\WinTR-55\Example 632. 1/18/2007 2:09 PM

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.

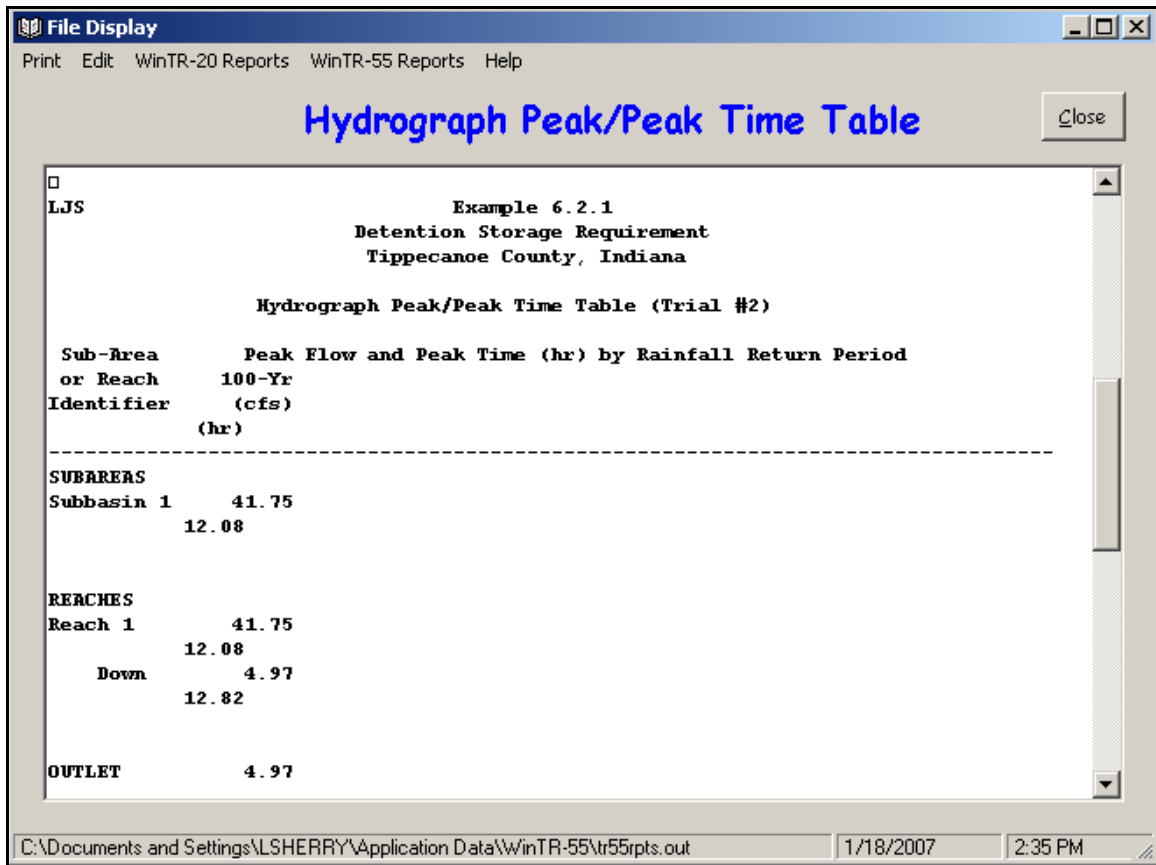


Figure 8.3.2  
Win TR-55 Output for Example 8.3.1

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).

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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.

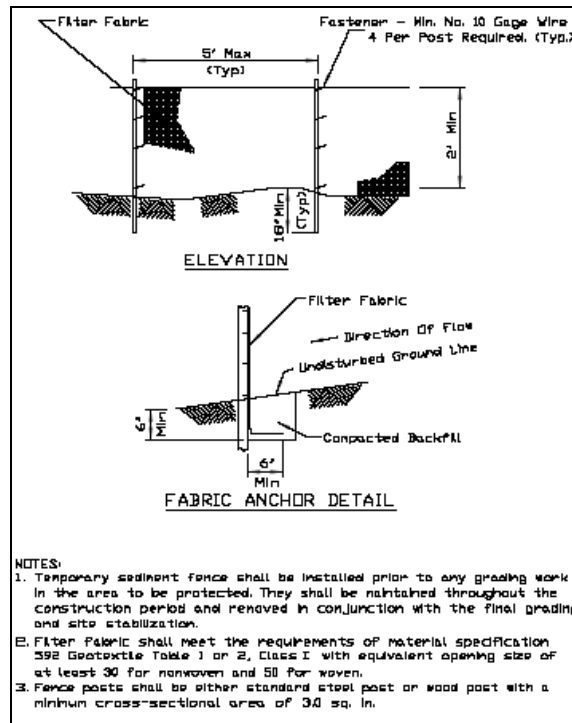


Figure 9.2.1  
Typical Silt Fence Plan (NRCS, 2002)

### 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.

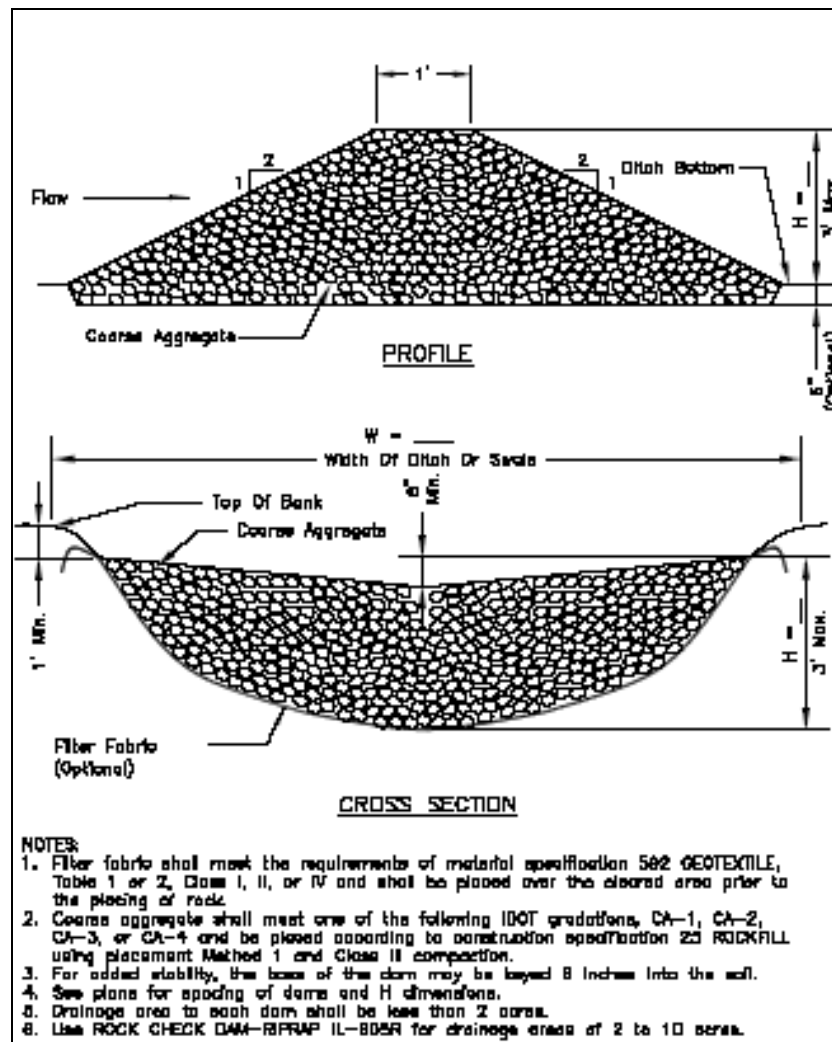


Figure 9.2.2  
Details of Rock Check Dam Installation (NRCS, 2002).



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.

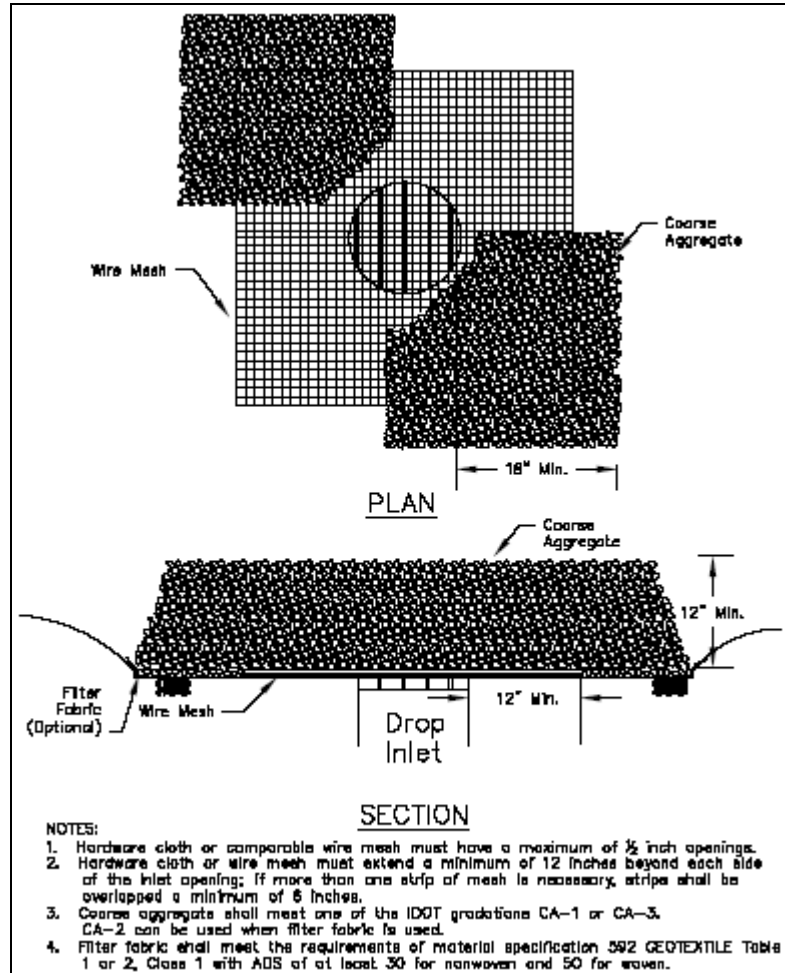


Figure 9.2.4

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.

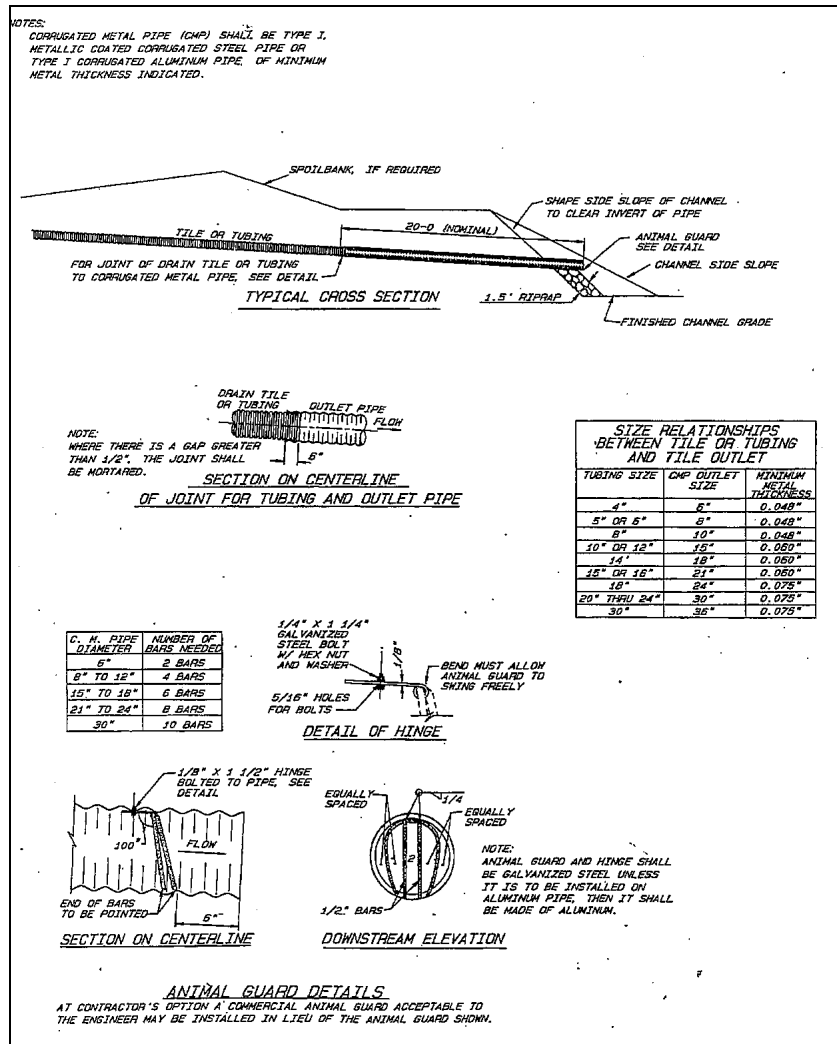


Figure 9.2.5  
 Specification for Typical Outlet Pipe Extension (NRCS, 2002)

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.

