CITY OF CINCINNATI
DEPARTMENT OF PUBLIC WORKS
DIVISION OF STORMWATER MANAGEMENT UTILITY

STORMWATER MANAGEMENT RULES AND REGULATIONS

PART 2
STORMWATER MANAGEMENT DESIGN MANUAL

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CHAPTER 6. STORMWATER RUNOFF

6.1 Introduction

Calculating the amount of stormwater runoff is fundamental to the design of the storm drainage system. The amount of stormwater runoff depends on a great number of factors. Some of these factors are reasonably fixed and subject to accurate determination, such as watershed area and shape, ground slope, and natural ponding. Others are seasonably variable, such as frozen soil, soil moisture condition, evaporation or transpiration. Still others vary by land use, such as type of ground cover and impervious area, or method of cultivation. Finally, rainfall is extremely variable and subject only to laws which govern natural occurrence. Despite the indeterminate nature of these factors, methods for obtaining useful information about stormwater runoff have been developed. The methods vary from the empirical Talbot Formula to deterministic models of the watershed, such as the Environmental Protection Agency's (EPA) Storm Water Management Model (SWMM) which requires a great amount of data about the physical condition of the watershed. This Chapter presents several acceptable runoff calculation methods. The design engineer will want to select the appropriate runoff calculation method depending on the information needed and the size of the area under study. This selection is outlined on Exhibit VI-1 which lists the preferred computation methods. When stormwater design involves agencies other than the Stormwater Management Utility and the design standards of the other agency differ from those of the Stormwater Management Utility, the Utility's criteria shall govern unless expressly agreed otherwise.

6.2 Peak Flow: Rational Method (Preferred Method for Drainage Areas Less than 10 Acres)

The Rational Method is the preferred method for determining peak rates of runoff from areas less than 10 acres. The other methods discussed later in this chapter may also be used to obtain the peak rate of runoff.

The basic formula for the Rational Method is: $Q = CiA$

where Q is the peak rate of runoff in cubic feet per second, C is the runoff coefficient which is an empirical coefficient representing a relationship between rainfall and runoff, i is the average intensity of rainfall in inches per hour for the time of concentration (T_C) for a selected frequency of occurrence or return period as provided on Exhibit V-1 or Exhibit V-2, and A is the drainage area in acres. The time of concentration is the estimated time required for runoff to flow from the most remote part of the drainage area under consideration to the point under consideration. It consists of the total of time for overland sheet flow, open channel flow, and pipe flow.

6.2.1 Adopted Runoff Coefficients

Table 6-1 lists the runoff coefficients adopted for use in the Rational Method for stormwater drainage in Cincinnati. These are based on average land use patterns and hydrologic soil group C.

The runoff coefficients used for the Rational Method differ from the intensity of development factor used in the monthly storm drainage service charge (SDSC). This is because the "Rational Method" runoff coefficient is used to estimate the total amount of runoff from an individual parcel of land and whereas the intensity of development factor (IDF) is the assignment of numerical values to various land uses, and the applications of standard land uses to individual properties based on random sampling of similar land uses under typical conditions for the purpose of computing the City's SDSC.

It is not contended that the land use assignment and associated IDF is identical with the Rational Method coefficient of each, but rather that the assigned IDF values appropriately reflect the differences among various classes of users of the stormwater systems and represent an equitable basis in determining the SDSC.

Table 6-1

Runoff Coefficients
For the Rational Method

Land Use	Runoff Coefficient
Residential	0.5
Multi-family	0.6
Commercial and Business Districts	0.85
Industrial Districts	0.75
Open Space (parks, golf courses, cemeteries, meadows, grass, woods, lawns, etc.)	0.3
Impervious Areas (parking lots, roads, rooftops)	0.9
Steep wooded hillside slope $\geq$10 percent	0.5

6.2.2 Alternate Runoff Coefficients

It is expected that the majority of developments can be designed using Table 6-1. However, if a development is shown to be on a soil group different from Group C or if a more detailed breakdown of land use is required, then runoff coefficients derived from Exhibit VI-2 may be used instead of the Table 6-1 runoff coefficients. Further information on the land use descriptors and hydrologic soil groupings may be found in Chapters 8 and 9 of Section 4, "Hydrology" of the Soil Conservation Service National Engineering Handbook.

Soils have been divided by infiltration rate characteristics into four hydrologic soil groups. These groups are:

Group A Represents soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well-drained sands and gravels.

Group B Represents soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well-drained soils with moderately fine to moderately coarse textures.

Group C Represents soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water, or soils with moderately fine to fine texture.

Group D Represents soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clay with high swelling potential, soils with permanently high water tables, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

Exhibit VI-3 lists Cincinnati soils and their hydrologic classifications. Soil maps showing soil series for different areas may be obtained from the Hamilton County Soil Conservation Service Office.

6.2.3 Composite Runoff Coefficients

In applying the Rational Method, the drainage area should be delineated on a topographic map, then divided into subareas for analysis. Each subarea would represent a point of contribution to a definite drainage course. If the runoff coefficient varies over a subarea, a composite coefficient can be calculated as an average, weighted by area of the various runoff coefficients. Exhibit VI-11 provides a composite runoff coefficient for areas containing impervious surfaces.

6.2.4 Time of Concentration

The minimum time of concentration used shall be 10 minutes. The time of concentration is the estimated time required for runoff to flow from the most remote part of the drainage area under consideration to the point under consideration.

The time of concentration is a combination of times of overland or sheet flow from the point of rainfall to an established drainage channel such as gutter, open ditch, or inlet and time of travel in the drainage course. The first part, sometimes referred to as inlet time, may be estimated from the overland flow time chart (Exhibit VI-4). The minimum inlet time used shall be 10 minutes. The time of flow in the natural channel and open or closed channel portion may be estimated by making a preliminary estimate of the quantities of flow and applying the Manning Formula for flow in an open channel (Exhibit VI-10). If in a gutter the velocity may be estimated by dividing the gutter capacity, as provided on Exhibit VI-10 by the gutter flow area; if in a ditch or natural channel, the shape must be known or measured. The value of "n," the coefficient of friction, will depend on the judgement of the designer but should be justified by a description of the channel.

6.2.5 Example - Rational Method Number 1

Determine the post-development 10-year discharge from a 5-acre watershed being developed for 3 acres of residential and 2 acres of commercial use. The overland flow will travel 300 feet over paved roads at a 0.5 percent slope and into a 700-foot-long drainage channel which has a 1 percent slope and an "n" of 0.03.

Step 1. Enter Table 6-1 and read:
residential runoff coefficient = 0.5
commercial runoff coefficient = 0.85

Step 2. Calculate the composite runoff coefficient = (the residential runoff coefficient x the residential area + the commercial runoff coefficient x the commercial area)/total area = ((0.5 x 3) + (0.85 x 2))/5 = 0.64

Step 3. Calculate the time of concentration:

a. Use Exhibit VI-4 and read overland time of concentration = 7.8 minutes.

b. Check calculated overland time of concentration is $\geq$10 minutes, 7.8 minutes < 10 minutes, therefore use 10 minutes.

c. Use Exhibit VI-10, assume a channel flow of 20.0 cfs and read channel velocity = 3.7 fps.

The channel time of concentration = length/velocity = 700/3.7 = 189/60 = 3.2 minutes.

d. Total time of concentration = overland time of concentration plus channel time of concentration = 10.0 + 3.2 = 13.2 minutes.

Step 4. Use Exhibit V-2, locate a duration of 13.2 minutes and read: 10-year rainfall intensity = 4.88 inches per hour.

Step 5. Calculate the 10-year discharge Q = CiA = 0.64 x 4.88 x 5 = 15.6 cfs.

Step 6. Check calculate 10-year discharge = assumed channel flow, 15.6 cfs = 20.0 cfs assumption in error, assume channel flow = 15.6 cfs go to Step 3c and continue.

Step 3. c. Use Exhibit VI-10, assume a channel flow of 15.6 cfs and read channel velocity = 3.5 fps.

The channel time of concentration = length/velocity = 700/3.5 = 200/60 = 3.3 minutes.

d. Total time of concentration = overland time of concentration plus channel time of concentration = 10.0 + 3.3 = 13.3 minutes.

Step 4. Use Exhibit V-2, locate a duration of 13.3 minutes and read: 10-year rainfall intensity = 4.86 inches per hour.

Step 5. Calculate the 10-year discharge Q = CiA = 0.64 x 4.86 x 5 = 15.6 cfs.

Step 6. Check calculate 10-year discharge = assumed channel flow, 15.6 cfs = 15.6 cfs assumption OK.

To design an open channel, see Chapter 7.

6.2.6 Example - Rational Method Number 2 (Simplified)

Determine the 10-year discharge from a 10-acre watershed assuming a runoff coefficient of 0.3 and a time of concentration of 12.0 minutes.

Step 1. Use Exhibit V-2, locate a duration of 12.0 minutes, and read: 10-year rainfall intensity = 5.09 inches per hour.

Step 2. Calculate the 10-year discharge Q = CiA = 0.3 x 5.09 x 10 = 15.3 cfs.

6.3 Regression Equations

The Ohio Department of Natural Resources has published a series of regression equations in Bulletin 45 for estimating peak flows from unregulated rural areas which have drainage areas equal to or greater than 6 acres. These equations relate pertinent physical and climatic parameters of the watershed (i.e., drainage area, slope, average annual precipitation) to peak discharges at the basin outlet. No attempt is made in Bulletin 45 to account for the effects of urbanization on the magnitude and frequency of floods; however, the need for such a study is noted therein.

The Ohio Department of Transportation has published urbanization adjustment factors which it applies to peak discharges calculated using Bulletin 45 methods. Similarly, Sauer presents a method of adjusting peak discharges in Oklahoma. Little actual data was available for derivation of these adjustments and Sauer states that "there are no urban flood data for Oklahoma to verify this method and consequently, the accuracy is questionable." Due to the rural nature of the original derivation of Bulletin 45 and the lack of strong proof of the validity of the urbanizing adjustments to Bulletin 45, its use is not permitted for stormwater design in the City of Cincinnati.

6.4 Hydrograph Methods

For areas larger than 10 acres, or where in addition to the peak rate of discharge it is necessary to know the rate of runoff with respect to time and/or the volume of water discharged, the Rational Method is not adequate. Appropriate calculation techniques include one of the various hydrograph methods, (SCS TR55, Modified Ration Method, or Unit Hydrograph Methods) or a deterministic computer model.

In designing stormwater systems for developing areas, there is usually insufficient information about future conditions for a meaningful use of a complex computer model. There are numerous adaptations of the hydrograph methods available which present a reasonable compromise considering data required, complexity of computations, and information obtained.

A hydrograph is a plot or tabulation of rate of stormwater runoff against time for a given watershed. A unit hydrograph is the hydrograph from 1 inch of runoff from the drainage area in a specified time. Standard unit hydrographs are derived from analysis of historical records correlated with watershed characteristics. Hydrographs from a given storm can then be obtained by scaling the height to the design rainfall and the base length to the duration of resulting runoff.

6.4.1 SCS TR55, Graphical Peak Discharge Method (Preferred Method for Drainage Areas Between 10 and 200 Acres)

Of the various hydrograph methods, the Graphical Peak Discharge Method is the preferred one in addressing problems relating to changes in stormwater runoff due to urbanization as outlined in "Urban Hydrology for Small Watersheds," Technical Release (TR) No. 55, Engineering Division, Soil Conservation Service (SCS), U.S. Department of Agriculture. This method is presented for use with this design manual. When use of an alternate method is requested, sample calculations shall be submitted to the Stormwater Management Utility for both the preferred and alternate methods in order to demonstrate the appropriateness of the alternate method.

The methodology of the SCS explained in TR No. 55 can be used to provide peak rates of runoff, estimated total volume of runoff, or estimated volume of storage required to reduce the rate of runoff to a desired value. The basic information required is:

1. Area of the watershed

2. Soil type, as for the Rational Method

3. Land use, as for the Rational Method

4. Time of concentration, as for the Rational Method

5. Travel time - Time of flow from the point of collection of the water in a defined channel in the design watershed to a point of evaluation or design. This is required in combining hydrographs from more than one watershed or where channel routing is considered.

6. Watershed factors of natural ponding and swamp areas

7. Design storm based on 24-hour duration rainfall for design recurrence interval.

Where peak rate of flow and total volume of runoff are needed, the Graphical Peak Discharge Method of Chapter 4 of SCS TR55 may be used.

The general procedure for determining peak discharge and total runoff volume from an area is as follows, with numbered steps corresponding to the worksheet used for Exhibit VI-9.

Step 1.

a. Determine the drainage area (DA) in acres for the study area.

b. Delineate the different soil types according to hydrologic soil groups using soil survey maps and Exhibit VI-3. (These maps are available at the Hamilton County Soil Conservation Service District Office.)

c. Delineate the various land use areas according to land use descriptions on Exhibit VI-5.

d. Subdivide the total area into subareas according to hydrologic soil groups and land use and assign the runoff curve numbers from Exhibit VI-5. If the subarea contains impervious surfaces, use Exhibit VI-11 in conjunction with Exhibit VI-5.

e. Calculate the weighted runoff curve number, i.e., Weighted CN = (CN of subarea x subarea)/$\sum$ subareas.

f. Determine the time of concentration (T_c) in hours for the study area.

g. Determine ponding and swampy areas not in the time of concentrations flood path as a percent of the study area acrage.

Step 2. Determine the total rainfall depth (P) for the design storm of chosen frequency and 24-hour duration. Rainfall data specific to Cincinnati is given in Table 5-1.

Step 3. Calculate the initial abstruction (Ia) using the equation Ia = 0.2[1,000/CN (weighted) -10] where CN (weighted) equals the weighted curve number.

Step 4. Calculate the initial abstruction (Ia) divided by the total rainfall depth (P) for the design storm of chosen frequency and 24-hour duration.

Step 5. Use the time of concentration (T_c) and the initial abstruction divided by rainfall depth (Ia/P) ratio and Exhibit VI-7, read the unit peak discharge in cfs per inch of runoff.

Step 6. Determine direct runoff in inches using Exhibit VI-6 for the weighted CN determined in Step 1. This may be done by any of three methods given: (a) reading from the plot; (b) interpolation from the table; or (c) substitution into the equations for P and CN. Any of these methods yield the same result as the other methods.

Step 7. Using the percent ponding and swampy areas and Exhibit VI-8, interpolate the ponding and swampy area adjustment factor.

Step 8. Calculate the adjusted peak discharge in cfs by multiplying the values determined in Steps 5, 6, and 7 by the drainage area in square miles.

Step 9. Calculate the total runoff volume in acre-feet as the runoff depth from Step 6 times the actual drainage area divided by 12.

The following example is included to illustrate the Graphical Peak Discharge Method of predicting runoff.

6.4.1.1 Example - Graphical Peak Discharge Method

Determine the anticipated future condition peak discharges and runoff volumes from development of a 34-acre watershed for the 10-, 25-, and 100-year, 24-hour storm events. The runoff curve number for the proposed development is computed to be 75. Approximately 40 percent of the watershed will be impervious. One acre or 3 percent of the drainage area is made of ponding or swampy areas that are not in the time of concentration's flow path. The time of concentration is computed to be 25 minutes.

Using Exhibit VI-1, the Graphical Peak Discharge Method was selected as the preferred method to determine the peak discharge and runoff volume from the 34-acre watershed.

Sample calculations for this example area shown on Exhibit VI-9, which follows the general procedure outlined above.

6.4.2 Modified Rational Method

The Modified Rational Method (MRM), an extension of the Rational Method, provides the peak discharge and a hydrograph of the runoff. This method is a means of quickly estimating the storage volume which will be required for stormwater detention/retention basins. Results of its use will be checked by the application of SCS TR55 methods.

6.4.3 SCS TR20 Method (Preferred Method for Drainage Areas Greater Than 200 Acres)

For drainage areas greater than 200 acres where considerable difference in the runoff characteristics is typical and where conduit or channel storage may deserve special attention, the dimensionless hydrograph method should be used to provide a better model of actual conditions. The Soil Conservation Service, Section 4, Hydrology, National Engineering Handbook, Chapter 16, amply describes the methodology of dimensionless unit hydrographs.

6.4.4 Computer Models

Numerous computer models exist for use in urban stormwater problems, both in analysis and control. Such models generally produce stormwater runoff hydrographs which can be used for design of stormwater management facilities. Two programs developed specifically for urban runoff analysis are the EPA SWMM and the STORM program developed by Water Resource Engineers. These models can provide information on water quality as well as quantity. They require a great amount of accurate input data for accurate results. They are valuable as tools for operation of an existing stormwater system where real time data is available and real time decisions are required for day-to-day management.

Other computer programs such as the U.S. Army Corps of Engineers HEC-1 Flood Hydrography Package and the Soil Conservation Service TR20 program may be used to compute stormwater runoff hydrographs for pre- and post-development conditions. Detention/retention basin sizing may be accomplished by routing the post-development hydrograph through the proposed basin using these models. Although not specifically designed to analyze urban stormwater runoff, these models provide useful results in the above applications.

With the increasing availability of microcomputers in design offices, many programs are being written to aid in the design of urban stormwater facilities, from determining inlet spacing and storm sewer sizes to routing discharge hydrographs through proposed stormwater detention/retention basins. These programs put the use of computer models within the reach of designers who may not have access to more sophisticated models such as SWMM and STORM.

The use of a computer program for system design must be approved by the Stormwater Management Utility prior to its use. The Utility may require the designer to submit sample calculations and program documentation as a part of the utility's program review and approval procedure. Prior to use of any computer model, the designer should verify the correctness of computations performed by the program for the various types of applications for which the program will be used. Upon submission of the drainage plan to the Stormwater Management Utility, the designer shall include sufficient data to allow the Utility to verify the design according to approved methods listed herein.

6.5 Bibliography

Department of Transportation, Federal Aviation Administration. Airport Drainage. Washington, D.C., 1965.

American Society of Civil Engineers. Design and Construction of Sanitary and Storm Sewers. Manuals and Reports on Engineering Practice No. 37. Water Pollution Control Federation Manual of Practice No. 9. New York, 1970.

Ohio Department of Natural Resources. Floods in Ohio - Magnitude and Frequency, Bulletin No. 45, May 1977.

Franzini, Joseph B., Ray K. Linsley. Water-Resources Engineering. 2nd ed. New York, NY: McGraw-Hill Book Company, 1972.

Handbook of Applied Hydrology, New York, NY: Mc-Graw Hill Book Company, Ven Te Chow, Editor-in-Chief, 1964.

American Society of Civil Engineers. Hydrology Handbook, Manual of Engineering Practice No. 28, New York, 1949.

U.S. Department of Agriculture, U.S. Government Printing Office. Hydrology, Soil Conservation Service National Engineering Handbook. Section 4. Washington, D.C., 1971.

Ohio Department of Transportation, Bureau of Location and Design. Location and Design Manual, 1983.

The National Association of Home Builders. The American Society of Civil Engineers. The Urban Land Institute. Residential Stormwater Management. Washington, D.C., 1975.

Sauer, V.B. U.S. Geological Survey. An Approach to Estimating Flood Frequency for Urban Areas in Oklahoma, 1974.

Harza Engineering Company. Storm Drainage Design Manual, Erie and Niagara Counties Regional Planning Board, 1972.

Maryland Department of Natural Resources. Weston, Roy F., Inc. Stormwater Management Design: A Manual of Procedures and Guidelines. 1976.

U.S. Department of Agriculture. Soil Conservation Service Regional Technical Service Center. Tabular Method of Flood Routing - 24-Hour Type II Storm Distribution. TSC Technical Note - Engineering - UD-20, Upper Darby, Pennsylvania, 1972.

U.S. Department of Agriculture. Soil Conservation Service Engineering Division. Urban Hydrology for Small Watersheds. Technical Release No. 55, 1986.

DESIGN FLOW CHART

TO DETERMINE THE PREFERRED METHOD FOR STORMWATER RUNOFF CALCULATIONS

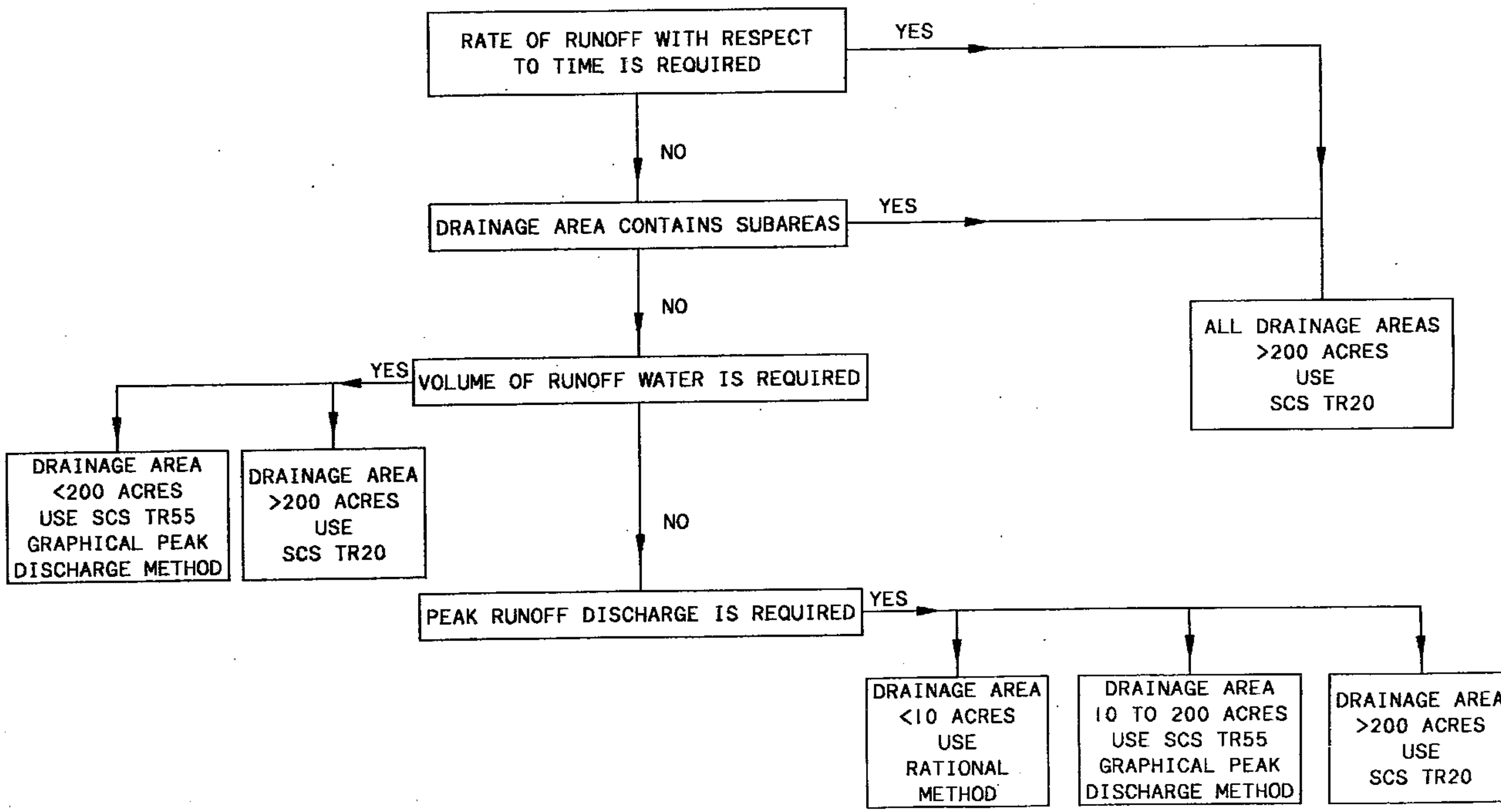


EXHIBIT VI-I

RUNOFF COEFFICIENTS
For the Rational Method

LAND USE DESCRIPTION		HYDROLOGIC SOIL GROUP A	B	C	D
Residential:					
Average Lot Size	Average % Impervious				
1/8 acre or less	65	0.41	0.59	0.72	0.77
1/4 acre	38	0.16	0.37	0.54	0.64
1/3 acre	30	0.12	0.32	0.50	0.61
1/2 acre	25	0.09	0.29	0.47	0.59
1 acre	20	0.06	0.26	0.45	0.57
2 acres		0.05	0.23	0.41	0.50
Commercial and business areas (85% impervious)		0.69	0.77	0.83	0.86
Industrial districts (72% impervious)		0.50	0.66	0.74	0.80
Paved parking lots, roofs, driveways, etc.		0.94	0.94	0.94	0.94
Open spaces, lawns, parks, golf courses, cemeteries, etc.		0.05	0.16	0.34	0.47
Cultivated land		0.17	0.30	0.43	0.50
Meadow		0.05	0.13	0.30	0.43
Wood or forest land		0.05	0.10	0.29	0.41

HYDROLOGIC SOIL GROUPS FOR HAMILTON COUNTY, OHIO

Soil Name	Identification Symbol	Hydrologic Classification
Ava silt loam	Ar	C
Ava-Urban land complex	As	C
Avonburg silt loam	Av	D
Avonburg-Urban land complex	Aw	D
Bonnel silt loam	Bo	C
Casco gravelly loam	Cd	B
Casco loam	Cd	B
Cincinnati silt loam	Cn	C
Dana silt loam	Da	B
Eden silty clay loam	Ec	C
Eden flaggy silty clay loam	Ed	C
Eden-Urban land complex	Ee	C
Eldean loam	Ep	B
Eldean-Urban land complex	Er	B
Fincastle silt loam	Fd	C
Fincastle-Urban land complex	Fe	C
Fox loam	Fo	B
Fox-Urban land complex	Fp	B
Genessee loam	Gn	B
Genessee-Urban land complex	Go	B
Hennepin silt loam	He	B
Henshaw silt loam	Ho	C
Huntington silt loam	Hu	B
Jules silt loam	Ju	B
Lanier sandy loam	Lg	B
Marklan silty clay loam	Ma	C
Martinsville silt loam	Mc	B
Miamian silt loam	Mn	C
Miamian-Hennepin silt loams	Mo	C/B
Miamian-Urban land complex	Mu	C
Parke silt loam	Pb	B
Parke-Urban land complex	Pc	B
Pate silty clay loam	Pf	C
Pate-Urban land complex	Ph	C
Patton silty clay loam	Pn	B/D
Pits, gravel	Po	B
Princeton sandy loam	Pr	B
Raub silt loam	Rd	B
Ross loam	Rn	B
Rossmoyne silt loam	Rp	C
Rossmoyne-Urban land complex	Rt	C
Russell silt loam	Rw	B
Russell-Urban land complex	Rx	B
Stonelick fine sandy loam	St	B
Switzerland silt loam	Sw	B
Switzerland-Urban land complex	Sx	B

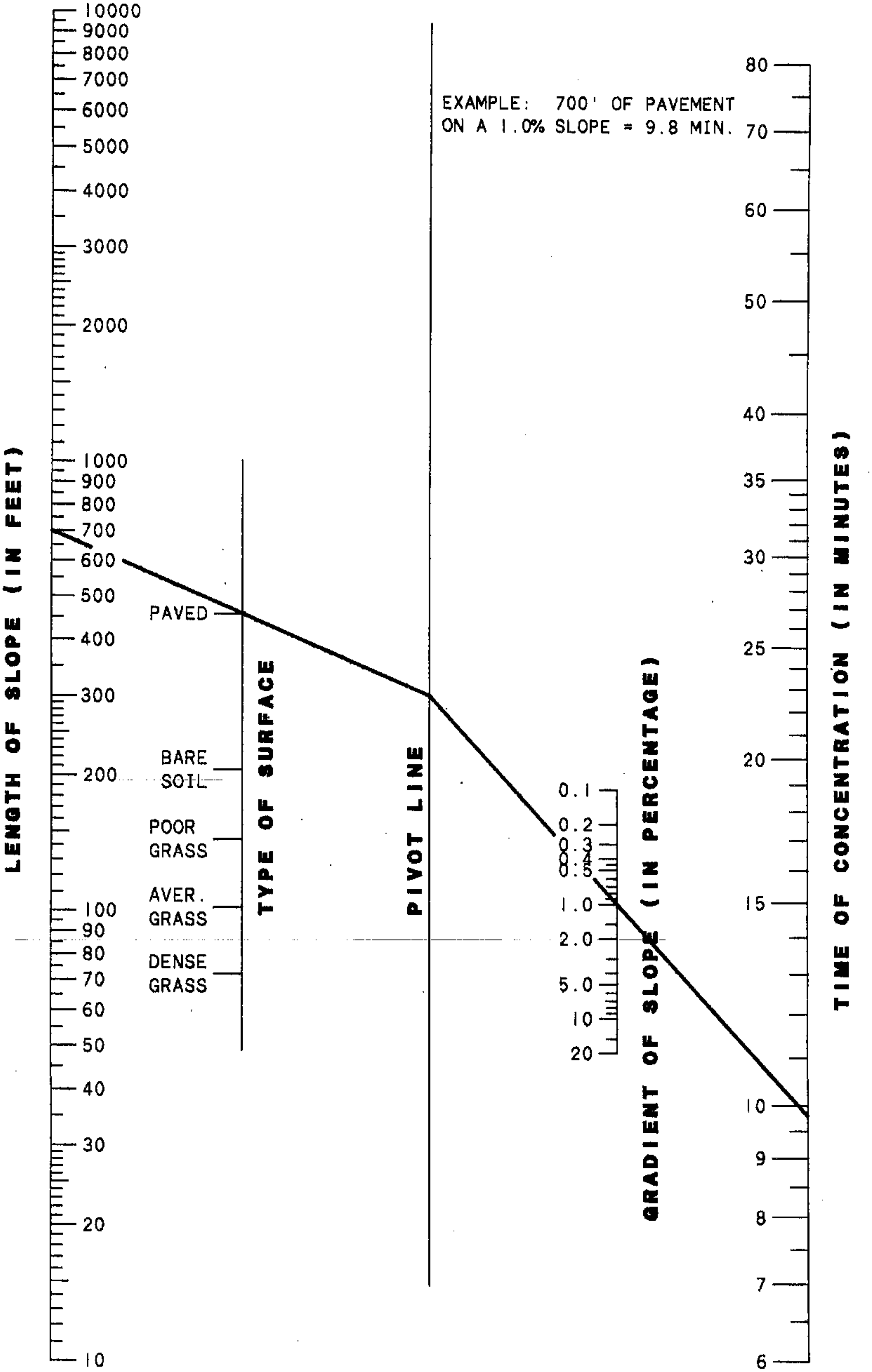
HYDROLOGIC SOIL GROUPS FOR HAMILTON COUNTY, OHIO

Soil Name	Identification Symbol	Hydrologic Classification
Udorthents, clayey	Ud	C
Udorthents, loamy	Uf	C
Urban land-Elkinsville complex	Ug	
Urban land-Huntington complex	Uh	
Urban land-Martinsville complex	Um	
Urban land-Patton complex	Uo	
Urban land-Rossmoyne complex	Ur	
Urban land-Stonelick complex	Ux	
Wakeland silt loam	Wa	B/D
Warsaw Variant sandy loam	Wb	B
Wea silt loam	We	B
Whitaker loam	Wh	C
Xenia silt loam	Xf	B

NOTES: Identification symbol indicates soil series only and not slope class.

Two soil groups such as B/D indicate the drained/undrained situation.

A blank indicates that the original soils have been significantly disturbed by urban development and no classified has been determined.



OVERLAND FLOW TIME

EXHIBIT VI-4

RUNOFF CURVE NUMBERS
(Antecedent Moisture Condition II)

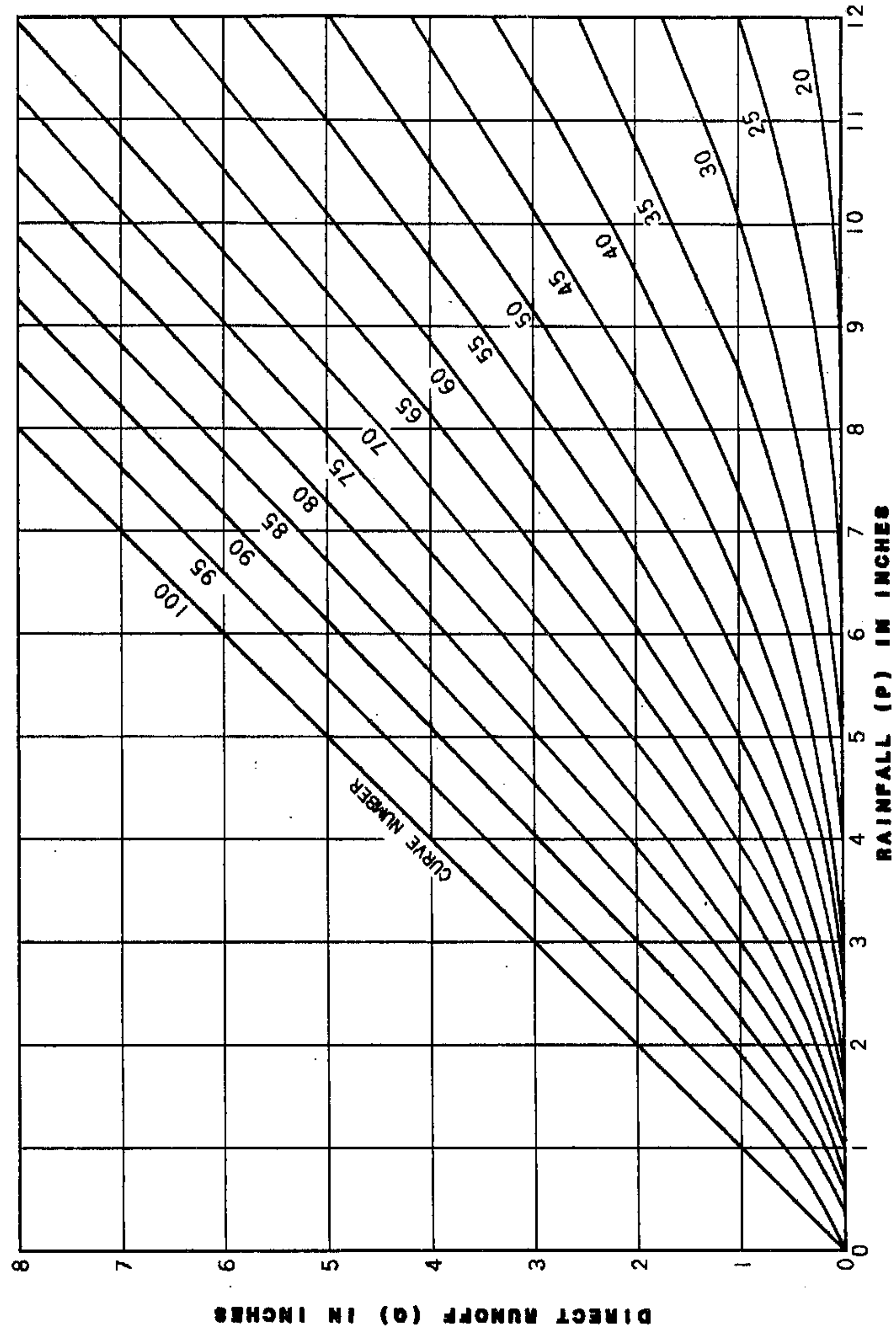
LAND USE DESCRIPTION		HYDROLOGIC SOIL GROUP A	B	C	D
Residential:[1]					
Average Lot Size	Average % Impervious[2,3]				
1/8 acre or less	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Commercial and business areas (85% impervious)[3]		89	92	94	95
Industrial districts (72% impervious)[3]		81	88	91	93
Paved parking lots, roofs, driveways, etc.		98	98	98	98
Streets and roads:					
paved with curbs and storm sewers		98	98	98	98
gravel		76	85	89	91
dirt		72	82	87	89
Open spaces, lawns, parks, golf courses, cemeteries, etc.		39	61	74	80
Cultivated land		62	71	78	81
Meadow		30	58	71	78
Wood or forest land		30[4]	55	70	77

[1]Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.

[2]The remaining previous areas (lawn) are considered to be in good pasture condition for these curve numbers.

[3]For land use with different percentage of impervious area or the impervious area does not flow directly into the drainage system, use Exhibit VI-17 to calculate a new composite runoff curve number.

[4]Actual curve number less than 30: use 30 for runoff computations.



HYDROLOGY: SOLUTION OF RUNOFF EQUATION

WHERE THE INITIAL ABSTRACTION IS EQUAL TO TWO TENTHS OF THE POTENTIAL MAXIMUM RETENTION AFTER RUNOFF BEGINS.

