CHAPTER TWELVE

DRAINAGE DESIGN

STATE OF MAINE

URBAN & ARTERIAL

HIGHWAY DESIGN GUIDE

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Chapter Twelve

DRAINAGE DESIGN

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Chapter Twelve
DRAINAGE DESIGN

12-1 *GENERAL DRAINAGE POLICY DIRECTIVES*

12-1.01 Introduction

This chapter will provide the drainage policies, criteria and references approved for use by the Maine Department of Transportation (MDOT). The objective is to provide the basic information needed by the highway designer, hydrologist, and/or hydraulic engineer to perform the hydrological and hydraulic analyses for which he/she is responsible. A list of references has been provided at then end of this chapter which provide more details about the methods and theory recommended in this chapter. This Guidance Document has been prepared by the Hydrology Section of the Environmental Office with assistance from other Maine DOT Staff.

In using this Manual, it should be understood that the intent is to provide guidelines dealing with situations commonly encountered in design. It is hoped that in most situations, the methods may be applied directly, in a straight-forward manner. That said, this Manual should not be seen as a "cookbook", as it cannot possibly anticipate every possible design issue. All design presumes professional judgment and experience on the part of the designer, informed by these guidelines and policies. Departures from these guidelines should be noted and documented by the person in responsible charge of the project design. Deviations from policy should be approved by appropriate MDOT staff.

Users of this manual are encouraged to share comments and suggestions for improvement with the Hydrology Section. Please direct all communication to

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Environmental Office - Hydrology Section
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Augusta, ME 04333-0016
Phone: 207-624-3100
Fax: 207-624-3101
Web address: http://www.state.me.us/mdot/env/homepage.html

12-1.02 Department Responsibility

The Urban/Arterial and Bridge Programs are both responsible for drainage analyses for projects under the jurisdiction of the Department. The Regional Program and Division offices will also have occasion to employ the drainage analysis and design methods described in this chapter.

Responsibility for different kinds of drainage systems is generally assigned as follows:

1) Closed Drainage Systems: Urban/Arterial Program
2) Bridges and Box Culverts: Bridge Program
3) Other Culverts: Bridge, spans $\geq$ 10 ft; Urban/Arterial or Regional, spans < 10 ft
4) Open Channels: responsibility is project-specific

12-1.03 Coordination with Other Agencies and DOT Offices

Modification to any drainage system can affect areas away from the area of construction. The analyst should be aware of future land-use plans and any expected changes to water courses, and should also investigate all future Federal, State, and local agency plans which may affect drainage.

Storm Water Memorandum of Agreement

The Maine Department of Environmental Protection (DEP) regulates storm water impacts due to site development. A Memorandum of Agreement (MOA) between DEP, MDOT, and the Maine Turnpike Authority (MTA) was signed in 1998 in order to streamline compliance by MDOT with state storm water quantity and quality regulations. Storm water quality issues are focused primarily on prevention and control of sedimentation and erosion; quantity issues are focused on controlling increases in peak flow from project sites. The Water Resources Section of the MDOT Environmental Office is responsible for water quality issues on MDOT projects; the Hydrology Section is responsible for storm water quantity.

The Storm Water Quantity Standard in the MOA states that

MDOT and MTA will calculate the peak flow from the site of a project if the project: 1) combines two or more sub drainage areas, and 2) includes 20,000 sq. ft. or more of new impervious area or five acres or more of disturbed are in the direct watershed of a waterbody most at risk from new development (as defined in DEP's Chapter 500 and 502), or one acre or more of new impervious area or five acres or more of disturbed area elsewhere. MDOT and MTA will design project ditches, culverts, and outlet areas to be stable and will minimize any increase in peak flow from the project site. In those instances in which a peak flow increase will result, MDOT and MTA shall take engineering measures to avoid adverse impacts to offsite property as a result of drainage increases resulting from the project.