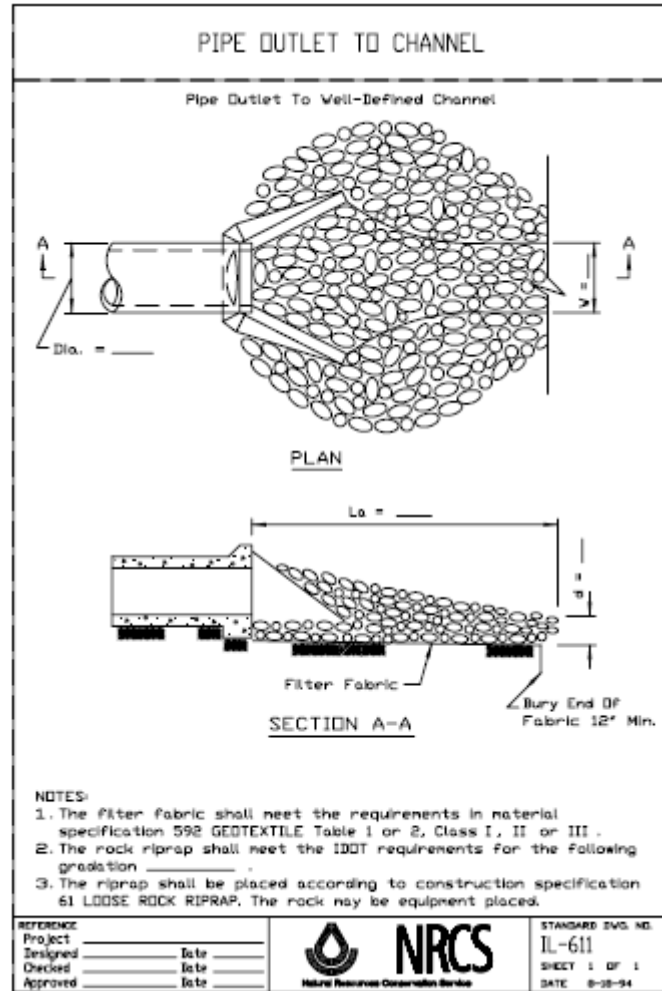


Figure 9.2.6  
Riprap-lined Apron Installation (NRCS, 2002)

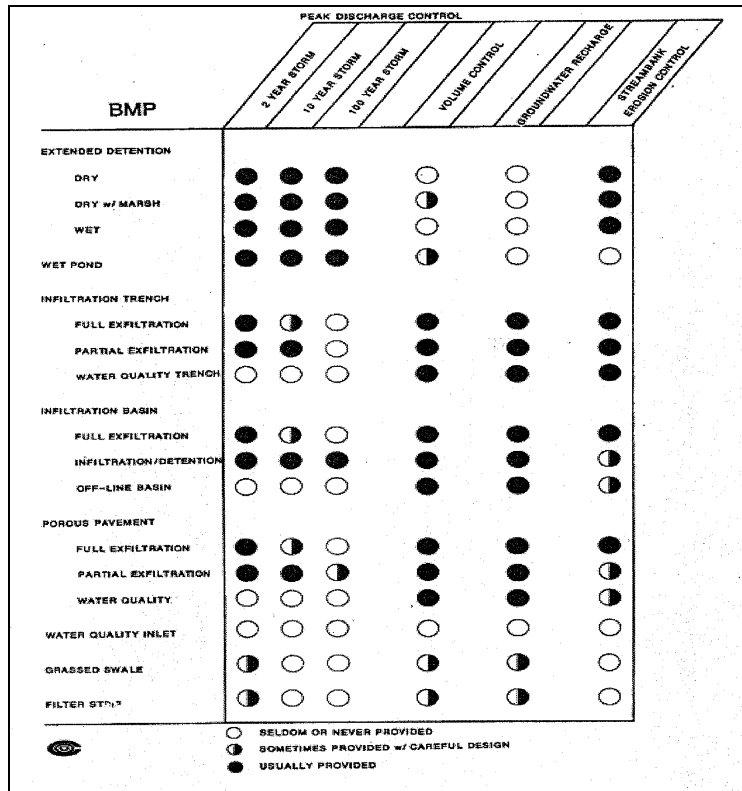


Figure 9.2.7

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.

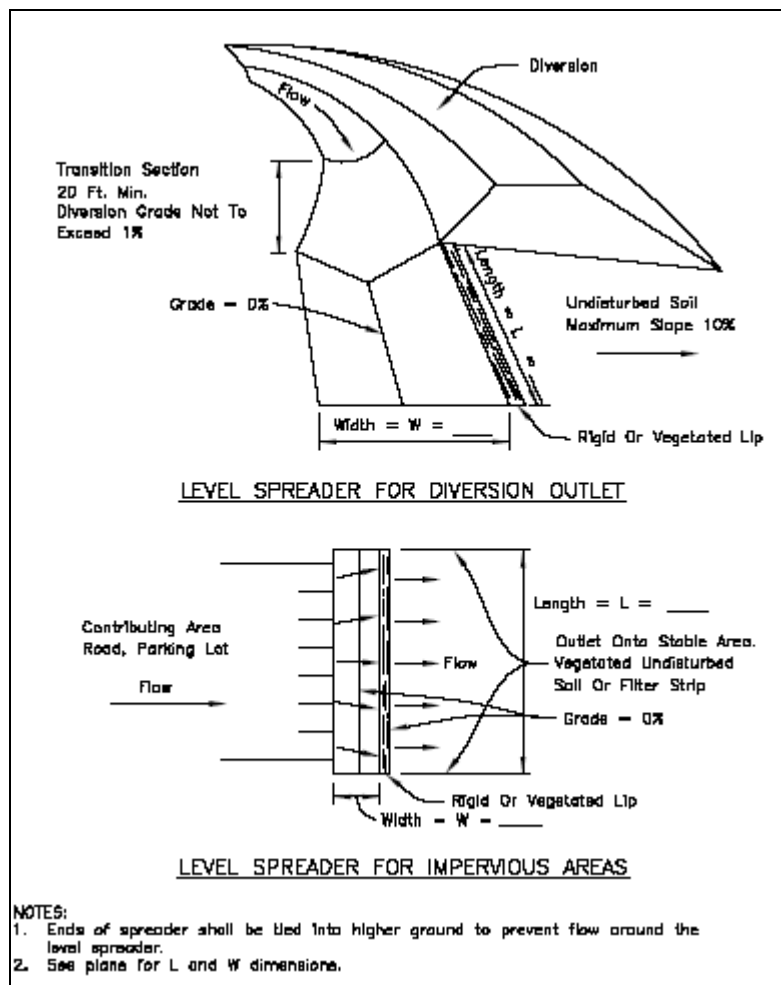


Figure 9.3.1  
 Level Spreader Installation Details (NRCS, 2002)

### 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.

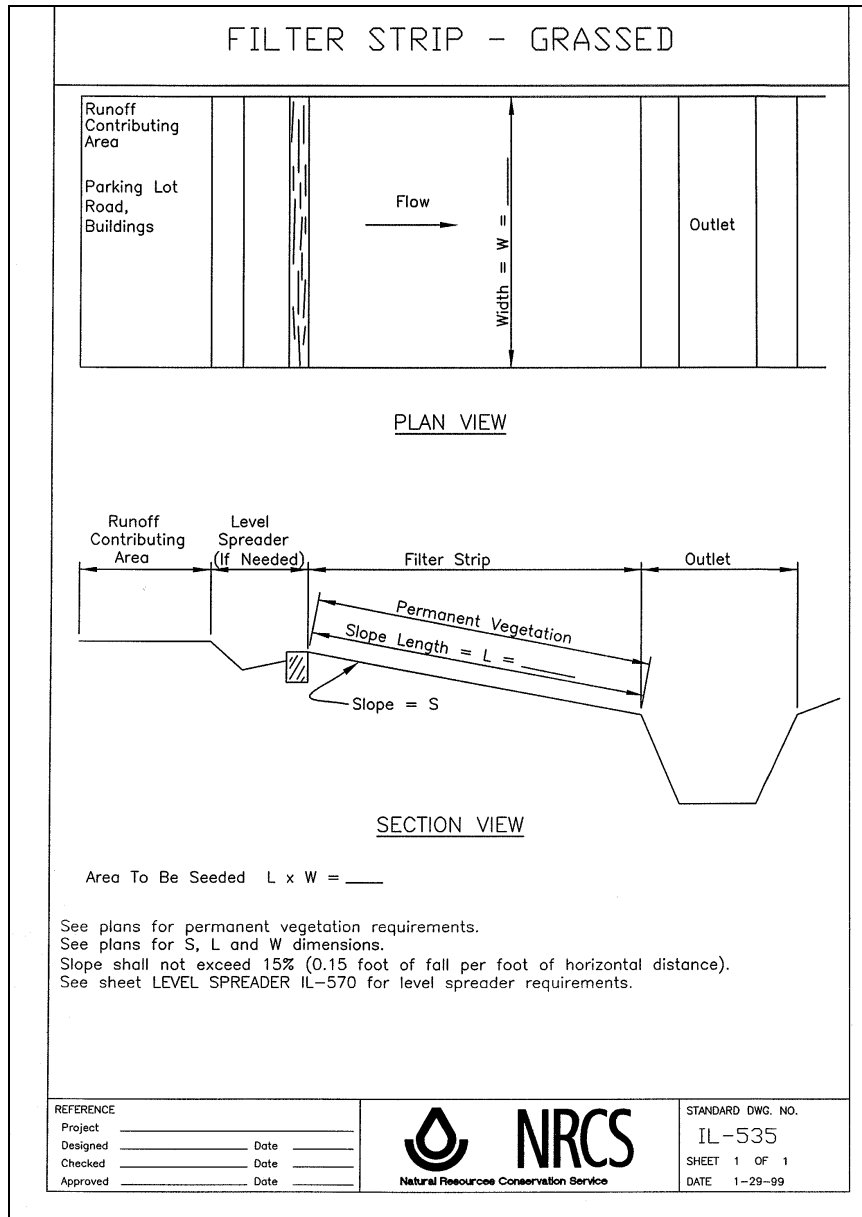


Figure 9.3.2  
Filter Strip Installation Details (NRCS, 2002)

### 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.

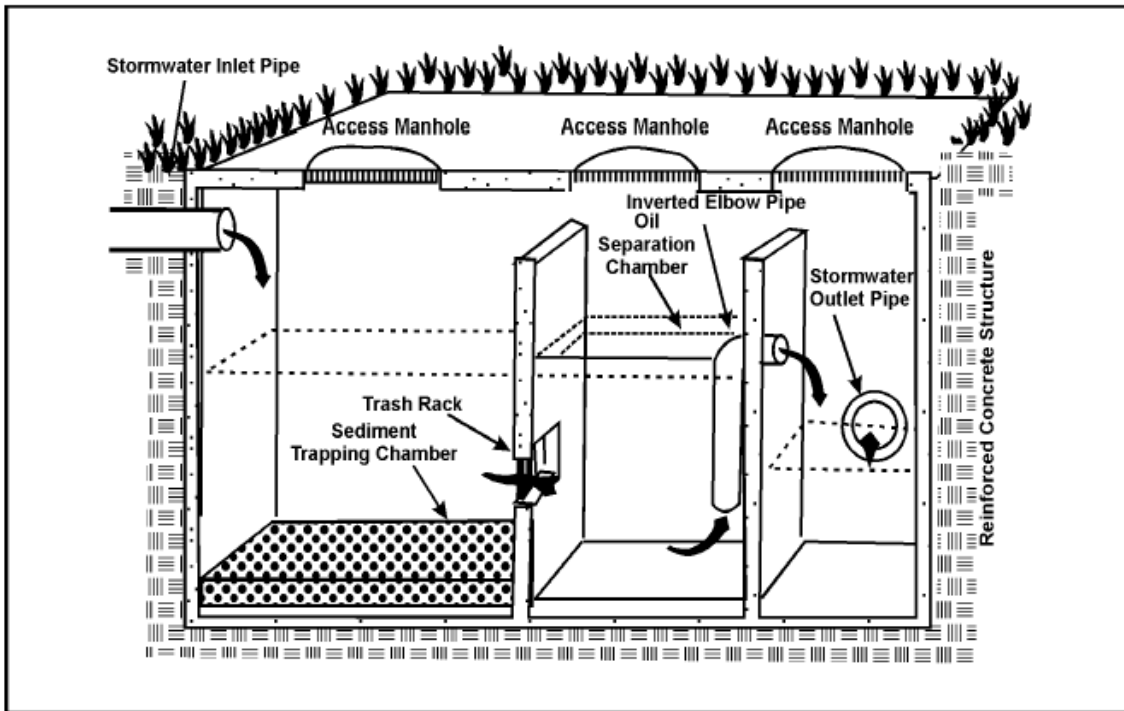


Figure 9.3.3  
Typical WQI (EPA, 1991)