EXHIBIT VI-6

HYDROLOGY: SOLUTION OF RUNOFF EQUATION
RUNOFF DEPTH IN INCHES FOR SELECTED CURVE NUMBERS AND RAINFALL AMOUNTS

Rainfall (inches)	Curve Number (CN)[1] 60	65	70	75	80	85	90	95	98
1.0	0	0	0	0.03	0.08	0.17	0.32	0.56	0.79
1.2	0	0	0.03	0.07	0.15	0.28	0.46	0.74	0.99
1.4	0	0.02	0.06	0.13	0.24	0.39	0.61	0.92	1.18
1.6	0.01	0.05	0.11	0.20	0.34	0.52	0.76	1.11	1.38
1.8	0.03	0.09	0.17	0.29	0.44	0.65	0.93	1.29	1.58
2.0	0.06	0.14	0.24	0.38	0.56	0.80	1.09	1.48	1.77
2.5	0.17	0.30	0.46	0.65	0.89	1.18	1.53	1.96	2.27
3.0	0.33	0.51	0.72	0.96	1.25	1.59	1.98	2.45	2.78
4.0	0.76	1.03	1.33	1.67	2.04	2.45	2.92	3.43	3.77
5.0	1.30	1.65	2.04	2.45	2.89	3.37	3.88	4.42	4.76
6.0	1.92	2.35	2.80	3.28	3.78	4.31	4.85	5.41	5.76
7.0	2.60	3.10	3.62	4.15	4.69	5.26	5.82	6.41	6.76
8.0	3.33	3.90	4.47	5.04	5.62	6.22	6.81	7.40	7.76
9.0	4.10	4.72	5.34	5.95	6.57	7.19	7.79	8.40	8.76
10.0	4.90	5.57	6.23	6.88	7.52	8.16	8.78	9.40	9.76
11.0	5.72	6.44	7.13	7.82	8.48	9.14	9.77	10.39	10.76
12.0	6.56	7.32	8.05	8.76	9.45	10.12	10.76	11.39	11.76

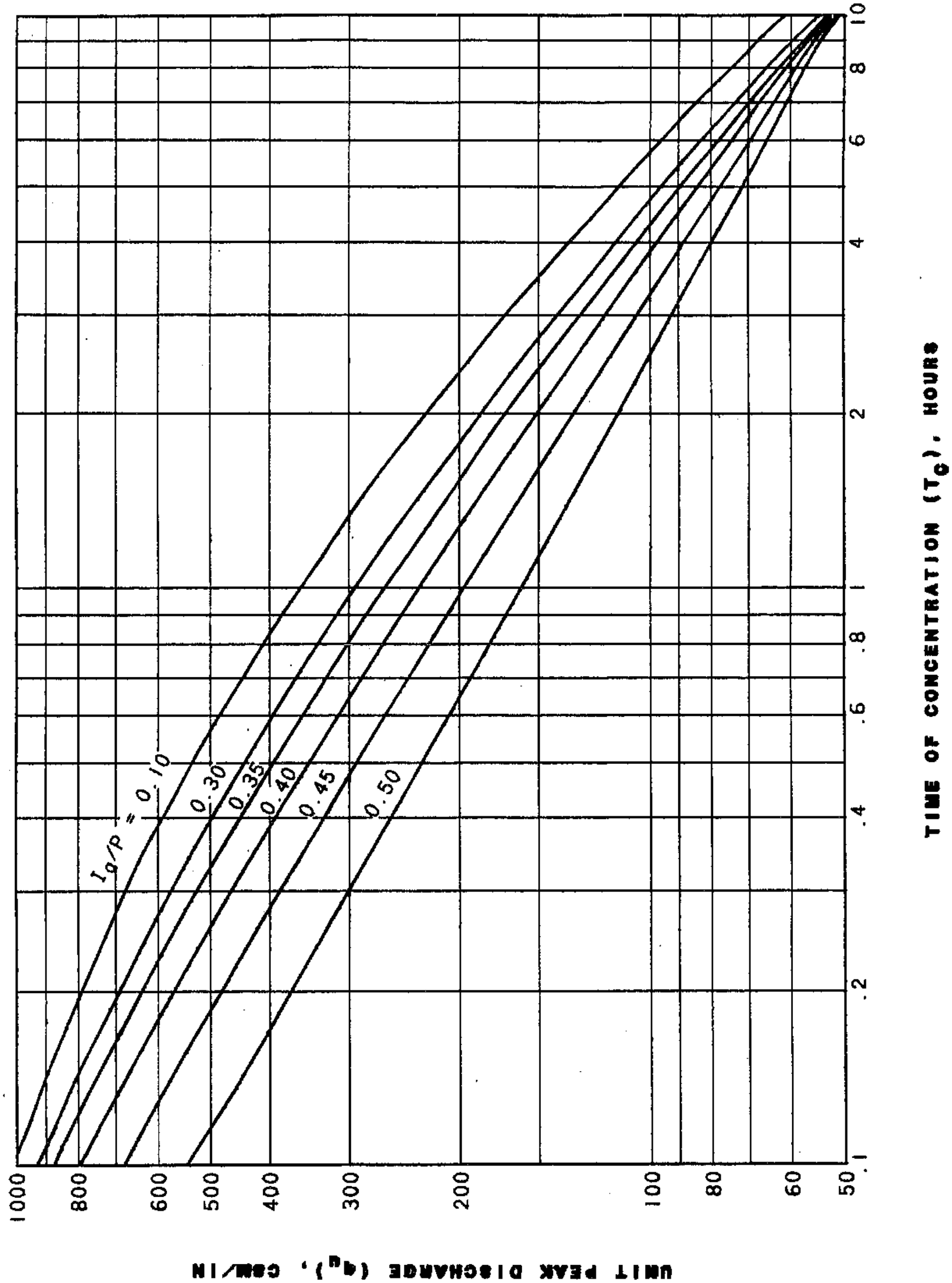
[1] To obtain runoff depths for CNs and other rainfall amounts not shown in this table, use an arithmetic interpolation.

Equation to determine the runoff from a rainfall with a particular curve number is:

$$Q = \frac{[P - 0.2\,(1{,}000/CN - 10)]^2}{[P + 0.8\,(1{,}000/CN - 10)]}$$

where

N = Curve Number
P = Rainfall in Inches
Q = Runoff in Inches



UNIT PEAK DISCHARGE (q_u) FOR SCS TYPE II RAINFALL DISTRIBUTION

EXHIBIT VI-7

Table 4-2

Adjustment factor (F_p) for pound and swamp areas that are spread throughout the watershed and not in the time of concentration's flow path.

Percentage of pond and swamp areas	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

STORMWATER RUNOFF
GRAPHICAL PEAK DISCHARGE COMPUTATIONS
ARTICLE 6.4.1.1 EXAMPLE

PROJECT ______________ DESIGNER ______________ DATE ______

1) DATA: WATERSHED CONDITION = FUTURE (PRESENT OR FUTURE) TYPE II STORM

DRAINAGE AREA (DA) = 34 ACRES.

Hydrologic Soil Group Exhibit VI-3	Land Use Description Include Treatment, Practice & Condition Exhibit VI-5	CN Exhibit (3) VI-5	CN Exhibit (3) VI-11	Area (Acres)	Area (%) (4)	Product (3) x (4) (5)
B	PARKING LOTS, ROOFS	98	NA	13.6	40	3920
B	POND OR SWAMP	100		1.0	3	300
B	MEADOW	58		19.4	57	3306
			Totals =	34	100	7526

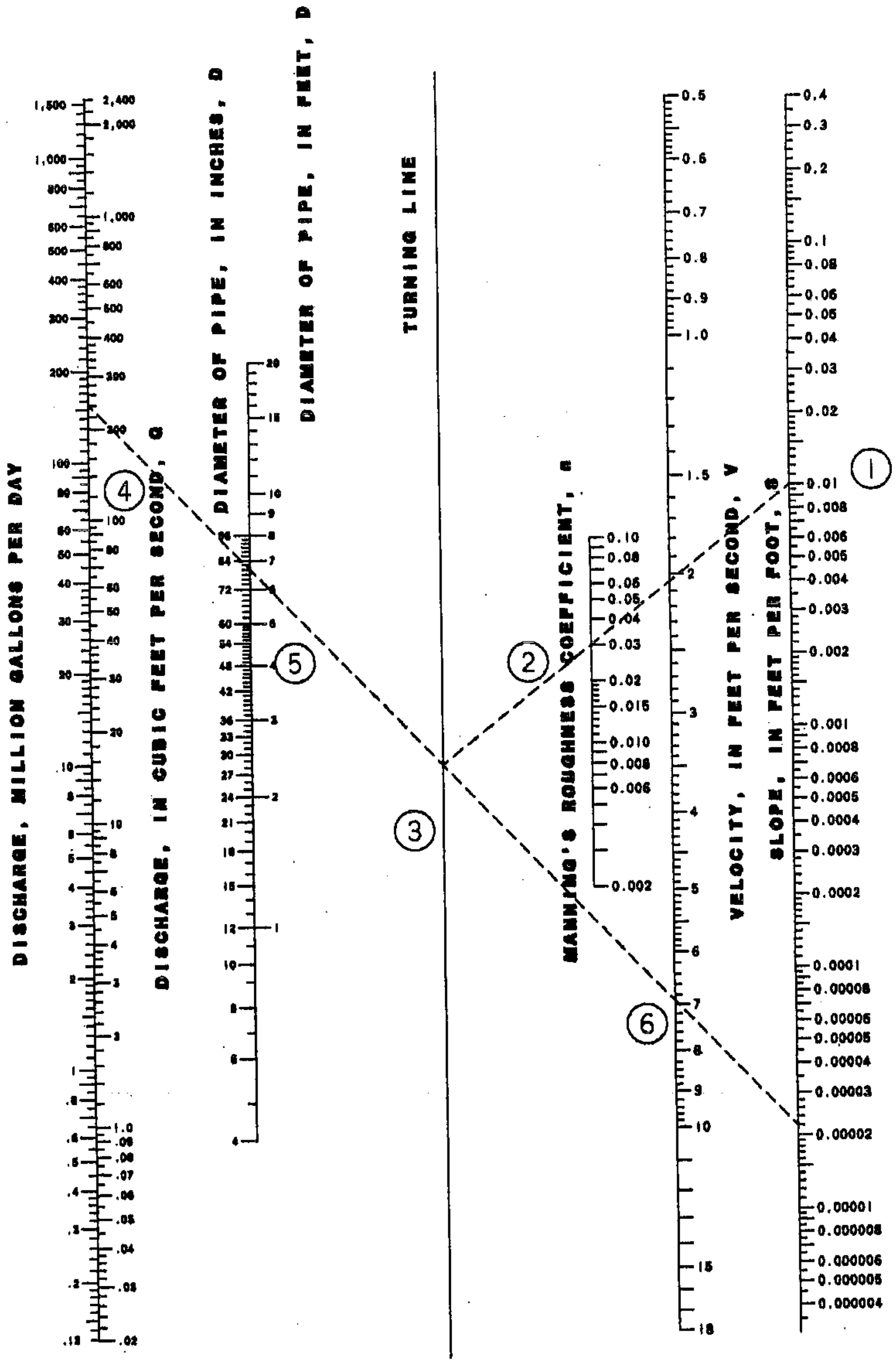
CN (weighted) = $\frac{\text{total col. (5)}}{\text{total col. (4)}} \left[\frac{7526}{100} \right] = 75.3$; use CN = $\boxed{75}$

Ponding and Swampy areas (PND) = 1 acres, 3 % of DA

Time of Concentration (TC) = 25 minutes 0.42 hours

		1st Storm	2nd Storm	3rd Storm	
2)	Rainfall Frequency (F)	10	25	100	yrs.
	Rainfall Depth (P) From Table 5-1	4.08	4.80	5.76	inches
3)	Initial Abstraction (Ia) $Ia = 0.2 \left[\frac{1000}{\text{weighted CN}} - 10 \right]$	0.67	0.67	0.67	
4)	(Ia) / (P)	0.16	0.14	0.12	
5)	Unit Peak Discharge Use TC, Ia/P, and Exhibit VI-7	560	565	570	CFS/Square Mile-inch
6)	Runoff Depth (Q) Use P, CN, and Exhibit VI-6	1.73	2.29	3.08	inches
7)	Ponding and Swampy Area Adjustment Factor Use % PND, and Exhibit VI-8	0.75	0.75	0.75	
8)	Adjusted Peak Discharge (q_g) Drainage area x step 5 x (step 6 x step 7) / 640	39	52	70	cfs
9)	Total Runoff Volume Step 6 x Drainage Area / 12	4.9	6.5	8.7	acre-feet

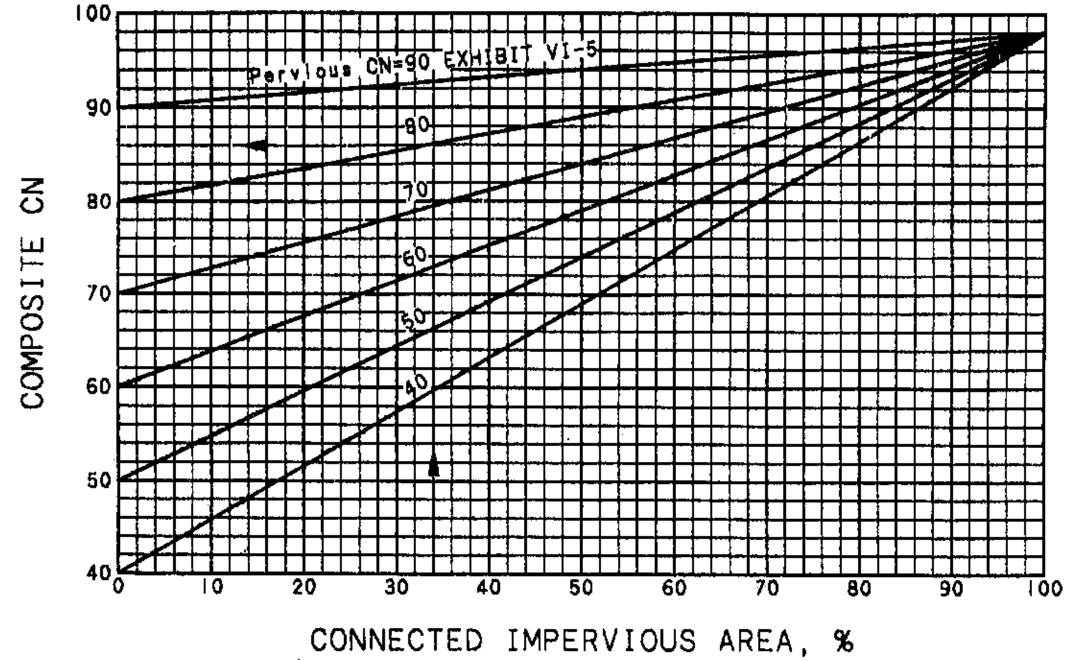
EXHIBIT V1-9



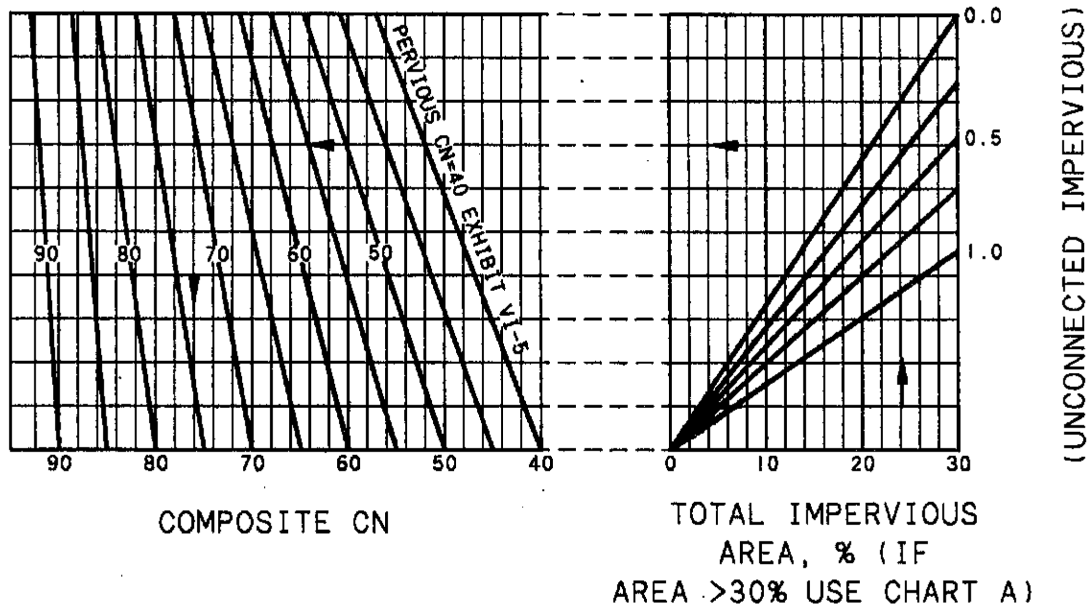
NOMOGRAPH FOR SOLUTION OF THE MANNING FORMULA

$$Q=AV=A\frac{1.49}{n}r^{2/3}s^{1/2}$$

EXHIBIT VI-10



A. COMPOSITE RUNOFF CURVE NUMBER WITH IMPERVIOUS AREA FLOWING DIRECTLY (CONNECTED) INTO THE DRAINAGE SYSTEM.



B. COMPOSITE RUNOFF CURVE NUMBER WITH IMPERVIOUS AREA NOT FLOWING DIRECTLY (UNCONNECTED) TO THE DRAINAGE SYSTEM

COMPOSITE RUNOFF CURVE NUMBER WITH IMPERVIOUS SURFACES

EXHIBIT VI-11

STORMWATER RUNOFF
GRAPHICAL PEAK DISCHARGE COMPUTATIONS

PROJECT ____________________ DESIGNER ____________ DATE ______

1) DATA: WATERSHED CONDITION = ____________________ (PRESENT OR FUTURE) TYPE II STORM

DRAINAGE AREA (DA) = __________ ACRES.

Hydrologic Soil Group Exhibit XI-3	Land Use Description Include Treatment, Practice & Condition Exhibit XI-5	CN Exhibit (3) XI-5	CN Exhibit (3) XI-11	Area (acres)	Area (%) (4)	Product (3) x (4) (5)
			Totals =		100	

CN (weighted) = $\frac{\text{total col. (5)}}{\text{total col. (4)}} \left[\frac{\quad}{\quad} \right]$ = ________ ; use CN = [ ]

Ponding and Swampy areas (PND) = ________ acres, ________ % of DA

Time of Concentration (TC) = ________ minutes ________ hours

		1st Storm	2nd Storm	3rd Storm	
2)	Rainfall Frequency (F)				yrs.
	Rainfall Depth (P) From Table 5-1				Inches
3)	Initial Abstraction (Ia) $Ia = 0.2 \left[\frac{1000}{\text{weighted CN}} - 10 \right]$				
4)	(Ia) / (P)				
5)	Unit Peak Discharge Use TC, Ia/P, and Exhibit XI-7				CFS/Square Mile-Inch
6)	Runoff Depth (Q) Use P, CN, and Exhibit XI-6				Inches
7)	Ponding and Swampy Area Adjustment Factor Use % PND. and Exhibit XI-8				
8)	Adjusted Peak Discharge (q_g) Drainage area x step 5 x (step 6 x step 7) / 640				cfs
9)	Total Runoff Volume Step 6 x Drainage Area / 12				acre-feet

FORM T6-1
(6.4.1)

CHAPTER 7. OPEN CHANNELS

7.1 Introduction

Properly designed and maintained, open channels become an efficient and cost-effective system for controlling and disposing stormwater runoff. This section of the manual addresses open channel design for roadside ditches, man-made channels, and natural streams in small drainage areas. Studies of large streams, being more complex, should be undertaken only with the assistance of a qualified staff of experienced hydrologists and experts in related fields.

One objective of open channel flow design is to determine a channel shape and size that will have sufficient capacity to prevent undue flooding damage during the anticipated peak runoff period and a velocity that does not cause erosion of the channel. In this Chapter, methods of open channel design will be discussed, while erosion control measures are included in Chapter 11.

7.1.1 Advantages and Disadvantages

Open channel design has many advantages in the management and control of stormwater runoff. It provides an opportunity for natural infiltration of stormwater into groundwater supply and probably most important, extends the time of concentration of runoff, helping to maintain the runoff rate nearer to that which exists prior to development. Other advantages of open channels include lower construction costs, opportunities for recreational activities, an aesthetically pleasing rural look, and the capability of being designed with an emergency overflow for major storms.

The opportunities presented by open channel flow must be measured against the problems, real or potential. Stream beds and banks require the use of valuable land. Channels must be maintained to assure their proper functioning and retain their aesthetic quality. Design factors must assure this natural water resource does not present health and safety hazards.

7.1.2 Guidelines for Evaluation

Open channels and swales should harmonize with the natural features of the site. By relating closely to individual lots, the lot owner will help with the maintenance and not be tempted to dispose of grass clippings and other debris on them. If installed in neighborhoods without a strong pride of ownership, they can become depositories for debris. Integrating the open channel into a linear corridor which is appropriately landscaped can help make a channel an aesthetic focal point, encourage care and maintenance, and discourage abuse.

Some of the features which should be considered when evaluating the appropriateness of open channel flow are:

Maximum anticipated discharge
Maximum allowable velocity
Slope of channel bottom
Available area in corridor
Ability to drain adjacent lots
Type of soil
Maintenance

Availability of material
Area for waste disposal
Neighborhood character
Green belt and open space requirements
Traffic patterns
Neighborhood children population
Pedestrian traffic
Recreational needs.

7.1.3 Selection of Shape

Open channels are usually designed with sections of regular geometric shapes. The most commonly used shapes are the trapezoid, rectangle, and triangle.

The trapezoid is the most common shape for channels with unlined earth banks, for it provides side slopes for stability. The rectangle and triangle are special cases of the trapezoid. Since the rectangle has vertical sides, it is commonly used for channels built of stable materials, such as lined masonry, rocks, metal, or timber. The triangular section is used only for small ditches and roadside gutters.

7.2 Design Criteria

The design criteria for open channels include minimum standards for the design storm, depth of flow, channel linings, and minimum bottom slope.

7.2.1 Design Storm

Roadside ditches shall be designed for the 10-year storm. All other open channels, except major channels as defined herein, shall be designed for the 25-year storm. Major channels shall be designed for the 100-year storm. Major channels are the following streams: Ohio River, Little Miami River, Mill Creek, Mill Creek-East Fork, Mill Creek-West Fork, Duck Creek, Muddy Creek, West Fork, Clough Creek, Congress Run, and Amberley Creek.

7.2.2 Bankful Depth of Flow

For subcritical flow, the bankful depth shall be equal to or greater than the design flow depth. For supercritical flow design, the channel shall be sized so that the bankful depth is equal to or greater than the critical depth for the design flow. A primary criterion for determining which state of flow exists is the depth of flow. If the flow depth (actual or as computed with the Manning equation) is deeper than the critical depth for the given discharge and channel shape, then subcritical flow exists. If the flow depth is less than critical depth, then supercritical flow exists. If the flow depth equals the critical depth, then critical flow exists. The depth of flow (discharge computation on open channel worksheet) shall be computed for the 50- and 100-year frequency storms for all open channels except roadside ditches and major channels.

7.2.3 Channel Linings

For channels with subcritical flow, channel bottoms shall be sodded and channel side slopes which are flatter than 2:1 may be sodded or seeded. Channel side slopes of 2:1 or steeper shall be protected with sod or lined with concrete, riprap, gabions, brick, asphalt, or other erosion-resistant lining. For channels with supercritical flow, the bottom and sides of the channel shall be concrete-lined.

7.2.4 Minimum Bottom Slope

The recommended minimum channel bottom slope shall be 0.50 percent for paved or lined channels and 1.00 percent for grass or sod-lined channels.

7.3 Types of Flow

Flow in an open channel is steady if the depth of flow is not varying with time, and is unsteady if the depth of flow is varying with time. These two main types of flow, steady and unsteady, can further be defined as being uniform, nonuniform, gradually varied, and rapidly varied. Steady uniform flow is the basic type of flow treated in open channel hydraulics and is the subject of the remainder of this chapter. The depth of flow does not change during the time interval under consideration.

Steady uniform flow can be further subdivided into three states of flows: subcritical (tranquil), critical, and supercritical (shooting) flow. Gravity forces dominate in subcritical flow, producing low velocity flow. Inertial forces dominate in supercritical flow, resulting in high velocity flow. Critical flow is the dividing point between subcritical and supercritical flow. A primary criterion for determining which state of flow exists is the depth of flow. If the flow depth (actual or as computed with the Manning equation) is deeper than the critical depth for the given discharge and channel shape, then subcritical flow exists. If the flow depth is less than critical depth, then supercritical flow exists. If the flow depth equals the critical depth, then critical flow exists.

At the critical state of flow, there exists certain relationships between specific energy and discharge and specific energy and depth of flow. Specific energy in a channel section is defined as the energy per pound of water at any section of a channel measured with respect to the channel bottom. For a given channel and discharge (Q), the specific energy in a channel section is a function of the depth of flow (d_f) only. The two conditions which describe critical flow are:

1. The discharge is maximum for a given specific energy head.

2. The specific energy head is minimum for a given discharge.

Changes in channel shape, slope, roughness, or alignment can be reflected by a drastic change in depth of flow. The designer should be aware that the depth in a given channel section may be influenced by conditions either downstream or upstream, depending on whether the slope is steep (supercritical) or mild (subcritical).

Uniform flow at or near critical depth is unstable. This results from the fact that the unique relationship between energy head and depth of flow which must exist in critical flow is substantially disturbed by minor changes in energy. Therefore, if any change in slope, roughness or sedimentation occurs, the flow would become wavy caused by appreciable changes in depth.

Because of this unstable flow, channels carrying uniform flow at or near critical depth should not be used unless the situation provides no alternative. In this case, allowance must be made in design for the height of wave generated. Often, when topography restricts the channel slope, the flow can be forced into subcritical stable or supercritical stable flow by varying the width of the channel.

Supercritical flow is difficult to control because its inertial forces become dominant; so the flow has a high velocity and is usually described as rapid, shooting, and turbulent. Changes in the channel slope (roughness, shape, or structures) could entail complex computations. For additional information regarding the design of open channels where supercritical flow could cause problems, Open-Channel Hydraulics by Ven Te Chow is a practical and simplified text.

Subcritical flow is least affected by channel changes. It will require a much larger cross-section, but will be much easier to control than the same quantity of water flowing at a supercritical velocity. Therefore, it is the preferred state of flow for design of open channels.

If a choice is possible between the two, consideration will have to be given to the amount of land available for channel, alignment, structures, and other channels or drains entering the system.

7.4 Design For Steady Uniform Flow

To calculate steady uniform flow in an open channel, the elements of the cross-section are needed. They are: (1) The cross-sectional area of flow (A) in square feet; (2) the wetted perimeter of the channel (W_p) in feet; and the hydraulic radius of the channel (r) in feet. The hydraulic radius (r) is calculated as the cross-sectional area (A) divided by the wetted perimeter (W_p).

With a given depth of flow (d_f) in an open channel having a uniform cross-section, a mean velocity (V) can be calculated by the Manning equation: $V = (1.49\ r^{2/3}\ S^{1/2})/n$, where V is the mean velocity in feet per second, "n" is Manning's coefficient of roughness, S is the slope of channel in feet per foot, and r is the hydraulic radius of the channel in feet.

The discharge (Q) is then calculated: $Q = VA$ where Q is the discharge in cubic feet per second, V is the mean velocity in feet per second, and A is the cross-sectional area of flow in square feet.

The Manning equation is simple to use and gives a reliable estimate of velocity as long as discharge, channel cross-section, and slope are constant.

General formulas for determining elements for various channel shapes are given in Exhibit VII-1.

7.4.1 Flow Depth and Velocity

To design open channels using Manning's equation, the channel cross-section must be known as well as the depth of flow (d_f), to determine velocity. The known factors are usually discharge and slope. To use Manning's formula, a method must be used to arrive at the proper channel configuration.

One method is to use channel charts similar to the ones developed by the Bureau of Public Roads[1], Design Charts for Open-Channel Flow, Hydraulic Design Series No. 3.

Knowing the discharge, the slope, and the coefficient of roughness, the depth of flow and velocity can be read directly for a given channel. It is a fairly simple method to use with reasonable accuracy. These charts are based on two to one (2:1) side slopes, and other charts must be developed for side slopes different from 2:1. The process to compute the curves and develop new charts is given in the manual referenced.

Another method which gives comparable results is tabulating the product of the area times the two-thirds power of the hydraulic radius ($Ar^{2/3}$) for various depths and areas of the section (see Table 7-1). The usefulness if apparent when the Manning formula is rearranged to: $Ar^{2/3} = (nQ)/(1.49S^{1/2})$.

Knowing the discharge Q, Manning's coefficient of roughness (n) and the channel slope (S), $Ar^{2/3}$ can easily be computed. Then, by looking at Table 7-1, a channel can be chosen corresponding to a depth of flow. It can be seen by the table that any channel can meet the design flow, but at different depths of flow. The table is a quick method to determine if a particular trapezoidal channel will flow at a desirable depth.

Also, where certain maximum or minimum depth requirements must be met, a channel can be selected that will give the required depth of flow. As one can see, a number of channel sections can readily be tried with reasonable results in a short time.

7.4.2 Example - Finding Flow Depth

Determine the depth and velocity of flow in a trapezoidal channel with a 2-foot bottom (b = 2), 3:1 side slopes (c = 3), a roughness coefficient of 0.03 (n = 0.03), a slope of 0.01 feet per foot (S = 0.01) and a discharge of 30 cfs (Q = 30).

Step 1. Calculate $Ar^{2/3} = (nQ)/(1.49S^{1/2}) = (0.030x30)/(1.49x0.01^{1/2}) = 6.04$.

Step 2. Use Table 7-1 with b = 2 and c = 3, and find d_f values closest to $Ar^{2/3} = 6.04$,

for $Ar^{2/3} = 5.30$, $d_f = 1.20$ feet

for $Ar^{2/3} = 6.33$, $d_f = 1.30$ feet.

Step 3. By interpolation, calculate depth of flow for $Ar^{2/3} = 6.04$.

$$d_f = 1.2 + \frac{6.04 - 5.30}{6.33 - 5.30} \times (1.30-1.20) = 1.2 + 0.07 = 1.27 \text{ feet.}$$

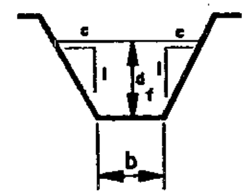
Step 4. Determine the flow area, using the area equation for a trapezoidal section given on Exhibit VII-1.

$$A = (b + c\, d_f)\, d_f = (2 + 3 \times 1.27)\, 1.27 = 7.38 \text{ square feet.}$$

[1]Can be obtained from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 70402.

d_f	b=2, c=2		b=2, c=3		b=2, c=4		b=3, c=2		b=3, c=3		b=3, c=4	
	$Ar^{2/3}$	A	$Ar^{2/3}$	A	$Ar^{2/3}$	A	$Ar^{2/3}$	A	$Ar^{2/3}$	A	$Ar^{2/3}$	A
0.5	0.75	1.50	0.85	1.75	0.95	2.00	1.05	2.00	1.15	2.25	1.24	2.50
0.8	1.85	2.88	2.21	3.52	2.56	4.16	2.50	3.68	2.85	4.32	3.19	4.96
1.0	2.90	4.00	3.56	5.00	4.20	6.00	3.83	5.00	4.47	6.00	5.10	7.00
1.1	3.53	4.62	4.38	5.83	5.21	7.04	4.60	5.72	5.44	6.93	6.26	8.14
1.2	4.23	5.28	5.30	6.72	6.35	8.16	5.47	6.48	6.53	7.92	7.56	9.36
1.3	5.00	5.98	6.33	7.67	7.63	9.36	6.41	7.28	7.73	8.97	9.01	10.66
1.4	5.86	6.72	7.48	8.68	9.06	10.64	7.44	8.12	9.05	10.08	10.61	12.04
1.5	6.79	7.50	8.74	9.75	10.64	12.00	8.56	9.00	10.49	11.25	12.38	13.50
1.6	7.81	8.32	10.13	10.88	12.38	13.44	9.77	9.92	12.07	12.48	14.32	15.04
1.7	8.91	9.18	11.64	12.07	14.29	14.96	11.07	10.88	13.78	13.77	16.43	16.66
1.8	10.10	10.08	13.28	13.32	16.37	16.56	12.47	11.88	15.63	15.12	18.71	18.36
1.9	11.38	11.02	15.05	14.63	18.63	18.24	13.97	12.92	17.62	16.53	21.19	20.14
2.0	12.76	12.00	16.97	16.00	21.07	20.00	15.56	14.00	19.76	18.00	23.85	22.00
2.1	14.23	13.02	19.03	17.43	23.70	21.84	17.27	15.12	22.05	19.53	26.71	23.94
2.2	15.81	14.08	21.23	18.92	26.53	23.76	19.07	16.28	24.49	21.12	29.77	25.96
2.3	17.48	15.18	23.59	20.47	29.55	25.76	20.99	17.48	27.09	22.77	33.04	28.06
2.4	19.26	16.32	26.10	22.08	32.78	27.84	23.01	18.72	29.85	24.48	36.51	30.24
2.5	21.14	17.50	28.77	23.75	36.22	30.00	25.15	20.00	32.78	26.25	40.21	32.50
2.6	23.13	18.72	31.61	25.48	39.87	32.24	27.41	21.32	35.88	28.08	44.13	34.84
2.7	25.24	19.98	34.61	27.27	43.75	34.56	29.78	22.68	39.15	29.97	48.28	37.26
2.8	27.45	21.28	37.78	29.12	47.85	36.96	32.27	24.08	42.59	31.92	52.65	39.76
2.9	29.79	22.62	41.21	31.03	52.18	39.44	34.88	25.52	46.22	33.93	57.27	42.34
3.0	32.24	24.00	44.64	33.00	56.75	42.00	37.62	27.00	50.03	36.00	62.13	45.00

TRAPEZOIDAL CHANNELS HYDRAULIC CHARACTERISTICS



$$Ar^{2/3} = \frac{n\ Q}{1.49\sqrt{s}}$$

r=A/wp

TABLE 7-1

Step 5. Calculate velocity of flow = V = Q/A = 30/7.38 = 4.07 fps.

7.4.3 Coefficient of Roughness (n)

The computed discharge for any given channel will only be as reliable as the estimated value of "n" used in making the computation. The value of "n" is not a fixed value and varies with the season, and from year to year. Each year, the value of "n" increases in the spring and summer months as vegetation grows, and diminishes in the fall as vegetation becomes dormant. The annual growth of vegetation, uneven accumulation of sediment in the channel, lodgment of debris, erosion, and sloughing of banks and other factors, tend to increase the value of "n" from year to year until the hydraulic efficiency of the channel is improved by clearing or cleaning out.

All of these factors should be studied and evaluated with respect to the type of channel, degree of maintenance, seasonal requirements, season of year design storm occurs, and other considerations before selecting the value of "n." The most conservative value of "n", usually when plants are in foliage, shall be used for design unless otherwise approved by the Stormwater Management Utility. In Exhibit VII-2, values for "n" have been tabulated to help the designer choose an appropriate value.

Because of the erosion effects velocity has on the channel, Exhibits VII-3 and VII-4 have been included to determine the maximum permissible velocity for a channel. The following is a general guide to determine when and what type of channel linings are required: for velocity below 2.5 fps, no special requirements; for velocity between 2.5 and 4.0 fps, seeded or sodded channels; and for velocity greater than 4.0 fps, special channel protection materials.

7.4.4 Critical Depth

Critical depth depends only on discharge and shape of the channel, and is independent of the slope or channel roughness. Therefore, in any given channel size, there is only one critical depth for a particular discharge. By using Exhibit VII-5, the critical depth can be found for a trapezoidal or rectangular channel.

To compute critical depth by Exhibit VII-5:

Step 1. Compute section factor (Z) $Z = Q/g^{1/2}$, where Q is discharge (cfs) and g is the acceleration of gravity (32.2 feet per second squared).

Step 2. Compute value of $Z/b^{2.5}$, where b is the bottom width of the channel in feet.

Step 3. Use Exhibit VII-5, with the value of $Z/b^{2.5}$ and intersect it with the curve corresponding to the appropriate side slope (c:1).

Step 4. Project horizontally from this point and read the value of d_c/b.

Step 5. Multiply this d_c/b value by the bottom width (b) to obtain the critical depth. $d_c = (d_c/b) \times b$.

An example is included here to illustrate critical depth computations.

7.4.5 Example - Finding Critical Depth

Find the critical depth (d_c) for a trapezoidal channel with a bottom width of 2 feet (b = 2.0) and side slopes of 2:1 (C = 2), discharging 25 cfs (Q = 25).

Step 1. Calculate $Z = Q/g^{1/2} = 25/32.2^{1/2}$ $25/5.67 = 4.41$.

Step 2. Calculate $Z/b^{2.5} = 4.41/(2)^{2.5} = 4.41/5.66 = 0.78$.

Step 3. Use Exhibit VII-5, find 0.78 on $Z/b^{2.5}$ scale and intersect with curve C = 2.

Step 4. Project horizontally and read on d_c/b scale: $d_c/b = 0.58$.

Step 5. Calculate critical depth = dc = $d_c/b \times b = 0.58 \times 2 = 1.16$ feet.

Knowing critical depth, critical velocity can be calculated by using $V_c = Q/A_c$. Where A_c = cross-sectional area for critical flow, Exhibit VII-1 gives an equation to calculate area. For a trapezoidal section $A_c = (b + cd_c)\ d_c = (2 + 2 \times 1.16) \times 1.16 = 5.01$ square feet. Therefore, the critical velocity = $V_c = Q/A_c = 25/5.01 = 4.99$ fps.

7.4.6 Summary of Design Procedures

The following summarizes general procedures for the design of open channels using the formatted Form T7-1. Exhibit VII-8 is an open channel computation worksheet that references how values are obtained for each column.

Step 1. Fill in frequency Q, "n", S, and V_{max} in columns 1, 2, 3, 4, and 12.

Step 2. Quantify value of $Ar^{2/3}$ for Discharge (Q) and Velocity (V), column 5: $Ar^{2/3} = (nQ)/(1.49S^{1/2})$.

Step 3. Calculate minimum area of channel (A_{min}) column 6 that will flow within limit set by V_{max}. A_{min} = Q(column 2)/V_{max} (column 12).

Step 4. Using Table 7-1, select channel configuration of bottom width = b (column 7) and side slope = c (column 7).

Step 5. Find d_f (column 8) by interpolating from Table 7-1, using $Ar^{2/3}$ (column 5).

Step 6. Calculate channel flow area A (column 9), from equation given on Exhibit VII-1.

Step 7. Calculate top width of flow T (column 10), using equation from Exhibit VII-1.

Step 8. Calculate channel velocity V (column 11) from area derived in Step 6 and discharge by V = Q/A. Check that this channel velocity (column 11) does not exceed maximum permissible velocity (column 12). If V (column 11) $\leq V_{max}$ (column 12), continue, if not, choose a different "n" or S and restart at Step 1, or choose a different channel cross-section and restart at Step 4.

Step 9. Calculate Z, for critical flow $Z = Q/g^{1/2}$ (column 13).

Step 10. Calculate $Z/b^{2.5}$ (column 14).

Step 11. Using Exhibit VII-5, find d_c/b (column 15).

Step 12. Multiply $d_c/b \times b$ to get d_c (column 16).

Step 13. Using d_c, calculate V_c (column 17), $V_c = (Q)/[(b+cd_c)d_c]$.

Step 14. Compare the critical flow depth and velocity values equal to the channel depth and velocity values.

If column 16 (d_c) < column 8 (d_f)
and column 17 (V_c) > column 11 (v) flow is subcritical.

If column 16 (d_c) > column 8 (d_f)
and column 17 (V_c) < column 11 (v) flow is supercritical.

If column 16 (d_c) = column 8 (d_f)
and column 17 (V_c) = column 11 (v) flow is critical.

Step 15. When an acceptable channel design for discharge and velocity has been selected with regard to discharge, capacity, and critical flow considerations, then the total channel depth as required by the design criteria is determined as:

a. for subcritical flow channel depth $\geq d_f$, or

b. for supercritical flow channel depth $\geq d_c$.

Step 16. Compute the anticipated depth of flow in the design channel for the 50- and 100-year frequency storm.

7.4.7 Example - Open Channel Design

The following example is included to further illustrate the open channel design procedures stated herein and follows the steps given in the preceding section. Exhibit VII-6 is a completed open channel computation worksheet for this example.