12-1.04 Information Required

Data typically developed as part of a drainage study will include, but is not limited to, watershed delineations, areas, slopes, flowpaths, times of concentrations, Rational Method runoff coefficients, and SCS Runoff Curve Numbers. In most cases, design flows will be calculated as part of the watershed characterization effort; flow computation will not ordinarily be separated from watershed characterization.

Information included in a drainage report will generally consist of

1) a brief description of the drainage basin under analysis
2) a brief description of how the data was obtained
3) a map(s) of the drainage area which provides pertinent information
4) a completed Drainage Study form (MS-Excel worksheet) containing pertinent watershed characteristics and computed design flows.

When at all feasible, drainage study documents should be prepared in digital format (spreadsheets, word processing documents, CAD files, scanned images). Current Department policy is to minimize the retention of paper "hard copy". All standard drainage reports and calculations should be executed using MS-Excel spreadsheet templates prepared specifically for this purpose by the Hydrology Unit.

12-1.05 Documentation Procedures

The following items should be included in the project file. The intent is not to limit the data to only those items listed, but rather to establish a minimum requirement consistent with the hydraulic design procedures outlined in this Guide. Any additional information essential to the hydraulic design should also be included.

Report Documentation

A summary report should be prepared for every drainage study. The recommended report format is discussed later in this chapter. The remainder of this section describes the kind of information that may be generated in the process of a study. This information will be summarized and presented in the report. This information may also be preserved in working documents such as spreadsheets, maps, photos, etc. To the greatest extent possible, information should be converted to digital form for final project archiving.

General Comment on Computer Files

It is anticipated that most information will be summarized and used in the form of computer files. All final, and other important, worksheets and computer program input/output files should be saved to the project directory. An index (as a MS-Word or MS-Excel file) of all drainage study computer files should be prepared, with a brief description of each file. Important paper documents that are not computer-originated (e.g., photos, USGS topographic

maps) should be scanned for archival with project drainage study computer files. Files created with software should be archived in native format (e.g., as MS-Word or MS-Excel files that can subsequently be changed) as well as "printed" to Adobe [.PDF] files corresponding to final results for design in the interests of preserving file integrity.

Hydrology

Drainage studies should be documented according to the guidance in Section 12-2.10, "Documentation of Hydrologic Studies".

Culverts

The following items should be included in the project file, when applicable:

1) culvert performance curves
2) allowable headwater elevation and basis for selection
3) tailwater elevation and basis of selection
4) cross-section(s) used in the design of highwater determinations
5) roughness coefficient assignments ("n" values)
6) observed highwater, dates, and estimated discharges
7) stage discharge curve for undisturbed, existing and proposed conditions to include the depth and velocity measurements or estimates and locations for the design, 50-year and check floods
8) performance curves showing the calculated backwater elevations, outlet velocities and scour for the design, 50-year and any historical floods
9) type of culvert entrance condition
10) culvert outlet appurtenances and energy dissipation calculations and designs
11) roadway geometry (plan and profile)
12) potential flood hazard to adjacent properties

Closed Systems (Stormwater Drainage Systems)

The following items should be included in the project file or project computer directory:

1) computations for inlets and pipes
2) drainage area map
3) design frequency
4) documentation of outfalls, existing storm drains, and other design considerations
5) a schematic indicating closed system layout

Open Channels

The following items should be included in the project file (if applicable):

1) stage-discharge curves for the design, 50-year, 100-year and any historical events
2) cross-section(s) used in the design water surface determinations and their locations

3) roughness coefficient assignments ("n" values)
4) information on the method used for design water surface determinations
5) observed highwater, dates, and discharges
6) channel velocity measurements or estimates and locations
7) design or analysis of materials proposed for the channel bed and banksenergy dissipation calculations and designs
8) copies of all computer analyses

12-1.06 **Design Event Frequency: Selection of Peak Discharge**

General policy is to design open drainage systems for the 50-year event and closed drainage (storm sewer) systems for the 10-year event. Roadway cross-culverts and open channels should also be checked for performance under the 100-year event; driveway culverts should be designed for the 10-year event. Rational method and rainfall-runoff models use storms of the specified frequency. Storm duration is assigned according to watershed time of concentration t_c, with a minimum value of t_c = 5 minutes. This policy is summarized in Table 12-1.1.