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 (%)	NO <sub>x</sub> -N (%)	Zn (%)	TPH-D (%)	Average Peak Flow Reduction (%)	Average Lag Time (Min.)
<b>ADS Water Quality Unit</b>	<b>UNH</b>	<b>66</b>	<b>N/T</b>	<b>74</b>	<b>47</b>	<b>N/T</b>	<b>N/T</b>
	www.ads-pipe.com	80	N/T	N/T			
<b>ADS Infiltration Unit</b>	<b>UNH</b>	<b>99</b>	<b>N/T</b>	<b>99</b>	<b>99</b>	<b>83</b>	<b>364</b>
<b>Surface Sand Filter</b>	<b>UNH</b>	<b>49</b>	<b>6</b>	<b>81</b>	<b>94</b>	<b>60</b>	<b>220</b>
	EPA: Sand Filters	70	N/T	45			
<b>Sand Filter</b>	Clayton & Schueler, 1996	85	N/T	71			
	Bell, W., et al, 1995	61-70	N/T	> 82			
<b>Retention Pond</b>	<b>UNH</b>	<b>81</b>	<b>64</b>	<b>92</b>	<b>61</b>	<b>85</b>	<b>554</b>
	EPA: Wet Detention Ponds	50-90	N/T	40-50			
	Winer, 2000	80 ± 27	43 ± 38				
<b>Bioretention System</b>	<b>UNH</b>	<b>97</b>	<b>44</b>	<b>99</b>	<b>99</b>	<b>85</b>	<b>615</b>
	EPA: Bioretention	90	N/T	N/T			
	Davis, et al, 1998	81	38				
	Winogradoff, 2001	N/T	N/T	87-99			
<b>Aqua-Swirl and Aqua-Filter</b>	<b>UNH</b>	<b>66</b>	<b>10</b>	<b>61</b>	<b>42</b>	<b>N/T</b>	<b>N/T</b>
	EPA website	84	N/T	N/T			
<b>VortSentry</b>	<b>UNH</b>	<b>29</b>	<b>37</b>	<b>42</b>	<b>53</b>	<b>N/T</b>	<b>N/T</b>
	Technical Bulletin 1	80	N/T	N/T			
<b>V2B1 Structural System</b>	<b>UNH</b>	<b>38</b>	<b>-43</b>	<b>35</b>	<b>40</b>	<b>N/T</b>	<b>N/T</b>
	www.env21.com	80					
<b>Continuous Deflective Separation Unit</b>	<b>UNH</b>	<b>41</b>	<b>N/T</b>	<b>26</b>	<b>26</b>	<b>N/T</b>	<b>N/T</b>
	various	52-84					
<b>Gravel Wetland</b>	<b>UNH</b>	<b>99</b>	<b>99</b>	<b>99</b>	<b>99</b>	<b>85</b>	<b>336</b>
	Clayton & Schueler, 1996	80-93	75-87	55-90			
<b>Stone (Riprap) Swale</b>	<b>UNH</b>	<b>52</b>	<b>-74</b>	<b>66</b>	<b>33</b>	<b>N/T</b>	<b>N/T</b>
<b>Vegetated Swale</b>	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)

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## CHAPTER 10 - GREEN INFRASTRUCTURE

### LIST OF PARAMETERS

$c, d, \alpha, \beta$	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)
$\epsilon$	
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 $\Delta T$ ) (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)
$\theta$	Volumetric water content of green roof
$\theta_s$	Water content of soil post rain fall
$\theta_{in}$	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

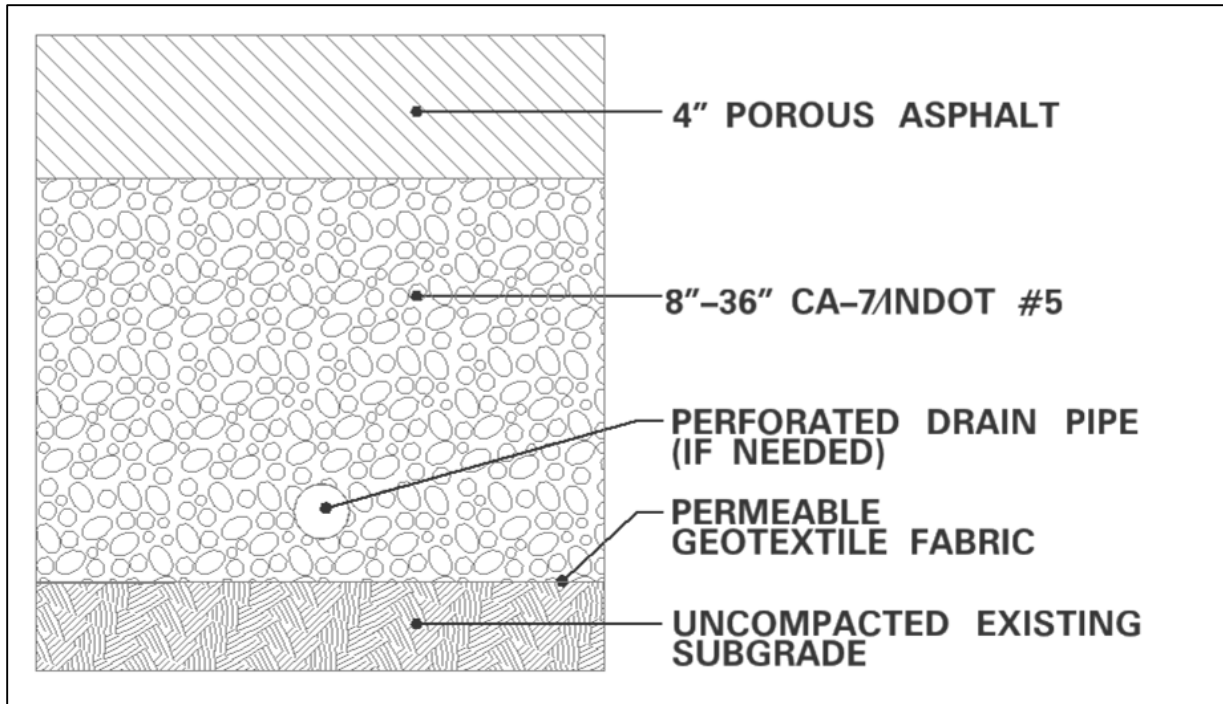


Figure 10.2.2  
 Typical Porous Asphalt Cross-Section Detail for Parking Lot

### 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<sup>3</sup>. 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.



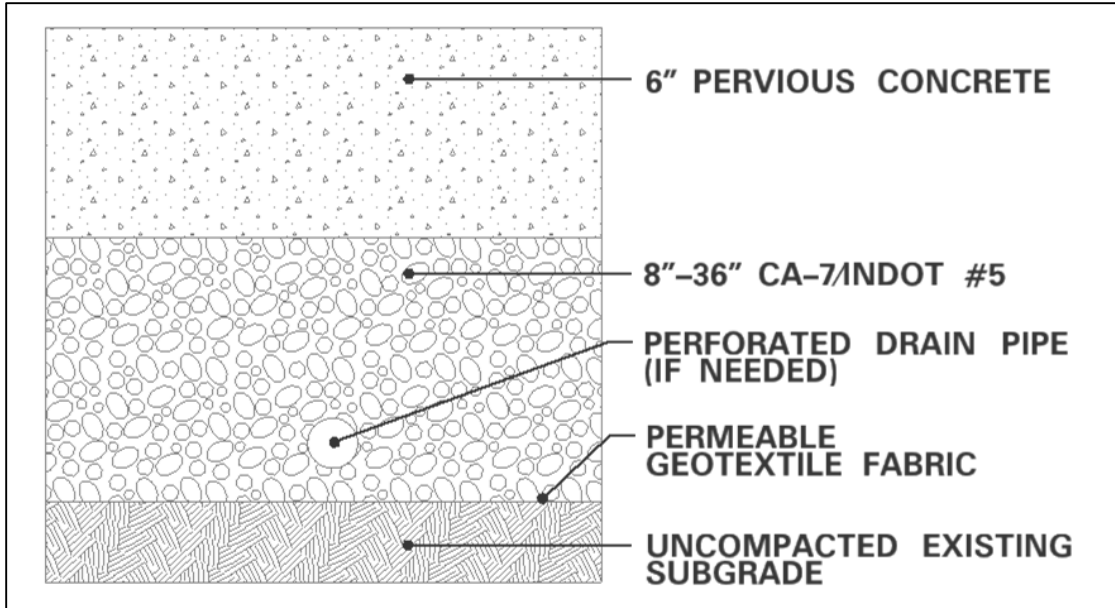


Figure 10.2.3  
 Typical Pervious Concrete Cross-Section Detail for Parking Lot

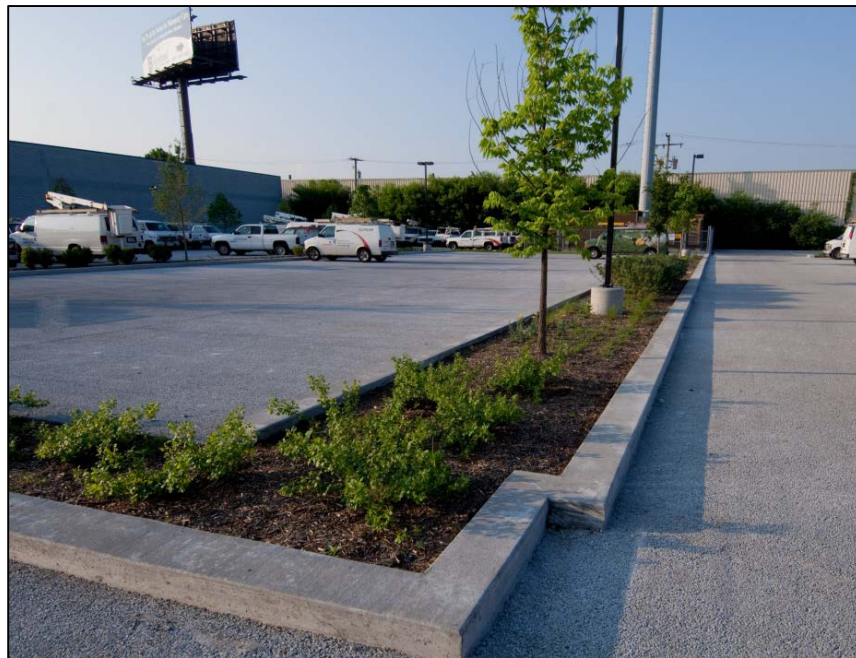


Figure 10.2.4  
 Example of Pervious Concrete Parking Lot (Lutey, 2011)

## 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 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.

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

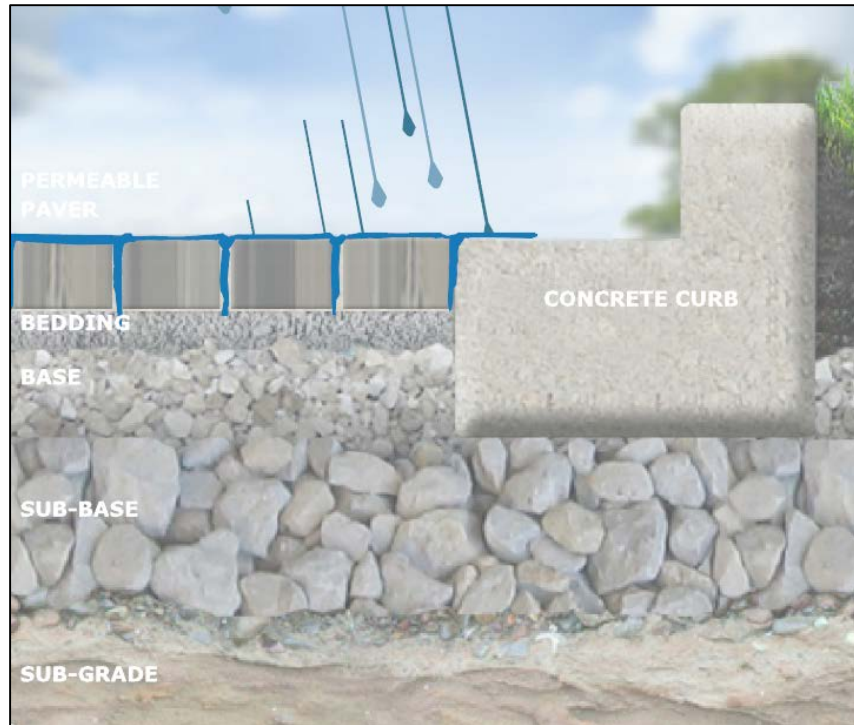


Figure 10.2.6  
Example of a Permeable Paver Parking Lot Cross-Section (APT, 2011)

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 3/4	60 - 70	2 3/8 - 2 3/4	60 - 70	2 3/8 - 2 3/4	60 - 70	2 3/8 - 2 3/4	60 - 70	2 3/8 - 2 3/4	60 - 70
	Bedding Sand	1 1/4	30	1 1/4	30	1 1/4	30	1 1/4	30	1 1/4	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 1/4	30	1 1/4	30	1 1/4	30	1 1/4	30	1 1/4	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 1/4	30	1 1/4	30	1 1/4	30	1 1/4	30	1 1/4	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 1/4	30	1 1/4	30	1 1/4	30	1 1/4	30	1 1/4	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 1/4	30	1 1/4	30	1 1/4	30	1 1/4	30	1 1/4	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 1/4	30	1 1/4	30	1 1/4	30	1 1/4	30	1 1/4	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)

<i>Event #</i>	<i>Date</i>	<i>Storm Duration (hours)</i>	<i>Total Rainfall (inches)</i>	<i>Observed Parking Lot Runoff (inches)</i>
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)

<i>Surface</i>	<i>Observed Mean Infiltration Rates (inches/hr)</i>
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.