Find the design channel depth for a trapezoidal channel that has a 3-foot bottom (b = 3), a 2:1 side slope (c = 2), a slope of 0.010 feet per foot. (S = 0.010 and an "n" being based on an excavated open channel, straight alignment, with clean sides, and a coarse gravel bottom to convey a 25-year discharge of 88 cfs (Q_{25} = 88), a 50-year discharge of 108 cfs (Q_{50} = 108), and a 100-year discharge of 129 cfs (Q_{100} = 129).

Open channel design for the 25-year storm.

Step 1. Use Exhibit VII-2 and Exhibit VII-3 and read: "n" = 0.03 and V_{max} = 5.5 fps. Complete columns 1, 2, 3, 4, and 12.

Step 2. Compute $Ar^{2/3} = (nQ)/(1.49S^{1/2}) = (0.03 \times 88)/(1.49 \times 0.010^{1/2}) = 17.72$.

Step 3. Determine $A_{min} = Q/V_{max} = 88/5.5 = 16.00$ square feet.

Step 4. Select channel with bottom width b = 3 and side slope c = 4.

Step 5. Use Table 7-1, with $Ar^{2/3}$ = 17.72 and find the largest d_f values closest to either $Ar^{2/3}$.

$Ar^{2/3}$	d_f
16.43	1.7
18.71	1.8

Calculate d_f for $Ar^{2/3}$ = 17.72 by interpolation
d_f = 1.7 + [(17.72 - 16.43)/(18.71 - 16.43)]
(1.8 - 1.7) = 1.76 feet.

Use d_f of 1.76.

Step 6. Calculate channel flow area = A = (b + cd_f) d_f = (3 + 4 x 1.76) 1.76 = 17.67 square feet.

Step 7. Calculate top width of flow = T = b + 2 cd_f = 3 + 2 + 4 x 1.76 = 17.08 feet.

Step 8. Calculate channel velocity = V = Q/A = 88/17.67 = 4.98 fps.

For velocity check, V $\leq$ V_{max} = 4.98 fps < 5.5 fps, therefore design is acceptable for velocity.

Step 9. Calculate Z = $Q/g^{1/2}$ = $88/32.2^{1/2}$ = 15.51.

Step 10. Calculate $Z/b^{2.5}$ = $15.51/3^{2.5}$ = 0.995.

Step 11. Use Exhibit VII-5 with $Z/b^{2.5}$ = 0.995 and c = 4 and read d_c/b = 0.55.

Step 12. Compute d_c = d_c/b x b = 0.55 x 3 = 1.65 feet.

Step 13. Compute V_c = $Q/((b + cd_c)d_c)$ = 88/((3 + 4 x 1.65)x1.65) = 5.56 fps.

Step 14. Compare calculated d_f and V to calculated d_c and V_c.

D_f = 1.76 > d_c = 1.65

and V = 4.98 < V_c = 5.56 therefore, flow is subcritical.

Step 15. Design channel depth for subcritical flow shall be 1.76 feet or greater.

To complete the design of the open channel, the anticipated depth of flow in the designed channel shall be computed for the 50- and 100-year frequency storm. The computations follow the steps given in the preceding section.

Step 1. Record the design channel "n" = 0.30, S = 0.010 feet per foot, and V_{max} = 5.5 fps from the preceding work in columns 3, 4, and 12. Record the Q = 108 cfs in column 2.

Step 2. Calculate $Ar^{2/3}$ = $(nQ)/(1.49S^{1/2})$ = $(0.03\times108)/(1.49\times0.010^{1/2})$ = 21.74.

Step 3. Determine A_{min} = Q/V_{max} = 108/5.5 = 19.64 square feet.

Step 4. Channel configuration has been chosen, with bottom width b = 3 and side slope c = 4.

Step 5. Use Table 7-1 with $Ar^{2/3}$ = 21.74 and find, by interpolation, the d_f value to satisfy $Ar^{2/3}$.

$$d_f = 1.9 + \frac{(21.74 - 21.19)}{23.85 - 21.19} (2.0 - 1.9) = 1.92 \text{ feet}$$

Step 6. Calculate channel flow area = A = $(b + cd_f)d_f$ = (3 + 4 x 1.92) 1.92 = 20.51 square feet.

Step 7. Calculate top width of flow = T = b + 2 cd_f = 3 + 2 x 4 x 1.92 = 18.36 feet.

Step 8. Calculate channel velocity V = Q/A = 108/20.51 = 5.27 fps.

Step 9. Calculate $Z = Q/g^{1/2} = 108/32.2^{1/2} = 19.03$.

Step 10. Calculate $Z/b^{2.5} = 19.03/3^{2.5} = 1.22$.

Step 11. Use Exhibit VII-5 with $Z/b^{2.5}$ = 1.22 and C = 2 read d_c/b = 0.60.

Step 12. Compute $d_c = d_c/b \times b = 0.60 \times 3 = 1.80$ feet.

Step 13. Compute $V_c = Q/((b + c\ (d_c)\ d_c) = 108/((3 + 4 \times 1.80)\ 1.80)$ = 5.88 fps.

Step 14. Compare calculated d_f and V to calculated d_c and V_c.

$d_f = 1.92 > d_c = 1.80$
$V = 5.27 < V_c = 5.88$

Therefore, flow is subcritical.

Step 15. The anticipated channel flow depth is 1.92 feet.

Anticipated depth of flow for the 100-year storm.

The calculations for the anticipated depth of flow for the 100-year storm are summarized on Exhibit VII-6 with the calculated depth of flow d_f = 2.08 feet and with a channel velocity V = 5.48 fps. The type of flow is subcritical, and the anticipated depth of flow for the 100-year storm is 2.08 feet.

7.4.8 Example - Roadside Ditch Design

Find the design channel depth for a trapezoidal roadside ditch that has a 2-foot bottom (b = 2), 4:1 side slopes (c = 4), a slope of 0.010 feet per foot (S = 0.010), and an excavated, straight alignment, lined with grass sides and a sod bottom where Manning's roughness coefficient is 0.025 (n = 0.025), a maximum permissible velocity of 4 fps (V_{max} = 4), and a 10-year discharge of 15 cfs (Q = 15). Exhibit VII-7 is a completed roadside ditch computation worksheet for this example.

Step 1. Complete columns 1, 2, 3, 4, and 12.

Step 2. Compute $Ar^{2/3} = (nq)/(1.49\ S^{1/2}) = (0.025 \times 15)/(1.49 \times 0.010^{1/2})$ = 2.52.

Step 3. Determine $A_{min} = Q/V_{max} = 15/4 = 3.75$ square feet.

Step 4. Channel configuration is bottom width b = 2 and side slope c = 4.

Step 5. Use Table 7-1, with $Ar^{2/3} = 2.52$ and find the d_f values closest to $Ar^{2/3}$.

$Ar^{2/3}$	d_f
0.95	0.5
2.56	0.8

Calculate d_f for $Ar^{2/3} = 2.52$. By interpolation $d_f = 0.5 + [(2.52 - 0.95)/(2.56 - 0.95)] (0.8 - 0.5) = 0.79$ feet.

Use d_f of 0.79.

Step 6. Calculate channel flow area = $A (b = cd_f) d_f = (2 + 4 \times 0.79)0.79 = 4.08$ square feet.

Step 7. Calculate top width of flow = $T = b + 2 cd_f = 2 + 2 \times 4 \times 0.79 = 8.32$ feet.

Step 8. Calculate channel velocity = $V = Q/A = 15/4.08 = 3.68$ fps. For velocity check, $V \leq V_{max} = 3.68$ fps < 4 fps; therefore, design is acceptable for velocity.

Step 9. Calculate Z for critical flow $Z = Q/g^{1/2} = 15/32.2^{1/2} = 2.64$.

Step 10. Calculate $Z/b^{2.5} = 2.64/2^{2.5} = 0.47$.

Step 11. Use Exhibit VII-5 with $Z/b^{2.5} = 0.47$ and $c = 4$ and read $d_c/b = 0.38$.

Step 12. Compute $d_c = d_c/b \times b = 0.38 \times 2 = 0.76$ feet.

Step 13. Compute $V_c = Q/([b + cd_c] d_c) = 15/([2 + 4 \times 0.76) 0.76] = 3.92$ fps.

Step 14. Compare calculated d_f and V to calculated d_c and V_c.

$$d_f = 0.79 > d_c = 0.76$$

and $V = 3.68 < V_c = 3.92$ Therefore, flow is subcritical.

Step 15. Design channel depth for subcritical flow shall be 0.79 feet or greater.

7.5 Capacity of Natural Channels

Determining the capacity of a natural channel is necessary when that stream will flow through an area where damage can occur when the channel capacity is exceeded. In addition, it is often necessary to determine the effect on channel capacity resulting from cleaning and landscaping the banks to obtain a more stable and aesthetic effect or from modifying the channel structure to obtain a more stable flow condition.

While natural channels are in a constant process of changing both their grade and course, a goal of stormwater management is to prevent acceleration of these changes as the land use of the tributary area changes. Such changes are caused by variations in: rate or duration of flow, sediment load, and/or channel geometry.

It is the change in channel geometry which can most significantly affect the flow capacity of the channel. Determination of flow capacity in a natural channel involves considerable judgement by the design engineer. The results cannot be determined with as great a certainty as for an artificial channel. Variations in the cross-section of the stream, alignment and roughness of the channel, and the changing quantities of flowing water make the determination of capacity an approximation, at best.

Two methods of determining channel capacity are presented below. First, application of computer backwater profile models and second, hand calculation of average channel capacity. The computer models provide a more sophisticated analysis of flow conditions caused by variations in channel geometry and roughness as well as bridges, culverts, and other obstructions to flow. The hand calculation method provides acceptable results when computer modeling is not available.

7.5.1 Computer Backwater Profile Models

Several computer models are available for computing backwater profiles in natural channels. These profiles provide the water surface elevations along the reach of the channel under analysis for a number of design discharges. Of the models available, the HEC-2 Water Surface Profile computer program, developed by the U.S. Army Corps of Engineers, is one of the most widely used and accepted programs. Basic data needed to run the program are:

a. Surveyed river cross-sections at all control locations, at selected intermediate locations, and at all bridges, culverts, dams, and other obstructions to flow

b. Estimates of Manning's roughness coefficients at each cross-section

c. Design discharges in the channel.

Using this data, the program computes water surface elevations at each cross-section which, when plotted, yield the flood profiles along the stream. These profiles can then be used to plot limits of flooding on topographic maps of the study area for determining the areal extent of flooding. Should the proposed design facilities result in inadequate capacity in the channel as determined by the HEC-2 model, then the facilities and channel should be reviewed for the causes of inadequate capacity (see Article 7.5.4).

For further information on use of the HEC-2 computer program, refer to the Corps of Engineers HEC-2 Users Manual.

7.5.2 Hand Calculation of Capacity

The following general procedure is not exact and gives only the bankfull capacity of each reach of stream without consideration of backwater effects. From this the rate of flow can be determined at which flooding would occur in any one reach and where bottlenecks may occur. The cross-section and profile data obtained can be used in a more sophisticated analysis of the flood profile if such is needed.

If accurate mapping is available, much of the necessary information can be obtained from it; if not, a detailed field survey will be required. In either event, field work will be required to determine the stream cross-sections, data on control points, and channel roughness.

7.5.3 Determining Capacity by Hand Calculation

The following summarizes general procedures for the determination of the natural channel capacity by hand calculation.

Step 1. From a base line established parallel to the principal course of the stream, obtain sufficient stream cross sections, at right angles to the centerline in each reach, to determine the average cross-section. The stream cross-section should be taken to sufficient width to include the major storm floodway.

Step 2. Plot the cross sections at a scale that will permit the cross-sectional area and wetted perimeter to be obtained.

Step 3. Plot a profile of the stream along the centerline of flow at design stage. A sufficiently accurate centerline can be determined by inspection. The elevation of the bottom of the channel (except for potholes) and the elevation at which the stream would leave the channel at each bank should be shown.

Step 4. Points of entry of major tributaries, significant changes in grade of the channel bottom, significant changes in stream cross-section, and bridges or culverts that would obstruct design flow should be used as control points.

Step 5. Divide the profile into reaches between each control point.

Step 6. Draw a preliminary hydraulic grade line on the profile for each reach. The hydraulic gradient should be parallel to the bottom of the channel for design flow conditions with changes in slope occurring at control points. The preliminary hydraulic grade line should be set with flow at bankful conditions or at an elevation at which flow could occur without damage to abutting land uses.

Step 7. Plot the preliminary hydraulic gradient on each cross-section.

Step 8. Determine the area and hydraulic radius of each section under the hydraulic gradient line and average these values for the reach. An alternative method is to superimpose the sections of a reach, using the hydraulic gradient as a common line, and sketch an average section by judgement. Then from this average section, determine the average area and hydraulic radius for the reach. If the latter method is used, the average section sketched can be used for further study as long as the slope of the preliminary hydraulic grade line is not changed.

Step 9. From the area and hydraulic radius so determined, the slope of the hydraulic gradient, and Manning's "n" value of the reach, a preliminary velocity of flow and hydraulic capacity of each reach can be determined.

Step 10. Using the preliminary hydraulic grade for the reach, determined as headwater or backwater conditions, determine capacities of culverts or bridges that may obstruct the channel. The capacities of culverts can be determined by the method outlined in Chapter 10.

7.5.4 Analysis of Channel Capacity Deficiencies

After the capacity of the natural stream has been determined, it should be compared with the design storm runoff. Deficiencies in the capacity of any reach should be examined to define an acceptable solution.

First, the culverts or bridges should be reviewed to determine the headwater that will result from the design storm. If this will not result in damage, change the hydraulic gradient of the upstream reach and recalculate this channel capacity with the backwater from the structure. If the backwater will result in flooding and damage, consideration could be given to increasing the size of the structure.

Next, if a few reaches of the stream channel do not have sufficient capacity to carry the design storm, they should be reviewed to determine the cause. Deficiencies along particular reaches may be eliminated without adversely affecting the entire stream.

7.6 Floodway Delineation and Regulation

The Federal Emergency Management Agency (FEMA) has published a Flood Insurance Study for the City of Cincinnati dated April 15, 1982. This study contains Flood Boundary and Floodway Maps showing the Regulatory Floodway Areas of special flood hazards, profiles of water surface elevations for the 10-, 50-, 100-, and 500-year floods along the Ohio River, Little Miami River, Muddy Creek, Mill Creek, Mill Creek-East Fork, Mill Creek-West Fork, West Fork, Clough Creek, Duck Creek, and Congress Run. Where such data exists, it should be utilized to provide floodway limits and control elevations in stormwater facility design.

The designer is referred to Title XI of the Cincinnati Municipal Code, the Cincinnati-Ohio Basic Building Code: Chapter 1156 - Floodplain Management for additional information.

7.7 Bibliography

Chow, Ven Te. Handbook of Applied Hydrology. McGraw-Hill Book Company. New York, New York 1964.

Chow, Ven Te. Open-Channel Hydraulics. McGraw-Hill Book Company. New York, New York 1959.

U.S. Department of Commerce, Bureau of Public Roads. Design Charts for Open-Channel Flow. Washington, D.C., August 1961.

U.S. Department of Agriculture. Engineering Field Manual for Conservation Practices. Soil Conservation Service, 1969.

Federal Emergency Management Agency. Guidelines and Specifications for Study Contractors. September 1982.

Keller, E. A. and E. K. Hoffman. "A Sensible Alternative to Stream Channelization." Public Works, October 1976, pp. 70-72.

Erie and Niagara Counties Regional Planning Board. Storm Drainage Design Manual. Harza Engineering Co., July 1972.

Maryland Department of Natural Resources. Stormwater Management Design: A Manual of Procedures and Guidelines, Roy F. Weston, Inc., April 1976.

Denver Regional Council of Government. Urban Storm Drainage Criteria Manual, Volume 2. Wright-McLaughlin Engineers. Denver, Colorado, 1969.

OPEN CHANNEL
SYMBOLS, EQUATION, AND GEOMETRIC FORMULA

SYMBOLS

Symbol	Units	Description
A	sq. ft.	Area of cross-section of flow
b	ft.	Bottom width of trapezoidal channel
c		Side slope of channel, c:l
d_c	ft.	Critical depth
d_f	ft.	Depth of flow
g	ft/sec^2	Acceleration of gravity = 32.2
n		Manning roughness coefficient
Q	cfs	Rate of discharge
r	ft.	Hydraulic radius = A/wp
s	ft./ft.	Slope of channel
s_c	ft./ft.	Critical slope
T	ft.	Top width of water surface in a channel
V	fps	Mean velocity of flow
Vc	fps	Critical velocity
wp	ft.	Wetted perimeter - length of line of contact between the flowing water and the channel
Z		Section factor for critical flow

Equations

$$V = \frac{1.49}{n} r^{2/3} s^{1/2} \qquad Q = AV \qquad Q = \frac{1.49}{n} Ar^{2/3} s^{1/2} \qquad Z = Q/g^{1/2}$$

Geometric Formula

Trapezoidal	Rectangle	Triangle
$A = (b + cd_f)\, d_f$	$A = bd_f$	$A = cd_f^2$
$wp = b + 2d_f\, (1 + c^2)^{1/2}$	$wp = b + 2d_f$	$wp = 2d_f\, (1 + c^2)^{1/2}$
$T = b + 2\, cd_f$	$T = b$	$T = 2\, cd_f$
$r = \dfrac{(b+cd_f)\, d_f}{b+2d_f\, (1 + c^2)^{1/2}}$	$r = \dfrac{bd_f}{b + 2d_f}$	$r = \dfrac{cd_f}{2\, (1 + c^2)^{1/2}}$

MANNING ROUGHNESS COEFFICIENTS

I. Open Channels Lined (Straight Alignment)	
A. Concrete	0.015
B. Concrete, bottom, sides, as indicated	
1. Stone in mortar	0.020
2. Riprap	0.025
C. Gravel bottom, sides as indicated	
1. Concrete	0.020
2. Riprap	0.028
D. Brick	0.017
E. Asphalt	0.015
II. Open Channels, Excavated (Straight Alignment, Natural Lining)	
A. Earth, fairly uniform section	
1. Grass, some weeds	0.025
2. Dense weeds	0.035
3. Sides clean, gravel bottom	0.030
B. Rock	
1. Based on design section	0.035
2. Based on actual mean section	
a. Smooth and uniform	0.040
b. Jagged and irregular	0.045
C. Channels not maintained, weeds and brush uncut	
1. Dense weeds, high as flow depth	0.100
2. Clean bottom, brush on sides	0.080
3. Dense brush, high stage	0.120

PERMISSIBLE VELOCITIES FOR CHANNELS WITH ERODIBLE LININGS, BASED ON UNIFORM FLOW IN CONTINUOUSLY WET, AGED CHANNELS

Soil type or lining (earth; no vegetation)	Maximum permissible velocities (fps)
Fine sand (noncolloidal)	3.0
Sand loam (noncolloidal)	3.0
Silt loam (noncolloidal)	3.0
Ordinary firm loam	3.0
Volcanic Ash	3.0
Fine gravel	5.0
Stiff clay (very colloidal)	5.0
Graded, loam to cobbles (noncolloidal)	5.0
Alluvial silts (noncolloidal)	5.0
Alluvial silts (colloidal)	5.5
Coarse gravel (noncolloidal)	5.5
Cobbles and shingles	5.5
Shales and hard pans	5.5

PERMISSIBLE VELOCITIES FOR CHANNELS LINED WITH UNIFORM STAND OF VARIOUS GRASS COVERS, WELL MAINTAINED

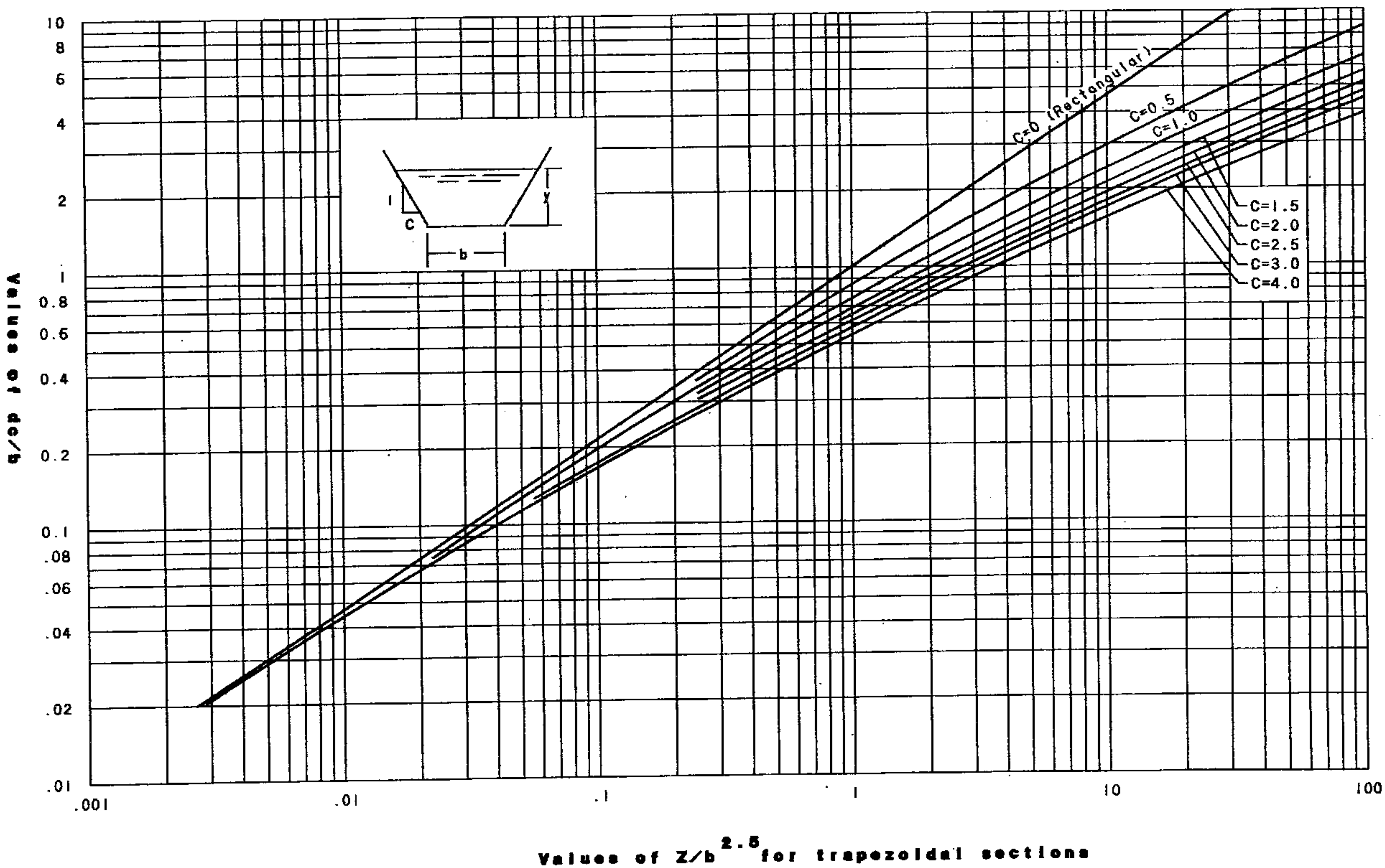
Cover	Slope Range[2]	Permissible Velocity on:[1] Erosion Resistant Soils	Easily Eroded Soils
Kentucky bluegrass	0-5	7	5
Smooth brome	5-10	6	4
Tall Fescue	Over 10	5	3
Grass Mixture	0-5	5	4
Grass Mixture[2]	5-10	4	3
Lespedeza sericea[3]	0-5	3.5	2.5
Weeping lovegrass[3]	0-5	3.5	2.5
Red Fescue[3]	0-5	3.5	2.5
Red top[3]	0-5	3.5	2.5
Alfalfa[3]	0-5	3.5	2.5
Crabgrass[3]	0-5	3.5	2.5
Common lespedeza[4]	0-5	3.5	2.5
Sundangrass[4]	0-5	3.5	2.5

[1]Use velocities exceeding 5 fps only where good covers and proper maintenance can be obtained. Erosion-resistant soils include those listed On Exhibit IX-3 with a maximum velocity of 5 fps.

[2]Do not use on slopes steeper than 10 percent except for vegetated side slopes in combination with a stone, concrete, or highly resistant vegetative center section.

[3]Do not use on slopes steeper than 5 percent except for vegetated side slopes in combination with a stone, concrete, or highly resistant vegetative center section.

[4]Annuals - use on mild slopes, less than 5 percent, or as temporary protection until permanent covers are established.



CURVES FOR DETERMINING CRITICAL DEPTH

EXHIBIT VII-5

OPEN CHANNEL COMPUTATIONS
ARTICLE 7.4.7 EXAMPLE

PROJECT ____________________ DESIGNER ____________________ DATE __________

	DISCHARGE											CRITICAL FLOW					
1 FREQUENCY	2 Q	3 n	4 s	5 $Ar^{2/3}$	6 Amin	7 b, c	8 d_f	9 A	10 T	11 V	12 Vmax	13 Z	14 $Z/b^{2.5}$	15 d_c/b	16 d_c	17 V_c	18 TYPE OF FLOW
	CFS		FT/FT		SQ FT		FT	SQ FT	FT	FPS	FPS				FT	FPS	
25 YEAR	88	0.030	0.010	17.72	16.00	3, 4	1.76	17.67	17.08	4.98	5.50	15.51	0.995	0.55	1.65	5.56	SUBCRITICAL
50 YEAR	108	0.030	0.010	21.74	19.64	3, 4	1.92	20.51	18.36	5.27	5.50	19.03	1.22	0.60	1.80	5.88	SUBCRITICAL
100 YEAR	129	0.030	0.010	25.97	23.45	3, 4	2.08	23.55	19.64	5.48	5.50	22.73	1.46	0.65	1.95	6.13	SUBCRITICAL

EXHIBIT VII-6

OPEN CHANNEL COMPUTATIONS
ARTICLE 7.4.8 EXAMPLE

PROJECT ____________________ DESIGNER ____________________ DATE __________

	DISCHARGE											CRITICAL FLOW					
1 FREQUENCY	2 Q	3 n	4 s	5 $Ar^{2/3}$	6 Amin	7 b, c	8 d_f	9 A	10 T	11 V	12 Vmax	13 Z	14 $Z/b^{2.5}$	15 d_c/b	16 d_c	17 V_c	18 TYPE OF FLOW
	CFS		FT/FT		SQ FT		FT	SQ FT	FT	FPS	FPS				FT	FPS	
10 YEAR	15	0.025	0.010	2.52	3.75	2, 4	0.79	4.08	8.32	3.68	4.0	2.64	0.47	0.38	0.76	3.92	SUBCRITICAL

EXHIBIT VII-7

OPEN CHANNEL COMPUTATIONS GUIDE

PROJECT ____________________ DESIGNER ____________________ DATE __________

	DISCHARGE											CRITICAL FLOW					
1 FREQUENCY	2 Q	3 n	4 s	5 $Ar^{2/3}$	6 Amin	7 b,c	8 d_f	9 A	10 T	11 V	12 Vmax	13 Z	14 $Z/b^{2.5}$	15 d_c/b	16 d_c	17 V_c	18 TYPE OF FLOW
	CFS		FT/FT		SQ FT		FT	SQ FT	FT	FPS	FPS				FT	FPS	
GIVEN	GIVEN	EXHIBIT VII-2	GIVEN	$\frac{\text{COLUMN 2 X COLUMN 3}}{\text{1.49 X (COLUMN 4)}^{\frac{1}{2}}}$	COLUMN 2 / COLUMN 12	SITE SPECIFIC	INTERPOLATE FROM TABLE 7-1 WITH COLUMN 5	CALCULATE BY GEOMETRIC FORMULA EXHIBIT VII-I WITH COLUMNS 7 AND 8	CALCULATE BY GEOMETRIC FORMULA EXHIBIT VII-I WITH COLUMNS 7 AND 8	COLUMN 2 / COLUMN 9	FROM EXHIBIT VII-3 OR VII-4	COLUMN 2 / $(32.2)^{\frac{1}{2}}$	COLUMN 13 / $(\text{COLUMN 7})^{2.5}$	EXHIBIT VII-5 WITH COLUMN 14	COLUMN 15 X COLUMN 7	COLUMN 2 / A WITH A CALCULATED FROM GEOMETRIC FORMULA EXHIBIT VII-I WITH COLUMNS 7 AND 16	SUPERCRITICAL IF COLUMN 11 < COLUMN 17 AND COLUMN 16 > COLUMN 8 CRITICAL IF COLUMN 11 = COLUMN 17 AND COLUMN 16 = COLUMN 8 SUBCRITICAL IF COLUMN 11 > COLUMN 17 AND COLUMN 16 < COLUMN 8

EXHIBIT VII - 8

OPEN CHANNEL COMPUTATIONS

PROJECT ______________________ DESIGNER ______________ DATE ________

	DISCHARGE											CRITICAL FLOW					
1 FREQUENCY	2 Q	3 n	4 s	5 $Ar^{2/3}$	6 Amin	7 b, c	8 d_f	9 A	10 T	11 V	12 Vmax	13 Z	14 $Z/b^{2.5}$	15 d_c/b	16 d_c	17 V_c	18 TYPE OF FLOW
	CFS		FT/FT		SQ FT		FT	SQ FT	FT	FPS	FPS				FT	FPS	

FORM T7-1
(7.4.6)

CHAPTER 8. STREETS AND INLETS

8.1 Introduction

This section of the manual outlines the role of streets and appurtenances as an integral part of the stormwater drainage system, in addition to their primary function for traffic movement. Gutter flow in streets serves to transport runoff from surface areas to storm inlets or to open drainage channels. Integration of drainage needs with street design can reduce considerably the cost of a storm sewer system.

Street inlets are included in this section as an appurtenant part of the street drainage system. The primary function of the street inlet is to provide stormwater runoff flow from the gutter or street side ditch into the storm sewer.

8.2 Design Criteria

The design criteria for streets and inlets include minimum standards for design storms, streets with curb and gutter, gutter inlet on continuous grade, combination inlet on sag or sump, maximum street spread and streets with side ditch swales.

8.2.1 Design Storms

For street and inlet design the design storm is the 5-year rainfall. After initially designing the streets and inlets for the design storm, a check shall be made to ensure that a 25-year rainfall does not exceed the maximum depth of flow (Article 8.2.2).

Final design shall indicate water surface elevations for the design storm. In addition, the 50- and 100-year water surface elevations for all streets and inlets shall be shown.

8.2.2 Streets with Curb and Gutter

The design roughness coefficient "n" value shall be 0.015 for paved streets. The minimum gutter slope shall be 0.50 percent. All street resurfacing shall be done to maintain a recommended minimum flow depth of 5 3/4 inches at the face of the curb.

Depth of flow shall not exceed the top of curb for the design storm. In addition, the 100-year water surface elevations shall not exceed 10-inch depth above the curb for local and collector streets and shall not exceed 6-inch depth at crown for arterial streets. In order for the depth of flow not to exceed the specified limit, especially for the 100-year rainfall, an open channel or storm sewer system may be required to convey some of the flow to a major channel. Where a drainageway is located outside a street right-of-way, easements shall be provided. In determining the required capacity of surface channels and storm sewer system, the street storm inlets and conduit provided shall be assumed to be carrying not more than one-half their design capacity. This is a safety factor to allow surcharge outlets, obstructed inlets, or other malfunctions.

8.2.3 Gutter Inlets: Continuous Grade

Gutter inlets shall be according to City of Cincinnati standard drawings Accession No. 49011 through 49015 and shall have a local depression of 3/4 inch below the normal gutter flow line. For calculations, the following values shall be used:

	Single Gutter Inlet	Double Gutter Inlet
local depression (a)	3/4 inch = 0.0625 feet	3/4 inch = 0.0625 feet
width of grade	1.4 feet	1.4 feet
length of grate (L)	2.5 feet	5.0 feet
perimeter of grate (P)	4.6 feet	6.9 feet
total clear opening area of grate (A)	1.6 square feet	3.3 square feet

Gutter inlets shall be located at all points where the maximum pavement spread or maximum flow depth is reached. No flow is permitted to cross street intersections for the initial design storm. Maximum inlet spacing on a continuous grade shall be 300 feet. Gutter inlets shall not be closer than 3 feet to the point of the drop curb to a driveway or handicap ramp.

8.2.4 Combination Inlets: Sag or Sump

Combination inlets shall be provided at all sag or sump locations. When combination inlets are used, the grate capacity alone shall be considered the capacity of the inlet. The curb opening serves as a relief in the event the grate is clogged.

Combination inlets with grate and curb opening shall be City of Cincinnati standard drawings Accession Nos. 49016 through 49018 and shall have a local depression of 3/4 inches below the gutter flow line. For calculations the following values shall be used:

	Combination Inlet
local depression (a)	3/4 inch = 0.0625 feet
width of grate	1.4 feet
length of grate (L)	5.0 feet
perimeter of grate (P)	6.9 feet
total clear opening area of grate (A)	3.3 square feet
height of curb opening (h)	5 5/4 inches = 0.479 feet

Recommended inlet locations are at the points of vertical curvature on each side of the sag at an elevation which is 0.2 feet higher than the low point and at least one inlet at the low point. Combination inlets shall not be closer than 3 feet to the point of the drop curb to a driveway or handicap ramp.

8.2.5 Maximum Street Spread

The following are maximum spread of the 5-year rainfall design storm onto the pavement.

For two-lane streets, maximum spread is 6 feet from the face of the curb.

For four-lane streets, maximum spread is 8 feet from the face of the curb.

The more restrictive condition for either maximum street spread or depth of flow (Article 8.2.2) shall control.

8.2.6 Streets with Side Ditch Swales

Side ditch swales shall be designed in accordance with the general procedures stated in Chapter 7, "Open Channels" of this manual. The minimum channel bottom slope shall be 1.00 percent for grass or sod lined channels and 0.50 percent for paved or lined channels.

8.3 General Design Procedures

A generalized design approach to inlet spacing is as follows:

Step 1. Locate the first inlet at the point where the maximum gutter capacity is reached based on the 10-year design storm.

Step 2. Determine the inlet capacity Q and percent carryover (gutter flow) to the next inlet (Exhibit VIII-2). As a general rule, the carryover flow should be no greater than 15 percent and the picked up flow no less than 85 percent.

Step 3. Locate the next inlet downstream at that point where the gutter capacity is again reached including the gutter flow (carryover) from the upstream inlet. Note that inlets so placed may or may not be located directly across from each other on each side of the street. The individual inlet spacing depends on the configuration of the tributary drainage area and the percent of carryover from the upstream inlet.

Step 4. Continue locating inlets at maximum gutter capacity points on continuous grades until a street intersection or a low point (sag) in the street profile is reached.

Step 5. At street intersections, inlet locations vary, depending on the respective street grades and pedestrian convenience. In general, inlets should be located at the upstream curb turnouts adjacent to crosswalks.

Step 6. Inlets located at the sags of vertical curves are designed for a capacity adequate to intercept 100 percent gutter flow.

The following design procedures are included to outline a uniform approach to the determination of gutter carrying capacity, and capacity of inlets. Examples are included in Article 8.4 to illustrate the design procedures.

8.3.1 Gutter Capacity

Step 1. Draw the street cross section and determine the maximum depth of flow and the permissible pavement spread.

Step 2. Determine the gutter slope in feet per foot, and "z," the reciprocal of the cross slope.

Step 3. Calculate the theoretical gutter (triangular channel) carrying capacity by using the modified Manning's formula: $Q = (0.56\ Z\ S^{1/2} d^{8/3})/n$ where Z is the reciprocal of the cross slope (T/d), n is Manning's coefficient of roughness, S is the longitudinal slope of the gutter in feet per foot, d is the depth of flow in the gutter at the deepest point in feet, and T is the top width of water surface in the gutter in feet. A nomograph for the solution of this formula is shown on Exhibit VIII-1. The nomograph may be used for all gutter configurations. Instructions for the capacity determination of composite sections are included on the nomograph.

8.3.2 Capacity of a Grate Inlet or Combination Inlet on Continuous Grade

Once the gutter capacity of the street section is determined, it is necessary to determine the capacity of a curb inlet to intercept part or all of the gutter flow. Economy of design dictates that the first inlet should be located at that point where the maximum permissible gutter capacity is reached. Downstream inlets are located at the most restrictive criterion for maximum inlet spacing, maximum gutter capacity, and maximum spread of flow on the roadway.

Capacity calculations for combination inlets (grate and curb opening) are performed considering only the grate as available for intercepting water, with the curb opening providing reserve capacity in the event the grate becomes clogged. The capacity of a grate or combination inlet is calculated as the amount of flow in the gutter cross section above the grate plus an amount of flow which will enter along the exposed edge of the grate.

When using two or more grates or combination inlets separated by short sections of paved gutter on a continuous grade, the grates should be spaced far enough apart, usually a minimum of 25 feet, so that the bypass flow from the first inlet will move into the curb before it reaches the second inlet. To determine the capacity of a given spacing, compute the flow over the end and side of the first grate based on total gutter flow. Compute capacity of the second grate based on carryover from the first grate by two methods: (1) flow over the end and side assuming the carryover to have reached normal gutter depth; (2) consider the distance from lower end of first grate to lower end of second grate as the length of a curb opening inlet and the depth of flow as the depth at the outer downstream corner of the first grate. Compute interception as a curb opening. Take the lower of the two values found in (1) and (2) as the interception of the second grate.

The general procedure for locating grate and combination inlets on a continuous grade follows:

Step 1. Determine the gutter flow at the inlet location (Q_a).

Step 2. Determine the gutter flow depth at the curb (d) and the spread of water on the roadway.

Step 3. Calculate the width, depth (d') and amount (Q_a') of flow outside of the grate.

Step 4. Determine the flow over the end of the grate as $QE = Q_a - Q_a'$.

Step 5. Calculate flow over the side of the grate using the following steps:

a. Using the depth (d') and the depression (a) of the grate, enter Chart A of Exhibit VIII-2 and read Q_a'/L_a.

b. Compute the 100 percent pickup length, $L_a = Q_a'/(Q_a'/L_a)$

c. Compute the ratio L/L_a where L is the distance along the outside edge of the grate. If this ratio is greater than or equal to 1.0, 100 percent of the flow is being intercepted, flow over the side of the grate is Q_a' and continue with step 6 below. If the ratio is less than 1.0, only a portion of the flow is being intercepted over the side of the grate. The amount intercepted is calculated beginning with step d.

d. Determine the ratio a/d'.

e. Using L/L_a and a/d' enter Chart B of Exhibit VIII-2 and read Q'/Q_a', the ratio of intercepted flow to total flow outside of grate.

f. Calculate the total flow intercepted over the outside edge of the grate, $Q' = Q_a' \times (Q'/Q_a')$.