Table 12-1.1: RECOMMENDED DESIGN FLOOD FREQUENCY

Highway Functional Class	Drainage Structure			
	Roadway Cross Culvert[1]	Driveway Culvert[2]	Closed System[3]	Open Channel[1]
Freeway	50-year	N/A	10-year	50-year
Other Arterials	50-year	10-year	10-year	50-year
Collectors & Locals	50-year	10-year	10-year	50-year

[1] *Impacts of the 100-year event should also be checked.*
[2] *If driveway culvert is part of main system, design for the 50-year event.*
[3] *Storm duration = catchment time of concentration, with 5 minute minimum duration.*

Exceptions to Recommended Design Frequency

Some roads may warrant design to a higher flood frequency, e.g., 100 years. These roads include major arterials that offer few or no detour options and roads that serve significant health, safety and other public facilities. The exceptions should be identified on a case-by-case basis in consultation with Division personnel and local officials.

Occasionally, some roads may warrant design to a lower risk, primarily in cases where design to the recommended frequency is too expensive. Such exceptions should be clearly identified and fully documented.

Ditches

Roadside ditches are not designed for a particular frequency, as they are usually not a limiting factor in system capacity. Based on the designer's experience and the requirements of a specific

project, ditches may be sized to a specific frequency following the above table. More often, ditches are placed to a standard depth to allow subbase drainage. In some cases, ditches cannot be placed to standard depth because it would require excessive backslope, thus impacting the adjacent property owner. A combination of shallow ditch and underdrain may be used to convey storm water and drain the subbase. In these situations, the shallow ditch capacity should be checked against the design flow.

Storm and Flood Frequencies

Unless otherwise stated, it is assumed that a flow of a particular frequency is produced by a rainfall event of the same frequency. While there are regular situations in Maine where this is not justified, the complexity of the underlying physical hydrology and climatic conditions as well as limited resources require this simplifying assumption. These other situations may be addressed on a project-specific basis where appropriate (e.g., based on past flooding experience, maintenance experience, designer concerns) and where the necessary data are available.

12-1.07 Execution of Work

Drainage analysis may be performed in-house by MDOT staff or by outside consultants, according to the particular needs of the MDOT. MDOT Drainage Policy shall be followed in either case. This manual is intended to document MDOT Drainage Policy as well as provide technical guidance for implementing that policy. However, this technical guidance is not intended to supercede the professional judgement of the licensed Professional Engineer in overall responsible charge of a drainage study.

12-1.08 Units of Measurement, Specification, and Analysis

MDOT policy is that design is performed in U.S. Customary units whenever possible and sensible, and that all designs are presented in U.S. Customary units exclusively. Most equations are given here in a general form valid for both Customary and metric units; appropriate conversion factors are also supplied. Even in Customary form, some equations may be cast in inconsistent units (e.g., ft^3/s, ac, and in/hr in the Rational formula). Preferred units for hydrologic and hydraulic reporting are given in Table 12-1.2. Common conversion factors are given Table 12-1.3. All tables and figures in the body of this chapter are given in U.S. Customary units; the same tables are given in metric in the appendix.

Table 12-1.2: Preferred Units for Hydrologic and Hydraulic Analysis and Design

Quantity	Symbol	U.S. Customary	Metric
Length	L	ft or mi	m or km
Area	A	ac, ft^2, or mi^2	ha, m^2, or km^2
Flow Rate	Q	ft^3/s	m^3/s
Velocity	v	ft/s	m/s
Rainfall Intensity	i	in/hr	mm/hr
Storm Depth	d	In	mm
Pipe Diameter	D	in or ft	mm or m
Time of Concentration	t_c	min	min
Slope	S	ft/ft or ft/mile	m/m or m/km

12-1.09 Vertical Datum

The Maine Department of Transportation, Survey Section, completed its conversion to the 1988 North American Vertical Datum (NAVD) in 1999. Since January, 2000, all new projects, with a few exceptions, are in the NAVD 1988 datum. If there is any doubt about what vertical datum was used for a project, the datum should be confirmed before proceeding.