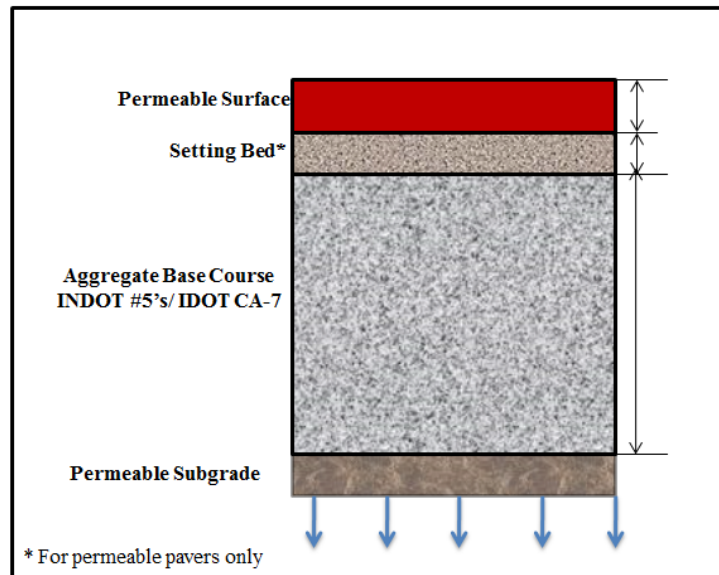


Figure 10.2.7  
Permeable Subgrade Condition

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.

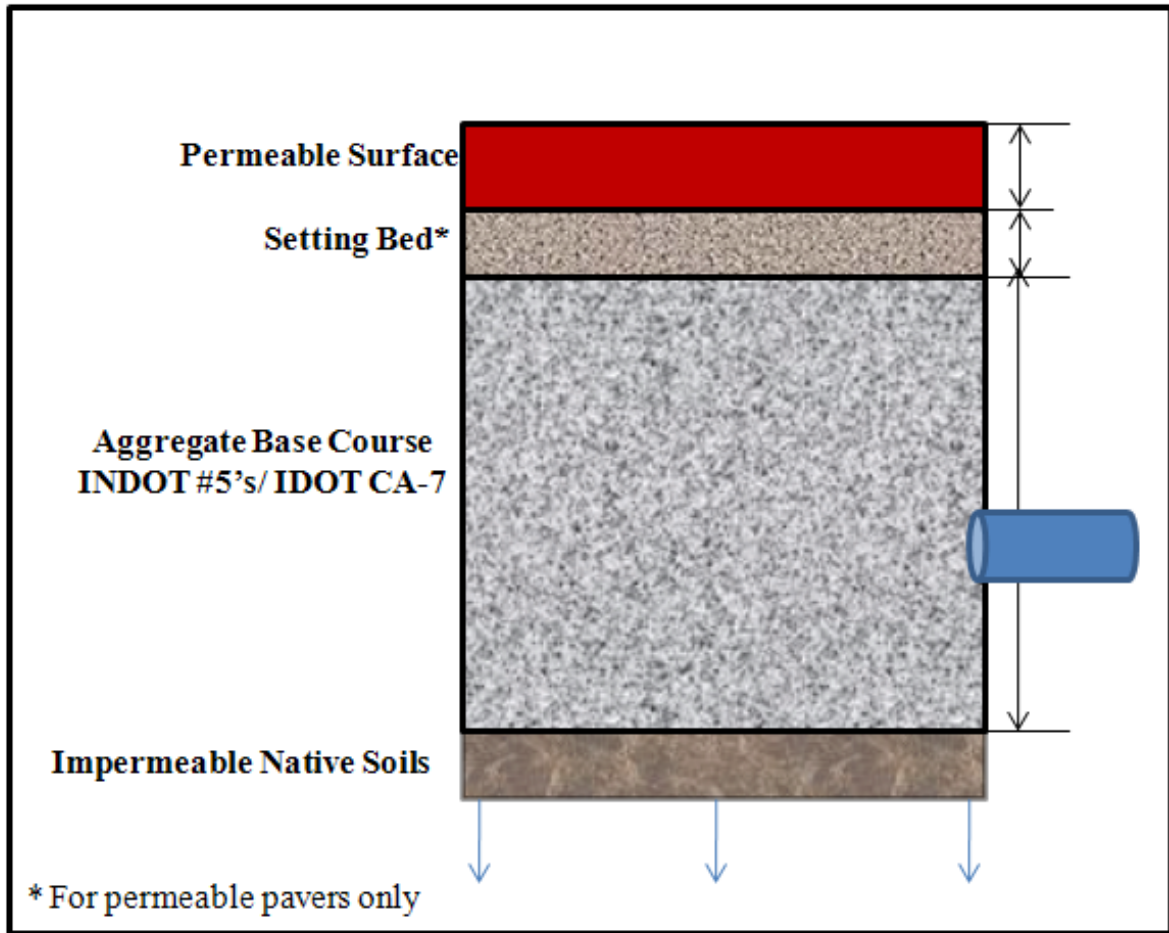


Figure 10.2.8  
Impermeable Subgrade Condition



## 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)

<i>Sample Type</i>	<i>Data Source</i>	<i>Porosity</i>	<i>Bulk Density (lbs/ft<sup>3</sup>)</i>
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
¾"	Anderson Eng. Cons.	38.5%	100.3
¾"	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  $\Delta t$ ).
- $O$  = 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

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### **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.

$$i = \frac{cT_r^\alpha}{(t+d)^\beta} \quad \text{Equation 2.2.13}$$

$$P = i \times d$$

$$\begin{aligned} P_{24}^{100} &= 6.47 \text{ inches} \\ P_{24}^{10} &= 3.93 \text{ inches} \\ P_{24}^5 &= 3.38 \text{ inches} \end{aligned}$$

Step 2: Calculate the stormwater runoff volume using Equation 3.3.1 and a CN of 98.

$$S = \left(\frac{1000}{CN}\right) - 10 \quad \text{Equation 3.3.2}$$

$$S_{98} = 0.20$$

$$R(t) = \frac{((P(t) - 0.2S)^2)}{P(t) + 0.8S} \quad \text{Equation 3.3.1}$$

$$\begin{aligned} R_{24}^{100} &= 6.23 \text{ inches} \\ R_{24}^{10} &= 3.70 \text{ inches} \end{aligned}$$

$$R_{24}^5 = 3.15 \text{ inches}$$

Multiply the runoff by the area of the parking lot to determine the total volume of the runoff.

$$Q_v = P(t) \times \frac{ft}{12 \text{ inches}} \times 200 \text{ ft} \times 200 \text{ ft}$$

$$Q_{24}^{100} = 20,778 \text{ ft}^3$$

$$Q_{24}^{10} = 12,319 \text{ ft}^3$$

$$Q_{24}^5 = 10,498 \text{ ft}^3$$

Step 3: Compute the volume of storage provided below the parking lot for the proposed sub-base depths.

$$V_{6"} = 200 \text{ ft} \times 200 \text{ ft} \times 6" \times \frac{ft}{12"} \times 0.36$$

$$V_{6"} = 7,200 \text{ ft}^3$$

$$V_{12"} = 14,400 \text{ ft}^3$$

$$V_{18"} = 21,600 \text{ ft}^3$$

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<sup>2</sup>. 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

<i>Roof Cover</i>	<i>Curve Number</i>
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.

$$i = \frac{cT_R^\alpha}{(t + d)^\beta}$$

$$i = \frac{1.5899 * 50^{0.2771}}{(2 + 0.725)^{0.8797}} = 1.95 \text{ in/hr}$$

$$P[t] = (1.95 \text{ in/hr}) * 2 \text{ hrs} = 3.89 \text{ in}$$

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

<i>Roof Cover</i>	<i>Runoff (in)</i>	<i>Total Runoff Volume (ft<sup>3</sup>)</i>
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.

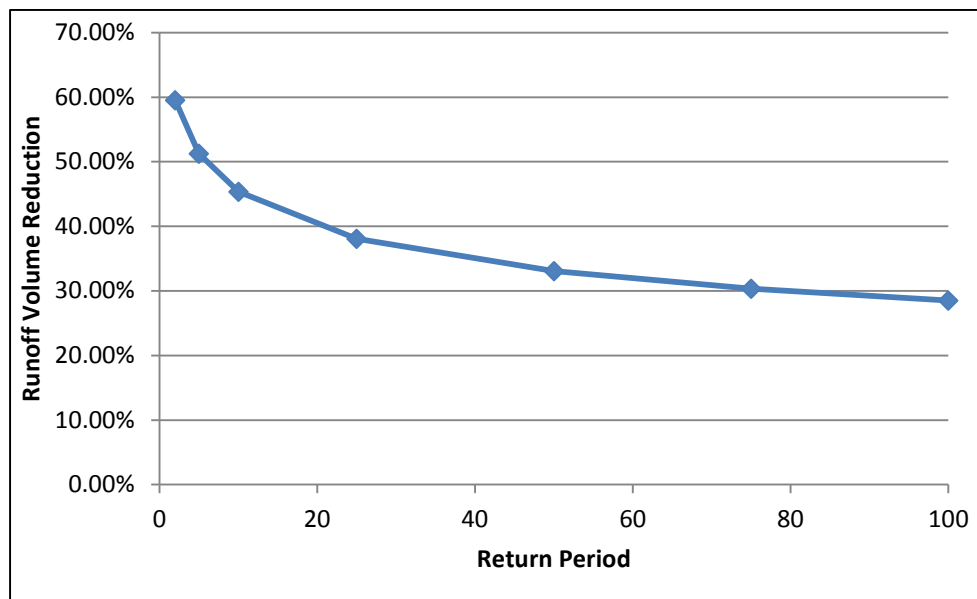


Figure 10.3.2  
Runoff Volume Reduction versus Return Period

## 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)

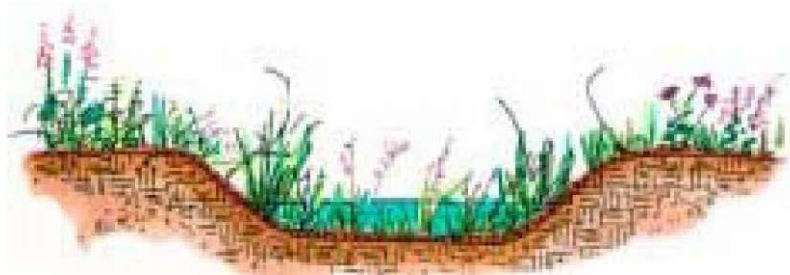


Figure 10.4.2  
Naturalized Infiltration Basin (Improving Stormwater Basin Design, 2008)

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

<i>Typical Soil</i>	<i>Permeability, K (ft/sec)</i>	<i>Relative Permeability</i>
Coarse, Gravel	Over $5.0 \times 10^{-1}$	Very Permeable
Sand, Fine Sand	$5.0 \times 10^1$ to $5.0 \times 10^{-3}$	Medium Permeability
Silty Sand, Dirty Sand	$5.0 \times 10^{-3}$ to $5.0 \times 10^{-5}$	Low Permeability
Silt	$5.0 \times 10^{-5}$ to $5.0 \times 10^{-7}$	Very Low Permeability
Clay	Less than $5.0 \times 10^{-7}$	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, reseeded 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 ( $\theta_s$ ) and soil initial water content ( $\theta_o$ ). The volume required can be determined using a number of different methods, including those discussed in Chapter 6.

$$A_{bottom} \geq \frac{V_{required}}{Z(\theta_s - \theta_o)} \quad \text{Equation 10.4.1}$$

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.

$$h \leq Z(\theta_s - \theta_o) \quad \text{Equation 10.4.2}$$

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 ( $f$ ).

$$T_p = \frac{h}{f} \quad \text{Equation 10.4.3}$$

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.