Step 6. Determine the total flow intercepted $Q = (Q_a - Q_a') + Q'$.

Step 7. Calculate the carryover flow to the next inlet, $Q_a - Q$.

Step 8. Calculate the percent of intercepted flow (% pickup), $(Q/Q_a) \times 100$.

Step 9. If the intercepted flow is less than 85 percent, try a different inlet location or type of gutter inlet.

8.3.3 Example - Capacity of a Grate Inlet on Continuous Grade

Find the discharge intercepted by the grate inlet of a City of Cincinnati single gutter inlet, Accession Nos. 49011, 49012, and 49015, on a four-lane street on a continuous longitudinal slope of 3 percent. The pavement cross slope is 2.5 percent and has a roughness coefficient value of 0.015. The gutter flow at the inlet location is 1.5 cfs.

Step 1. Gutter flow at the inlet location is 1.5 cfs.

Step 2. Use Exhibit VIII-1 with $Z/n = (1/0.025)/0.015 = 2667$, $S = 0.03$ ft/ft and $Q_a = 1.5$ cfs read $d = 0.15$ ft.

Calculate spread of water = d/cross slope = 0.15/0.025 = 6 ft.

Check maximum street spread 6 ft < 8 ft.

Step 3. Calculate width, depth (d'), and flow (Q_a') outside of the grate width = spread - width of grate = 6 - 1.4 = 4.6 ft. Depth (d') = width x cross slope = 4.6 x 0.025 = 0.115 ft. Use Exhibit VIII-1 with S = 0.03 ft/ft d' = 0.115 feet and Z/n = 2667. Read $Q_a' = 0.8$ cfs.

Step 4. Calculate the flow over the end of the grate $QE = Q_a - Q_a' = 1.5 - 0.8 = 0.7$ cfs (flow at inlet location - flow outside of grate).

Step 5a. Use Exhibit VIII-2 Chart A with $d' = 0.115$ ft and $a = 0.0625$ ft and read $Q_a'/L_a = 0.047$.

Step 5b. Calculate 100 percent pickup length (L_a). $L_a = Q_a'/(Q_a'/L_a) = 0.8/0.047 = 17.02$ ft.

Step 5c. Calculate ratio of length of grate $L/L_a = 2.5/17.02 = 0.15 < 1.0$ only portion of flow is intercepted.

Step 5d. Calculate ratio of local depression $a/d' = 0.0625/0.115 = 0.54$.

Step 5e. Use Exhibit VIII-2 Chart B with $L/L_a = 0.15$ and $a/d' = 0.54$ and read $Q'/Q_a' = 0.27$.

Step 5f. Calculate flow over the side of the grate $Q' = (Q_a')(Q'/Q_a') = (0.8)(0.27) = 0.22$ cfs.

Step 6. Calculate total flow intercepted $Q = (Q_a - Q_a') + Q' = (1.5 - 0.8) + 0.22$ cfs $= 0.92$ cfs.

Step 7. Calculate carryover flow $= Q_a - Q = 1.5 - 0.92 = 0.58$.

Step 8. Calculate percent of intercepted flow $= (Q/Q_a) \times 100 = (0.92/1.5) \times 100 = 61$.

Step 9. Check that intercept flow picks up 85 percent of flow in gutter. $(1.5)(0.85) = 1.28$ cfs > 0.92 cfs. Therefore, change location of inlet or try a double gutter inlet.

8.3.4 Capacity of Grate Inlet or Combination Inlet in Sag or Sump (Water Ponded on Grate)

The following general procedures are stated for a combination grate inlet. The same procedures would apply to a grate only inlet; however, in consideration of possible clogging of the grate it is recommended the design perimeter and the design area of the grate be one-half of the effective perimeter (P) and effective area (A) determined below.

Step 1. Calculate the total inflow (Q) to the inlet.

Step 2. Determine the effective perimeter of the grate opening (P) in feet ignoring the bars and omitting any side of the grate over which water does not enter; e.g., side against face of curb.

Step 3. Calculate the discharge per foot of perimeter (Q/P). Q is the total gutter discharge from each side of the grate.

Step 4. Determine the total clear opening area (A), excluding the area of the bars.

Step 5. Calculate the discharge per square foot of effective area (Q/A).

Step 6. Enter Exhibit VIII-4 with the values of Q/P and Q/A and read the required head (H) in feet using the appropriate weir or orifice curve.

Step 7. Compare the two head values to determine the type of flow; i.e., weir flow or orifice flow.

Step 8. If the required head (H) is between 0.4 and 1.4 feet, the actual head may be anywhere in this head range. Use the value that gives the more conservative result (highest H).

Step 9. Compare the value of H determined in the preceding steps to the maximum allowable gutter depth (d) including local depression (a).

a. $H > (d + a)$ indicates that the allowable ponding limits are exceeded and that additional inlets are required.

b. $H < (d + a)$ indicates the inlet has ample grate capacity and the maximum allowable ponding limits will not be exceeded.

As stated earlier, the curb opening of a combination inlet serves as an emergency overflow in the event the grate becomes clogged; and, therefore, the curb opening capacity is not included in the capacity determination of combination type inlets in a sump.

8.3.5 Capacity of Combination Inlet on Continuous Grade

Capacity of a curb opening on a continuous grade is done as follows:

Step 1. Determine the length (L) of the inlet opening and the depth of local flow line depression (a) at the inlet.

Step 2. Calculate the design gutter discharge (Q_a) for the initial design storm as stated in the preceding Article 8.3.1, including carryover from previous inlets.

Step 3. Determine the gutter flow depth (d) at design Q_a for the particular street section using Exhibit VIII-1.

Step 4. Enter Chart A of Exhibit VIII-2 with depth of flow in gutter (d) and local depression (a) and determine the interception per foot of inlet opening (Q_a/L_a).

Step 5. Calculate the length, $L_a = Q_a(Q_a/L_a)$. If length L_a is less than actual inlet length L, 100 percent of the flow is being intercepted. If L_a is greater than L, determine the percentage intercepted following Steps 6 through 10.

Step 6. Calculate the ratio of actual inlet length (L) in feet to length of inlet required to intercept 100 percent of gutter flow (L_a). The ratio is expressed as (L/L_a). Also, calculate the ratio a/d.

Step 7. Enter Chart B of Exhibit VIII-2 with the ratios calculated in Step 6 and determine Q/Q_a (the ratio of total flow intercepted by the inlet to gutter flow).

Step 8. Calculate the total intercepted flow, $Q = Q/Q_a \times Q_a$.

Step 9. The carryover flow to the next inlet is $Q_a - Q$.

Step 10. Calculate the percent of intercepted flow (percent pickup) = $Q/Q_a \times 100$.

Step 11. If the intercepted flow is less than 85 percent, try a different inlet location.

8.3.6 Capacity of Gutter Inlet or Combination Inlet at Street Intersections

Inlets are usually placed immediately upstream from pedestrian crosswalks and street intersections and should intercept 100 percent of the gutter flow. The ponded water depth at such low points of street intersections is determined in terms of curb opening height (Exhibit VIII-3) as follows:

Step 1. Calculate the inflow (Q) to the inlet.

Step 2. Determine the vertical height of curb opening (h) at the curb face, including local depression (a).

Step 3. Calculate the required capacity of the inlet per foot of length of opening, Q/L in cfs/foot.

Step 4. Determine the ratio of ponded water depth (H) to vertical curb opening height (h), H/h, using Exhibit VIII-3.

Step 5. Calculate the ponded water depth, $H = H/h \times h$ (in feet).

Step 6. The ponded water depth (H) is compared to the maximum allowable depth of flow in the gutter including local depression (a).

a. If H is less than (d + a) using the same units of depth, the curb opening inlet is intercepting 100 percent of the initial design storm discharge.

b. If H is greater than (d + a), the physical design criteria are exceeded and adjustments in the design are necessary.

8.4 Example - Design Calculations

The following example is included to further illustrate the street and inlet design procedures stated herein and follows the steps given in the preceding Article. Exhibit VIII-6 is a pavement drainage computation worksheet that references how values are obtained for each column. Exhibit VIII-5 is a completed streets and inlets computation worksheet for the initial design storm for this example. The major storm drainage calculations are included in Chapter 9 "Storm Sewer."

Determine the inlet type and location for the proposed residential development as shown on Figure 8-1. The typical local street half-section dimensions are shown on Figure 8-2. The inlets selected for streets on a continuous grade are the City of Cincinnati standard Single Gutter Inlet (SGI), drawing Accession No. 49011 and the City of Cincinnati standard Double Gutter Inlet (DGI), drawing Accession No. 49013. The inlet selected for street intersections and for the sag vertical curve is the City of Cincinnati standard grate and curb opening Combination Inlet (CI), drawing Accession No. 49016.

The following design data are given:

1. Tributary drainage area along street B is 6.35 acres.

2. Runoff coefficient for the Rational Method is C = 0.50.

3. Average land slope is moderate, 5.0 percent.

The following design criteria are from the preceding Articles:

1. Initial design storms return period is 5 years.

2. Maximum allowable pavement encroachment for the initial design storm on this two-lane road is 6 feet from the face of curb.

3. Roughness coefficient value is n = 0.015.

4. Minimum gutter slope is 0.50 percent.

Step 1. Locate the first inlet at the point of maximum gutter flow. Therefore, determine the maximum gutter flow. Following Steps 1.1, 1.2, and 1.3 are the steps given in the design procedures for gutter capacity, Article 8.3.1.

Step 1.1 Use Figure 8-2 and determine the maximum depth of gutter flow at the curb d = depth for maximum allowable encroachment d = 6'-0" x 0.0340 = 0.204 feet.

Step 1.2 Use Figure 8-1 and read gutter slope of 1 percent. Use Figure 8-2 for the composite idealized section and determine the reciprocal of the cross section (Z).

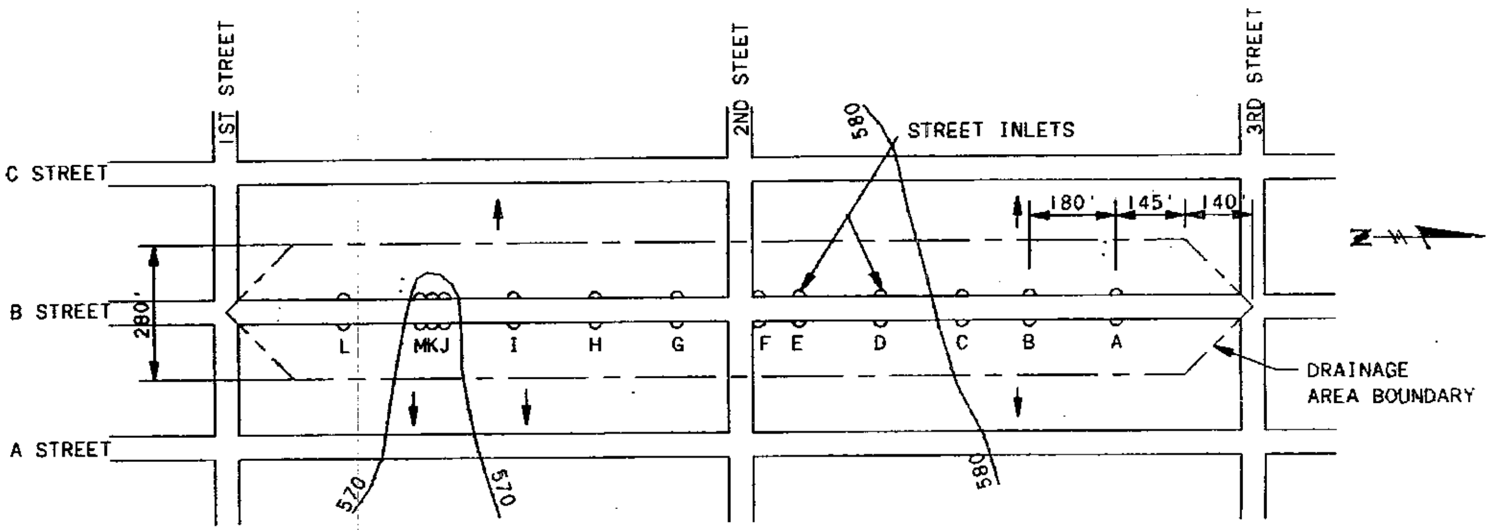
$$Z = 1/0.0340 = 29.41$$

Step 1.3 Use Exhibit VIII-1, with a channel slope = 1 percent, d = 0.204 feet, Z = 29.41, and n = 0.015 and determine the maximum gutter flow = Q.

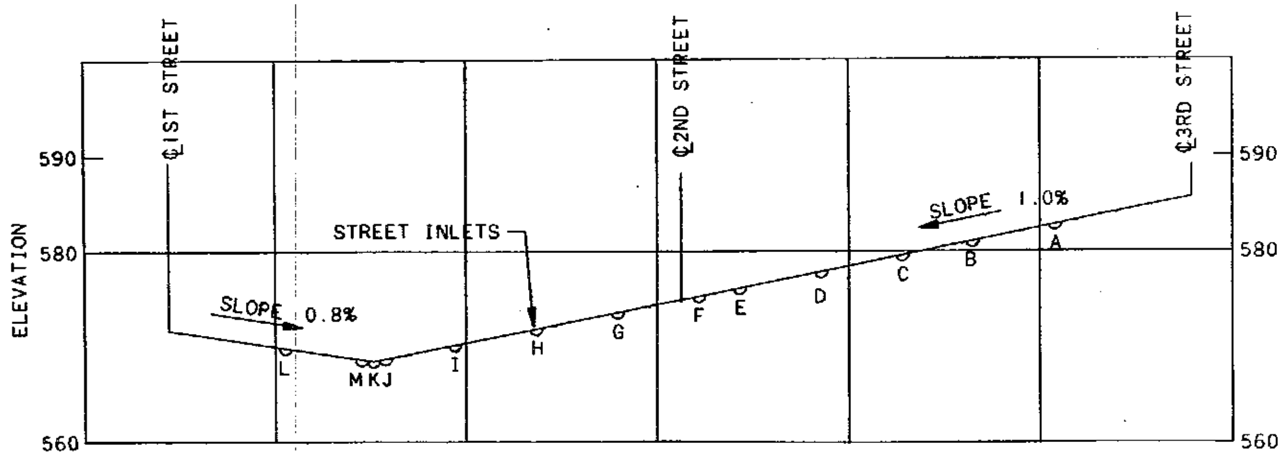
Calculate Z/n = 29.41/0.015 = 1960.

Read from Exhibit VIII-1 Q_1 = 1.58 cfs.

Step 1.4 Determine the drainage area, overland flow time, and rainfall intensity so the peak flow at this design point as calculated by the Rational Method = maximum gutter flow. This is a trial and error method involving selecting a drainage area and determine the overland flow time, rainfall intensity and calculating a peak flow for comparison.



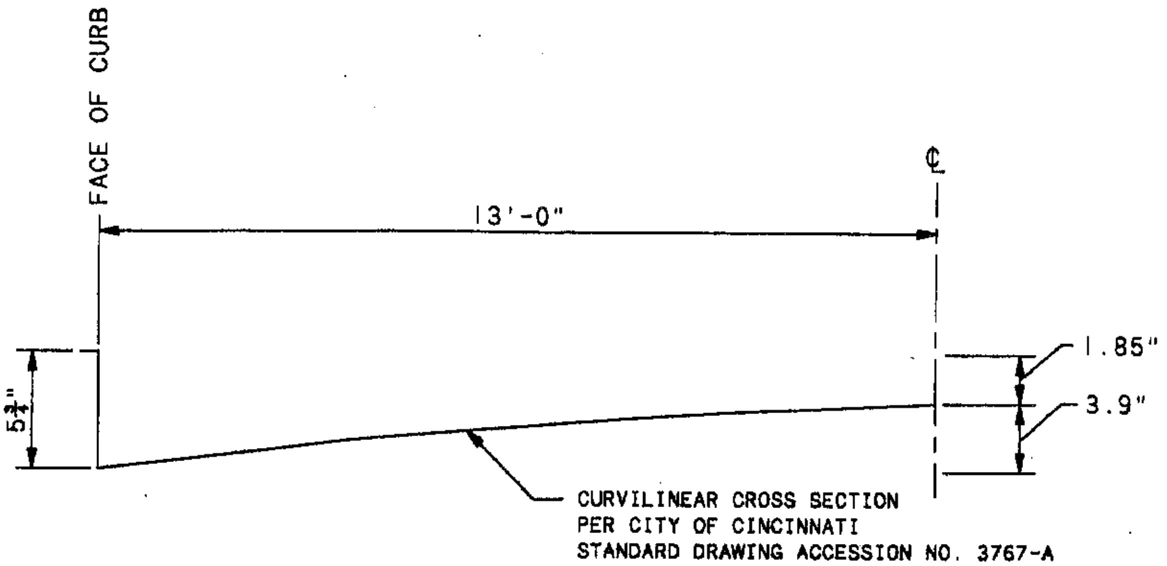
PLAN
NO SCALE



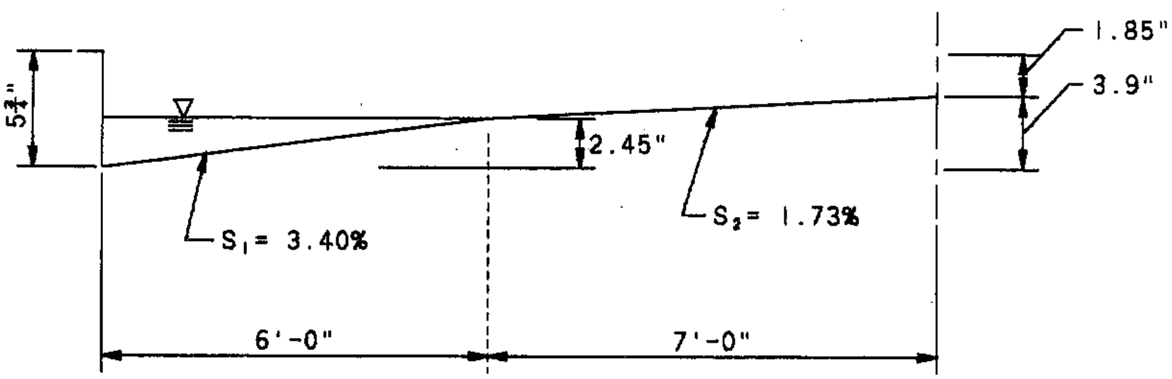
B STREET PROFILE
NO SCALE

TYPICAL RESIDENTIAL AREA

FIGURE 8-1



ACTUAL SECTION



IDEALIZED SECTION FOR DISCHARGE COMPUTATIONS

TYPICAL LOCAL 26 FOOT STREET HALF SECTION

FIGURE 8-2

Use inlet station A, calculate peak flow Q = CiA. Use Figure 8-1 and calculate drainage area to inlet A, A = length x width/43,560 = (155 x 140 + 1/2 x 140 x 140)/ 43,560 = 0.723 acres; C = 0.50; from Exhibit VI-4 with length of slope = 140 - 13 = 127 feet, average grass and read overland flow time = 11.9 minutes; gutter travel time = 155/(60)(1.58/(0.5 x 6 x 0.204)) = 1.00 minutes; TC = 11.90 + 1.00 = 12.90; from Exhibit V-2 with T_c = 12.90 read i = 4.36 inches per hour; therefore, Q = CiA = 0.5 x 4.36 x 0.723 = 1.58 cfs = maximum gutter flow.

Step 2. Determine the inlet capacity Q to the first inlet station A and percent carryover (gutter flow) to the next inlet station B. To minimize the number of inlets select a double gutter inlet for station A. The calculations follow steps given in the design procedures for a grate inlet on a continuous grade, Article 8.3.2.

Step 2.1 Calculate the gutter flow Q_a = 1.58 cfs see the preceding Step 1.4.

Step 2.2 For Q_a = 1.58 cfs, gutter slope = 1 percent, and cross slope of 3.40 percent as shown on Figure 8-2, calculate the gutter flow depth (d) and spread of water on the roadway. The spread cannot exceed the design criteria maximum street encroachment. See the preceding Steps 1.1, 1.2, and 1.3 where d = 0.204 feet and spread = 6 feet.

Step 2.3 Calculate the width, depth (d') and flow (Q_a') outside of the grate. Width = spread - width of grate = 6'-1.4" = 4.6 feet.

Depth = d' = length x slope = (6'-0" - 1.4') x 0.0340 = 0.156 feet.

Use Exhibit VIII-1 with a channel slope = 1 percent, d' = 0.156 feet, Z = 29.41, and n = 0.015 and determine flow = Q_a'. Calculate Z/n = 29.41/0.015 = 1960. Read from Exhibit VIII-1 Q_a' = 0.77 cfs.

Step 2.4 Calculate the flow over the end of the grate = QE = Q_a - Q_a' = 1.58 - 0.77 = 0.81 cfs.

Step 2.5 Calculate flow over the side of the grate Q'.

Step 2.5a Use Chart A of Exhibit VIII-2 with d' = 0.156 and a = 3/4 inch and read Q_a'/L_a = 0.069.

Step 2.5b Calculate the 100 percent pick-up length = L_a = $Q_a'/(Q_a'/L_a)$ = 0.77/0.069 = 11.16 feet.

Step 2.5c Calculate the ratio of length of grate $(L)/L_a$ = 5.0/11.16 = 0.45.

Step 2.5d Calculate the ratio of local depression (a)/d' = 0.0625/0.156 = 0.40.

Step 2.5e Use Chart B of Exhibit VIII-2 with $L/L_a = 0.45$ and $a/d' = 0.40$ and read $Q'/Q_a' = 0.67$.

Step 2.5f Calculate total flow over the side of the grate = $Q' = Q_a' \times (Q'/Q_a') = 0.77 \times 0.67 = 0.52$ cfs.

Step 2.6 Calculate the total flow intercepted $Q = (Q_a - Q_a') + Q' = 1.58 - 0.77 + 0.52 = 1.33$.

Step 2.7 Calculate carryover flow = $Qa - Q = 1.58 - 1.33 = 0.25$.

Step 2.8 Calculate percent of intercepted flow = $Q/Q_a \times 100 = 1.33/1.58 \times 100 = 84 < 85$. Therefore, try a different inlet location. The calculations are shown on Exhibit VIII-5.

Step 3. Locate the next inlet station B downstream at that point where the gutter capacity is again reached including the gutter flow (carryover) from the upstream inlet. Note the inlets so placed may or may not be located directly across from each other on each side of the street. The individual inlet spacing depends on the configuration of the tributary drainage area and the percent of carryover from the upstream inlet. The calculation would be similar to calculation of Step 2 and the values are shown on Exhibit VIII-5.

Step 4. Continue locating inlets at maximum gutter capacity points on continuous grades until a street intersection or a low point (sag) in the street profile is reached.

At street intersections, inlet locations vary depending upon the respective street grades and pedestrian convenience. In general, inlets should be located at the upstream curb turnout adjacent to cross walks and should have a capacity for 100 percent of the gutter flow. The calculations for inlet station G, at the intersection of B street and 2nd street, follow the steps given in the design procedures for curb opening inlet at street intersection, Article 8.3.6, and the results are summarized on Exhibit VIII-5.

Inlets are located at the sag vertical curve. The inlets are designed for a capacity adequate to intercept 100 percent gutter flow. The calculations for inlet station N at the sag in B street follow the steps given in the design procedures for grate inlet in a sag, Article 8.3.4 and the results are summarized on Exhibit VIII-5.

Again, the design procedures discussed herein for inlet locations are a generalized approach to design and are not intended as a cook book procedure. Engineering experience and judgment is an important factor in any design and in many instances will justify deviation from usual design procedures to achieve the desired system performance with respect to the unique site features.

8.5 Bibliography

U.S. Department of Commerce. Design Charts for Open-Channel Flow. Bureau of Public Roads. Washington, D.C., August 1961.

U.S. Department of Commerce. Design of Roadside Drainage Channels. Bureau of Public Roads. Washington, D.C., May 1965.

Johns Hopkins University. The Design of Storm-Water Inlets. Department of Sanitary Engineering and Water Resources. Baltimore, Maryland, June 1956.

U.S. Department of Transportation. Drainage of Highway Pavements. Bureau of Public Roads. Washington, D.C., March 1969.

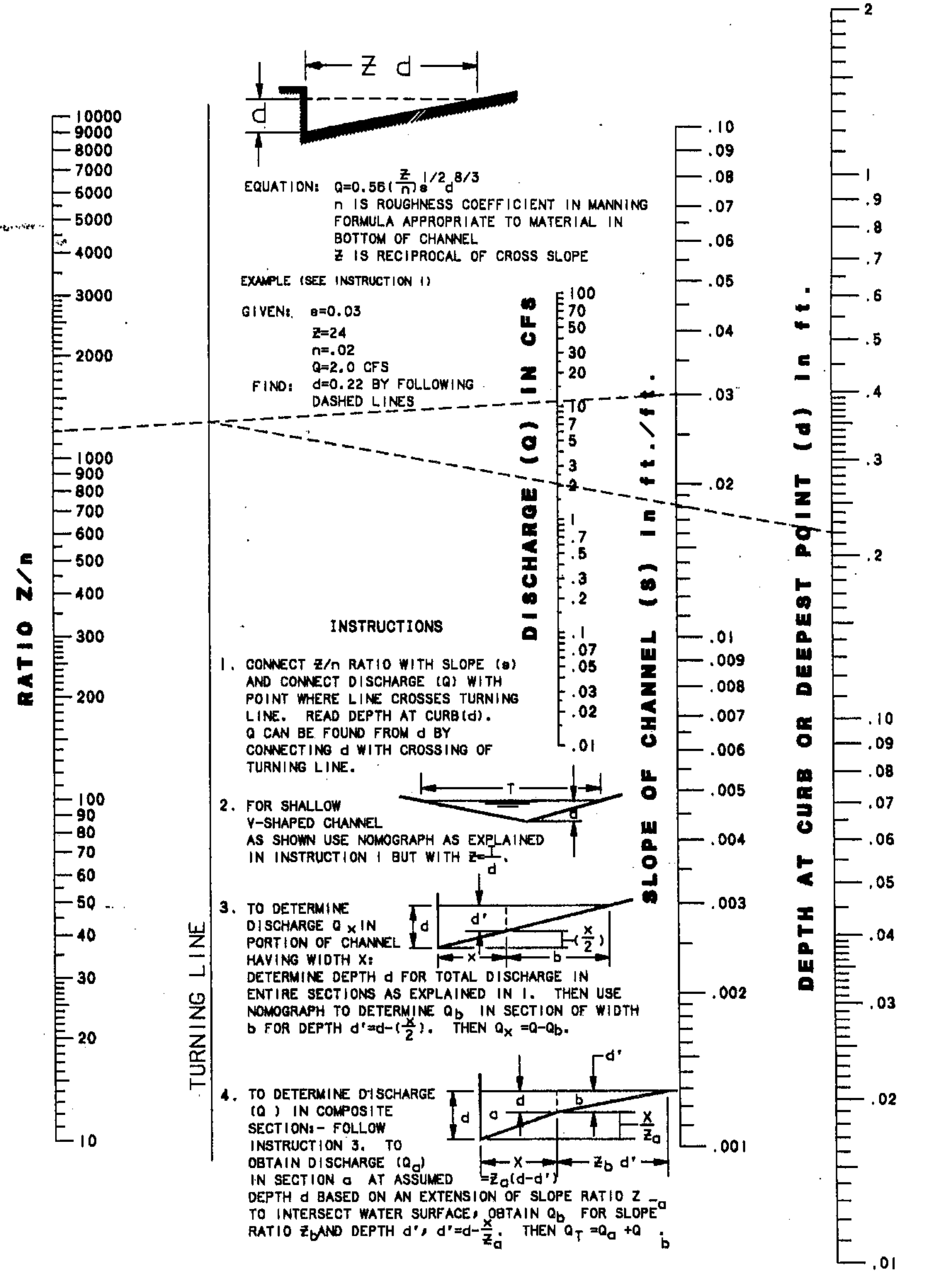
Ohio Department of Transportation. Manual of Location and Design. Bureau of Location and Design.

The Urban Land Institute. The American Society of Civil Engineers. The National Association of Home Builders. Residential Streets. Washington, D.C., 1974.

City of Cincinnati, Ohio. Standard Drawings. Department of Public Works, Division of Engineering.

Erie and Niagara Counties Regional Planning Board. Storm Drainage Design Manual. Harza Engineering Co., July 1972.

Denver Regional Council of Governments. Urban Storm Drainage Criteria Manual, Volume 1. Wright-McLaughlin Engineers. Denver, Colorado, March 1969.



NOMOGRAPH FOR FLOW IN TRIANGULAR CHANNELS

EXHIBIT VIII-1

CHART A

DISCHARGE PER FOOT OF LENGTH OF CURB OPENING INLETS WHEN INTERCEPTING 100% OF GUTTER FLOW

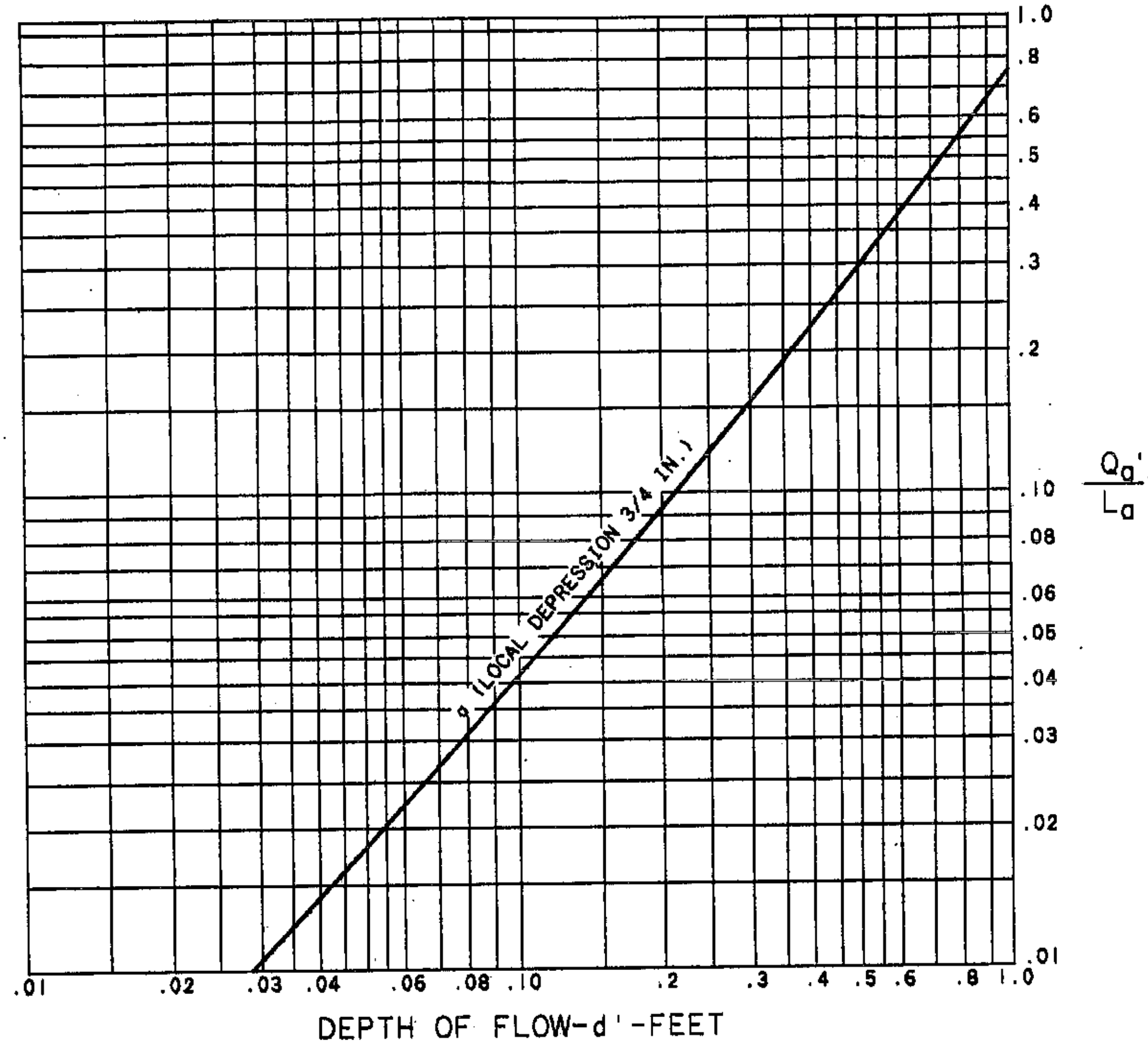
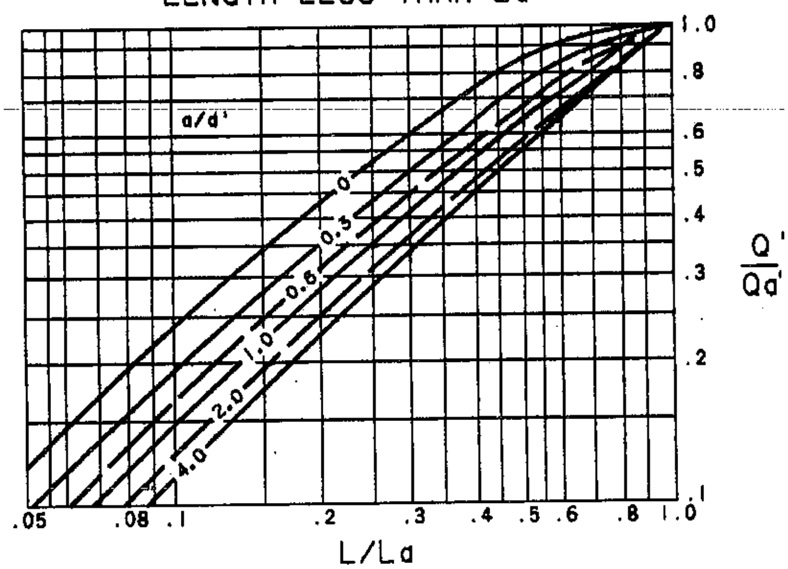


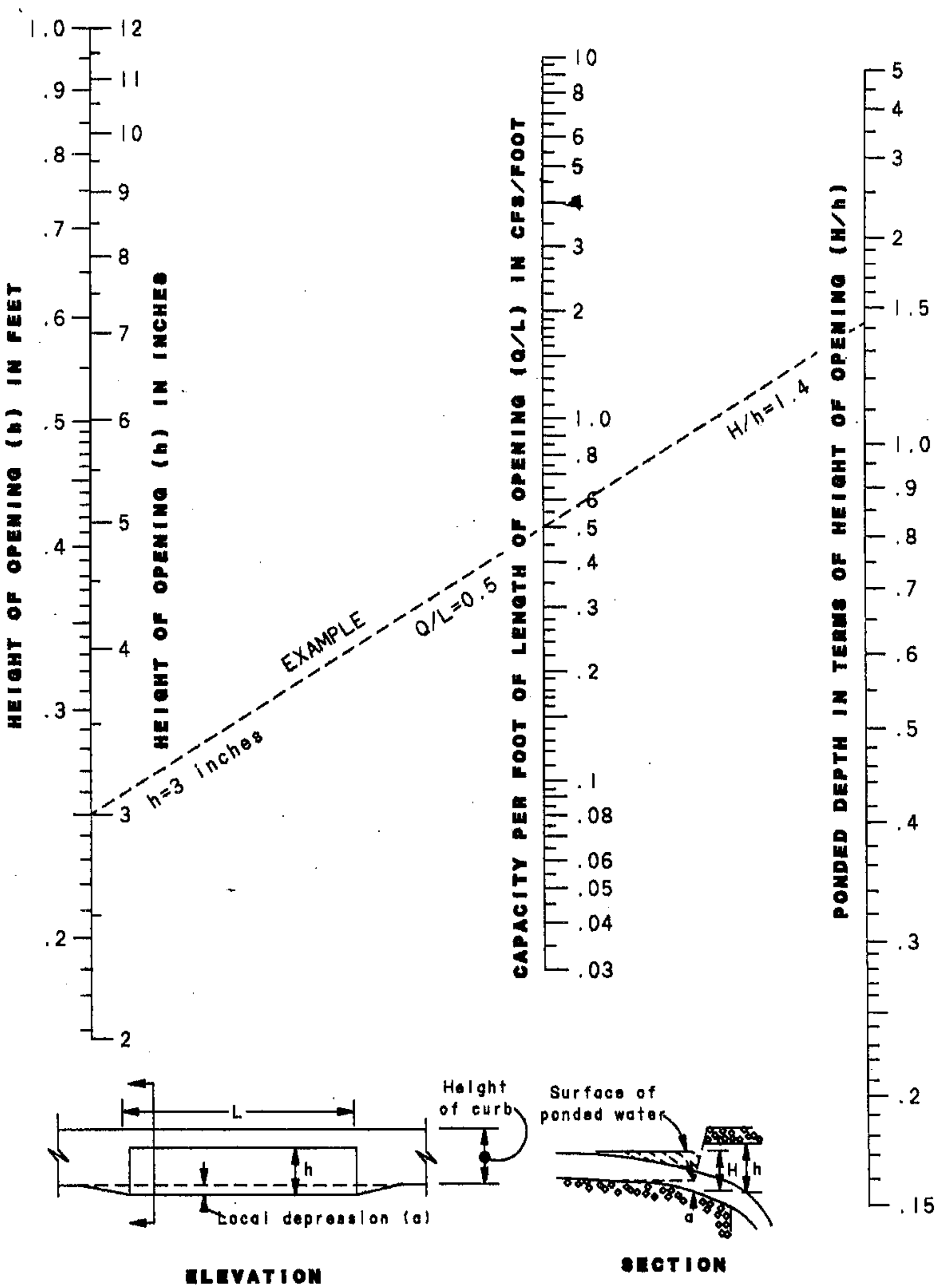
CHART B

PARTIAL INTERCEPTION RATIO FOR INLETS OF LENGTH LESS THAN La



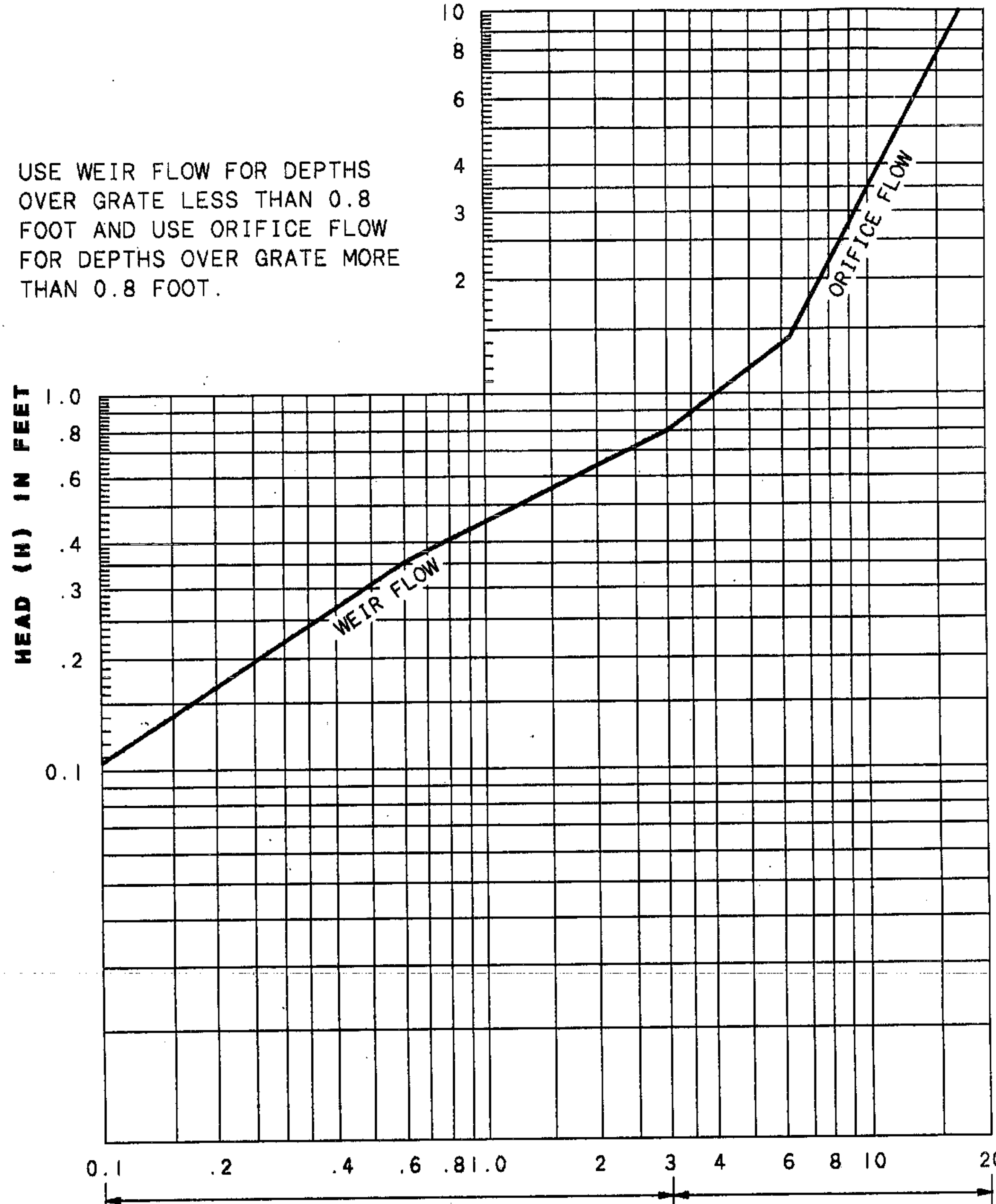
CAPACITY OF CURB OPENING INLETS ON CONTINUOUS GRADE