Most existing MDOT plans, historical flood information, and U.S. Geological Survey topographical maps are based on the 1929 National Geodetic Vertical Datum (NGVD). Elevations *Z* based on the older NGVD 1929 need to be converted to the newer NAVD 1988 using the following equation:

$$Z_{NGVD} - Z_{shift} = Z_{NAVD}$$

The datum shift (correction factor) Z_{shift} ranges between 0.591 ft (0.18 m) and 0.722 ft (0.22 m). The exact datum shift for a specific location in Maine can be calculated using the *VERTCON* program at the following National Geodetic Survey (NGS) Internet website:

http://www.ngs.noaa.gov/cgi-bin/VERTCON/vert_con.prl

The following data is required as input on the web page:

1) North Latitude (Required)
2) West Longitude (Required)
3) Orthometric Height (Optional)

Latitude and Longitude may be entered in any of the following three formats:

1) degrees, minutes and decimal seconds (ddd mm ss.sss)
2) degrees and decimal minutes (ddd mm.mmm)
3) decimal degrees (ddd.ddddd)

There must be one or more blanks between entry fields. Decimals can be keyed commensurate with the field's precision, but are not required.

Example Problem: Presque Isle, Gouldville Bridge #3881

Q_{100} Elevation = 431 ft (131.37 m) from Flood Insurance Study based on NGVD 1929. Determine the NAVD flood elevation.

Step 1.) Go to website and get datum shift by entering latitude and longitude for the location you are interested in.

Latitude 46.667^o
Longitude 68.00^o
Datum shift = 0.627 ft (0.191 m)

Step 2.) Subtract datum shift from elevation based on NGVD 1929 to convert to NAVD 1988.

$$Z_{NGVD} - Z_{shift} = Z_{NAVD}$$

431 ft - 0.627 ft = 430.373 ft

All hydrology/hydraulics reports should state what vertical datum is used. For example, the following statement can be added at the end of the hydraulics report:

<u>Note:</u> All elevations based on North American Vertical Datum (NAVD) 1988. Elevations based on the National Geodetic Vertical Datum (NGVD) 1929 were converted to NAVD by the appropriate shift (0.627 ft) using NGS Vertcon program.

Table 12-2.3: Common Conversion Factors

Length	1 in	=	25.4 mm
	1 m	=	3.28 ft
	1 mi	=	5,280 ft
	1 mi	=	1.61 km
	1 m	=	10^2 cm
	1 m	=	10^3 mm
	1 km	=	10^3 m
Area	1 ac	=	43,560 ft^2
	1 mi^2	=	640 ac
	1 mi^2	=	2.59 km^2
	1 km^2	=	247 ac
	1 hectare	=	2.47 ac
	1 hectare	=	10^4 m^2
	1 km^2	=	10^6 m^2
	1 km^2	=	10^2 hectare
Volume	1 m^3	=	35.29 ft^3
	1 ft^3	=	7.48 gal
	1 ac-ft	=	43,560 ft^3
	1 gal	=	3.785 liters
Time	1 day	=	86,400 sec
	1 day	=	1,440 min
	1 hr	=	3,600 sec
Pressure	1 atm	=	10.35 m H_2O
	1 atm	=	101.4 kiloPascals (kPa)
	1 atm	=	14.7 lbs/in^2

Density of Water $\rho_w = 1\ g/cm^3 = 10^3\ kg/m^3 = 62.4\ lb/ft^3$

Gravity $g = 9.81\ m/s^2 = 32.2\ ft/s^2$

12-2 *HYDROLOGY*

12-2.01 Introduction

The hydrologic analysis will determine the peak discharge for the selected design frequency and site conditions. This discharge will be used in determining the required size of highway facilities in the hydraulic analysis. On occasion, complete flood hydrographs for storage routing may also be determined to support design of detention ponds and size culverts.