$$Q = fA \quad \text{Equation 10.4.4}$$

## 10.4 - EXAMPLE PROBLEMS

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### **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:

$$A_{\text{bottom}} \geq \frac{0.48 \text{ ac} - \text{ft}}{12 \text{ ft} * (0.35 - 0.10)}$$

$$A_{\text{bottom}} \geq 0.16 \text{ ac}$$

$$A_{\text{bottom}} \geq 6,970 \text{ ft}^2$$

$$h \leq 12 (0.35 - 0.10)$$

$$h \leq 2.4 \text{ ft}$$

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:

$$f = \frac{2.4 \text{ ft} * 12 \frac{\text{in}}{\text{ft}}}{24 \text{ hrs}}$$

$$f = 1.2 \text{ in/hr}$$

**Step 3:** If the basin is designed as an 85-foot by 85-foot square, the infiltration volume rate can be estimated:

$$Q = fA$$

$$Q = 1.2 \frac{\text{in}}{\text{hr}} * (85 * 85) \text{ ft}^2$$

$$Q = 0.20 \text{ cfs}$$

### **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?

$$Q = fA = f\pi R^2$$

$$Q = 1.5 \frac{\text{in}}{\text{hr}} * \pi * (50 \text{ ft})^2$$

$$Q = 0.27 \text{ cfs}$$

## 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.

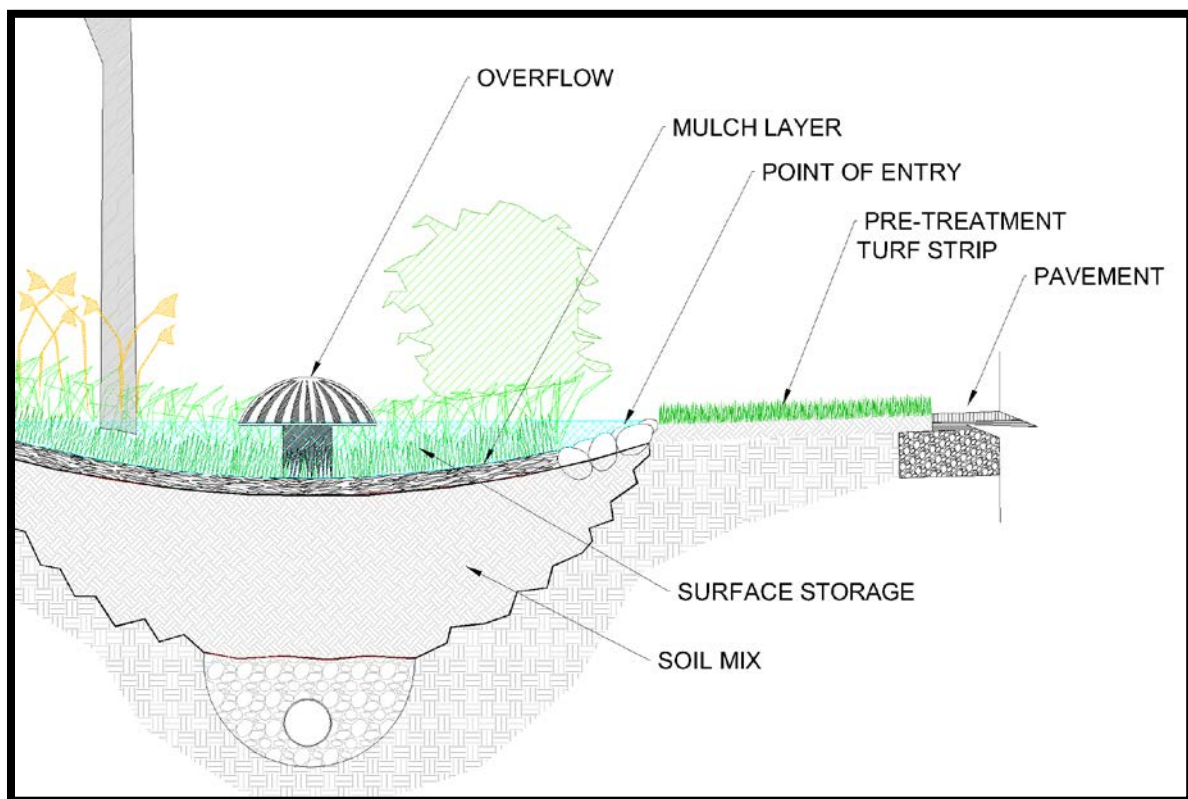


Figure 10.5.1  
Typical Rain Garden Cross-Section

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)

<i>Rainfall Depth (inches)</i>	<i>Indianapolis International Airport</i>	<i>Fort Wayne Airport</i>	<i>South Bend Regional Airport</i>	<i>Evansville Museum</i>
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.

$$\begin{array}{ll} \text{Area of Roof} = 500 \text{ ft}^2 & \text{CN}_{\text{Roof}} = 98 \\ \text{Area of Grass} = 5,000 \text{ ft}^2 & \text{CN}_{\text{Grass}} = 80 \end{array}$$

Step 1: Calculate the stormwater runoff volume from both the grassed and roof area using Equation 3.3.1 for varying rainfall depths.

$$S = \left( \frac{1000}{CN} \right) - 10 \quad \text{Equation 3.3.2}$$

$$S_{98} = 0.20 \text{ inches} \quad S_{80} = 2.50 \text{ inches}$$

$$R(t) = \frac{((P(t) - 0.2S)^2)}{P(t) + 0.8S} \quad \text{Equation 3.3.1}$$

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

<i>Rainfall Depth (inches)</i>	<i>Runoff from Grassed Area (inches)</i>	<i>Runoff from Roof Area (inches)</i>
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.

$$Q_v = R_{\text{Grass}} \times \frac{1 \text{ ft}}{12 \text{ in}} \times A_{\text{Grass}} + R_{\text{Roof}} \times \frac{1 \text{ ft}}{12 \text{ in}} \times A_{\text{Roof}}$$

$$\begin{aligned} Q_{0.25''} &= 4.41 \text{ ft}^3 \\ Q_{0.50''} &= 13.25 \text{ ft}^3 \\ Q_{0.75''} &= 24.08 \text{ ft}^3 \\ Q_{1.00''} &= 37.12 \text{ ft}^3 \end{aligned}$$

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.

$$\begin{aligned} \text{Volume} &= \frac{\text{Area of Top} + \text{Area of Bottom}}{2} \times \text{Depth} \\ \text{Volume} &= \frac{(10 \text{ ft} \times 10 \text{ ft}) + (4 \text{ ft} \times 4 \text{ ft})}{2} \times 0.5 \text{ ft} = 29 \text{ ft}^3 \end{aligned}$$

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<sup>3</sup> 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.

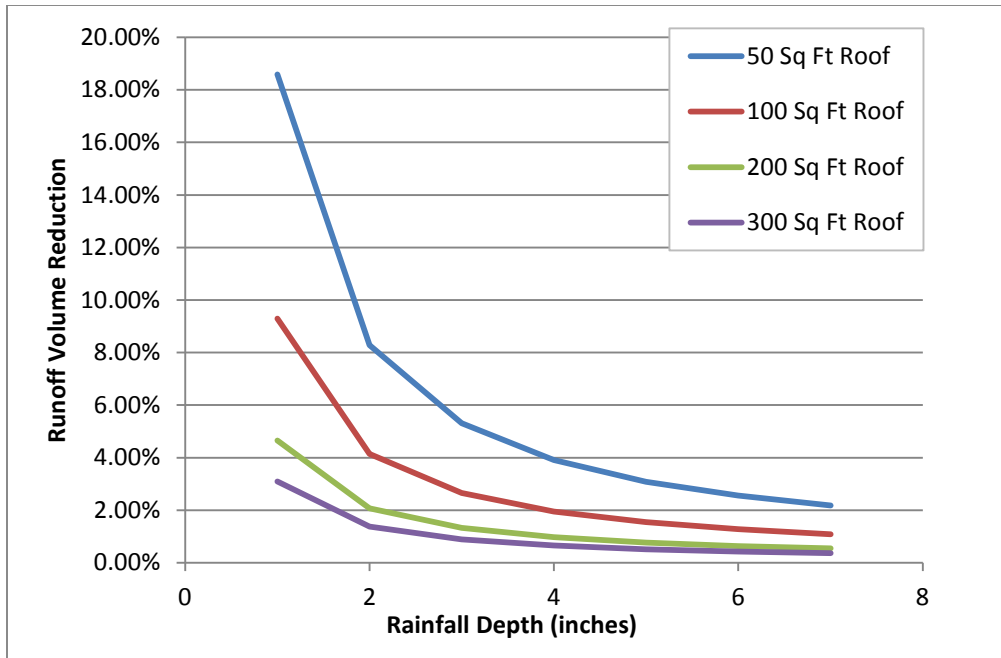


Figure 10.6.3  
Runoff Reduction on Varying Sized Roofs Due to One Rain Barrel

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

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### 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.

$$55 \text{ gallons} \cong 7.35 \text{ ft}^3$$

$$CN_{\text{ROOF}} = 98$$

$$S_{\text{roof}} = 0.20$$

$$R = 0.79 \text{ inches} = 0.066 \text{ feet}$$

$$V = (0.066 \text{ ft} * 1000 \text{ sq ft})$$

$$V = 66 \text{ ft}^3$$

$$\# \text{ of barrels needed} = \frac{66 \text{ ft}^3}{7.4 \text{ ft}^3}$$

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:

$$R_W = \frac{(P-0.2S)^2}{P+0.8S} \quad \text{Equation 3.3.1}$$

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:

$$S = \left(\frac{1000}{CN}\right) - 10 \quad \text{Equation 3.3.2}$$

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:

$$V_W = R_W \times \frac{1 \text{ ft}}{12 \text{ in}} \times A_W$$

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.

## 10.8 - EXAMPLE PROBLEM

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### 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:

$$V_w = \frac{R_w}{12} \times A_w = \frac{5}{12} \times 10 \text{ ac} = 4.17 \text{ ac} - \text{ft}$$

Step 3: The volume associated with the total impervious area that must be stored is:

$$V_R = 3 \text{ ac} \times \frac{1 \text{ ft}}{12 \text{ in}} = 0.25 \text{ ac} - \text{ft}$$

For this example,  $V_{GI} = 0$ , so the adjusted runoff volume is:

$$V_{ADJ} = 4.17 \text{ ac} - \text{ft} - 0.25 \text{ ac} - \text{ft} = 3.92 \text{ ac} - \text{ft}$$

Step 4: And therefore:

$$12 \times \frac{V_{ADJ}}{A_w} = \frac{(P - 0.2S)^2}{(P + 0.8S)} = 4.70 \text{ in}$$

Since  $P = 7.58''$ :

$$4.70 \text{ in} = \frac{(7.58'' - 0.2S)^2}{(7.58'' + 0.8S)}$$

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

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### **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

<i>Rainfall Depth (in)</i>	<i>R(t) (inches)</i>	<i>Runoff Volume (ft<sup>3</sup>)</i>
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

<i>Rainfall Depth (in)</i>	<i>R(t) (inches)</i>	<i>Runoff Volume (ft<sup>3</sup>)</i>	<i>Runoff Volume Provided in 2 Rain Barrels (ft<sup>3</sup>)</i>	<i>Total Runoff Volume (ft<sup>3</sup>)</i>
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

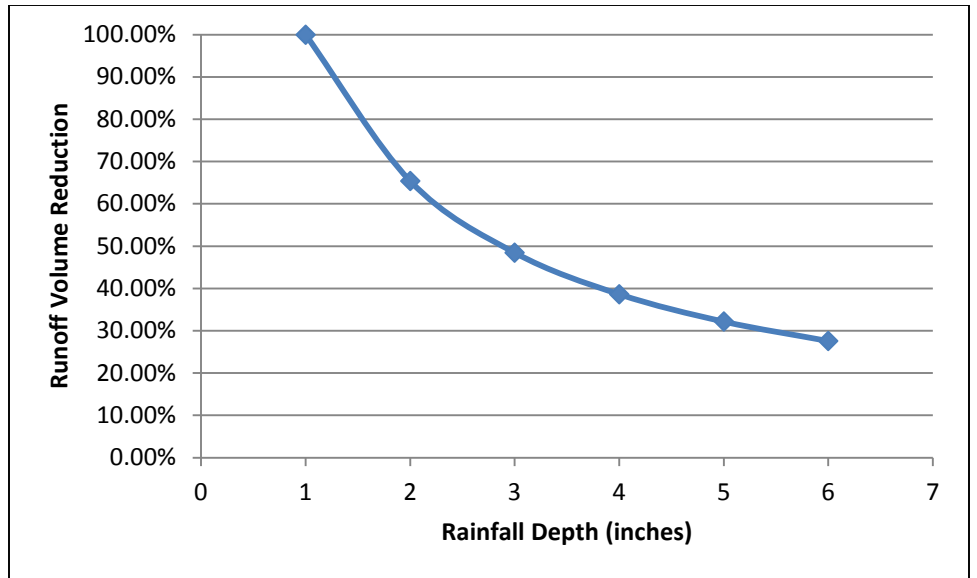


Figure 10.9.1  
Surface Runoff Volume Reduction

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%.

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### 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.

$$\begin{aligned}
 7.48 \text{ gallons} &= 1.00 \text{ ft}^3 \\
 500 \text{ gallons} &= 66.8 \text{ ft}^3 \\
 1,000 \text{ gallons} &= 133.7 \text{ ft}^3 \\
 2,000 \text{ gallons} &= 267.4 \text{ ft}^3
 \end{aligned}$$

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.