EXHIBIT VIII-2



CAPACITY OF CURB OPENING INLET AT LOW POINT IN GRADE (STREET INTERSECTION)

EXHIBIT VIII-3



CAPACITY OF GRATE INLET IN SUMP
(WATER PONDED ON GRATE)

EXHIBIT VIII-4

PAVEMENT DRAINAGE COMPUTATIONS
ARTICLE 8.4 EXAMPLE

PROJECT ______________ DESIGNER ______________ DATE ______________

DISCHARGE										GUTTER FLOW				GRATE OPENINGS INLETS					CURB OPENING INLETS									CHECK
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29
INLET STATION	UPSTREAM STATION	LENGTH	WIDTH	A	C	CA	TIME	I 5 YEAR FREQUENCY	Qo	GUTTER SLOPE (S)	CROSS SLOPE (Z)	DEPTH AT CURB (d)	SPREAD	TYPE OF INLET	WIDTH OF FLOW OUTSIDE OF GRATE	DEPTH OUTSIDE EDGE OF GRATE (d')	FLOW OUTSIDE OF GRATE (Q'o)	FLOW AT END OF GRATE (Qo-Q'o)	Qo/Lo CURB OPENING Q'o/Lo GRATE OPENING	100% PICKUP LENGTH (Lo)	L/Lo	o/d CURB OPENING o/d' GRATE OPENING	Q/Qo CURB OPENING Q'/Q'o GRATE OPENING	FLOW OVER SIDE OF GRATE (Q')	TOTAL FLOW INTERCEPTED (Q)	CARRY OVER FLOW	% PICKUP	INLET OK
		FT	FT	ACRES		ACRES	MIN	IN/HR	CFS	FT/FT	FT/FT	FT	FT		FT	FT	CFS	CFS		FT				CFS	CFS	CFS		
A		145	140	.691	0.50	.346	12.84	4.37	1.51	0.010	29.41	0.200	5.88	DG1	4.48	0.152	0.72	0.79	0.067	10.78	0.46	0.41	0.68	0.49	1.28	0.23	85	
B		180	140	.579	0.50	.290	13.06	4.34	1.49	0.010	29.41	0.199	5.85	DG1	4.45	0.151	0.71	0.78	0.066	10.71	0.47	0.41	0.68	0.48	1.26	0.23	85	
C		180	140	.579	0.50	.290	13.06	4.34	1.49	0.010	29.41	0.199	5.85	DG1	4.45	0.151	0.71	0.78	0.066	10.71	0.47	0.41	0.68	0.48	1.26	0.23	85	
D		180	140	.579	0.50	.290	13.06	4.34	1.49	0.010	29.41	0.199	5.85	DG1	4.45	0.151	0.71	0.78	0.066	10.71	0.47	0.41	0.68	0.48	1.26	0.23	85	
E		180	140	.579	0.50	.290	13.06	4.34	1.49	0.010	29.41	0.199	5.85	DG1	4.45	0.151	0.71	0.78	0.066	10.71	0.47	0.41	0.68	0.48	1.26	0.23	85	
F (STREET)	INTER.	25	140	.077	0.50	.039	12.06	4.49	0.40	0.010	29.41	0.122	3.59	C1 h=0.479 FT a/L=.40/5 H/h=0.215 H=0.479×0.215=0.103<(0.204+0.06=0.26)													100	
G		180	140	.579	0.50	.290	13.06	4.34	1.26	0.010	29.41	0.187	5.50	DG1	4.10	0.139	0.57	0.69	0.060	9.47	0.53	0.45	0.75	0.43	1.12	0.14	89	

PAVEMENT DRAINAGE COMPUTATIONS
ARTICLE 8.4 EXAMPLE

PROJECT ____________ DESIGNER ____________ DATE ________

DISCHARGE										GUTTER FLOW				GRATE OPENINGS INLETS					CURB OPENING INLETS									CHECK
1 INLET STATION	2 UPSTREAM STATION	3 LENGTH	4 WIDTH	5 A	6 C	7 CA	8 TIME	9 I 5 YEAR FREQUENCY	10 Qo	11 GUTTER SLOPE (S)	12 CROSS SLOPE (Z)	13 DEPTH AT CURB (d)	14 SPREAD	15 TYPE OF INLET	16 WIDTH OF FLOW OUTSIDE OF GRATE	17 DEPTH OUTSIDE EDGE OF GRATE (d')	18 FLOW OUTSIDE OF GRATE (Q'o)	19 FLOW AT END OF GRATE (Qo-Q'o)	20 Qo/Lo CURB OPENING Q'o/Lo GRATE OPENING	21 100% PICKUP LENGTH (Lo)	22 L/Lo	23 o/d CURB OPENING o/d' GRATE OPENING	24 O/Qo CURB OPENING O'/Q'o GRATE OPENING	25 FLOW OVER SIDE OF GRATE (Q')	26 TOTAL FLOW INTERCEPTED (Q)	27 CARRY OVER FLOW	28 % PICKUP	29 INLET OK
		FT	FT	ACRES		ACRES	MIN	IN/HR	CFS	FT/FT	FT/FT	FT	FT		FT	FT	CFS	CFS		FT				CFS	CFS	CFS		
H		180	140	.579	0.50	.290	13.06	4.34	1.40	0.010	29.41	0.195	5.73	DG1	4.33	0.147	0.66	0.74	0.064	10.27	0.49	0.43	0.72	0.47	1.22	0.18	87	
I		180	140	.579	0.50	.290	13.06	4.34	1.44	0.010	29.41	0.197	5.79	DG1	4.39	0.149	0.69	0.75	0.065	10.57	0.47	0.42	0.69	0.48	1.23	0.21	85	
J		115	140	.370	0.50	.185	12.64	4.40	1.02	0.010	29.41	0.173	5.09	DG1	3.69	0.125	0.43	0.59	0.053	8.09	0.62	0.50	0.80	0.34	0.93	0.09	91	
L		115	140	.594	0.50	.297	12.64	4.40	1.31	0.008	29.41	0.198	5.82	DG1	4.42	0.150	0.62	0.69	0.066	9.43	0.53	0.42	0.71	0.44	1.13	0.18	86	
M		150	140	.482	0.50	.241	12.87	4.37	1.23	0.008	29.41	0.193	5.68	DG1	4.28	0.146	0.58	0.65	0.064	9.10	0.55	0.43	0.71	0.41	1.06	0.17	86	
K (SUMP)		50	140	.161	0.50	.081	12.06	4.49	0.62	0.008	29.41	0.150	4.41	C1	Q/P=0.62/6.9 = 0.09 Q/A=0.62/3.3 = 0.19 H<0.1<(0.204+0.06=0.26)												100	

EXHIIT VIII-5 (CONT.)

PAVEMENT DRAINAGE COMPUTATIONS GUIDE

PROJECT ______________ DESIGNER ______________ DATE ______________

DISCHARGE	1	INLET STATION		
DISCHARGE	2	UPSTREAM STATION		
DISCHARGE	3	LENGTH	FT	SITE SPECIFIC
DISCHARGE	4	WIDTH	FT	SITE SPECIFIC
DISCHARGE	5	A	ACRES	DRAINAGE AREA TO INLET STATION
DISCHARGE	6	C		SITE SPECIFIC SEE CHAPTER 6
DISCHARGE	7	CA	ACRES	COLUMN 5 x COLUMN 6
DISCHARGE	8	TIME	MIN	TIME OF CONCENTRATION TO INLET STATION SEE CHAPTER 6
DISCHARGE	9	I YEAR FREQUENCY	IN/HR	EXHIBIT V-1 OR V-2 WITH COLUMN 8
DISCHARGE	10	Qo	CFS	COLUMN 7 x COLUMN 9
GUTTER FLOW	11	GUTTER SLOPE (S)	FT/FT	SITE SPECIFIC
GUTTER FLOW	12	CROSS SLOPE (Z)	FT/FT	SITE SPECIFIC
GUTTER FLOW	13	DEPTH AT CURB (d)	FT	EXHIBIT VIII-1 WITH COLUMNS 10, 11 & 12
GUTTER FLOW	14	SPREAD	FT	COLUMN 12 x COLUMN 13
GRATE OPENINGS INLETS	15	TYPE OF INLET		SELECT
GRATE OPENINGS INLETS	16	WIDTH OF FLOW OUTSIDE OF GRATE	FT	COLUMN 14 - WIDTH OF GRATE
GRATE OPENINGS INLETS	17	DEPTH OUTSIDE EDGE OF GRATE (d')	FT	COLUMN 16 + COLUMN 12
GRATE OPENINGS INLETS	18	FLOW OUTSIDE OF GRATE (Q'o)	CFS	EXHIBIT VIII-1 WITH COLUMNS 11, 12 & 17
GRATE OPENINGS INLETS	19	FLOW AT END OF GRATE (Qo-Q'o)	CFS	COLUMN 10 - COLUMN 18
CURB OPENING INLETS	20	Qo/Lo CURB OPENING Q'o/Lo GRATE OPENING		EXHIBIT VIII-2a WITH COLUMNS 13 OR 17
CURB OPENING INLETS	21	100% PICKUP LENGTH (Lo)	FT	COLUMN 18 / COLUMN 20 OR COLUMN 10 / COLUMN 20
CURB OPENING INLETS	22	L/Lo		LENGTH OF GRATE / COLUMN 21
CURB OPENING INLETS	23	a/d CURB OPENING a/d' GRATE OPENING		LOCAL DEPRESSION (3/4) / COLUMN 13 OR LOCAL DEPRESSION (3/4) / COLUMN 17
CURB OPENING INLETS	24	Q/Qo CURB OPENING Q'/Q'o GRATE OPENING		EXHIBIT VIII-2b WITH COLUMNS 22 AND 23
CURB OPENING INLETS	25	FLOW OVER SIDE OF GRATE (Q')	CFS	COLUMN 10 x COLUMN 24 OR COLUMN 18 x COLUMN 24
CURB OPENING INLETS	26	TOTAL FLOW INTERCEPTED (Q)	CFS	COLUMN 19 + COLUMN 25
CURB OPENING INLETS	27	CARRY OVER FLOW	CFS	COLUMN 10 - COLUMN 26
CURB OPENING INLETS	28	% PICKUP		COLUMN 26 / COLUMN 10 x 100
CHECK	29	INLET OK		

EXHIIT VIII-6

PAVEMENT DRAINAGE COMPUTATIONS

PROJECT ______________________ DESIGNER ______________________ DATE __________

DISCHARGE										GUTTER FLOW				GRATE OPENINGS INLETS					CURB OPENING INLETS									CHECK
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29
INLET STATION	UPSTREAM STATION	LENGTH	WIDTH	A	C	CA	TIME	I YEAR FREQUENCY	Q_o	GUTTER SLOPE (S)	CROSS SLOPE (Z)	DEPTH AT CURB (d)	SPREAD	TYPE OF INLET	WIDTH OF FLOW OUTSIDE OF GRATE	DEPTH OUTSIDE EDGE OF GRATE (d')	FLOW OUTSIDE OF GRATE (Q'_o)	FLOW AT END OF GRATE ($Q_o-Q'_o$)	Q_o/L_o CURB OPENING Q'_o/L_o GRATE OPENING	100% PICKUP LENGTH (L_o)	L/L_o	o/d CURB OPENING o/d' GRATE OPENING	Q/Q_o CURB OPENING Q'/Q'_o GRATE OPENING	FLOW OVER SIDE OF GRATE (Q')	TOTAL FLOW INTERCEPTED (Q)	CARRY OVER FLOW	% PICKUP	INLET OK
		FT	FT	ACRES		ACRES	MIN	IN/HR	CFS	FT/FT	FT/FT	FT	FT		FT	FT	CFS	CFS		FT				CFS	CFS	CFS		

FORM T8-1
(8.4)

CHAPTER 9. STORM SEWERS

9.1 Introduction

Storm sewer systems are designed to collect and convey stormwater runoff from street inlets, from runoff control structures, and from other locations where the accumulation of stormwater is undesirable. Storm sewers are closed conduits which convey stormwater runoff from a drainage area to an outlet. Although the storm sewer is a basic part of the initial storm drainage system, it also serves to carry a significant portion of the runoff during the infrequent major storms.

The objective of a storm sewer system is to remove runoff from an area fast enough to avoid unacceptable amounts of ponding damage and inconvenience. Therefore, design criteria and general design procedures are presented herein to meet these objectives. Example calculations are included for illustration of the general design procedures.

9.2 Design Criteria

9.2.1 Design Frequency

Storm sewer sizing shall be based on the just full capacity for a 10-year frequency rainfall. After initial sizing, a hydraulic grade line (HGL) check shall be made for a 25-year frequency rainfall. If the check shows water flowing out of the system, then the system needs to be revised to contain the rainfall.

Final design shall indicate water surface elevations for the design storm. In addition, 100-year water surface elevations for all storm sewers shall be shown on the storm sewer profile.

9.2.2 Depth

The minimum cover for storm sewers in or within the right-of-way of streets with curb and gutter shall be the deeper of (a) 1 foot (clearance) from the bottom of the curb or underdrain to the top of the conduit; or (b) 2 feet below the bottom of the roadway base. A minimum cover of 2 feet of unfinished ground surface, or as recommended by manufacturer or as required for structural adequacy, is recommended at all other locations.

9.2.3 Velocity

A minimum velocity of 2.5 feet per second (fps) is recommended to insure self cleaning. The maximum allowable velocity shall be 12 fps unless special materials are included for protection against scouring.

9.2.4 Time of Concentration

The minimum inlet time of concentration for storm sewers shall be 10 minutes.

9.2.5 Design Discharge Method

The Rational Method or the Soil Conservation Service method may be used for drainage areas less than 10 acres to determine peak discharge. For areas between 10 and 200 acres, the Graphical Peak Discharge method of calculations shall be used as presented in Soil Conservation Service, Technical Release No. 55. For areas greater than 200 acres, the hydrograph method of calculation shall be used as presented in Soil Conservation Service, Technical Release No. 20.

9.2.6 Hydraulic Design

The hydraulic design of storm sewers shall be based on the Manning Equation: $V = (1.49\ r^{2/3}\ S^{1/2})/(n)$.

Special slide rules are available for the solution of this equation and nomographs are included herein (see Exhibit IX-1). The hydraulic grade line for both the initial and major design storms shall be considered.

9.2.7 Roughness Coefficients

Table 9-1 lists the Manning roughness coefficients (n) to be used for different conduit materials.

Table 9-1

Manning Roughness Coefficients

Closed Conduit Material	Manning "n"
Concrete, vitrified clay or bituminous lined corrugated metal	.013
Concrete (monolithic)	
Smooth forms	.013
Rough forms	.017
Corrugated metal pipe (1/2 inch x 2 3/4 inch corrugations)	
Plain	.024
Paved invert	.022

9.2.8 Manhole Spacing

Manholes should be located at junctions of conduits, at changes in conduit direction, at changes in slope, and at changes in pipe size. Maximum manhole spacing should be 300 feet for storm sewers with diameters up to and including 36 inches and 500 feet for storm sewers larger than 36 inches.

9.2.9 Conduit Size

The minimum conduit size shall be 12 inches in diameter. Although open channel design is an alternative which should always be considered for its appropriateness, drainage systems requiring 72 inch and larger conduits may be especially suited for open channel flow.

9.2.10 Hydraulics at Structures

The inverts of curb inlets, manholes, and other structures shall be rounded and sloped to minimize turbulence and collection of debris.

9.2.11 Location of Sewers

Location of sewers in street right-of-way shall be as approved by the City Engineer and City Stormwater Engineer.

9.3 General Design Procedures

The following general design procedures provide a uniform approach to storm sewer design. The procedure as outlined is for a storm sewer system serving an urban area with curbed streets. With minor modifications, it can apply as well to streets with side ditch swales.

The general procedures for street and inlet design, and a generalized approach to inlet spacing are discussed in the preceding chapter of the manual (Chapter 8). Street and inlet design is a basic part of the storm sewer drainage system. Maximum use should be made of the street gutter capacity to transport storm water runoff to inlets, and thereby reduce the size of the storm sewers.

The following basic data is required:

1. Map of drainage area for which the storm sewer system is to serve (Subdivision plan supplemented by United States Geological Survey [USGS] maps or Hamilton County topographic maps for off-site area, if required).

2. Typical street cross-sections and profiles.

3. Pavement Drainage Computations (use form T9-1 provided).

4. Soil maps and data.

5. Outfall Elevation (from field measurement).

6. Rainfall intensity-duration-frequency curves or tabulation (Exhibit V-2).

Step 1. Determine proposed curb inlet locations based on gutter capacity (Article 8.3).

Calculate the initial storm sewer pipe size starting at the most upstream inlet location and working downstream as follows.

Step 2. Calculate the initial storm, 10-year frequency, total runoff (Q) to the storm sewer inlet.

Step 3. Estimate the slope of the storm sewer to the next manhole and using Exhibit IX-1 with flow, pipe roughness, and pipe slope determine the standard size storm sewer pipe diameter required. For this size, read flowing full capacity and the corresponding pipe velocity.

Step 4. Check that flowing full discharge is greater than total runoff discharge and that the pipe velocity meets the design criteria velocity.

Step 5. Calculate the flow travel time between the two manholes. Travel time equals pipe length/pipe velocity. For each successive downstream manhole, the time of concentration used should be the greater of the preceding manhole's time of concentration plus the flow travel time between the two manholes or the time of concentration of the intermediate area between the two manholes.

Step 6. Calculate the manhole bottom elevation which equals manhole elevation minus pipe length times pipe slope and check depth meets the design criteria.

Step 7. Go back to Step 2 and repeat Steps 2 through 6 for each length of storm sewer throughout the system until initial sizing of all the storm sewer pipes has been completed.

Calculate the hydraulic gradient for the 25-year frequency rainfall starting at the storm sewer system outlet and working upstream as follows:

Step 8. The control elevation (hydraulic gradient elevation) at the system outlet can be taken as the conduit crown for a freely discharging sewer or as the pool elevation for a submerged outlet.

Step 9. Determine the 25-year rainfall intensity at the outlet. This intensity will be used throughout the storm sewer system so long as the system is under pressure. If the system is not under pressure then the intensity would change at that manhole and would be used so long as the system is not under pressure.

Step 10. Calculate the 25-year frequency total runoff at the storm sewer inlet.

Step 11. Using Exhibit IX-1 with flow, pipe size, and pipe roughness determine the friction slope to the next inlet.

Step 12. Calculate head loss to next inlet which equals friction slope times pipe length.

Step 13. Determine the hydraulic gradient elevation at the next inlet which equals hydraulic gradient elevation of inlet plus head loss. This hydraulic gradient elevation must be lower than the inlet gutter elevation. If it is not, the pipe must be resized. Go to Step 2 and begin calculations again. For the next pipe upstream, the HGL is assumed to be at the crown of the conduit at the downstream end of that conduit. Ordinarily, the hydraulic gradient will be above the top of the pipe causing the system to operate under pressure. If, however, any run in the system does not flow full (pipe slope steeper than friction slope), the hydraulic gradient slope will follow the friction slope until it reaches normal depth of flow in the steep run. From that point, it will coincide with normal depth of flow until it reaches a run flatter than the friction slope for the run.

Step 14. Go back to Step 9 and repeat Steps 8 through 11 for each length of storm sewer throughout the system.

Step 15. The hydraulic effects of the 100-year storms on the drainage system are determined for compliance with the physical design criteria presented herein.

Step 16. The final design is drawn on prepared plan and profile sheets.

9.4 Example Calculations (Initial Design Storm)

The following example is included to further illustrate the streets and invert design procedures stated herein and follows the steps given in the preceding Article. Since most storm sewers in the Cincinnati area will probably be designed by the Rational Method, the example design calculations included, Exhibit IX-2, are based on that method. Exhibit IX-3 is a storm sewer design computations worksheet that references how values are obtained for each column. Exhibit IX-2 is a completed storm sewer design computations worksheet for initial design storm and the 25-year frequency rainfall hydraulic grade line check for this example. The major storm calculations are included in the following Article.

Determine the storm sewer concrete pipe sizes for the residential development as shown on Figure 9-1. The street inlets type and location where determined in Article 8.4. Refer to Article 8.4 for subarea delineation and other design data. Note the inlets are on both sides of the street and the storm sewer pipe will be located on one side of the street.

Step 1. The proposed curb inlet location was determined in Article 8.4.

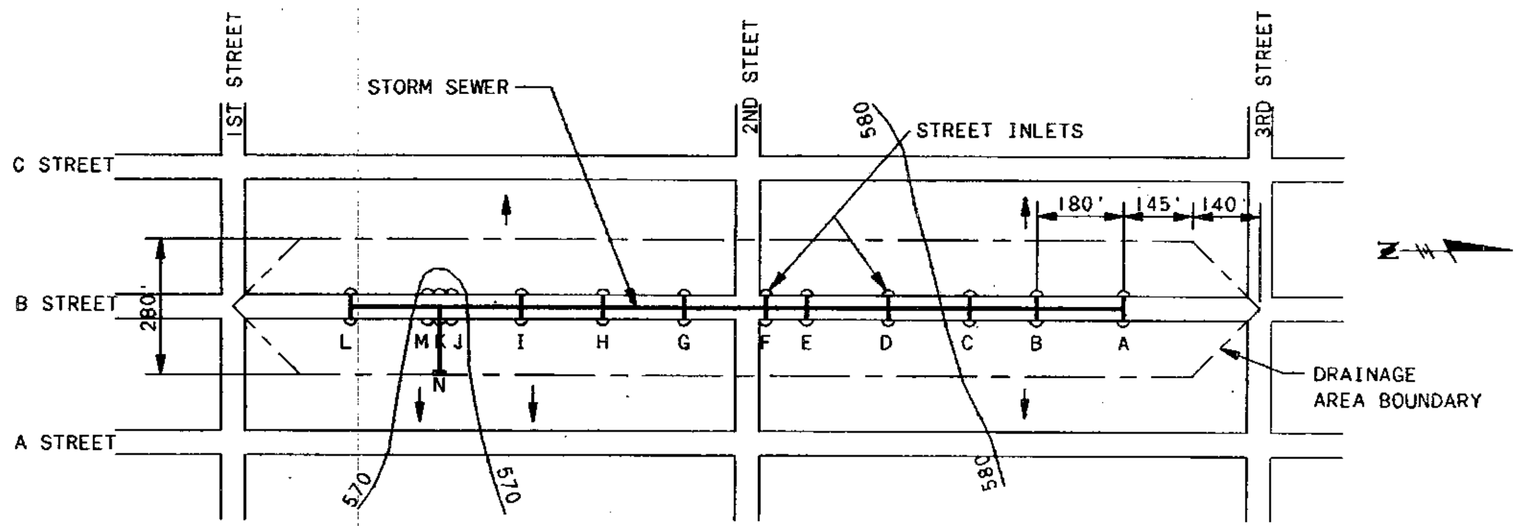
Calculate the initial storm sewer pipe size starting at the uppermost inlet location A and going to the next downstream inlet location B.

Step 2. For inlet station A calculate peak flow Q = CiA. From Example 8.4 A = 2 x 0.691 = 1.38; c = 0.50; inlet time = 12.84 minutes and 10-year rainfall i = 4.94; therefore, Q = CiA = 0.50 x 4.94 x 1.38 = 3.41 cfs. For this example there is no other controlled runoff, so total runoff = 3.41 cfs.

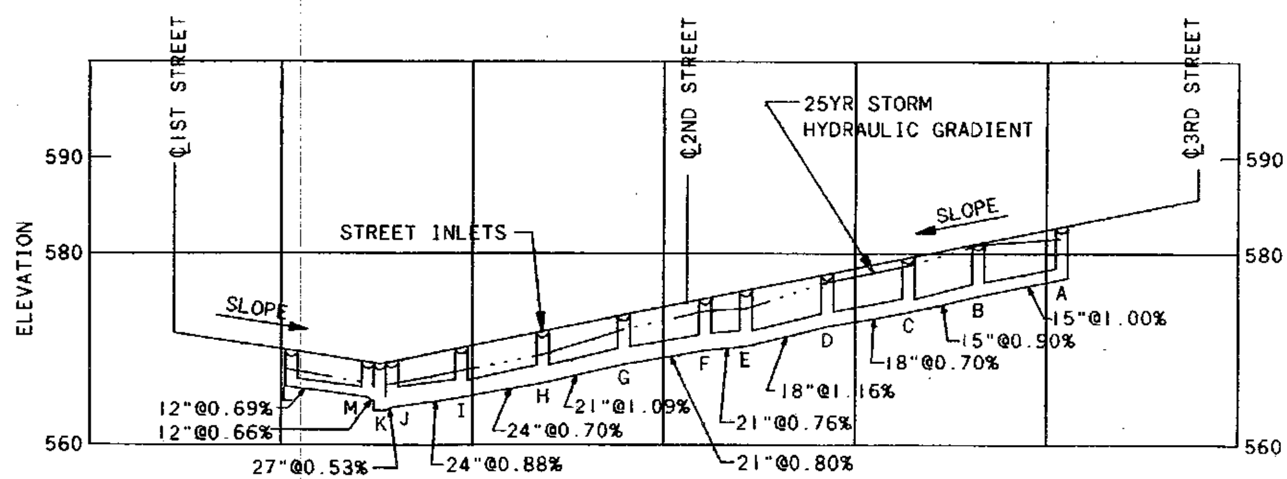
Step 3. Estimate the pipe slope s = 0.01 feet/feet. Enter Exhibit IX-1 with Q = 3.41 cfs; n = 0.015 and s = 0.01 feet/feet read next larger standard size of D = 15 inches. For D = 15 inches read velocity full capacity Q_f = 6.46 cfs and velocity v = 5.26 fps.

Step 4. Check that $Q_f > Q$ = 6.46 cfs > 3.41 cfs check is OK. Check that 2.5 fps < V < 12 fps = 2.5 fps < 5.26 fps < 12 fps check is OK.

Step 5. Calculate flow travel time between inlets A and B = L/V = 180/5.26 = 34.22 sec./60 = 0.57 minutes.



PLAN
NO SCALE



B STREET PROFILE
NO SCALE

TYPICAL RESIDENTIAL AREA

FIGURE 9-1

Step 6. Calculate inlet bottom elevation. For inlet location A the value is set by the design = 577.33. For inlet location B = inlet elevation of A - (length x slope) = 577.33 - (180 x .01) = 575.53.

Depth cover = 582.70 - (577.33 + 15/12) = 4.12 feet > 2 feet below roadway base.

Step 7. Steps 2 through 6 would be repeated for the remaining inlet. These calculations are shown on Exhibit IX-2.

Step 8. Starting at the outlet on location N. The hydraulic gradient elevation = crown of outlet conduit = 562.41 + (27/12) = 564.66.

Step 9. At the outlet pipe T_C = 17.28 minutes. Enter Table V-2 with a duration of 17.28 minutes; read 25-year rainfall intensity = 5.00 inches per hour.

Step 10. Calculate 25-year discharge Q = CiA. From column 5, CA = 6.43 and Q = CiA = 6.43 x 5.00 = 32.2 cfs.

Step 11. Enter Exhibit IX-1 with Q = 32.2, D = 27. n = 0.015 and read S = 0.0110 feet/feet.

Step 12. Head loss from inlet location N to inlet location K = friction slope x pipe length = 0.0110 x 140 = 1.54 feet.

Step 13. Hydraulic gradient at next inlet K equals hydraulic gradient at inlet N plus head loss = 564.66 + 1.54 = 566.20 feet. 566.20 < 568.48 flow still in system. 566.20 > 563.55 + pipe diameter = 563.55 + 27/12 = 565.80 still pressure flow.

Step 14. Steps 10 through 13 would be repeated for the remaining inlets. If in Step 13 flow is not pressure flow, go to Step 8 and restart calculation. If flow goes out of system then go to Step 2 and determine a new size pipe and continue with Steps 2 through 14.

Step 15. The major design storm would be checked. Article 9.6 presents an example of the major design storm calculations.

Step 16. Final design is drawn on prepared plan and profile sheet.

The final design conduit sizes and slopes together with the 25-year HGL are shown in the profile of Figure 9-1.

9.5 Major Storm Considerations

The 100-year storm runoff is routed through the drainage system to determine if the combined capacity of the street and storm sewer system is sufficient to maintain surface flows within permissible limits. The maximum allowable flow depth of a storm is stated in Article 8.2.2. The capacity of the storm sewer conduit at any given point for the 100-year storm is assumed to be one-half of the design storm capacity for determining the required capacity of surface channels as stated in Article 8.2.2. If the 100-year storm runoff exceeds the combined capacity of the street and storm sewer drainage system, revision in the design is required. Where a drainageway is located outside a street right-of-way, easements shall be provided.

9.6 Example - Major Storm Calculations

Determine the effect of the major storm on the residential development as shown on Figure 9-1. The street inlet type and location were determined in Article 8.4 and the storm sewer pipe sizes were determined in Article 9.4. It is assumed that runoff from other drainage areas is served by other streets and storm sewers, and that "B" Street is designated as an arterial street.

9.6.1 Determine Peak Discharge and Total Runoff Volume

Determine the major storms peak discharge and runoff volume from the residential development as shown on Figure 9-1. Both the peak discharge and total runoff volume need to be determined so that the street flow capacity and available storage volume at the sump area can be evaluated against the design criteria. The calculations are shown on Figure 9-2 and follow the general procedure presented in Article 6.4.1.1.

9.6.2 Determine Allowable Street Carrying Capacity

Determine the allowable street carrying capacity of "B" street. The major storm shall not exceed 6-inch depth at crown. For "B" street the most restrictive criteria is not exceeding 6-inch depth of flow at crown. Therefore, the street capacity will be calculated for this condition. Figure 9-1 shows the active flow area for "B" Street.

Step 1. The capacity is calculated using the Manning's Formula

$$V = \frac{1.49}{n} r^{2/3} S^{1/2} \text{ and } Q = VA$$

Step 2. From Article 8.4 s = 0.01 feet per foot and n = 0.015.

Step 3. Calculate active flow area for 1/2 the street, as shown on Figure 9-3.

A_1 = 1/2 LH = 1/2 x 6.00 x 2.45/12 = 0.61 square feet.
A_2 = LH' + 1/2 L'H' = 6.00 x (3.90 - 2.95)/12 + 1/2 x 7.00 x (3.90 - 2.45)/12 = 1.15 square feet.
A_3 = LH = 13 x 6/12 = 6.50 square feet.

Total for 1/2 street A = A_1 + A_2 + A_3 = 0.61 + 1.15 + 6.50 = 8.26 square feet.

Total A for "B" Street = A x 2 = 8.26 x 2 = 16.5 square feet.

Step 4. Calculate hydraulic radius = area/wetted perimeter (WP) referring to Figure 9-1 going from top of curb to centerline of street WP = 5.75/12 + 13 = 13.48 feet.
WP for "B" Street = WP x 2 = 13.48 x 2 = 26.96 feet.
r = A/WP = 16.5/26.96 = 0.61.

STORMWATER RUNOFF
GRAPHICAL PEAK DISCHARGE COMPUTATIONS

PROJECT ______________ DESIGNER ______________ DATE ______

1) DATA: WATERSHED CONDITION = FUTURE (PRESENT OR FUTURE) TYPE II STORM

DRAINAGE AREA (DA) = 12.85 ACRES.

Hydrologic Soil Group Exhibit VI-3	Land Use Description Include Treatment, Practice & Condition Exhibit VI-5	CN Exhibit (3) VI-5	CN Exhibit (3) VI-11	Area (acres)	Area (%) (4)	Product (3) x (4) (5)
C	1/4 ACRE LOTS, RESIDENTIAL	83	–	12.85	100	8300
			Totals =		100	8300

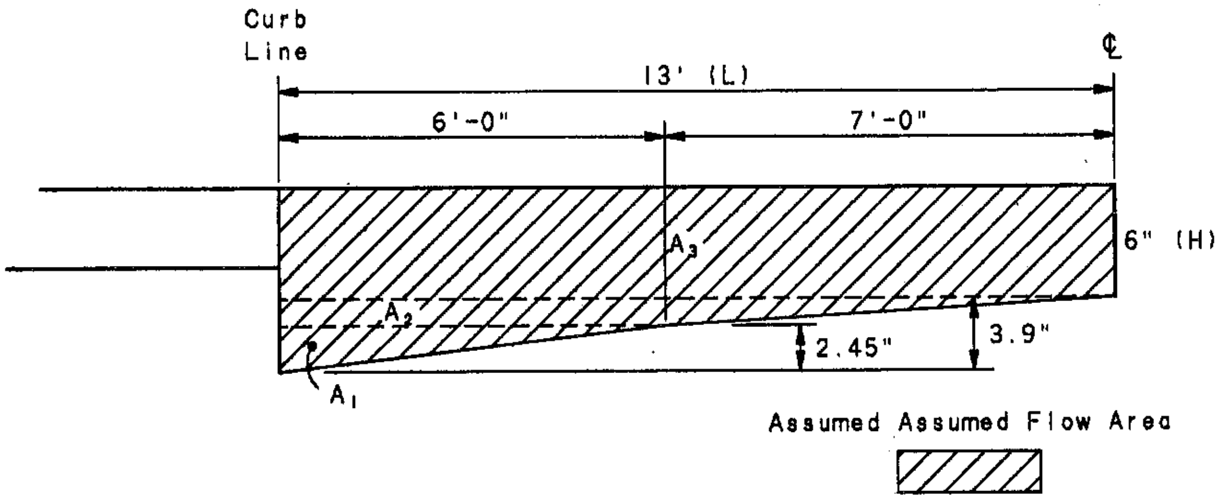
$$\text{CN (weighted)} = \frac{\text{total col. (5)}}{\text{total col. (4)}}\left[\frac{8300}{100}\right] = 83 \; ; \quad \text{use CN} = \boxed{83}$$

Ponding and Swampy areas (PND) = 0 acres, 0 % of DA

Time of Concentration (TC) = 17.28 minutes 0.30 hours

		1st Storm	2nd Storm	3rd Storm	
2)	Rainfall Frequency (F)	100			yrs.
	Rainfall Depth (P) From Table 5-1	5.76			inches
3)	Initial Abstraction (Ia) $Ia = 0.2\left[\frac{1000}{\text{weighted CN}} - 10\right]$	0.41			
4)	(Ia) / (P)	0.07			
5)	Unit Peak Discharge Use TC, Ia/P, and Exhibit VI-7	700			CFS/Square Mile-Inch
6)	Runoff Depth (Q) Use P, CN, and Exhibit VI-6	3.86			inches
7)	Ponding and Swampy Area Adjustment Factor Use % PND. and Exhibit VI-8	–			
8)	Adjusted Peak Discharge (q_g) Drainage area x step 5 x (step 6 x step 7) / 640	54.3			cfs
9)	Total Runoff Volume Step 6 x Drainage Area / 12	4.13			acre-feet

FIGURE 9-2



HALF - SECTION

MAJOR FLOODWAY AREA
ARTICLE 9.6.2 EXAMPLE

FIGURE 9-3

Step 5. Calculate Q using Manning's Formula

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

$$1.49/0.015 \times 0.61^{2/3} \times 0.01^{1/2} \times 16.5 = 118 \text{ cfs.}$$

The maximum street drainageway capacity (118 cfs) is much greater than the major storm peak discharge (54.3 cfs). Therefore, the actual depth of flow in the street on a continuous slope is within allowable limits.

9.6.3 Determine Ponding at Sump (Design Point "K")

Determine if the ponding storage at sump, design point "K" has enough storage to handle major storm runoff without an overflow channel. The street low point actually functions as a structural runoff control facility with the combination inlets and proposed 27-inch storm sewer being the flow control device. The graphical flow routing method discussed in Article 12.9.4 can be used to determine the approximate storage volume.