12-2.02 Methods of Estimating Peak Discharge

The Maine Department of Transportation (MDOT) uses several methods of estimating design event peak flow rates, volumes, and timing. Experience has shown these methods to be practical, economical, and within the limits of attainable accuracy. The following methods are regularly employed in MDOT practice:

1. Rational Method
2. U.S. Geological Survey (USGS Hodgkins) Regression Equations
3. Rainfall-Runoff Hydrograph Modeling (including, but not limited to, TR-20)
4. Published Flow Records
5. Flood Reports

Additional sources of valuable information are

1. Documented testimony
2. Maintenance experience
3. Local anecdotal experience
4. High water marks and observations
5. River/stream geomorphologic observations

More complex hydrology and hydraulic computer simulation models might be applied in project-specific cases. However, these methods typically require costly field data, large amounts of data preparation, model setup and calibration, and interpretation, not to mention staff expertise to execute these tasks. Therefore, such methods can only be justified in unusual or sensitive situations.

In the discussion that follows, TR-20 is identified explicitly in view of its widespread use in the Maine engineering community. However, its use in this manual should be understood as a surrogate for all rainfall-runoff modeling approaches.

12-2.03 Selecting a Flood Estimation Procedude

The choice of an estimation method is driven by the specific technical needs of a project subject to the usual requirements of watershed characteristics, accuracy and cost-efficiency. In many highway design applications, just the peak instantaneous flows are needed and the Rational Method and USGS Regression Equations will be the methods of choice. Even when other methods are employed (typically, event rainfall-runoff modeling as in TR-20), Rational and USGS should be used to check the reasonableness of model results. In all cases, the analyst should consult with maintenance staff, local authorities, and local residents for their experience with flooding at the site under study. Table 12-2.1 outlines the appropriate choice of method for particular situations when only the peak instantaneous flow is needed.

Table 12-2.1: Methods[1] for Peak Flow Calculation for Culverts & Ditches

Watershed Area	Rural	Urban
< 0.5 mi^2 (320 ac = 1.25 km^2)	Rational and Modeling	Rational and Urban USGS[3]
0.5 – 1 mi^2 (320 ac - 640 ac or 1.3 km^2 – 2.6 km^2)	Rational, USGS[2,5] and Modeling[4]	Rational, Urban USGS and Modeling
> 1 mi^2 (640 ac = 2.6 km^2)	USGS	Urban USGS

Notes:

1) At the discretion of MDOT, other methods may be used on a project-specific basis
2) USGS indicates USGS (Hodgkins, 1999) regression equations (Water-Resources Investigations Report 99-4008, USGS, Augusta, ME, 1999)
3) Urban USGS indicates regression equations with Sauer correction for urbanization as documented in Hodgkins (1999)
4) Modeling will generally be performed with TR-20 or equivalent. MDOT may approve alternative models on a project-specific basis. See further comments on use of modeling.
5) USGS is only a secondary method for small watersheds (< 1 mi^2)

Rational and USGS Regression are the simplest methods to apply. TR-20 event modeling is moderately more data intensive. Even though TR-20 gives the appearance of being a physically based, detailed representation of hydrologic processes, its performance in comparable situations is no better (and often worse) than Regression and Rational. In contemporary Maine practice, event modeling is often performed with *HydroCad* (Applied Microcomputer Systems, Inc., 2001), an easy-to-use proprietary derivative of TR-20 methodology, as well as other models with highly developed user interfaces. While internal MDOT practice is to use Rational Method on small watersheds, event models will also be accepted.

<u>USGS Regression</u>

USGS Regression offers several advantages over other methods:

1) is only method with established accuracy limits
2) is at least as accurate as the other methods
3) is demonstrably more accurate than rainfall-runoff modeling in comparable situations.

4) is based directly on annual maximum data and thus does not depend on the questionable assumption (inherent in rainfall-runoff modeling) that the T-year storm produces the T-year flood event.