---



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### Example 10.9.3

A residential dwelling is located on a 15,000 ft<sup>2</sup> lot. The driveway is 45 feet long and 20 feet wide, the house footprint is 1,800 ft<sup>2</sup> 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.

$$S = \left( \frac{1000}{CN} \right) - 10 \quad \text{Equation 3.3.2}$$

$$R = \frac{(P - 0.2S)^2}{P + 0.8S} \quad \text{Equation 3.3.1}$$

In Equation 3.3.1, if  $P \leq 0.2S$ , then R is equal to zero.

$$Q_v = R \times \frac{1 \text{ ft}}{12 \text{ in}} \times A$$

Table 10.9.4  
Runoff Volume (ft<sup>3</sup>) for Corresponding Rainfall Depths

<i>Surface</i>	<i>Area (ft<sup>2</sup>)</i>	<i>CN</i>	<i>Rainfall Depth (in)</i>						
			<i>0.25</i>	<i>0.50</i>	<i>1.00</i>	<i>1.50</i>	<i>2.00</i>	<i>2.50</i>	<i>3.00</i>
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
<i>Total</i>	<i>15,000</i>	<i>---</i>	<i>58.04</i>	<i>71.92</i>	<i>249.87</i>	<i>553.06</i>	<i>934.77</i>	<i>1,369.28</i>	<i>1,841.04</i>

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.

$$\begin{aligned} \text{Porous Pavement Storage Volume} &= 45 \text{ ft} \times 20 \text{ ft} \times 24 \text{ inches} \times \frac{\text{ft}}{12 \text{ inches}} \times 0.36 \\ &= 648 \text{ ft}^3 \end{aligned}$$

$$\text{Rain Barrel Storage Volume} = 4 \times 55 \text{ gallons} \times \frac{\text{ft}^3}{7.4 \text{ gallons}} = 29.7 \text{ ft}^3$$

$$\text{Rain Garden Storage Volume} = 29 \text{ ft}^3 \text{ (see Example 10.5.1)}$$



Table 10.9.5  
Total Runoff Volume Reduction from Green Infrastructure System

<i>Rainfall Depth</i>	<i>0.25</i>	<i>0.50</i>	<i>1.00</i>	<i>1.50</i>	<i>2.00</i>	<i>2.50</i>	<i>3.00</i>
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<sup>3</sup>. 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

$\alpha, u$	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:

$$\bar{x} = x_m = \frac{1}{n} \sum_{i=1}^n x_i \quad (\text{A.2.1})$$

where  $x_m$  is the mean of the  $x_i$  data set.

The log mean is written as:

$$\log x_m = \sum_{i=1}^n \frac{\log x_i}{n} \quad (\text{A.2.2})$$

### 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:

$$s_d = \left[ \frac{1}{n-1} \sum_{i=1}^n (x_i - x_m)^2 \right]^{1/2} \quad s'_d = \left[ \frac{1}{n-1} \sum (\log x_i - \log x_m)^2 \right]^{1/2} \quad (\text{A.2.3})$$

The skewness indicates the weighting of the distribution to either side of the mean. It is determined by:

$$g = \frac{1}{s_d^3} \frac{n}{(n-1)(n-2)} \sum_{i=1}^n [x_i - x_m]^3 \quad (\text{A.2.4})$$

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.

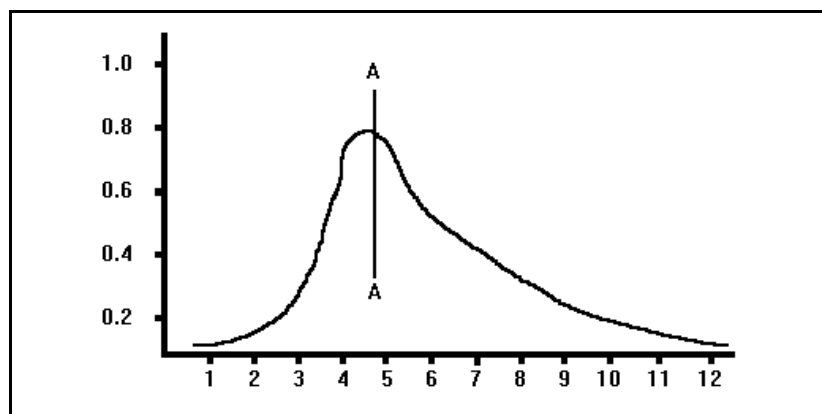


Figure A.3.1  
Probability Distribution Function

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.

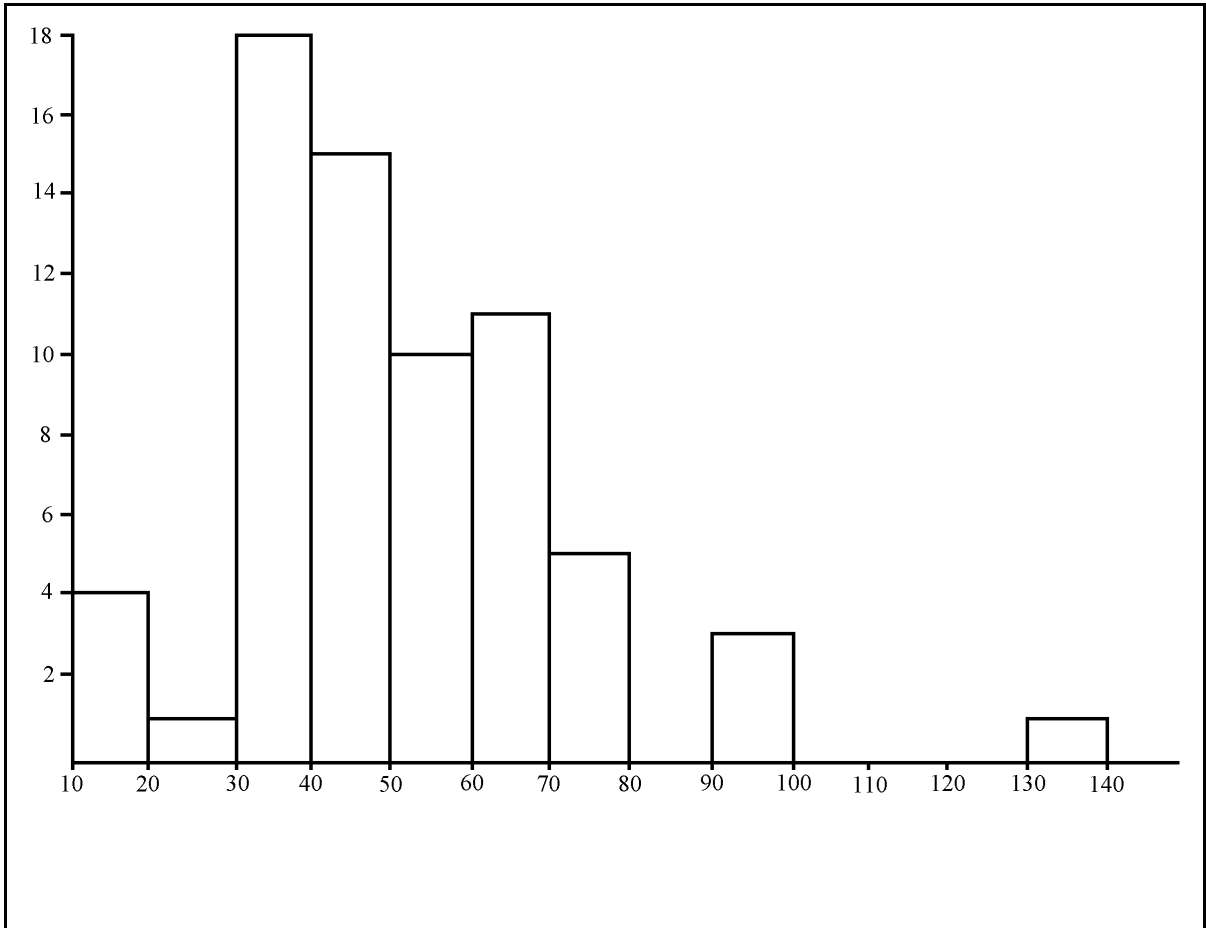


Figure A.3.2  
Histogram of the Annual Maximum Flows of the  
Wabash River at Lafayette, Indiana (1924 – 1992)

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.

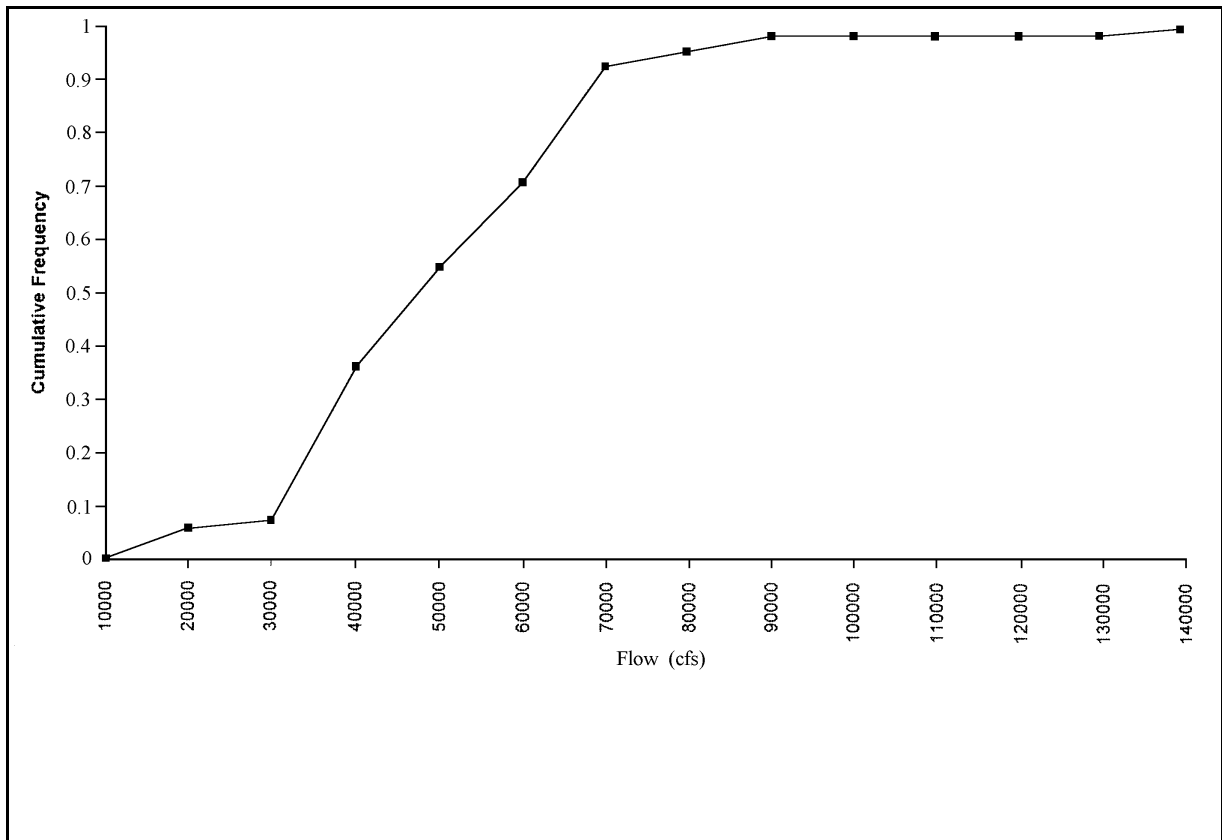


Figure A.3.3  
Cumulative Distribution of Maximum Annual Flows of the  
Wabash River at Lafayette, Indiana (1924 – 1992)

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:

$$0 \leq P(E_1) \leq 1 \quad (\text{A.3.1})$$

It is observed from Figure A.3.3 that the sum of the probability estimates of the events is equal to one.

$$P(E_1) + P(E_2) + P(E_3) + \dots + P(E_n) = \sum_{i=1}^n P(E_i) = 1 \quad (\text{A.3.2})$$

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:

$$P(E'_1) + P(E_1) = 1 \quad (\text{A.3.3})$$

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:

$$P(X \leq x) = F(x) = \exp[-\exp[-\alpha(x - u)]] \quad (\text{A.4.1})$$

where  $\alpha$  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,  $b = (50,000 - 51,000 + 0.45(21,000))/(0.78 \times 21,000) = 0.516$ . 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:

$$P(\bar{E}) = 1 - P(E) = 1 - \frac{1}{T_r} \quad (\text{A.5.1})$$

It follows that the probability that E will not occur for  $n$  successive years is:

$$P_1(E_m) \times P_2(E_m) \times \dots \times P_n(E_m) = P(E_m)^n = \left[1 - \frac{1}{T_r}\right]^n \quad (\text{A.5.2})$$

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:

$$J = 1 - \left[1 - \frac{1}{T_r}\right]^n = 1 - [P(E_m)]^n \quad (\text{A.5.3})$$

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.

$$0.10 = 1 - \left[1 - \frac{1}{T_r}\right]^{30} \rightarrow 0.90^{\frac{1}{30}} = 1 - \frac{1}{T_r}$$

$$0.9965 - 1 = \frac{-1}{T_r} \rightarrow T_r = 285 \text{ years}$$



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.