Step 1. The 100-year frequency major storm has a peak discharge of 54.3 cfs and runoff volume of 4.13 acre-feet, or 179,900 cubic feet, as calculated in Article 9.6.1.

Step 2. The outlet pipe has a capacity of 28.0 cfs as calculated in Article 9.4. As stated in the design criteria Article 8.2.2, the allowable major storm sewer capacity = 0.5 x capacity = 0.5 x 28.0 = 14.0 cfs.

Step 3. Calculate ratio of peak outflow discharge to peak inflow discharge = Q_0/Q_1 = 14.0/54.3 = 0.26.

Step 4. Enter Exhibit XII-5 with Q_0/Q_1 = 0.26 and read V_s/V_r = 0.41.

Step 5. Calculate $V_s = V_r \times (V_s \times V_r)$ = 0.41 x 179,900 = 73,800 cubic feet.

Step 6. Estimate the amount of storage available at the sump. The maximum depth can only be 6 inches above the crown of the roadway as per design criteria, therefore, storage available at slope is approximately equal to cross-section area of road x maximum depth/ road slope = 16.5 x (6/12)/0.01 + 16.5 x (6/12)/0.008 = 1,856 cubic feet.

The maximum storage volume provided at the street sump is approximately 1,856 cubic feet which is less than the required storage volume of 73,800 cfs. Therefore, an overflow channel is needed to carry the excess runoff volume (Figure 9-1).

9.6.4 Overflow Channel Design

Determine an overflow channel design to handle the major design storm. A triangular channel lined with Kentucky Bluegrass will be used for the channel. According to Exhibit VII-4, the maximum permissible velocity is 5 fps.

Step 1. Calculate required discharge of channel - major storm discharge - storm sewer capacity = 54.3 - 28.0/2 = 40.3 cfs.

Step 2. Calculate required minimum area of channel = A_c = discharge/maximum velocity = 40.3/5 = 8.1 square feet.

Step 3. From the site condition the channel would have a slope of 0.07 feet per foot. The maximum depth over the curb into the channel would be 6-inches. According to Exhibit VII-2 Manning's n of channel = 0.025.

Enter Exhibit VIII-1 with d = 0.5 feet, S = 0.03 feet per foot, and Q = 40.3 and read Z/n = 2,700.

Step 4. Calculate Z = Z/n x n = 2,700 x 0.025 = 67. This is steeper than the natural ground of 100 (slope of 0.01 feet per foot).

Step 5. Calculate top width of the triangular channel during passage of the major design storm = Z x depth = 67 x 0.5 = 33 feet.

Step 6. Check actual flow velocity of channel = Q/A = 40.3/(0.5 x 33 x 0.5) = 4.9 fps.

This is less than the maximum permissible velocity of 5 fps, so design is acceptable.

9.7 Bibliography

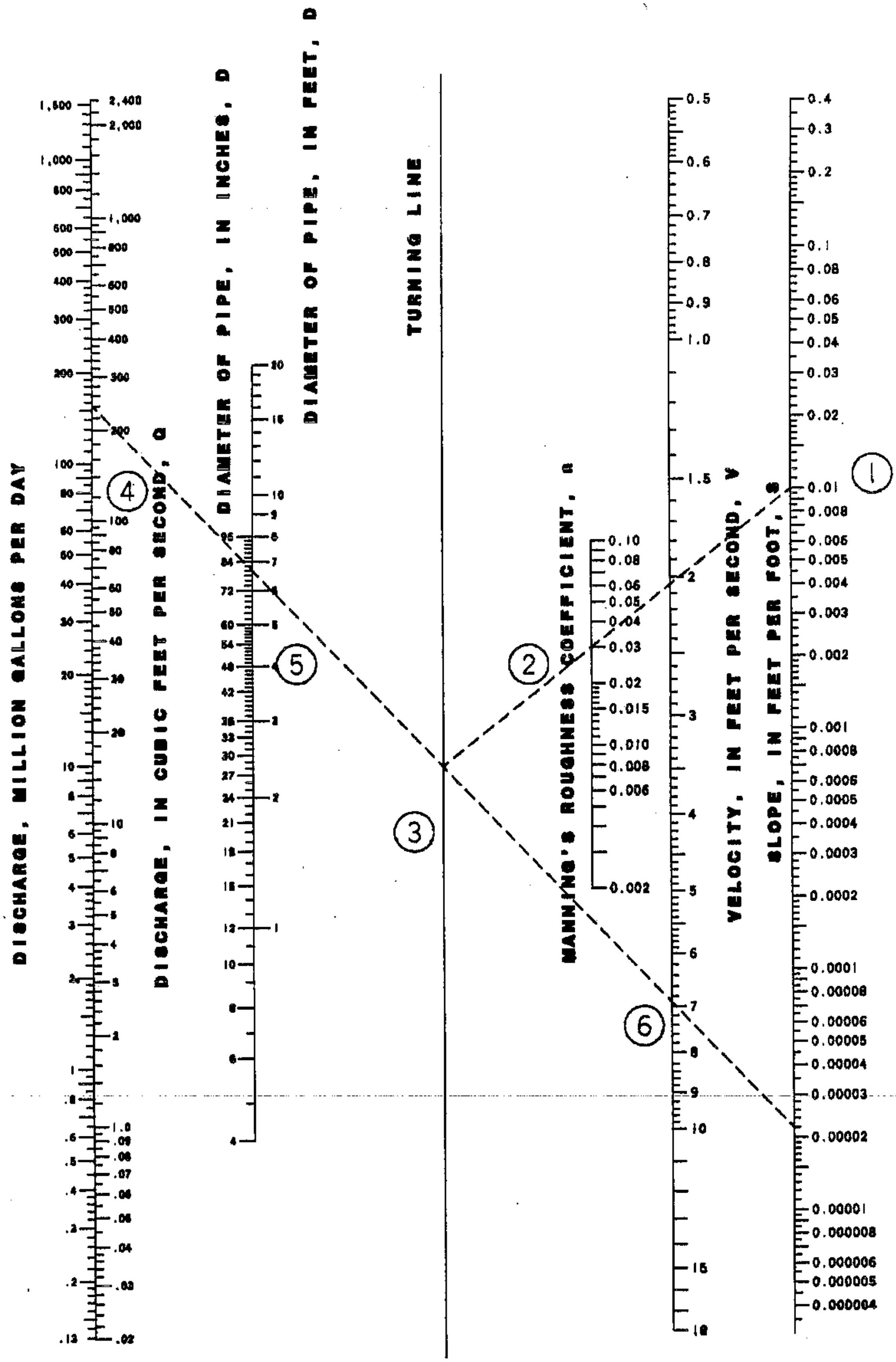
National Clay Pipe Institute. Clay Pipe Engineering Manual. Washington, D.C., 1974.

American Concrete Pipe Association. Concrete Pipe Design Manual. Arlington, Virginia. February 1970.

American Society of Civil Engineers. Design and Construction of Sanitary and Storm Sewers. Manuals and reports on Engineering Practice, No. 37. (Water Pollution Control Federation Manual of Practice No. 9). New York, 1970.

County of Fairfax, Virginia. Policies and Guidelines for the Preparation of Subdivision Plans and Site Development Plans. January 1964.

Denver Regional Council of Governments. Urban Storm Drainage Criteria Manual, Volume 1. Wright-McLaughlin Engineers. Denver, Colorado. March 1969.



NOMOGRAPH FOR SOLUTION OF THE MANNING FORMULA

$$Q=AV=A\frac{1.49}{n}r^{2/3}s^{1/2}$$

EXHIBIT IX-1

STORM SEWER COMPUTATIONS
ARTICLE 9.4 EXAMPLE

PROJECT ______________________ DESIGNER ______________________ DATE ____________

DISCHARGE											STORM SEWER SIZE											HYDRAULIC GRADIENT						
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29
INLET LOCATION (DESIGN POINT)	RUNOFF COEFFICIENT C	DRAINAGE AREA A	CA	ΣCA	INLET OR CONDUIT TRAVEL TIME	TIME OF CONCENTRATION T_c	RAINFALL INTENSITY 10 YEAR FREQUENCY I	RUNOFF IΣCA	OTHER CONTROLLED RUNOFF	TOTAL RUNOFF, Q	PIPE LINE DESIGNATION	PIPE DIAMETER	LENGTH, L	SLOPE	GUTTER AT INLET OR COVER ELEV.	INLET OR MANHOLE BOTTOM	PIPE COVER	MEETS COVER DESIGN CRITERIA	JUST FULL CAPACITY	VELOCITY	MEETS VELOCITY DESIGN CRITERIA	RAINFALL INTENSITY 25 YEAR FREQUENCY I	TOTAL RUNOFF Q	SLOPE	HEAD LOSS	ELEVATION OF HYDRAULIC GRADIENT	CROWN PIPE	PRESSURE FLOW
		ACRES			MIN.	MIN.	IN/HR	CFS	CFS	CFS		IN	FT	FT/FT			FT		CFS	FPS		IN/HR	CFS	FT/FT	FT			
A	0.50	1.38	0.69	0.69		12.84	4.94	3.41	-	3.41					582.70	577.33	4.12	Y				5.69	3.9	.004		581.56	578.58	Y
					0.57						AB	15	180	0.010					6.46	5.26	Y				.067			
B	0.50	1.16	0.58	1.27		13.41	4.85	6.16	-	6.16					580.90	575.53	4.12	Y				5.59	7.1	.012		580.89	576.78	Y
					0.60						BC	15	180	0.009					6.16	5.02	Y				2.18			
C	0.50	1.16	0.58	1.85		14.01	4.76	8.80	-	8.80					579.10	573.89	3.96	Y				5.49	10.2	.009		578.71	575.14	Y
					0.60						CD	18	180	0.007		573.64			8.80	4.98	Y				1.68			
D	0.50	1.16	0.58	2.43		14.61	4.67	11.33	-	11.33					577.30	572.38	3.42	Y				5.39	13.1	.016		577.03	573.88	Y
					0.47						DE	18	180	0.012					11.33	6.41	Y				2.79			
E	0.50	1.16	0.58	3.01		15.08	4.60	13.83	-	13.83					575.50	570.29	3.71	Y				5.32	16.0	.010		574.24	571.79	Y
					0.07						EF	21	25	0.008		570.04			13.83	5.75	Y				0.26			
F	0.50	0.15	0.08	3.08		15.15	4.59	14.15	-	14.15					575.25	569.85	3.65	Y				5.31	16.4	.011		573.98	571.60	Y
					0.51						FG	21	180	0.008					14.15	5.88	Y				1.92			
G	0.50	1.16	0.58	3.66		15.66	4.52	16.55	-	16.55					573.45	568.41	3.29	Y				5.23	19.2	.015		572.06	570.16	Y
					0.44						GH	21	180	0.011					16.55	6.88	Y				2.63			

EXHIBIT IX-2

STORM SEWER COMPUTATIONS
ARTICLE 9.4 EXAMPLE

PROJECT ______________ DESIGNER ______________ DATE ________

DISCHARGE											STORM SEWER SIZE											HYDRAULIC GRADIENT						
1 INLET LOCATION (DESIGN POINT)	2 RUNOFF COEFFICIENT C	3 DRAINAGE AREA A	4 CA	5 ΣCA	6 INLET OR CONDUIT TRAVEL TIME	7 TIME OF CONCENTRATION T_c	8 RAINFALL INTENSITY 10 YEAR FREQUENCY I	9 RUNOFF IΣCA	10 OTHER CONTROLLED RUNOFF	11 TOTAL RUNOFF, Q	12 PIPE LINE DESIGNATION	13 PIPE DIAMETER	14 LENGTH, L	15 SLOPE	16 GUTTER AT INLET OR COVER ELEV.	17 INLET OR MANHOLE BOTTOM	18 PIPE COVER	19 MEETS COVER DESIGN CRITERIA	20 JUST FULL CAPACITY	21 VELOCITY	22 MEETS VELOCITY DESIGN CRITERIA	23 RAINFALL INTENSITY 25 YEAR FREQUENCY I	24 TOTAL RUNOFF Q	25 SLOPE	26 HEAD LOSS	27 ELEVATION OF HYDRAULIC GRADIENT	28 CROWN PIPE	29 PRESSURE FLOW
		ACRES			MIN.	MIN.	IN/HR	CFS	CFS	CFS		IN	FT	FT/FT			FT		CFS	FPS		IN/HR	CFS	FT/FT	FT			
H	0.50	1.16	0.58	4.24		16.10	4.46	18.92	-	18.92					571.65	566.45	3.45	Y				5.17	21.9	.009		569.43	568.20	Y
					0.50						HI	24	180	.007		566.20			18.92	6.02	Y				1.69			
I	0.50	1.16	0.58	4.82		16.59	4.39	21.18	-	21.18					569.85	564.94	2.91	Y				5.10	24.6	.012		567.74	566.94	Y
					0.28						IJ	24	115	.009					21.18	6.74	Y				1.36			
J	0.50	0.74	0.37	5.19		16.88	4.36	22.62	-	22.62					568.70	563.93	2.77	Y				5.06	26.2	.007		566.38	565.93	Y
					0.07						JK	27	25	.005		563.60			22.62	5.69	Y				0.18			
K	0.50	0.16	0.08	5.27		16.95	4.35	22.92	-	22.92					568.45	563.55	2.65	Y				5.05	26.6			566.20	565.80	Y
					-																							
												12																
L	0.50	1.19	0.59	0.59		12.64	4.97	2.95	-	2.95					569.85	566.00	2.85	Y				5.72	3.4	.009		567.78	567.00	Y
					0.66						LM	12	150	.007					2.95	3.76	Y				1.36			
M	0.50	0.96	0.48	1.08		13.30	4.87	5.24	-	5.24					568.65	564.97	2.68	Y				5.61	6.0	.009		566.42	565.97	Y
					0.10						MK	15	25	.007		564.72			5.24	4.27	Y				0.22			
K	0.50	0.16	0.08	6.43		16.95	4.35	27.96	-	27.96					568.45	564.55	2.65	Y				5.05	32.5	.011		566.42	565.80	Y
					0.33						KN	27	140	.008		563.55			27.96	7.03	Y				1.54			
N	0.50	-	0.00	6.43		17.28	4.31	27.69	-	27.69					567.05	562.41	2.39	Y				5.00	32.5	-		564.66	564.66	Y

EXHIBIT IX-2
CONTINUE

STORM SEWER COMPUTATIONS GUIDE

PROJECT ______________ DESIGNER ______________ DATE ______________

DISCHARGE	1	INLET LOCATION (DESIGN POINT)		
DISCHARGE	2	RUNOFF COEFFICIENT C		SITE SPECIFIC SEE CHAPTER 6
DISCHARGE	3	DRAINAGE AREA A	ACRES	DRAINAGE AREA TO INLET
DISCHARGE	4	CA		COLUMN 2 x COLUMN 3
DISCHARGE	5	ΣCA		SUMMATION OF COLUMN 4
DISCHARGE	6	INLET OR CONDUIT TRAVEL TIME	MIN.	TIME OF CONCENTRATION TO INLET SEE CHAPTER 6 OR COLUMN 14 / COLUMN 20
DISCHARGE	7	TIME OF CONCENTRATION T_c	MIN.	CURRENT INLET'S INLET TIME COLUMN 6 OR PRECEEDING INLET TIME + CONDUIT TRAVEL TIME COLUMN 6 WHICHEVER IS GREATER
DISCHARGE	8	RAINFALL INTENSITY ___ YEAR FREQUENCY I	IN/HR	EXHIBIT V-1 OR V-2 WITH COLUMN 7
DISCHARGE	9	RUNOFF IΣCA	CFS	COLUMN 5 x COLUMN 8
DISCHARGE	10	OTHER CONTROLLED RUNOFF	CFS	ANY OTHER RUNOFF ENTERING STORM SEWER PIPE
DISCHARGE	11	TOTAL RUNOFF, Q	CFS	COLUMN 9 + COLUMN 10
STORM SEWER SIZE	12	PIPE LINE DESIGNATION		SITE SPECIFIC
STORM SEWER SIZE	13	PIPE DIAMETER	IN	ASSUME
STORM SEWER SIZE	14	LENGTH, L	FT	SITE SPECIFIC
STORM SEWER SIZE	15	SLOPE	FT/FT	SITE SPECIFIC
STORM SEWER SIZE	16	GUTTER AT INLET OR COVER ELEV.		SEE STREET INLET CALCULATION
STORM SEWER SIZE	17	INLET OR MANHOLE BOTTOM		SET FOR FIRST INLET THEN CALCULATE AS INLET ELEVATION - COLUMN 14 x COLUMN 15
STORM SEWER SIZE	18	PIPE COVER	FT	COLUMN 16 - COLUMN 17
STORM SEWER SIZE	19	MEETS COVER DESIGN CRITERIA		
STORM SEWER SIZE	20	JUST FULL CAPACITY	CFS	EXHIBIT IX-1 WITH COLUMNS 13, 15 & PIPE ROUGHNESS
STORM SEWER SIZE	21	VELOCITY	FPS	EXHIBIT IX-1 WITH COLUMNS 11 AND 13 AND EXHIBIT X-6 WITH RATIO OF COLUMN 11 TO COLUMN 20
STORM SEWER SIZE	22	MEETS VELOCITY DESIGN CRITERIA		
HYDRAULIC GRADIENT	23	RAINFALL INTENSITY ___ YEAR FREQUENCY I	IN/HR	EXH. V-1 OR V-2 W/COL.7 FOR LAST DN. STRM. INL. IS CONST. TILL NO LONGER PRESS. FLOW THEN START AGAIN W/NEW COL.7
HYDRAULIC GRADIENT	24	TOTAL RUNOFF Q	CFS	COLUMN 5 x COLUMN 23
HYDRAULIC GRADIENT	25	SLOPE	FT/FT	EXHIBIT IX-1 WITH COLUMN 13, 24 & PIPE ROUGHNESS
HYDRAULIC GRADIENT	26	HEAD LOSS	FT	COLUMN 14 x COLUMN 25
HYDRAULIC GRADIENT	27	ELEVATION OF HYDRAULIC GRADIENT		DOWNSTREAM HYDRAULIC GRADIENT PLUS COLUMN 26 IF NOT PRESSURE FLOW USE CROWN OF PIPE
HYDRAULIC GRADIENT	28	CROWN PIPE		COLUMN 13 + COLUMN 17
HYDRAULIC GRADIENT	29	PRESSURE FLOW		

EXHIBIT IX-3

STORM SEWER COMPUTATIONS

PROJECT ____________________ DESIGNER ____________________ DATE __________

DISCHARGE											STORM SEWER SIZE											HYDRAULIC GRADIENT						
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29
INLET LOCATION (DESIGN POINT)	RUNOFF COEFFICIENT C	DRAINAGE AREA A	CA	ΣCA	INLET OR CONDUIT TRAVEL TIME	TIME OF CONCENTRATION t_c	RAINFALL INTENSITY ___ YEAR FREQUENCY I	RUNOFF IΣCA	OTHER CONTROLLED RUNOFF	TOTAL RUNOFF, Q	PIPE LINE DESIGNATION	PIPE DIAMETER	LENGTH, L	SLOPE	GUTTER AT INLET OR COVER ELEV.	INLET OR MANHOLE BOTTOM	PIPE COVER	MEETS COVER DESIGN CRITERIA	JUST FULL CAPACITY	VELOCITY	MEETS VELOCITY DESIGN CRITERIA	RAINFALL INTENSITY ___ YEAR FREQUENCY I	TOTAL RUNOFF Q	SLOPE	HEAD LOSS	ELEVATION OF HYDRAULIC GRADIENT	CROWN PIPE	PRESSURE FLOW
		ACRES			MIN.	MIN.	IN/HR	CFS	CFS	CFS		IN	FT	FT/FT			FT		CFS	FPS		IN/HR	CFS	FT/FT	FT			

CHAPTER 12. RUNOFF CONTROL METHODS

12.1 Introduction

The runoff control criteria stated below necessitates the use of stormwater runoff control facilities in many development situations. While the success of such facilities for accomplishing a desirable level of runoff control cannot be denied, it is often found these same facilities have the potential for adding to neighborhood blight or a threat to public health and safety.

It is not necessary that the "cure" be worse than the "disease." Stormwater storage facilities can be functional and wholly unobtrusive. If desired, or in some cases, their presence can offer an added emenity to the urban environment. This positive impact can be achieved by adherence to four basic steps in the implementation of stormwater storage facilities. These are:

1. Proper selection of runoff control mechanism.

2. Proper design of facility.

3. Construction maintenance program and designated responsibility for maintenance.

4. Regular maintenance program and designated responsibility for maintenance.

This chapter discusses these steps which result in a rewarding and often cost-saving approach to stormwater management.

12.2 Design Criteria for Runoff Control

Peak flow runoff controls shall be required on all land developments and redevelopments except for those which contain only Class A and/or Class B property as defined in Section 720-55 of the Stormwater Management Code and which comprise less than four contiguous properties. Exemption from runoff controls will also be granted for developments of less than one-half acre if the difference between the predevelopment and post-development runoff coefficient is less than 20 percent. When phased construction is planned or occurs, the total land area to be developed shall be considered when planning the stormwater facilities.

For all developments or redevelopments, except those exempted above, stormwater detention shall be in accordance with the Stormwater Management Utility's Master Plan and shall be accepted and approved by the Utility Engineer. Detention shall be provided to assure that the peak rate of runoff from the area after development does not exceed the peak rate of runoff from the same area before development for the 2-, 10-, 25-, and 100-year frequency, 24-hour storms. The 24-hour rainfall amounts are given in Table 5-1.

The recommended method for determining the amount of runoff control is based on the size of the area under study. For sites with drainage areas less than 10 acres, the storage equation method is the preferred method. For sites with drainage areas between 10 and 200 acres, the graphical flow routing method, as defined in the Soil Conservation Services' Technical Release No. 55, is the preferred method. For sites with drainage areas greater than 200 acres, the

Soil Conservation Services' Technical Release No. 20 Method is the preferred method. All routing calculations shall account for tailwater conditions of the receiving facility and shall be submitted to the utility.

12.3 Approaches to Runoff Control

The runoff control requirements may be achieved in a variety of ways. A basic differentiation is the nonstructural approach versus the structural runoff control mechanism.

12.3.1 Nonstructural Approach

The nonstructural approach relies on land use management techniques and site design methods to reduce the impact of accelerated storm runoff or to minimize the quantity of storm runoff. Such approaches may be broadly classified as watershed management.

Watershed management is directed at land use in areas outside the flood plain but within naturally occurring, geographically defined drainage areas. Watershed management emphasizes control of stormwater close to its source to reduce undesirable impacts on lower, servient lands.

Watershed management is not a euphemism for land use control. Such control is both unrealistic and unnecessary. However, the rationale of considering runoff impact relationships is valid. Negative runoff impacts can be minimized by orienting roads, buildings, and open space to the unique topographic and drainage characteristics of a site.

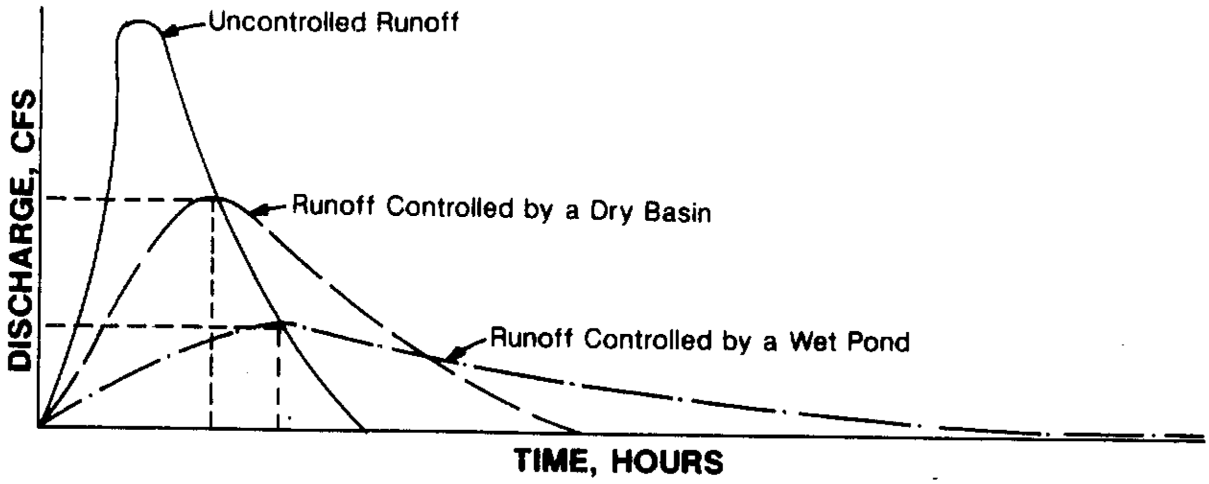
12.3.2 Structural Approaches

Structural approaches are those that rely on physical facilities designed and constructed for the purpose of controlling stormwater runoff. These can be placed in three major categories in terms of their runoff control mechanisms: detention structures, infiltration structures, and conduit structures. While both structural and nonstructural approaches are effective tools for stormwater management, the most effective runoff control will result when both approaches are considered and applied collectively. Chapter 12 confines itself to a more detailed discussion of structural solutions to stormwater runoff control.

12.4 Detention Structures

Structures for detention of stormwater may be considered together since the major control structures function the same for each. The principal difference is that while the control mechanism for dry basin may be entirely passive, there should be some manner of movable gate provided in a wet basin so that it can be drained for occasional maintenance. Any detention basins should be checked for compliance with Chapter 1521 of the Ohio Revised Code and Ohio Department of Natural Resources (ODNR) Division of Water Regulations and, if required, a construction permit must be obtained from the ODNR.

The results of applying detention storage methods for peak discharge control are illustrated by the typical hydrographs shown on the following page. The objective of both methods is to reduce peak rate of discharge by storage and



TYPICAL DETENTION HYDROGRAPHS

controlled release. The difference in discharge rates shown is a function of the control devices discussed in Article 12.9 rather than the type of structure.

Infiltration of the stored stormwater will occur to some degree depending on underlying soil characteristics. However, in most cases, the amount of infiltration is minimal and is not considered as a part of the structural design. Thus, the total area under the hydrographs representing total volume of runoff will be substantially the same for all three cases.

Land requirements are associated with any storage facilities. The demand for land is a major economic factor. For this reason, consideration of the multi-use concept for either wet or dry detention structures is strongly encouraged. Some types of detention and retention structures are discussed.

Detention structures can be categorized as: dry basins, wet ponds, storage tanks, and multi-use storage areas such as rooftops, parking lots, roadway embankments, and other shallow holding areas. Design criteria of the various structural types are discussed in Article 12.6. A general discussion of the specific types of detention (dry) basins, (wet) ponds storage follows.

12.4.1 Detention (Dry) Basins

Dry basins are surface storage areas created by constructing a typical excavated or embankment basin. There is no normal pool level and a specific controlled release feature is included to control the rate of discharge. The detention flow control structure is usually a multi-stage device, and the retention flow control structure is usually a single-stage device.

Dry basins are the most widely used structural method of stormwater management. The soil permeability and water storage potential are not as important with dry basins as with wet ponds. Therefore, dry basins have the greatest potential for broad applications. They can be utilized in small developments because they can be designed and constructed as small structures. Dry basins are often less costly than wet ponds because they do not require extensive design and construction considerations. They can be designed for multi-use purposes such as recreation and parks.

12.4.2 Detention (Wet) Ponds

Wet ponds are permanent ponds where additional storage capacity is provided above the normal water level and special features for controlled release are included. Historically, wet ponds have proven extremely effective in abating increased runoff and channel erosion from urbanized areas. They are a major Soil Conservation Service land treatment practice.

Some problems often encountered with wet ponds are: site reservation (land requirements), permanent easements, complexity of design and construction, safety hazards, and maintenance problems. Because of large land requirements, and the necessity of maintaining a permanent pool of water, wet ponds have a broader application for instream control where large watershed areas are involved compared to their use as on-site facilities for small urban areas. However, the recreational and aesthetic benefits of permanent wet ponds are very often considerable and may be justifiable in certain on-site applications.

12.4.3 Tank Storage

Tank storage is an underground tank or chamber, either prefabricated or constructed in place, which has a special controlled release feature. This method is most applicable where land area is very valuable, such as in industrial and commercial areas. Construction cost and operation costs which may include pumps make this method relatively expensive and, therefore, undesirable in most cases. Storage trenches, a variation on basic tank storage are rock-filled underground storage tanks. The exception is that storage is provided within the void spaces between granular material, rather than in concrete structures.

12.4.4 Rooftop Storage

Rooftop storage is surface storage provided on flat rooftops designed with provisions for temporary ponding and with special roof-drain-controlled release features. Rooftop storage utilizes the built-in structural capability of rooftops to store certain amounts of rainfall.

Existing structures conforming to local building codes meet the requirement of being able to support a specified snow and live loading. This loading allowance may be utilized for stormwater storage without additional support. Modification of the roof drains to a controlled runoff device is usually all that is required for existing structures.

Rooftop storage can usually be incorporated into the design of new buildings. This method is probably one of the most convenient and economical to design and construct. It usually performs a retention function since the method is to hold the stored water for a relatively long time while draining gradually. For this reason, directing the drained water to lawn areas and infiltration trenches where it can percolate is possible.

The main disadvantages of rooftop storage are the inspection and maintenance requirements. Routine inspections are difficult when the installations are not readily accessible. Clogging of the roof drains make routine inspections a necessity. Another disadvantage of this method is the possibility of the roof drain flow control devices being removed after construction in the event the roof develops some leaks.

12.4.5 Parking Lot Storage

Parking lot storage is surface storage where shallow ponding is designed to flood specifically graded areas of the parking lot. Controlled release features are incorporated into the surface drainage system of the parking lot. Parking lot storage is a convenient multi-use structural control method where impervious parking lots are planned. Design features include small ponding areas with slotted controlled release structures and/or pipe-sizing reduction, and increased curb heights. This method can easily be incorporated into a site development at approximately the same cost as that of a conventional parking lot.

The major disadvantage is the inconvenience to users during the ponding function. This inconvenience can be minimized with proper design consideration. Clogging of the flow control device and icy conditions during cold weather are maintenance problems. Parking lot design and construction grades are critical factors. For these reasons, the functional effectiveness of parking lot storage is questionable. This method is intended to control the runoff directly from the parking area, and is usually not appropriate for storing large runoff volumes.

12.5 Other Control Methods

Stormwater control methods other than those listed above may be used only with the approval of the utility engineer. Such methods include, but are not limited to, the following:

12.5.1 Special Fill Impoundments

These are created where fills such as roads, parking lots, and other similar special use embankment fills are modified and designed to cause surface ponding and controlled release of stormwater runoff.

12.5.2 Infiltration Methods

Infiltration methods include dry wells, infiltration trenches, and storage trenches. These are generally holes drilled or excavated into the ground and then refilled with crushed stone or washed gravel. Runoff is collected in these facilities and infiltrates into the groundwater, thereby reducing the total volume of surface runoff which must be carried at the ground surface. Due to the limited applicability and potential operational problems related to infiltration methods, they are not generally recommended.

12.6 Evaluation and Selection of Alternative Detention Structures

It is not possible to state which detention facility is best in any given design situation. However, evaluation criteria are provided to assist the designer in the selection process. It is recommended that the features of each structure be examined with respect to the stages of facility development. The four basic stages of facility development are as follows:

1. Consideration of Alternatives

   a. Types of runoff control facilities
   b. Economic considerations
   c. Multi-use features
   d. Design complexity
   e. Effective life.

2. Site Selection and Design

   a. Land costs
   b. Site investigation requirements
   c. Soil limitations
   d. Accessibility to facility
   e. Aesthetic appear and siting convenience.

3. Construction

   a. Installation costs
   b. Complexity of construction
   c. Structural integrity
   d. Construction inspection.

4. Operation and Maintenance

   a. Operation costs
   b. Maintenance plans and cost
   c. Ownership and maintenance responsibility
   d. Maintenance inspection needs.

A 1972 survey of engineering firms (Practices in Detention of Urban Storm Water Runoff) revealed dry basins and wet ponds are prevalent over all other structural control methods. Detention of rainfall on parking lots ranked next, followed by detention of rainfall on rooftops.

12.7 Design Criteria for Detention Structures

The following criteria are recommended as standards of design. The criteria are not intended to provide a "cook book" method of design, but rather to place limits on design of structures so performance objectives can be achieved. Drawings of typical structures are included. Again, these drawings are not to be taken as the actual design configuration for all applications, but are included as a guide for design.

12.7.1 Detention (Dry) Basins, (Wet) Ponds

Typical design configurations of on-site detention basin are included in Exhibits X11-1, X11-2, and X11-3. Orifice pipe control is shown on Exhibit X11-2 where flow enters the basin when the upstream sewer system becomes surcharged.

1. Discharge Control Facilities

   The outlet structure shall be designed to minimize the transport of floating debris, oil, and grease through the detention facility. The design of the facility shall also include adequate provisions to minimize erosion in the vicinity of the inlet and outlet, and on the side slopes of surface facilities.

2. Detention Period

   A minimum of 50 percent of the total storage volume required to attenuate the peak discharge of the facility shall be recovered within a 24-hour time period. The remaining 50 percent shall be recovered within an additional 72-hour time period.

3. Surface Slope

   The bottom of a dry detention pond shall be at least 3 feet above the seasonal high water table. For wet ponds, the level of zero storage will be taken as the water surface elevation of the dead storage pool. The minimum bottom width for ponds and open drainage ways shall be 4 feet.

4. Stability Analysis

   Where berms constructed of fill are proposed, calculations supporting the stability of the fill berms are to be submitted by a Geotechnical Engineer.

5. Barriers to Access

   For fenced facilities, the maximum side slop inside the fence may be 3:1, but the maintenance berm, if required, must be a minimum of 12 feet wide all around the perimeter. Fenced facilities are discouraged.

6. Rights-of-Way and Easements

   Outfall ditches and channels shall have sufficient right-of-way for the facility plus an unobstructed maintenance berm on one or both sides. Vehicular access from a public road to the maintenance berm shall be provided. Detention facilities shall have sufficient easement to allow for the installation and maintenance of a maintenance berm. The Utility Engineer may require a maintenance berm all around the perimeter of the pond. If required, top widths of maintenance berms shall be 10 feet, and cross slopes shall be no steeper than 3/8-inch per foot.

7. Aesthetics

Areas adjacent to ponds shall be graded to preclude the entrance of stormwater, except at planned locations. Where detention areas are located on the project periphery, the developer may be required to provide additional landscaping or screening to adequately protect abutting properties. Grading should take into consideration ease of maintenance, such as mowing.

8. Combined Sewer Areas

Detention/retention ponds that discharge to combined sewers may only discharge the 10-year predevelopment flow to the combined sewer. Storage outflows greater than that rate must discharge to the downstream surface drainage system to ensure that the pond discharge does not overload the combined sewer.

12.7.2 Underground Storage Tanks

1. Materials

Underground storage tanks may be constructed in lieu of surface storage facilities. Storage tanks may be constructed of reinforced concrete either prefabricated, poured-in-place, or of corrugated steel. The structural design shall be in accordance with current design practice. A typical concrete storage tank is shown on Exhibit XII-4.

2. Pipe Size

The minimum size drain pipe shall be 12 inches.

3. Access

An access hatch or manway shall be provided for inspection and maintenance. All openings shall be properly secured to minimize safety hazards.

4. Capacity

The required storage volume, discharge release rate, and detention period shall be as determined for surface storage facilities. A flow control device such as a simple weir or orifice shall be included.

5. Overflow

Overflow provisions shall be included to accommodate the less frequent storms up to and including the 100-year storm runoff.

6. Draining

The storage tank shall include provisions for completely draining the tank. The minimum slope of the tank bottom shall be 0.50 percent.

Pumping facilities, where proposed, shall be designed in accordance with the requirements set forth for wet ponds.

12.7.3 Parking Lot Storage (Storage of Runoff in Depressions)

1. Maximum Water Depth

   The water depth in the storage area shall not exceed 7 inches.

2. Discharge Period

   The parking lot shall be completely drained in the minimum time permissible within the constraints of the runoff control standards.

3. Outlet Structure

   The outlet structure shall be located at the absolute lowest point or points in the storage area.

4. Storage Location

   The storage area should be located in the more remote, least used portion of the parking facility. It is desirable that the ponded area at the maximum water depth be at least 50 feet distant from building structures.

5. Slope

   The maximum surface slope of the storage area shall be 4 percent and the recommended minimum slope is 1 percent.

6. Overflow

   Provisions shall be included for overflow of runoff above the design storm, such as uncontrolled drainage structures near the ponding area or surface outlets. When the latter is provided, special consideration shall be given to control of runoff velocities. Damage from soil erosion shall be minimized.

7. Storage Volume

   The required storage volume shall be determined in accordance with the same design criteria as for dry basins.

8. Alternate Designs

   Consideration shall be given to the use of semi-paved/semi-grassed areas for remote parking areas. These can include infiltration methods and thereby reduce total runoff.

   The use of pervious pavement materials is possible in certain applications, although widespread use is discouraged unless appropriate means to prevent clogging the pervious areas are provided.

12.7.4 Summary of Design Criteria

A summary of the basic design criteria parameters for detention basins and parking lot storage is given in Table 12-1. They are intended to establish general limits of design and are not all inclusive. Hopefully, these criteria will be refined and further expanded to suit specific applications. In the final analysis, engineering judgment and actual experience are important factors of any design. The detailed criteria should be based on a thorough investigation of site conditions and design limitations applied to all stages of facility development. The Soil Conservation Service Engineering Field Manual is a valuable source for typical design of embankments, ponds, spillways, and outlet structures and should be consulted for detailed information not specified herein.

Table 12-1

Summary of Design Criteria For On-Site Detention Structures

Control Method	Maximum Side Slope	Maximum/ Minimum Water Depth	Top Width of Embankment	Permanent Easement Width
Detention (Dry) Basin	4:1	Maximum 4-6 feet	8 feet	10 feet
Detention (Wet) Pond	4:1	Minimum Pool 10 feet	10 feet	10 feet
Parking Lot Storage	--	Maximum 7 inches	--	--

12.8 Conduit Structures

Conduit structures include storm drainage pipes, open ditches (natural and man-made), natural streams, and channelized streams. They serve primarily as a conveyance system to transport stormwater from one point to another. Conduits are used to direct flows to a particular point in the stormwater management system and may be used to bypass flows. Flow bypassing is a viable method in certain applications and in some instances is unavoidable. When stormwater flows are bypassed, the impact at the point of reentry into the drainage system should be investigated. Details of channel design and storm sewer design are discussed in Chapter 7 and 9 of this manual.

Conduit storage as a structural control method is underground storage provided by enlarged storm drain pipes, or storm inlet structures that include storage capacity and a flow control device. It compares closely with tank storage except for construction requirements. Conduit storage is generally applied to urban high land value areas where storage basins and ponds are undesirable, such as small industrial and commercial areas. Storm pipe manholes provide easy access for maintenance purposes. This method is generally expensive and impractical for runoff control of large areas or for storm runoff from high intensity storms where emergency overflow must be provided.

Conduit structures (oversized or enlarged storm sewer pipes) shall be designed in accordance with the criteria specified for underground storage tanks, except as modified herein.

Special consideration shall be given to the structural strength and load-carrying capacity of the conduit as well as the bearing capacity of the soil. Perforated pipe may be used where soil conditions are favorable for infiltration methods.

12.9 Methods for Determining Storage Capacity

This section examines several methods for estimating the volume requirements of different types of stormwater storage facilities. The criteria used to choose the appropriate runoff prediction method (Chapter 6) may also be used to determine the method of estimating storage requirements except the storage equation can be used on drainage areas less than 10 acres. Generally, if the rational or the Peak Discharge Method of runoff calculation is appropriate for a given situation, it may be assumed that Graphical Flow Routing Methods are similarly applicable. If the Tabular Method or Unit Hydrograph Method are used for estimating runoff then the Storage Indication Flow Routing Method is most accurate for determining storage capacity.

A flow control device is needed for each method used in estimating the required storage volume for the stormwater storage facility. Flow control devices can be either fixed or variable. For purposes of stormwater management, the most reliable is the fixed type, either an orifice, a simple weir, or a combination of the two. The theoretical flow characteristics for these types of devices are as follows:

Circular orifice, (submerged discharge)[1] $Q = Ca\sqrt{2gh}$ where: Q is the orifice discharge in cfs, C is the coefficient of discharge, a is the orifice cross-sectional area in square feet, g is the gravitational acceleration consistent equal to 32.2 ft/sec^2, and h is the height of water surface over center of orifice.

When the storage water surface falls below the top of the orifice opening, the flow characteristics are equivalent to those of a weir.

Weir discharge[2] $Q = CLH^{3/2}$ where: Q is the weir discharge in cfs, C is the weir coefficient of discharge, L is the effective length of weir in feet equal to $L^1 - 0.1\,NH$, L^1 is the measured length of crest in feet, N is the number of end contractions, usually equal to 2, and H is the hydraulic head above weir crest in feet.

Values for the constants to be used in the above equations for different configurations of weir may be found in the reference given. Orifice pipe and weir discharge curves are shown on Exhibits XII-6 and XII-7, respectively.

Multi-stage outlet control structures are possible by including both the orifice and weir in one structure. A common configuration used in detention structures is a pipe orifice as a primary discharge near the bottom of a reservoir with a simple weir located a few feet above the orifice or near the

(1) Handbook of Hydraulics, Fifth Edition, King & Brater, pp. 4-3.

(2) Handbook of Hydraulics, Fifth Edition, King & Brater, pp. 5-3 et. seq.

top of the structure as a secondary discharge. When both control devices are used together, the resulting discharge is the sum of the individual discharges for a given storage elevation.

12.9.1 Storage Equation (Preferred Method for Determining the Storage Requirement from a Drainage Area of Less Than 10 Acres)

The storage equation method presented here is a simplified method for computing the required storage for developments of less than 10 acres. The storage equation says the required storage volume equals the 25-year post-development peak discharge with a rainfall intensity of 2.42 inches per hour minus the 10-year predevelopment peak discharge with a rainfall intensity of 2.03 inches per hour times 60 minutes times 1.15 (a 15 percent safety factor). The procedure is as follows:

Step 1. Determine the predevelopment and post-development peak discharges for the 2-, 10-, 25-, and 100-year frequency event using the Rational Method.

Step 2. Determine the required storage volume by the storage equation. The required storage for each frequency event shall equal the frequency event's post-development peak discharge divided by the 100-year post-development peak discharge times the required storage volume as determined by the storage equation.

Step 3. Size the outlet facility so that for each frequency event the outlet discharge does not exceed the predevelopment discharge.

Step 4. Design the detention/retention structure according to the design criteria given in this chapter.

12.9.2 Example - Storage Equation

Determine the required storage volume for a 5-acre watershed and a pre-development runoff coefficient equal to 0.3 and a post-development runoff coefficient equal to 0.65. The time of concentration (TC) has been calculated to be 15 minutes for predevelopment conditions and 8.0 minutes for post-development conditions. Since for post-development, the TC of 8.0 is less than TC of the minimum value, use 10 minutes. Because the area is less than 10 acres, the storage equation can be used. For this example, the 2-, 10-, 25-, and 100-year frequency predevelopment discharges are not to be calculated as they are not required for the storage volume although these discharges are needed to size the outlet facility.

Step 1. Use Exhibit I-2 with TC post = 10 and read.

Post-development rainfall intensity I_2 = 3.90, I_{10} = 5.45, I_{25} = 6.36, and I_{100} = 7.81

Calculate discharge Q = CIA

Post-development

2-year event: Q_2post = 0.65 x 3.90 x 5 = 12.7 cubic feet per second (cfs);

10-year event: Q_{10}post = 0.65 x 5.45 x 5 = 17.7 cfs;

25-year event: Q_{25}post = 0.65 x 6.36 x 5 = 20.7 cfs;

100-year event: Q_{100}post = 0.65 x 7.81 x 5 = 25.4 cfs.

Step 2. Calculate storage volume by the storage equation = (0.65 x 2.42 x 5 - 0.3 x 2.03 x 5) x 60 x 1.15 = (7.86 - 3.04) x 60 x 1.15 x 60 sec/min = 19,950 cubic feet (cf).

Required storage for each frequency event

2-year event (12.7/25.4) x 19.950 = 9,970 cf;

10-year event (17.7/25.4) x 19.950 = 13,900 cf;

25-year event (20.7/25.4) x 19.950 = 16,260 cf;

100-year event (25.4/25.4) x 19.950 = 19.950 cf.

The 100-year storage could be developed as a 1/5-acre detention basin with a maximum storage depth of 3 feet, assuming 4:1 side slopes or as an underground pipe, 7 feet diameter and 520 feet in length.

12.9.3 Graphical Flow Routing (Preferred Method for Determining the Storage Requirement from a Drainage Area Between 10 and 200 Acres)

The graphical method presented herein was developed by the Soil Conservation Service and is found in Chapter 6 of the Urban Hydrology for Small Watersheds, Technical Release No. 55. It is based on average storage and routing effects using the storage-indication method of routing. The graphs relate inflow (Q_i) and release rate (Q_o) to storage requirements for single or multiple stage outlet structures. Emergency spillway flow (overflow) is not considered in this method.

Use of this graph will result in rough approximation since this method is based on several general assumptions and the procedure may significantly overestimate the required storage requirements. The results of the Graphical Flow Routing method should be interpreted accordingly.

For any application where the graphical method is not appropriate, a more accurate flow routing method, such as the storage-indication method, is needed to determine storage requirements.

The following summarizes general procedures for the determination of storage capacity using the graphical flow routing method.

Step 1. Determine the post-development volume of runoff (V_r), the peak inflow discharge (Q_i), and the peak outflow discharge (Q_o) for the 2-, 10-, 25-, and 100-year frequency events using the graphical peak discharge method (see Article 6.4.1.1). Normally the peak inflow discharge equals the post-development peak discharge and the peak outflow discharge equals the predevelopment peak discharge.

Step 2. Calculate the ratio of peak outflow discharge to the peak inflow discharge (Q_o/Q_i) for the 2-, 10-, 25-, and 100-year frequency events.

Step 3. Use Exhibit XII-5 and determine the ratio of storage volume to the post-development volume of runoff (V_s/V_r) for each event in Step 2.

Step 4. Calculate the required storage volume (V_s) $V_s = V_r \times (V_s/V_r)$. The required storage volume (V_s) is expressed in the same units as the post-development volume of runoff (V_r). If V_r is expressed in inches of runoff, the conversion to acre feet is: multiply V_r times 53.33 (the conversion factor from inches per square mile to acre feet) times drainage area in square miles. To convert acre feet to cubic feet, multiply V_r expressed in acre feet times 43,560 (the conversion factor from acre feet to cubic feet).

Step 5. Check that for each frequency event the outflow structure releases the desired rate of outflow at the corresponding storage elevation. This can be done by constructing a stage-discharge and stage-storage relationship for the outlet structure. To be a satisfactory design, the stage of each frequency event should result in a release rate that is equal to or less than the desired rate of outflow which would result in the actual storage being equal to or greater than the required storage.

Step 6. Design the detention/retention basin according to the design criteria given in this chapter.

12.9.4 Example - Graphical Flow Routing

Determine the required storage volume for the 100-year frequency event using the graphical flow routing method for a 34 acre forested watershed that is being developed. The runoff curve numbers for the development are 55 for pre-development and 75 for post-development. The times of concentrations for the development are 0.60 hours for predevelopment and 0.42 hours for post-development. Approximately 40 percent of the watershed will be impervious. One acre, or 3 percent of the drainage area, is made of ponding or swampy areas spread throughout the watershed. The average watershed slope is estimated to be 7 percent. Exhibit XII-8 is a completed graphical flow routing worksheet for this example. Exhibit XII-16 is a graphical flow routing worksheet that references how values are obtained for each column.

Step 1. Calculate for the 100-year frequency event the predevelopment peak (outflow) discharge (Q_o), the post-development peak (inflow) discharge (Q_i), and the post-development volume of runoff (V_r). Refer to Article 6.4.1 for general procedures for calculations and Article 6.4.1.1 for the post-development calculations. Q_o = 18 cfs, Q_i = 70 cfs, and V_r = 3.08 inches.

Step 2. Calculate for the 100-year frequency event the ratio of Q_o/Q_i = 18/70 = 0.257.

Step 3. Use Exhibit XII-5, with Q_o/Q_i = 0.257 for the 100-year frequency events and read V_s/V_r = 0.405.

Step 4. Calculate for the 100-year frequency event $V_s = V_r \times (V_s/V_r) = 3.08 \times (0.405) = 1.25$ inches or $1.25 \times 53.3 \times 34/640 = 3.5$ acre-feet.

Step 5. The outlet structure needs to be sized so that for the calculated storage (V_s) the outflow does not exceed the desired outflow. See Article 12.9.2 on how to size an outlet structure.

Step 6. The detention basin would now be designed to meet the other design criteria given in Article 12.7.1.

12.9.5 Flood Routing

Flow routing problems are solved by using the continuity equation. The continuity equation is based on the concept of conservation of mass. For a given time interval the volume of inflow minus the volume of outflow equals the change in volume of storage. The continuity equation expressed in this simplest form is: $\Delta t\ (\bar{I} - \bar{O}) = S$ where: Δt is the time interval, $\bar{I}$ is the average rate of inflow during the time interval, $\bar{O}$ is the average rate of outflow during the time interval, and S is the change in volume of storage during the time interval.

The rate of inflow is determined by the inflow hydrograph. The rate of discharge is determined by the elevation versus discharge characteristics of the reservoir outlet structure. The change in storage volume is represented by the elevation versus storage curve of a reservoir. For routing studies, the inflow hydrograph is determined once a design storm and watershed parameters have been established. Also, the elevation-storage curve is determined by the reservoir site. Therefore, for a particular site, design storm, and drainage area, the only variable in the continuity equation is the elevation-discharge curve. The elevation-discharge curve is dependent on the hydraulic characteristics of the flow control device (outlet structure).

12.9.6 Storage-Indication Flow Routing

Several techniques have been developed for solving flow routing problems. One method which is relatively simple and gives accurate results for many applications is the Storage-Indication Method. This method is recommended for use when the graphical storage method is not applicable.

For sufficiently small time intervals, (Δt) the continuity equation can be expressed: $(I_1 + I_2)/2 - (O_1 + O_2)/2 = (S_2 - S_1)/\Delta t$. Subscripts "1" and "2" represent the beginning and end of the time interval. Usually, the complete inflow hydrograph is known or can be determined, as well as the initial storage and outflow values. Therefore, the remaining unknown values are the outflow at the end of the time interval (O_2) and the storage volume (S_2). The above equation is rearranged as follows: $(I_1 + I_2)/2 + S_1/\Delta t - O_1/2 = S_2/\Delta t + O_2/2$. The left side of this equation can be determined for successive increments of time and, therefore, the right side of the equation can be quantified. Storage-indication curves are developed to determine outflow (O_2) for known values of $S_2/\Delta t + O_2/2$.

The general procedures for flow routing by the Storage-Indication Method is as follows:

Step 1. Develop the inflow hydrograph. Hydrographs can be developed using the SCS TR20 Method, by synthetic hydrograph procedures or by actual stream measurement.

Step 2. Estimate storage requirements, make a preliminary design of a storage facility and develop an elevation-storage curve for the storage site. Data may be obtained from field survey and topographic maps. (The graphical flow routing method is helpful in preliminary design of the storage facility.)

Step 3. Develop an elevation-discharge for the reservoir discharge control structure. This is based on the hydraulic characteristics of the flow control device. (The size of the flow control device is based on the design discharge rate and the design high water level.)

Step 4. Select the time interval (Δt). Trial routing interval (Δt) is chosen based on the following considerations:

a. Shorter Δt increases the labor involved in computation but can detect abrupt changes in the inflow rate of discharge.

b. Longer Δt saves computation time, but may lose the inflow discharge information stated above.

c. Δt may be determined by examining the characteristics of the inflow hydrograph. It may be necessary to use two different time intervals; one time interval for small changes in inflow, and one for abrupt changes.

The value of Δt may need to be decreased if the storage-indication curve developed in the following step exceeds the equal value line represented by: $O_2 = S_2/\Delta t + O_2/2$.

Step 5. Develop the storage-indication curve by plotting O_2 versus $S_2/\Delta t + O_2/2$. Check the choice of Δt as explained in the preceding step. The suggested format for storage-indication computation and operation tables is included to facilitate calculations of this and subsequent steps.

Step 6. Tabulate inflow values for each routing interval (Δt) from the design inflow hydrograph, and calculate the average inflow $\bar{I}$ by averaging successive values. Record on the operations table worksheet. $\bar{I} = (I_1 + I_2)/2$

Step 7. Perform the routing by using the operations table worksheet and the storage-indication curve. The calculations start when the inflow equals zero, and all the remaining values across the operational table worksheet are zero.

For computational purposes, the first calculated tabular time of inflow is the start of the routing. At this time it is assumed that there is no storage and no outflow occurring. In actuality, some outflow does occur prior to this time, but the value is too small to warrant its calculation. Where a high degree of accuracy is required, computer analysis of flow routing through structures is recommended.

The calculation, after starting with inflow and storage equaling zero, follows the solution of the rearranged continuity equation for the next time interval. The rearranged continuity equation can be written as: $(I_1 + I_2)/2 + S_1/\Delta t + O_1/2 - O_1 = S_2/\Delta t + O_2/2$ by substitution of $O_1/2 - O_1$ for $- O_1/2$ which are equal.

From this point on, average inflow $\overline{I}$ is determined from Step 6, and values for $S_1/\Delta t + O_1/2$ and O_1 are taken from proceeding values of $S_2/\Delta t + O_2/2$ and O_2 of the preceding routing interval.

Referring to the operations table, the value of $S_2/\Delta t + O_2/2$ is obtained by summing average inflow column 5 and $S_1/\Delta t + Q_1/2$ column 6 and subtracting O_1 column 7. The outflow value (O_2) is determined using the developed storage-indication curve (Step 5).

Step 8. Develop the outflow hydrograph by plotting O_2 versus time from the operations table.

Step 9. Graphically determine the required storage volume which equals the area between the inflow and outflow hydrographs.

Step 10. Check that the maximum outflow does not exceed the maximum design release rate.

The same principles and procedures of flow routing by the storage-indication method can be applied to channel routing as well as reservoir routing. The effects of channel storage can be determined, if necessary, for a particular design situation. As with reservoir routing, the stage-storage and stage-discharge relationships must be determined for the channel in question.

As the preceding flow routing example calculations indicate, manual calculations of the storage-indication method can become quite lengthy. Computer analysis can minimize computation time and is recommended whenever there are several storage facilities and/or channel reaches to be analyzed within a watershed.

12.9.7 Example - Storage-Indication Flow Routing

Determine the required storage volume for a detention basin to be placed on a 210 acre (0.328 square mile) drainage area using the Storage-Indication Flow Routing Method. The predevelopment runoff curve number is 55 and the post-development runoff curve number is 75. The time of concentration for the pre and post-development conditions is 1.05 hours and 0.85 hours, respectively. Travel time equals zero. For this example only the 100-year frequency event will be presented, although the other frequency events would also need to be analyzed.

Exhibit XII-12 is a completed storage-indication operation table worksheet for this example. Exhibit XII-15 is a storage-indication operation table worksheet that references how values are obtained for each column.

Step 1. Develop the predevelopment and post-development inflow by hydrographs by the SCS TR20 Method. Article 6.4.3 illustrates the general procedure for developing the inflow hydrographs, and the detailed calculations would follow these procedures. The peak predevelopment discharge is 122 cfs and is the maximum release rate. The post-development inflow hydrograph is shown on Exhibit XII-9 and the peak value is 364 cfs.

Step 2. A storage facility having vertical side slopes and a surface area of 4.2 acres will be used with the maximum storage depth of 6 feet. The elevation-storage curve for this facility is shown on Figure 12-1.

Step 3. A multi-stage outlet control structure will be used utilizing a 27 inch orifice pipe as the first (low flow) stage and a 10 foot weir as the second stage with its crest 4 feet above the invert of the outlet pipe. The elevation-discharge curve is shown on Figure 12-2. Article 12.9.2 shows calculation in developing an elevation-discharge curve for an outlet structure.

Step 4. The selected Δt is 15 minutes. Record the hydrograph time by each routing interval Δt, and the routing interval t time.

Step 5. Using Figures 12-1 and 12-2, the storage-indication curve is developed and plotted on Exhibit XII-11. The computations are shown on Exhibit XII-10, and the storage-indication curve is a plot of columns 2 and 6 from the computation table. Exhibit XII-14 is a storage-indication operation table worksheet that references how values are obtained for each column.

As seen on Exhibit XII-11, the developed storage-indication curve is less than the equal value line so the choice of Δt should be alright.

Step 6. Use Exhibit XII-9 and read for each Δt the post-development inflow value. Calculate the average inflow $\overline{I}$ values for each routing interval. For hydrograph time 11.25 $\overline{I}$ = (beginning inflow I_1 + ending inflow I_2)/2 = $(I_1 + I_2)/2$ (14 + 20)/2 = 17.0.

Step 7. The flow routing is performed as shown in the operations table worksheet, Exhibit XII-12. The following Table 12-2, a portion of the storage-indication operations worksheet, is included to indicate the routing operation.

Table 12-2

Partial Operations Table

(3) Time	(4) Inflow	(5) $\overline{I}$	(6) $\frac{S_1}{t} + \frac{O_1}{2}$	(7) O_1	(8) $\frac{S_2}{t} + \frac{O_2}{2}$	(9) O_2
0	14.0	--	0	0	0	0
15	20[(a)]	17.0[(b)]	0	0	17.0[(c)]	0.2[(d)]
30	27	23.5	17.0	0.2	40.3	0.8

(a) Step 6 above.
(b) Step 6 above.
(c) Columns 5 + 6 - Column 7.
(d) From Storage-Indication curve (Exhibit X-11).

Step 8. The storage outflow, O_2 (Column 9 of Exhibit XII-12) is plotted versus Hydrograph Time, (Column 1 of Exhibit XII-12) to determine the configuration of the outflow hydrograph as shown on Exhibit XII-13.

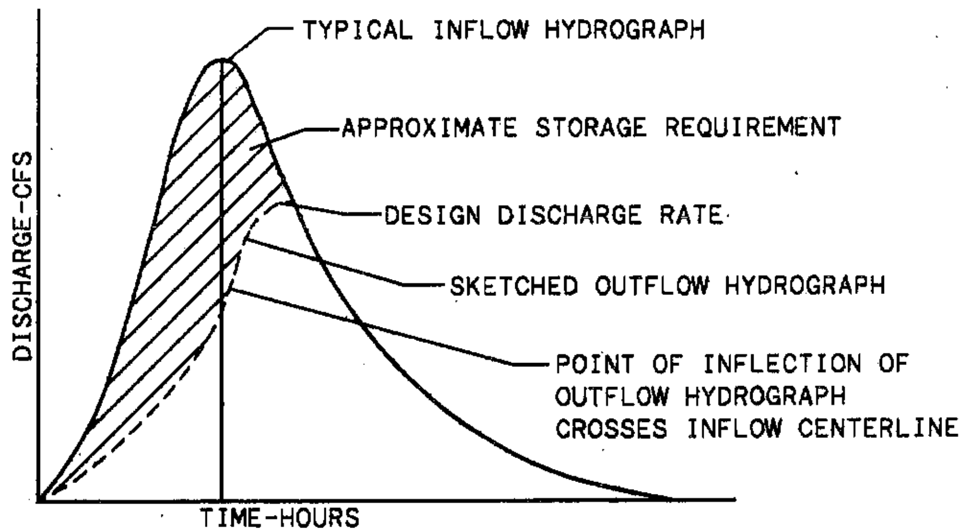
Exhibit XII-13 includes the inflow hydrograph and the storage volume determination.

Step 9. Using Exhibit XII-13, the area between the inflow and outflow hydrographs equals 305 cfs-hours. Thus, the required storage volume equals approximately 305 cfs-hours or, 25.2 acre-feet.

Step 10. The maximum outflow rate (O_2) is 115.4 cfs (Exhibit XII-13) which is less than the maximum design release rate of 122 cfs. Thus, the design is adequate for the 100-year event.

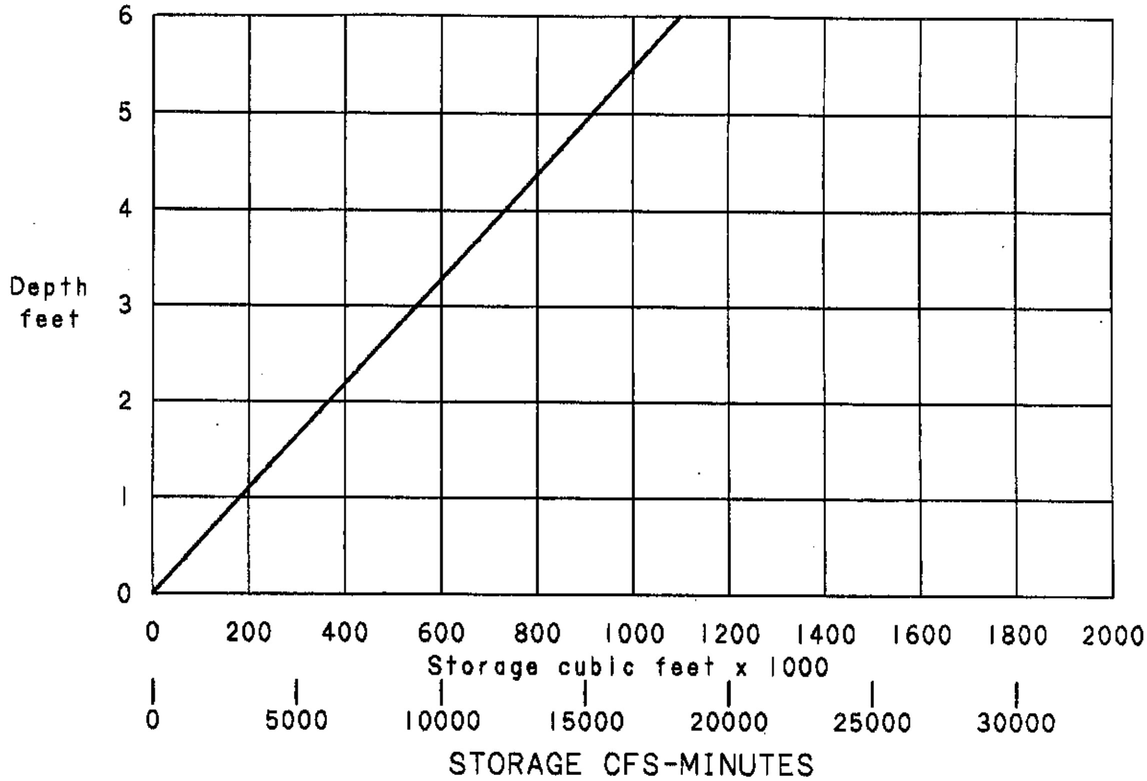
12.9.8 Graphical Approximations of Storage Requirements

Preliminary determinations of storage volume may be found using various graphical methods. One such method is a graphical version of the Mass-Curve Method of flow routing through structures described in Chapter 17 of the National Engineering Handbook, Section 4, Hydrology. Another approximate method is by sketching the outflow hydrograph on a graph together with the inflow hydrograph similar to the hydrographs shown on Exhibit XI-13. The design discharge rate (peak of outflow hydrograph) is plotted on the falling limb of the inflow hydrograph, the point of inflection on the rising limb of the outflow hydrograph crosses the centerline of the inflow hydrograph, and the beginning point (zero flow) is the same for both hydrographs. The subtended area is the approximate storage volume of the proposed facility. The procedure is illustrated graphically on Figure 12-3.



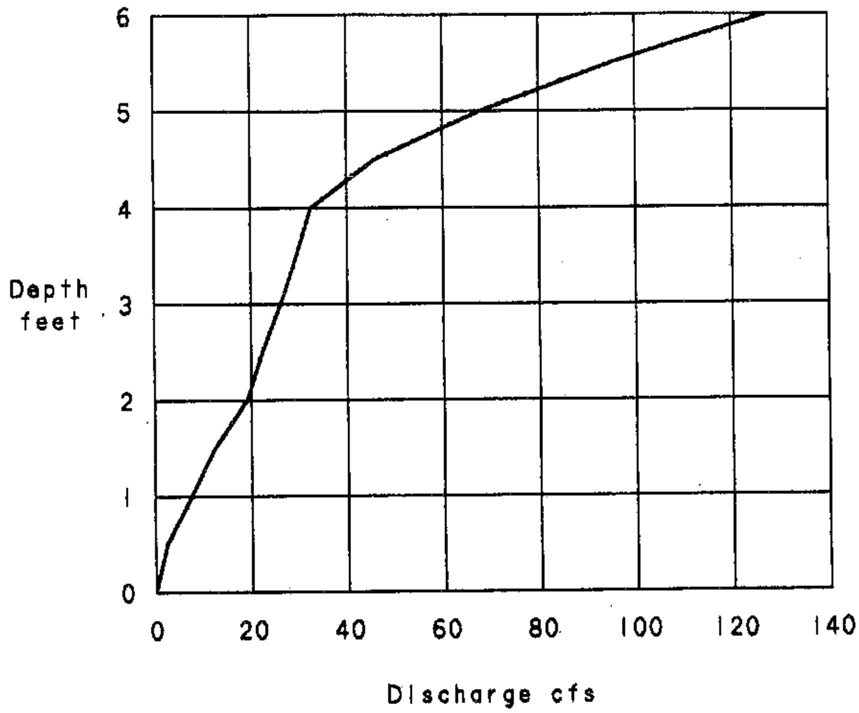
GRAPHICAL STORAGE VOLUME DETERMINATION BY SKETCHING OUTFLOW HYDROGRAPH

FIGURE 12-3



STORAGE ELEVATION CURVE
ARTICLE 12.9.7 EXAMPLE

FIGURE 12-1

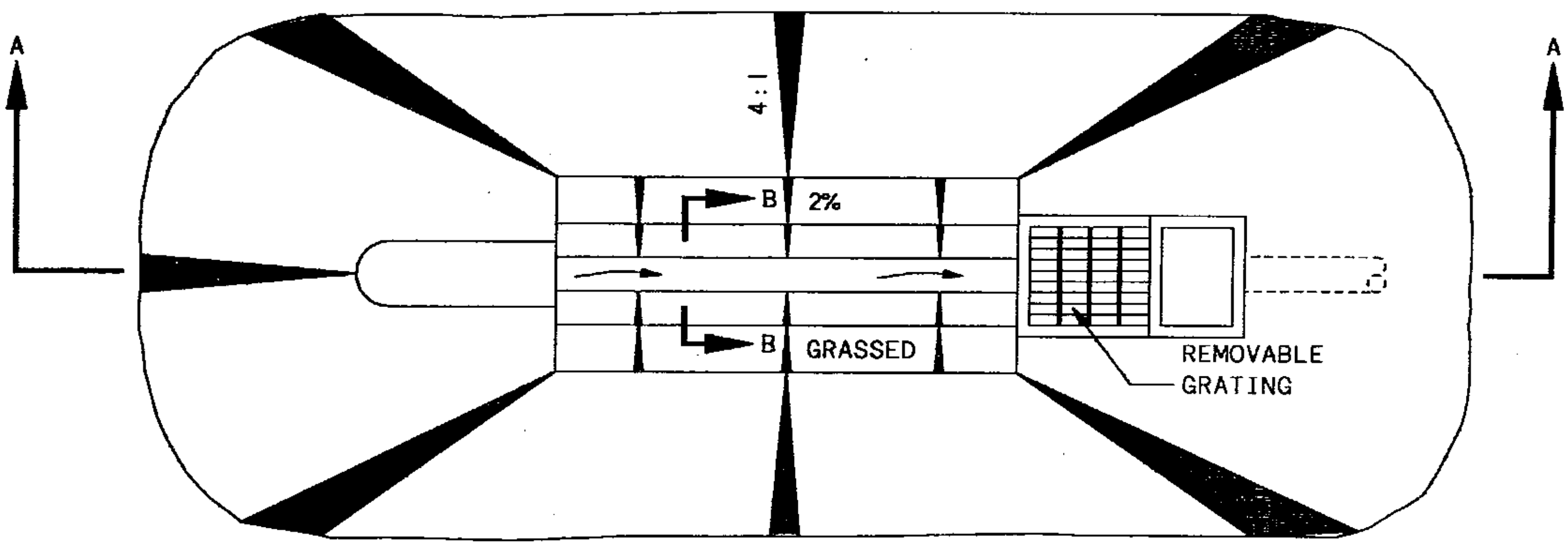


Depth feet	Discharge cfs
0	0
0.5	2.4
1.0	7.5
1.5	12.4
2.0	19.1
2.5	22.4
3.0	26.2
3.5	29.5
4.0	32.5
4.5	45.8
5.0	67.7
5.5	95.1
6.0	127.2

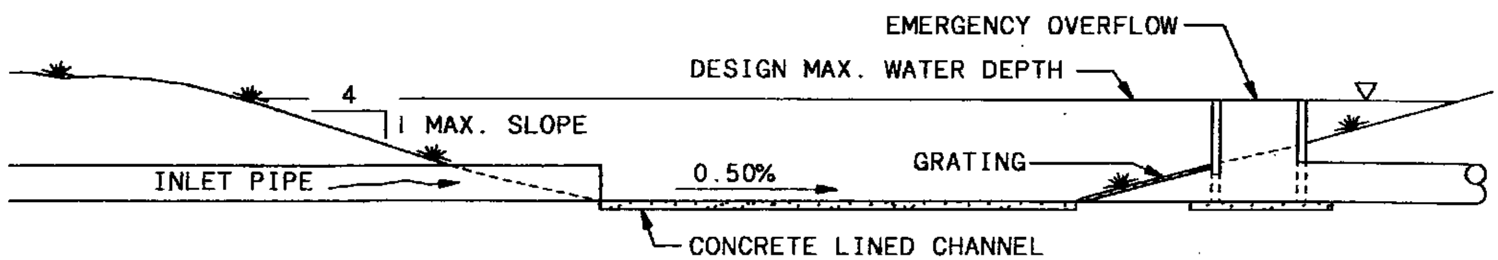
ELEVATION DISCHARGE CURVE
ARTICLE 12.9.7 EXAMPLE