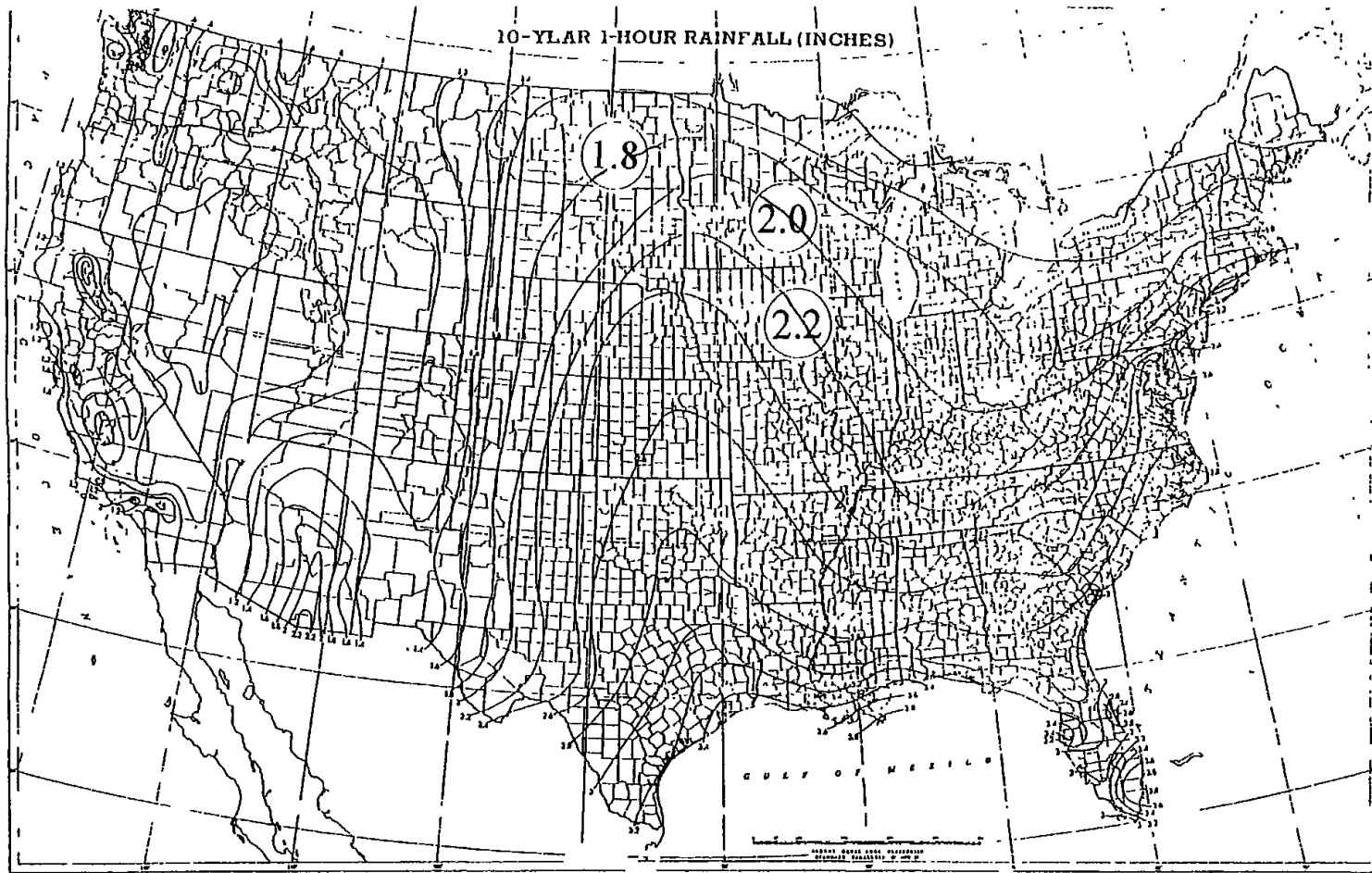


Figure A.6.1  
10-Year 1-Hour Rainfall Depth Map (U.S. Weather Bureau Technical Paper 40)

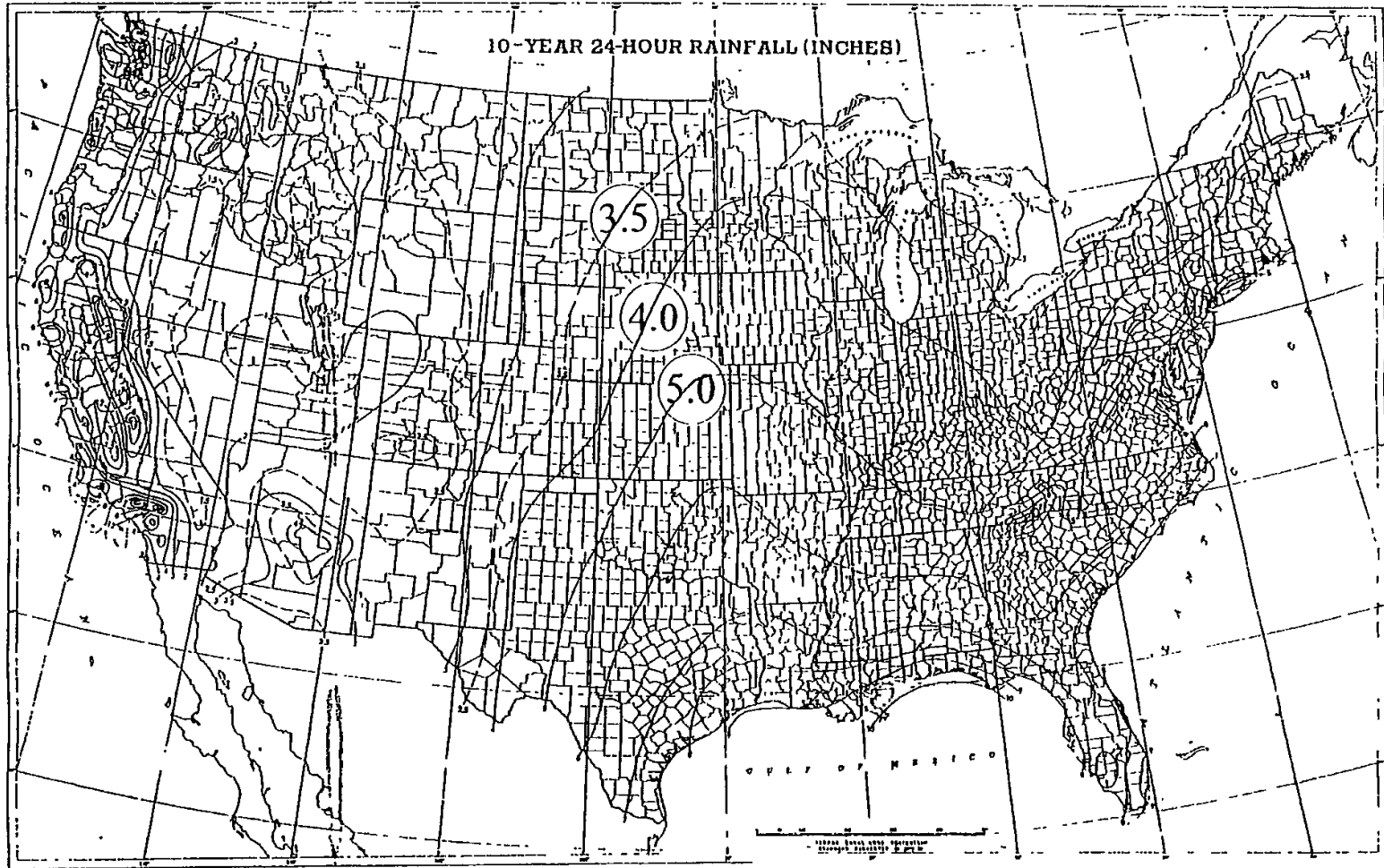


Figure A.6.2  
10-Year 24-Hour Rainfall Depth Map (U.S. Weather Bureau Technical Paper 40)

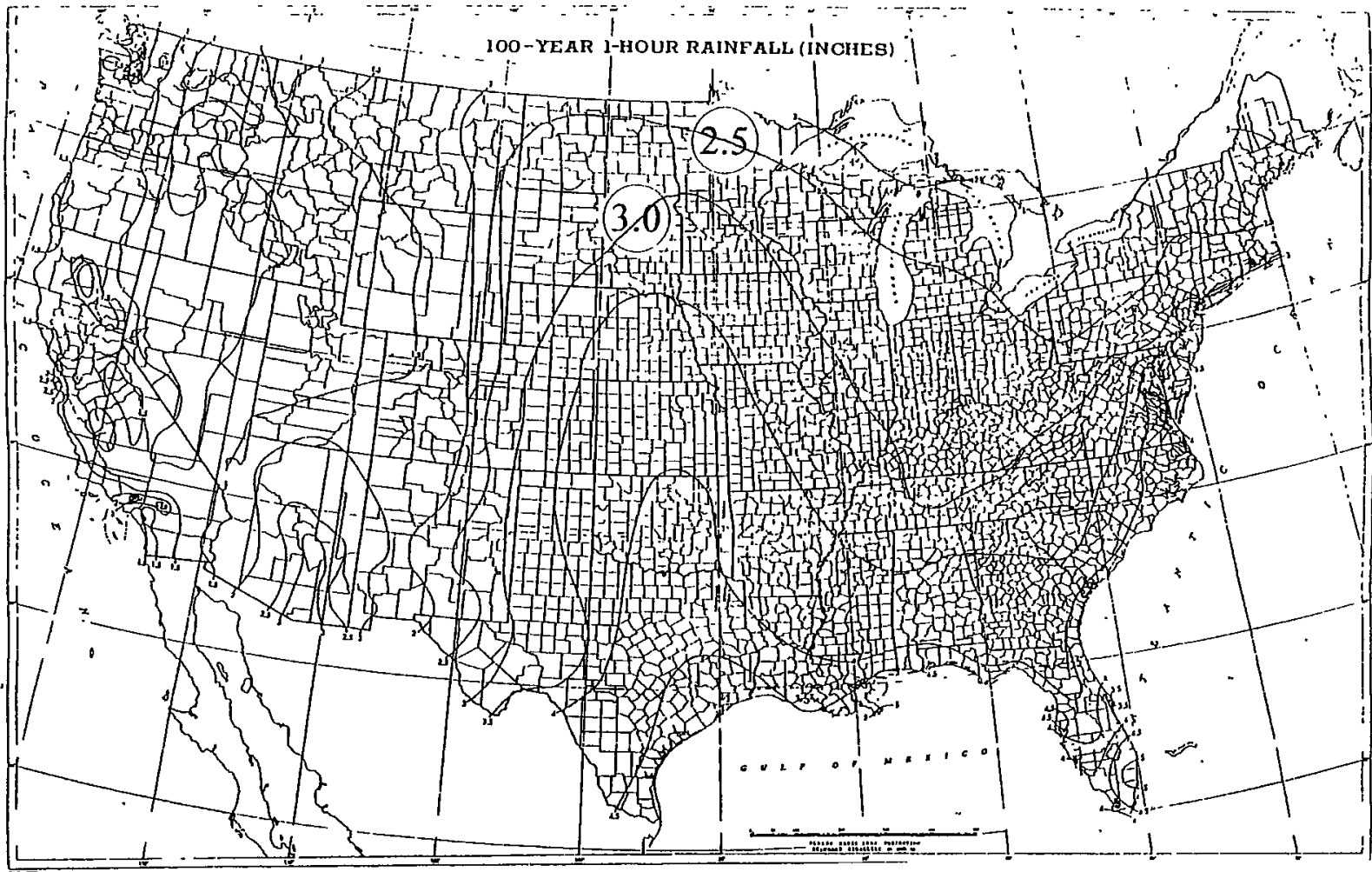


Figure A.6.3  
100-Year 1-Hour Rainfall Depth Map (U.S. Weather Bureau Technical Paper 40)

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Appendix B - FUNDAMENTALS OF HYDRAULICS

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## Appendix B - FUNDAMENTALS OF HYDRAULICS

### LIST OF PARAMETERS

A	Area (ft <sup>2</sup> )
C	Coefficient for Hazen-Williams equation
C <sub>d</sub>	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 <sup>2</sup> )
γ	Specific weight of the fluid (lbs/ft <sup>3</sup> )
h	Vertical distance from the fluid surface (ft)
h <sub>f</sub>	Pipe friction
h <sub>L</sub>	Head loss due to pipe friction (ft)
h <sub>m</sub>	Minor head losses (ft)
h <sub>w</sub>	Height of water above weir crest (ft)
H <sub>A</sub>	Mechanical energy head added to the system
H <sub>R</sub>	Mechanical energy head removed from the system
HGL	Hydraulic grade line
L	Length of pipe (ft)
L <sub>e</sub>	Equivalent length of pipe (ft)
m	Mass flowrate (slugs/sec)
n	Manning's roughness coefficient
ν	Kinematic viscosity of the fluid (ft <sup>2</sup> /sec)
P	Hydrostatic pressure (lbs/ft <sup>2</sup> )
P'	Wetted perimeter (ft)
Q	Flowrate (cfs)
R	Hydraulic radius (ft)
R <sub>e</sub>	Reynolds number
ρ	Fluid density (slugs/ft <sup>3</sup> )
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:

$$m = \rho_1 A_1 V_1 = \rho_2 A_2 V_2 \quad (\text{B.1.1})$$

where subscripts 1 and 2 denote two sections in the flow,  $m$  = mass flow rate (slugs/sec),  $\rho$  = fluid density (slugs/ft<sup>3</sup>),  $V$  = average velocity across the section (ft/sec), and  $A$  = area normal to the section (ft<sup>2</sup>).