FIGURE 12-2

TYPICAL DRY BASIN



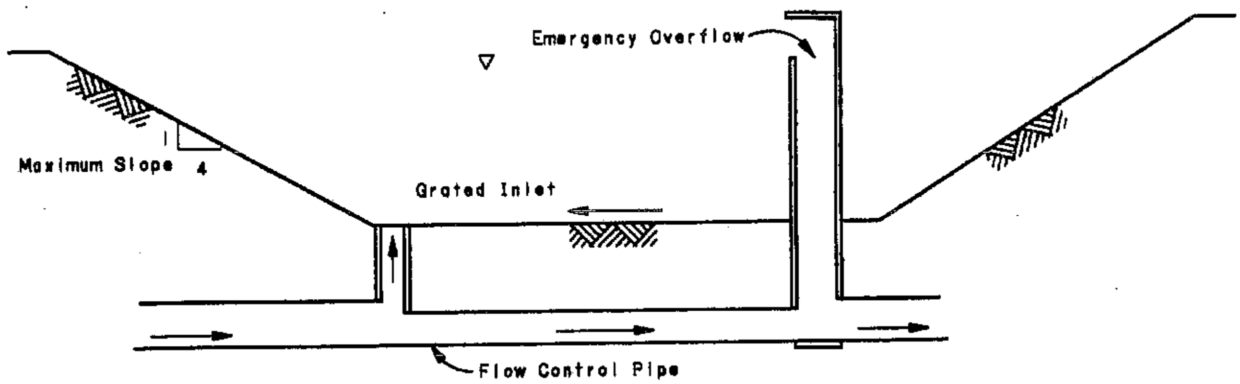
PLAN



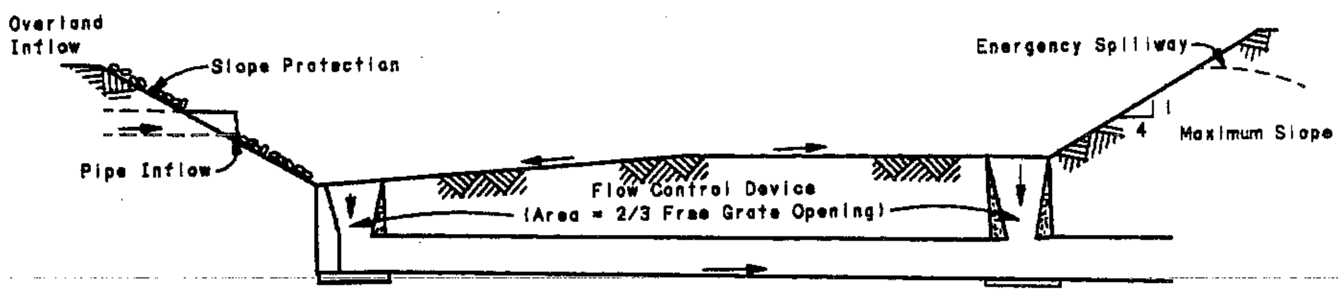
SECTION A-A

SECTION B-B

EXHIBIT XII-I



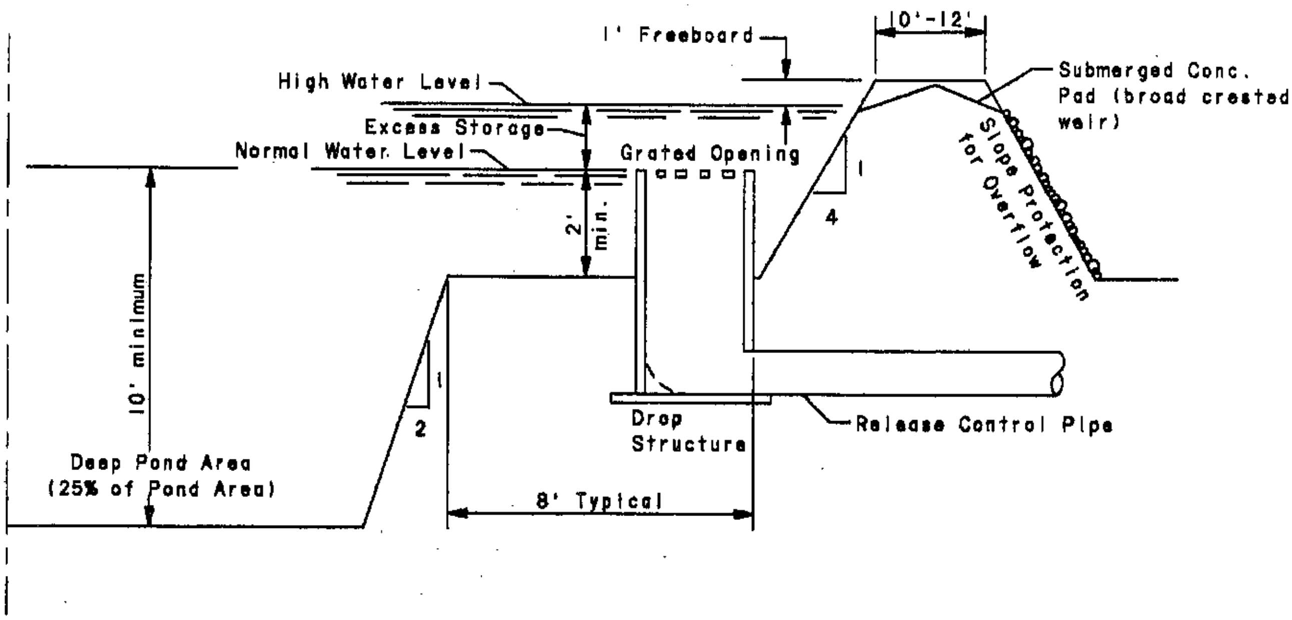
ORIFICE PIPE CONTROL



CONTROLED RATE INLETS

TYPICAL DRY BASIN
ALTERNATE DESIGN PROFILES

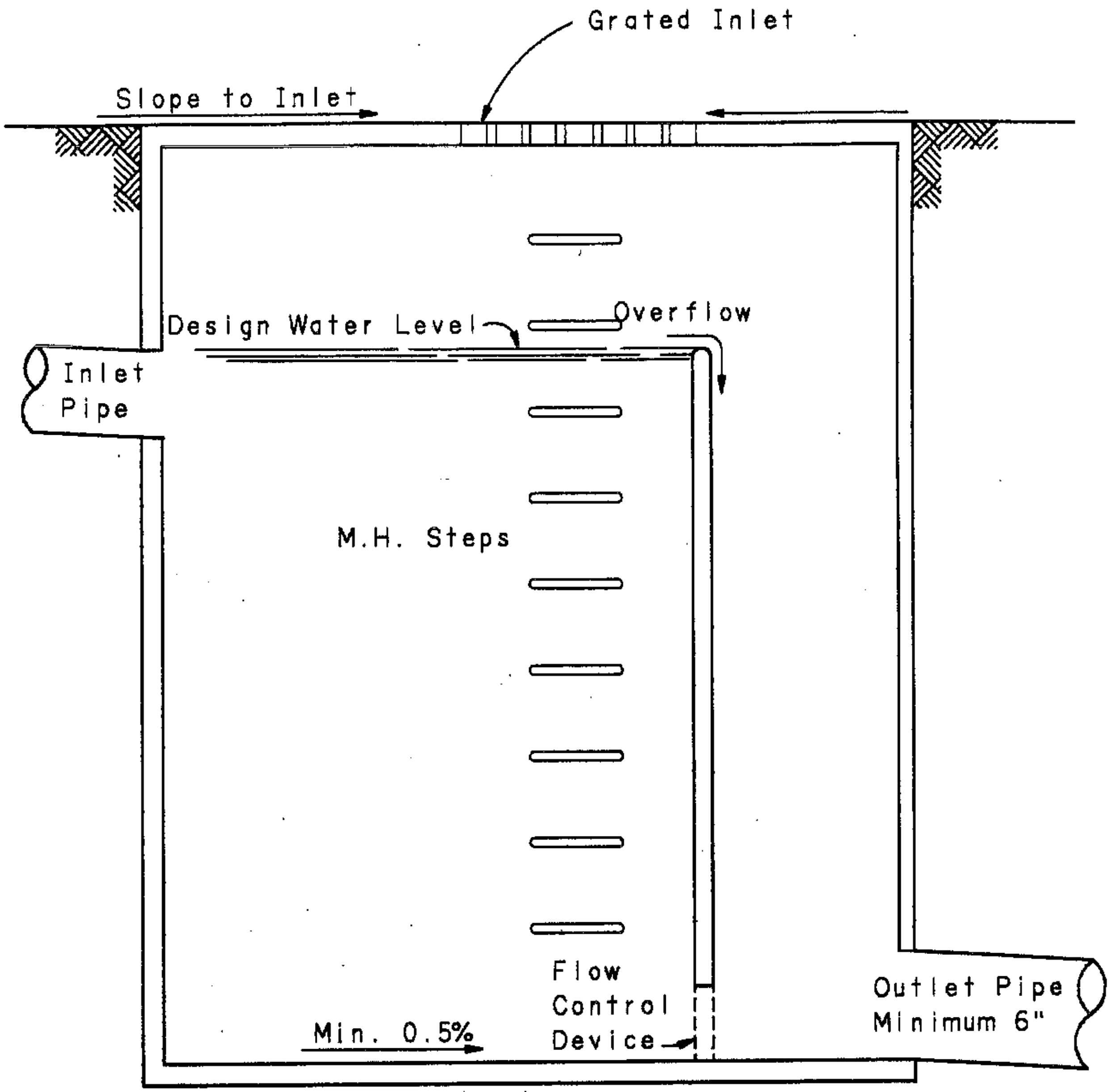
EXHIBIT XII-2



HALF PROFILE

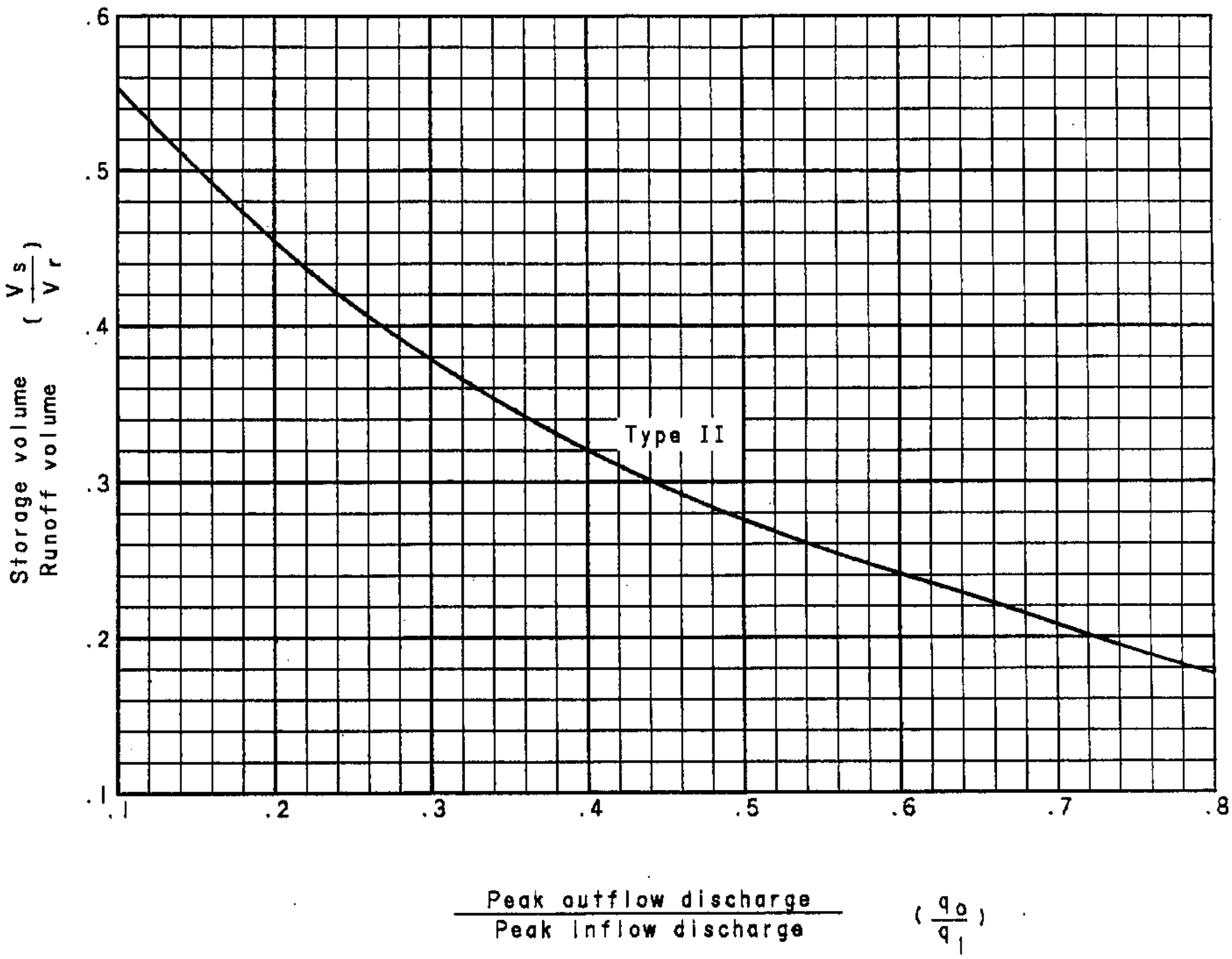
TYPICAL WET POND

EXHIBIT XII-3



TYPICAL STORAGE TANK

EXHIBIT XII-4



APPROXIMATE DETENTION BASIN ROUTING

EXHIBIT XII-5

ORIFICE PIPE DISCHARGE CURVES

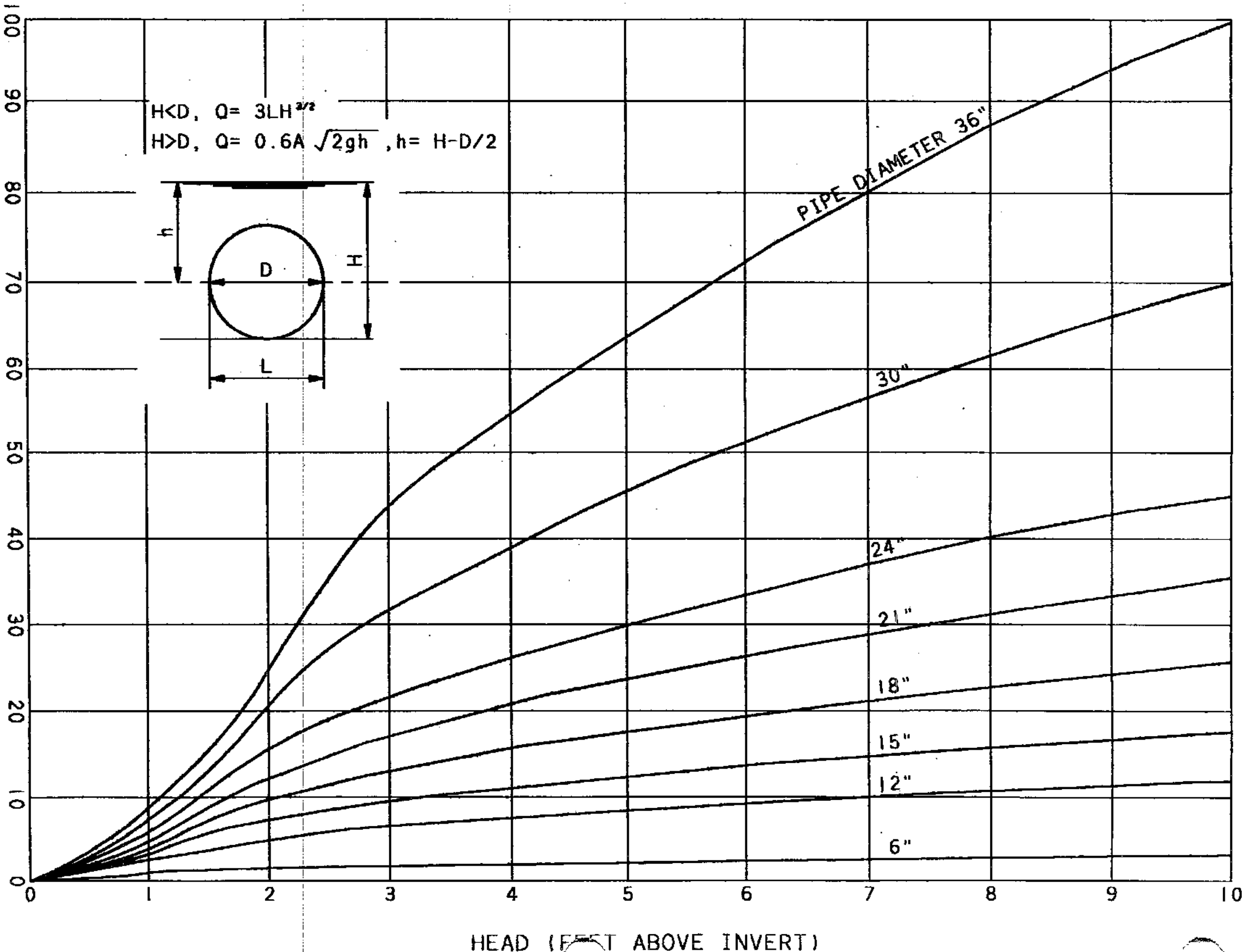


EXHIBIT XII-6

WEIR DISCHARGE CURVES

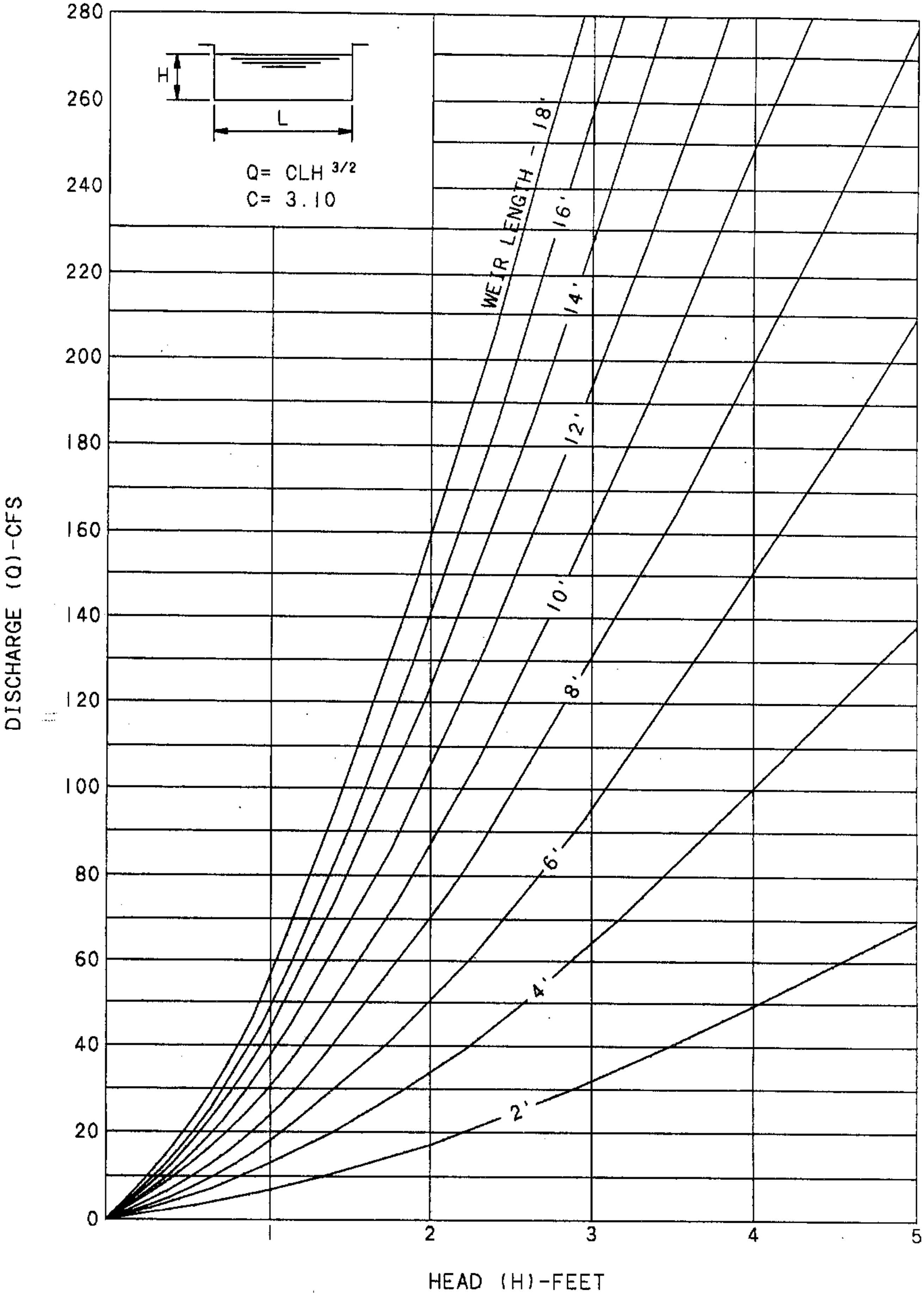
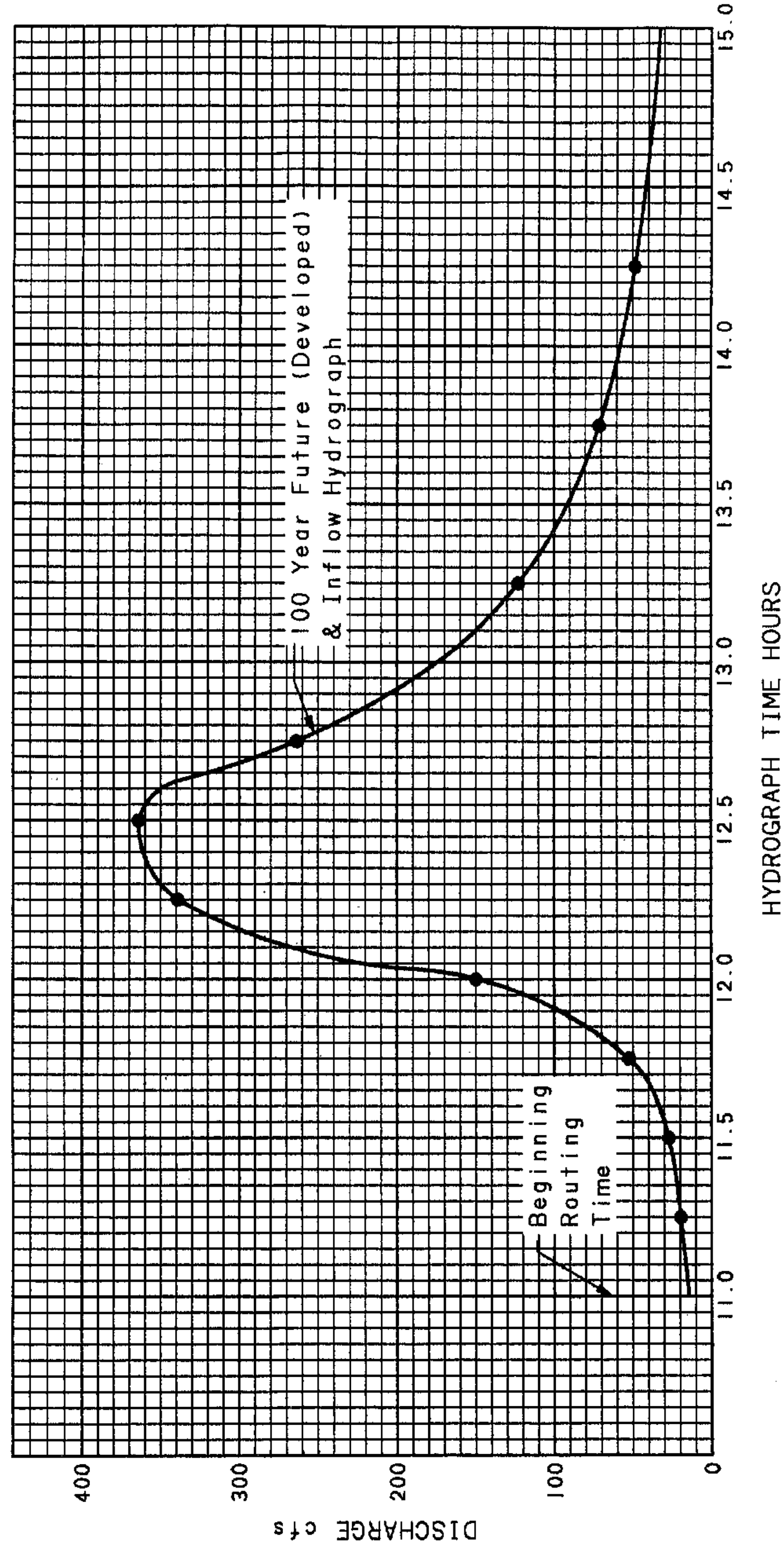


EXHIBIT XII-7

GRAPHICAL FLOW ROUTING COMPUTATIONS
ARTICLE 12.9.4 EXAMPLE

1 STORM FREQUENCY	PRE-DEVELOPMENT 2 PEAK DISCHARGE Qo	POST-DEVELOPMENT 3 PEAK DISCHARGE Qi	POST-DEVELOPMENT 4 VOLUME RUNOFF Vri	5 RELEASE RATE Qo	DISCHARGE RATIO 6 Qo/Qi	VOLUME RATIO 7 Vs/Vri	REQUIRED STORABLE VOLUME EXHIBIT XII-5 8 Vs	REQUIRED STORABLE VOLUME EXHIBIT XII-5 9 Vs
YEARS	CFS	CFS	INCHES	CSM			INCHES	ACRE FT
100	26	129	3.08	26	0.202	0.45	1.48	1485

EXHIBIT XII-8



INFLOW HYDROGRAPH
ARTICLE 12.9.7 EXAMPLE

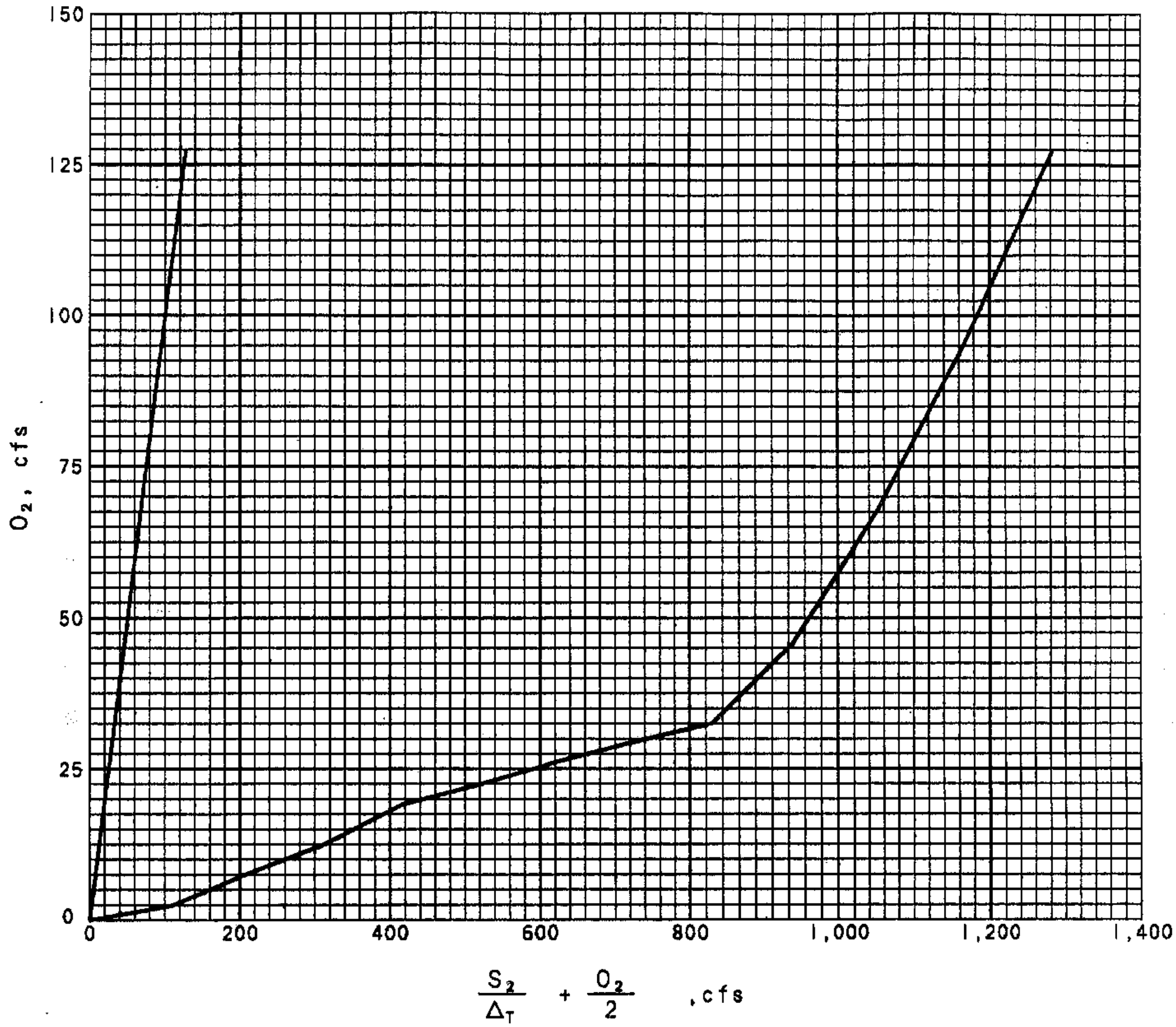
EXHIBIT XII-9

STORAGE-INDICATION COMPUTATION TABLE
ARTICLE 12.9.7 EXAMPLE

PROJECT ____________ DESIGNER ____________ DATE ____________

1	2	3	4	Δt = 15 min.	
				5	6
Elevation	O_2	S_2	$\frac{O_2}{2}$	$\frac{S_2}{\Delta t}$	$\frac{S_2}{\Delta t}+\frac{O_2}{2}$
FT	CFS	CFS-MIN	CFS	CFS	CFS
0	0	0	0	0	0
0.5	2.4	1525	1.20	101.7	102.9
1.0	7.5	3050	3.75	203.3	207.1
1.5	12.4	4575	6.20	305.0	331.2
2.0	19.1	6100	9.55	406.7	416.3
2.5	22.4	7625	11.20	508.3	519.5
3.0	26.2	9150	13.10	610.0	623.1
3.5	29.5	10675	14.75	711.7	726.5
4.0	32.5	12200	16.25	813.0	829.6
4.5	45.8	13725	22.90	915.0	937.9
5.0	67.7	15250	33.85	1016.7	1051.0
5.5	95.1	16775	47.55	1118.3	1166.0
6.0	127.2	18300	63.60	1220.0	1284.0

EXHIBIT XII-10



STORAGE - INDICATION CURVE
ARTICLE 12.9.7 EXAMPLE

EXHIBIT XII-11

STORAGE-INDICATION OPERATING TABLE
ARTICLE 12.9.7 EXAMPLE

PROJECT ____________ DESIGNER ____________ DATE ______

1 Hydrograph Time	2 Routing Interval	3 Time	4 Inflow	5 Ave. Inflow	6 $S_1/\Delta t+O_1/2$	7 O_1	8 $S_2/\Delta t+O_2/2$	9 O_2
HRS		MINUTES	CFS	CFS	CFS	CFS	CFS	CFS
11.00	-	0	14		0	0	0	0
11.25	1	15	20	17.0	0	0	17.0	0.4
11.50	2	30	27	23.5	17.0	0.4	40.1	0.9
11.75	3	45	52	39.5	40.1	0.9	78.7	1.8
12.00	4	60	159	105.5	78.7	1.8	182.4	6.3
12.25	5	75	339	249.0	182.4	6.3	425.1	19.4
12.50	6	90	364	351.5	425.1	19.4	757.2	30.4
12.75	7	105	264	314.0	757.2	30.4	1040.8	65.7
13.00	8	120	175	219.5	1040.8	65.7	1194.6	102.9
13.25	9	135	123	149.0	1194.6	102.9	1240.7	115.4
13.50	10	150	91	107.0	1240.7	115.4	1232.3	113.1
13.75	11	165	71	81.0	1232.3	113.1	1200.2	104.4

EXHIBIT XII-12

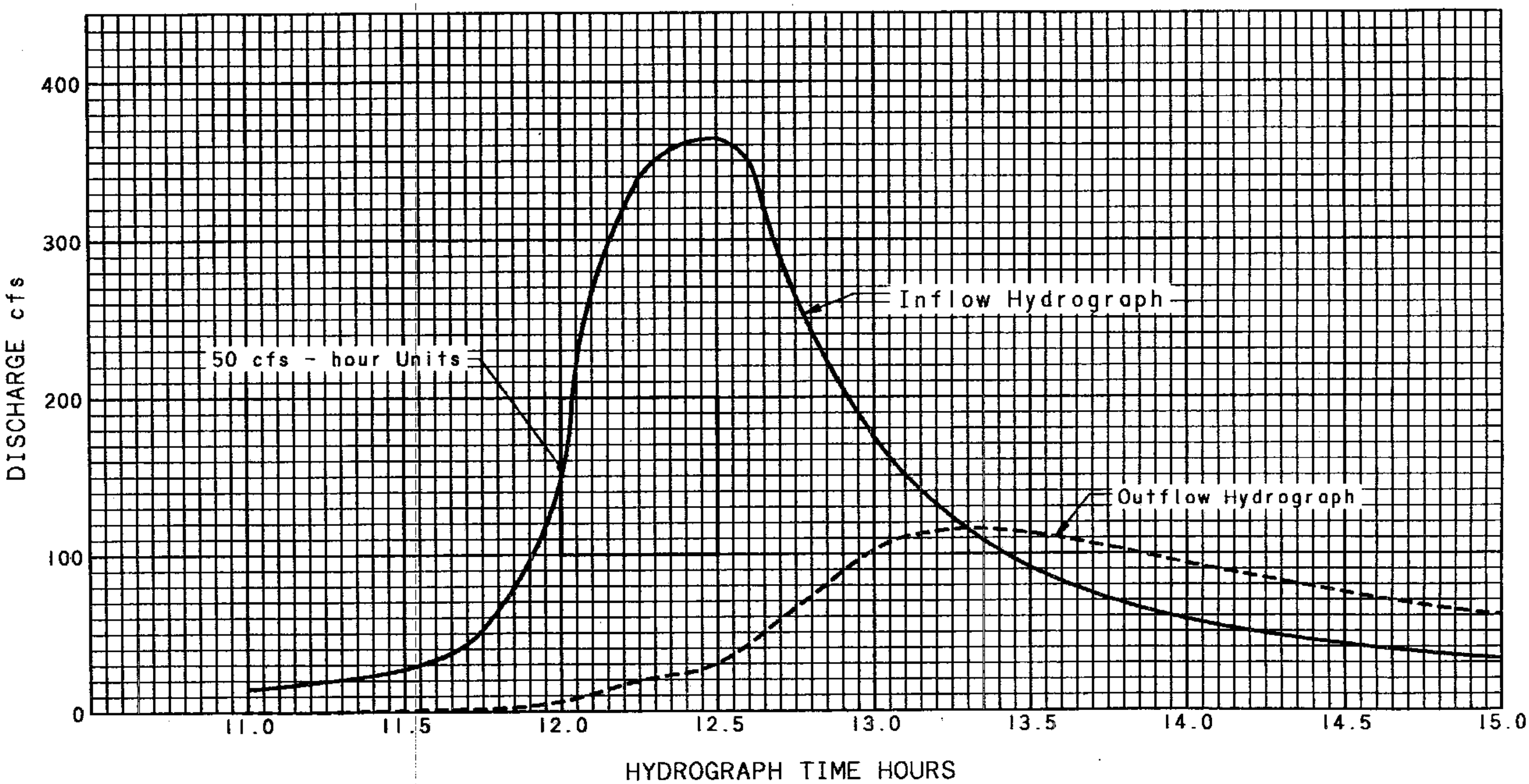
STORAGE-INDICATION OPERATING TABLE
ARTICLE 12.9.7 EXAMPLE

PROJECT ______________ DESIGNER ______________ DATE ______

1 Hydrograph Time	2 Routing Interval	3 Time	4 Inflow	5 Ave. Inflow	6 $S_1/\Delta t+O_1/2$	7 O_1	8 $S_2/\Delta t+O_2/2$	9 O_2
HRS		MINUTES	CFS	CFS	CFS	CFS	CFS	CFS
14.00	12	180	58	64.5	1200.2	104.4	1160.3	93.7
14.25	13	195	49	53.5	1160.3	93.7	1120.1	84.2
14.50	14	210	42	45.5	1120.1	84.2	1081.4	74.9
14.75	15	225	37	39.5	1081.4	74.9	1046.0	66.7
15.00	16	240	33	35.0	1046.0	66.7	1014.3	60.6
15.25	17	255	30	31.5	1014.3	60.6	985.2	55.0
15.50	18	270	28	29.0	985.2	55.0	959.2	49.9
15.75	19	285	27	27.5	959.2	49.9	936.8	45.7
16.00	20	300	25	26.0	936.8	45.7	917.1	43.2
16.25	21	315	24	24.5	917.1	43.2	898.4	40.9
16.50	22	330	24	24.0	898.4	40.9	881.5	38.9
16.75	23	345	24	24.0	881.5	38.9	866.6	37.0
17.00	24	360	23	23.5	866.6	37.0	853.1	35.4

EXHIBIT XII-12
(CONTINUED)

UNIT CONVERSION: 1 UNIT = 1 SQUARE INCH = (100)(0.5) = 50 CFS-HOURS = (50)(60)(60) = 180,000 CUBIC FEET = 180,000 / 43,560 = 4.132 ACRE-FEET



INFLOW AND OUTFLOW HYDROGRAPH
ARTICLE 12.9.7 EXAMPLE

EXHIBIT XII-13

STORAGE-INDICATION COMPUTATION TABLE GUIDE

PROJECT ____________________ DESIGNER ______________ DATE __________

1	2	3	4	Δt = __ min.	
				5	6
Elevation	O_2	S_2	$\frac{O_2}{2}$	$\frac{S_2}{\Delta t}$	$\frac{S2}{\Delta t}+\frac{O2}{2}$
FT	CFS	CFS-MIN	CFS	CFS	CFS
GIVEN	FROM ELEVATION DISCHARGE CURVE	FROM ELEVATION STORAGE CURVE	COLUMN 2/2	COLUMN 3/ΔT	COLUMN 4 & COLUMN 5

EXHIBIT XII-14

STORAGE-INDICATION OPERATING TABLE GUIDE

PROJECT ______________________ DESIGNER ______________ DATE ______

1 Hydrograph Time	2 Routing Interval	3 Time	4 Inflow	5 Ave. Inflow	6 $S_1/\Delta t + O_1/2$	7 O_1	8 $S_2/\Delta t + O_2/2$	9 O_2
HRS	MINUTES	MINUTES	CFS	CFS	CFS	CFS	CFS	CFS
GIVEN	ASSUMED	COLUMN 2 x INTERVAL TIME INCREMENT	FROM INFLOW HYDROGRAPH WITH TIME GIVEN IN COLUMN	(COLUMN 4 FOR CURRENT TIME + COLUMN 4 FOR NEXT TIME) +2	COLUMN 8 FOR PROCEEDING TIME INTERVAL	COLUMN 9 FOR PROCEEDING TIME INTERVAL	COLUMN 5+ COLUMN 6 - COLUMN 7	FROM STORAGE - INDICATION CURVE

EXHIBIT XII-15

GRAPHICAL FLOW ROUTING COMPUTATIONS GUIDE

1 STORM FREQUENCY	PRE-DEVELOPMENT	POST-DEVELOPMENT		5 RELEASE RATE	DISCHARGE RATIO	VOLUME RATIO	REQUIRED STORABLE VOLUME EXHIBIT XII-5	
	2 PEAK DISCHARGE Qo	3 PEAK DISCHARGE QI	4 VOLUME RUNOFF Vri	Qo	6 Qo/QI	7 Vs/Vri	8 Vs	9 Vs
YEARS	CFS	CFS	INCHES	CSM			INCHES	ACRE FT
	CALCULATED BY GRAPHICAL PEAK DISCHARGE METHOD	CALCULATED BY GRAPHICAL PEAK DISCHARGE METHOD	CALCULATED BY GRAPHICAL PEAK DISCHARGE METHOD	COLUMN 2÷ DRAINAGE AREA IN SQUARE MILES	COLUMN 2÷COLUMN 3	EXHIBIT XII-5 WITH COLUMN 6	COLUMN 7x COLUMN 4	COLUMN 7 x COLUMN 4 x 53.3x DRAINAGE AREA IN SQUARE MILES

EXHIBIT XII-16

GRAPHICAL FLOW ROUTING COMPUTATIONS

ARTICLE 129.4

1 STORM FREQUENCY	PRE-DEVELOPMENT	POST-DEVELOPMENT		5 RELEASE RATE Qo	DISCHARGE RATIO	VOLUME RATIO	REQUIRED STORABLE VOLUME EXHIBIT XII-5	
	2 PEAK DISCHARGE Qo	3 PEAK DISCHARGE QI	4 VOLUME RUNOFF VrI		6 Qo/QI	7 Vs/VrI	8 Vs	9 Vs
YEARS	CFS	CFS	INCHES	CSM			INCHES	ACRE FT

FORM T12-1
(12.9.4)

STORAGE-INDICATION COMPUTATION TABLE

PROJECT ______________________ DESIGNER ______________ DATE ________

				Δt = min.	
1	2	3	4	5	6
Elevation	O_2	S_2	$\frac{O_2}{2}$	$\frac{S_2}{\Delta t}$	$\frac{S_2}{\Delta t} + \frac{O_2}{2}$
FT	CFS	CF	CFS	CFS	CFS

FORM T12-2
(12.9.7)

STORAGE-INDICATION OPERATING TABLE

PROJECT ____________________ DESIGNER ____________________ DATE ________

1 Hydrograph Time	2 Routing Interval	3 Time	4 Inflow	5 Ave. Inflow	6 $S_1/\Delta t+O_1/2$	7 O_1	8 $S_2/\Delta t+O_2/2$	9 O_2
HRS	MINUTES	MINUTES	CFS	CFS	CFS	CFS	CFS	CFS

FORM T12-3
(12.9.7)

12.10 Bibliography

Soil Conservation Service Central Technical Unit, U.S. Department of Agriculture. Computer Program for Project Formulation - Hydrology. Technical Release No. 20. Washington, D.C., May 1965.

U.S. Department of Agriculture, Soil Conservation Service. Engineering Field Manual for Conservation Practices. Washington, D.C., 1969.

Stem, George L. Proceedings of National Symposium on Urban Hydrology and Sediment Control. "An Evaluation of Practical Experience in Storm Water Management. University of Kentucky, Lexington, Kentucky. July 1975.

McGraw-Hill Book Company. Handbook of Applied Hydrology. Ven Te Chow, Editor-in-Chief. New York, New York, 1964.

The Urban Land Institute. Lakes and Ponds, Technical Bulletin 72. Washington, D.C., 1976.

Lindley, R. William. University of Kentucky, Lexington, Kentucky. "A Case Study: Green Trails Development, Lisle, Illinois." Precedings of National Symposium on Urban Hydrology, Hydraulics, and Sediment Control. July 1976.

Linsley, Ray K., Joseph B. Franzini. Water-Resources Engineering, Second Edition. McGraw-Hill Book Company. New York, New York, 1972.

National Association of Home Builders, NAHB Research Foundation, Inc. A Manual of Residential Storm Water Management Development Standards. Rockville, Maryland. April 1973.

The Maryland-National Capital Park and Planning Commission. A Manual of Storm Water Management for Evaluating and Mitigating the Effects of Land Use Changes on Runoff. Roy F. Weston, Inc., Pennsylvania. June 1974.

Poertner, Herbert G. "Urban Stormwater Detention and Flow Attenuation." Public Works. August 1976. pp. 83-85.

American Public Works Association. Practices in Detention of Urban Stormwater Runoff. Special Report No. 43. Chicago, Illinois, 1974.

The Urban Land Institute. The American Society of Civil Engineers. Residential Storm Water Management. The National Association of Home Builders. Washington, D.C., 1975.

Soil Conservation Service. U.S. Department of Agriculture, U.S. Government Printing Office. "Section 4, Hydrology", National Engineering Handbook. Washington, D.C. January 1971.

Maryland Department of Natural Resources. Storm Water Management Design: A Manual of Procedures and Guidelines. Roy F. Weston, Inc., Pennsylvania. April 1976.

Soil Conservation Service Regional Technical Service Center, U.S. Department of Agriculture. Tabular Method of Flood Routing - 24-Hour Type II Storm Distribution, TSC Technical Note. Engineering - UD-20. Upper Darby, Pennsylvania. January 1972.

Soil Conservation Service Engineering Division, U.S. Department of Agriculture. Urban Hydrology for Small Watersheds, Technical Release No. 55. January 1975.

Daugherty, David L. "Urban Storm Runoff Controls" Proceedings of Planning and Design for Urban Runoff and Sediment Management. University of Kentucky, Lexington, Kentucky, 1973.