For fluids such as water which are incompressible, the density remains a constant ( $\rho_1 = \rho_2$ ). Thus, the continuity equation is obtained.

$$Q = A_1 V_1 = A_2 V_2 \quad (\text{B.1.2})$$

where  $Q$  = volumetric flowrate (ft<sup>3</sup>/s) (cfs).

### B.2 - PRESSURE

The hydrostatic (stationary fluid) pressure varies directly with the fluid depth:

$$P = \gamma h \quad (\text{B.2.1})$$

where  $P$  = hydrostatic pressure (lbs/ft<sup>2</sup>),  $\gamma$  = specific weight of the fluid (lbs/ft<sup>3</sup>), and  $h$  = vertical distance from the fluid surface (ft).

This relationship is shown graphically in Figure B.2.1.

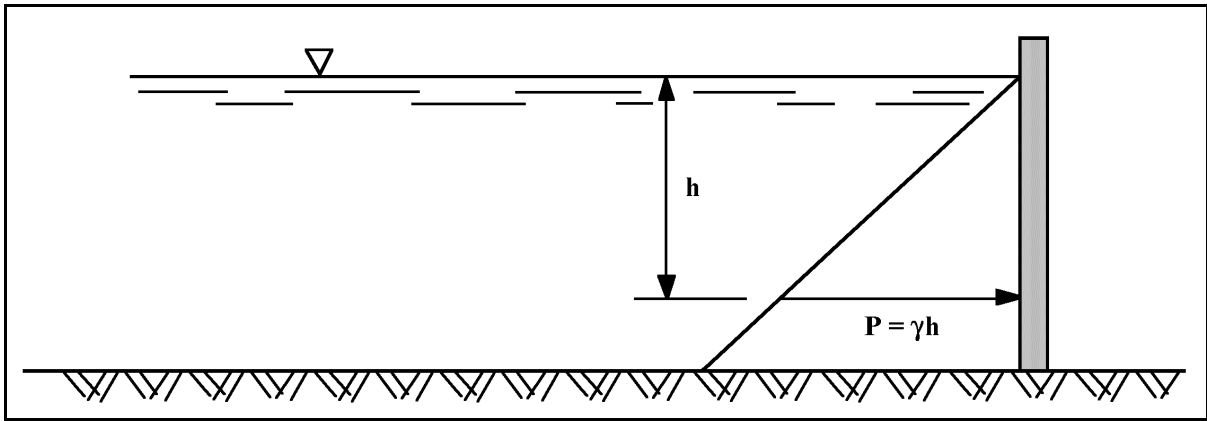


Figure B.2.1  
Graphical Representation of Pressure Variation with Depth

The pressure determined by Equation B.2.1. and shown in Figure B.2.1, is termed gauge pressure. Gauge 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). Gauge pressure is denoted as psig and absolute as psia.

Gauge 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<sup>2</sup>)

- 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.

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma} + z_1 + H_A = \frac{V_2^2}{2g} + \frac{P_2}{\gamma} + z_2 + H_R + h_L \quad (\text{B.3.1})$$

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 [ $P/\gamma + z$ ].

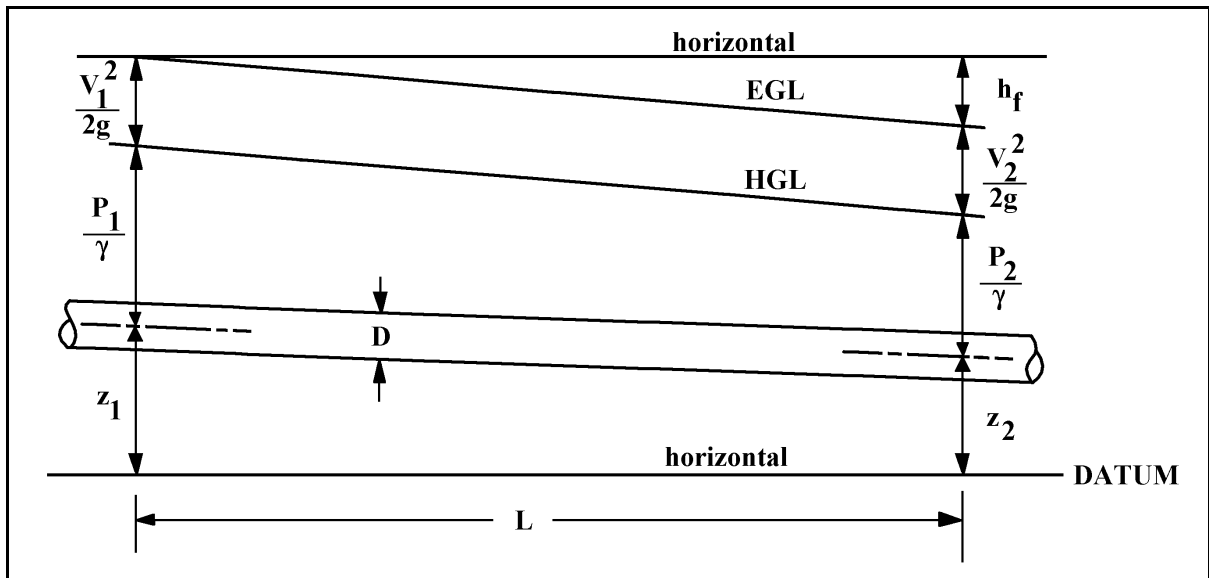


Figure B.3.1

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:

$$h_f = f \frac{L V^2}{D 2g} \quad (\text{B.4.1})$$

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,  $Re$ , and the relative roughness,  $\varepsilon/D$ , where  $\varepsilon$  = the equivalent roughness of the pipe wall, (ft), given in Table B.4.1,  $D$  = pipe diameter (ft),  $Re = VD/\nu$ , and  $\nu$  = kinematic viscosity of the fluid ( $\text{ft}^2/\text{sec}$ ) (values for water are given in Table B.4.2).

The friction factor is determined by entering the graph along with the  $\varepsilon/D$  curve and moving left along the curve until the curve intersects a vertical line for the given  $Re$  (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.

$$D = \left[ \frac{8 L Q^2 f}{\pi^2 g h_f} * f \right]^{1/5} \quad (\text{B.4.2})$$

$$Q = \left[ \frac{\pi^2 D^5 g h_f}{8 L f} \right]^{1/2} \quad (\text{B.4.3})$$

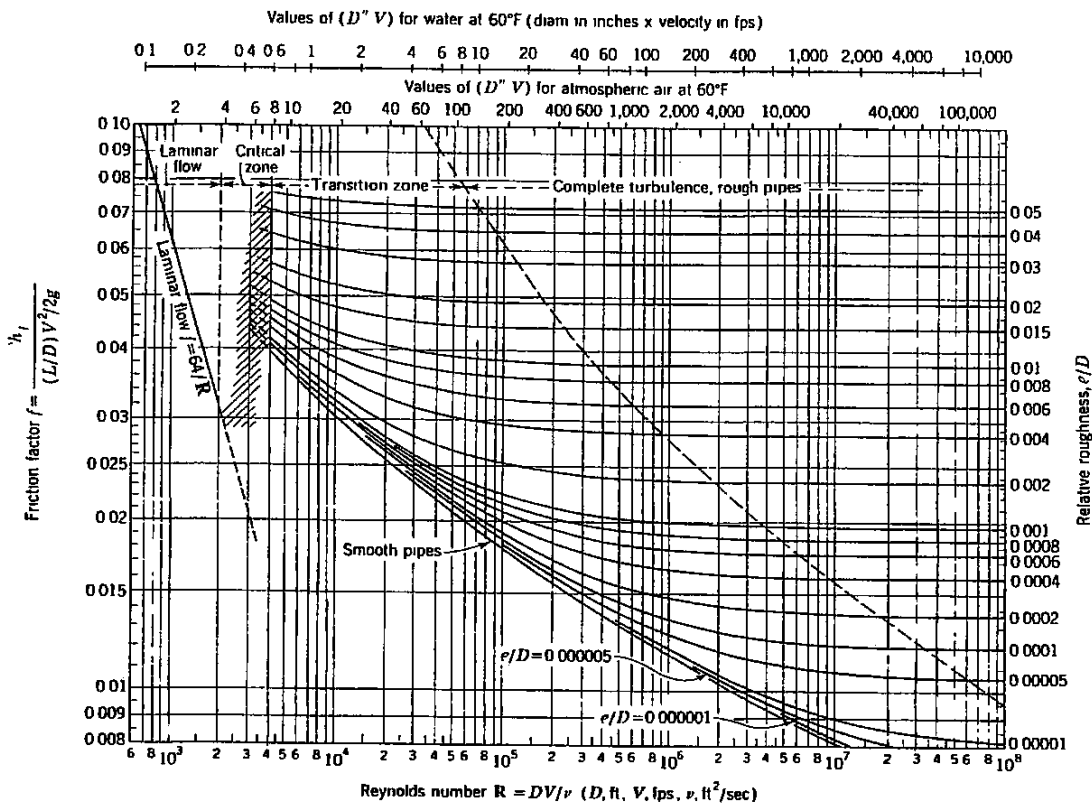


Figure B.4.1  
Moody Diagram (Daugherty et al., 1985)

Another approach in estimating  $f$  is to use the explicit formula developed by Swamee and Jain (1976).

$$f = \frac{1.325}{\left[ \ln \left( \frac{\varepsilon}{3.7D} + \frac{5.74}{R_e^{0.9}} \right) \right]^2} \quad (\text{B.4.4})$$

$$\text{valid for } 10^{-6} \leq \frac{\varepsilon}{D} \leq 10^{-2} \quad 5000 \leq R_e \leq 10^8$$

Table B.4.1  
Equivalent Roughness

Pipe Material	$\epsilon$ (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 (ft <sup>2</sup> /sec)
40	$1.67 \times 10^{-5}$
50	$1.40 \times 10^{-5}$
60	$1.21 \times 10^{-5}$
70	$1.05 \times 10^{-5}$
80	$9.15 \times 10^{-6}$
90	$8.39 \times 10^{-6}$
100	$7.36 \times 10^{-6}$

#### 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:

$$h_f = 10.6 (Q_{\text{mgd}}/C)^{1.85} L/(D)^{4.87} \quad (\text{B.4.5})$$

$$h_f = 4.73 (Q_{\text{cfs}}/C)^{1.85} L/(D)^{4.87} \quad (\text{B.4.6})$$

$$Q_{\text{cfs}} = 0.432 C D^{2.63} (h_f/L)^{0.54} \quad (\text{B.4.7})$$

$$Q_{\text{mgd}} = 0.279 C D^{2.63} (h_f/L)^{0.54} \quad (\text{B.4.8})$$

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 expressed 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.

$$h_f = h_{f1} + h_{f2} + \dots + h_{fn}$$

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 Flow through branch	44 140
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.

$$Q_{\text{TOTAL}} = Q_1 + Q_2 + Q_3 + \dots + Q_n$$
$$h_{f1} = h_{f2} = h_{f3} = h_{fn}$$

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<sup>2</sup>)

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:

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2} \quad (\text{B.5.1})$$

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  $AR^{2/3}$ .

$$AR^{2/3} = \frac{Q * n}{1.49 S^{1/2}} \quad (\text{B.5.2})$$

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:

$$Q = \frac{2}{3} C_d \sqrt{2g} L h_w^{2/3} \quad (\text{B.5.3})$$

where  $C_d$  = discharge coefficient,  $L$  = width of weir crest (ft), and  $h_w$  = depth upstream of weir (ft). ( $C_d$ , 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

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## **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 Service (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 visiting 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

**South West Area Office**  
2017 Hart Street  
Vincennes, IN 47591-6329  
(812) 882-8210 Ext. 8

**North East Area Office**  
3718 New Vision Drive  
Fort Wayne, IN 46845  
(260) 484-5848

**South East Area Office**  
2600 North State Highway 7  
North Vernon, IN 47265-9109  
(812) 346-3411 Ext. 8