Strictly speaking, the regression equations are subject to several limitations:

1) limited to watersheds greater than 1 mi^2 (2.6 km^2) in area (the smallest watershed in the underlying data set)
2) flatter than 260 ft/mi (50 m/km) in Benson slope
3) wetlands (by National Wetlands Inventory) less than 27% of total area.
4) watersheds should be rural, undeveloped, and unregulated.

However, regression estimates can be calculated for watersheds outside these limits because the equations are so simple to apply. Much of the basic data preparation for USGS is also needed for Rational and Modeling. Therefore, the USGS estimate should always be calculated. Even for small watersheds, USGS can be used as one piece of information to judge the reasonableness of other estimates provided the watersheds are only marginally outside the limits of strict applicability.

Rational Method

Rational Method has been in use for 150 years and remains the tool of choice for hydrologic design on small watersheds and paved surfaces. It is impossible to assign a precise upper limit on watershed size for reliable Rational application. References cite upper limits ranging from 200 acres (Dunne and Leopold, 1978, p. 298) and "several hundred" acres (Viessman, et al, 1977, p. 512) to 1000 acres (WSDOT, 1997, p. 2-3) (with 1 km^2 approximately = 250 ac). General consensus is that best results are obtained when area is less than 200 ac (0.8 km^2; WSDOT suggests an upper limit of 100 ac (0.4 km^2) for best accuracy) and the surface is largely impervious or, failing that, at least homogeneous. Indeed, under such conditions Rational is likely to be as good or better than more complicated methods. For the purposes of MDOT design, Rational estimates should be calculated for all watersheds smaller than 640 ac (1 mi^2 = 2.6 km^2). At the lower end of this range, Rational estimates should be most reliable. Above 320 ac (0.5 mi^2 = 1.3 km^2, the designer should combine Rational, USGS and other estimates in some manner (not necessarily simple averaging) to arrive at a single design flow. As watershed area approaches 1 mi^2 (= 640 ac = 2.6 km^2), greater emphasis should be given to the USGS estimate. Rational estimates should not be used for areas larger than 1 mi^2.

Rainfall-Runoff Modeling

Rainfall-Runoff Modeling can be a useful technique under certain well-defined circumstances. However, it is wrong to assume that because these models purportedly capture more detailed hydrology, they are somehow more accurate. The assumptions underlying much of TR-20 hydrology are almost as severe as those underpinning Rational Method. Furthermore, the paucity of flow and other process-based hydrologic data in most highway design situations requires major assumptions about parameter values and distributions. With Rational Method the

assumptions and simplifications are impossible to ignore, whereas in TR-20 it is easy to be seduced by the "complexity" into believing that a higher degree of accuracy has been achieved. Therefore, TR-20 (and other rainfall-runoff) modeling should only be used when Rational and USGS are clearly inappropriate. Some of these situations are

1) watershed is too heterogeneous for Rational application
2) watershed is heavily developed
3) hydrograph routing is required (as in detention pond design and some culvert analysis)
4) watershed is between 320 - 640 ac (0.5 - 1 mi^2; 1.3 - 2.6 km^2) and other estimates are highly inconsistent
5) watershed is smaller than 640 ac and has significant storage

The different methods for peak flow estimation will now be described in more detail. Following these descriptions, suggestions will be given for combining several estimates into a single design flow. As this document is primarily a manual for MODT highway design staff, emphasis will be given to the Rational Method. Modeling is a specialized topic that is beyond the scope of this manual.

12-2.04 USGS Regression Equations

Introduction

Regression equations are the preferred method for peak flow estimation, provided they are appropriate for the given situation (see table above). Generally speaking, for undeveloped watersheds larger than 1 mi^2 (640 ac = 2.6 km^2), USGS regression should be the only method employed when peak flows are needed.

Advantages of regression equations are

1) cost-efficient
2) minimal data requirements (just area and percentage wetlands)
3) uncertainty (error) is quantified
4) more accurate than modeling
5) based on actual flood events as opposed to design storms

Primary limitations of the Maine equations are

1) accuracy decreases in an unknown way as watersheds become smaller than 1 mi^2 (640 ac = 2.6 km^2)
2) accuracy decreases in an unknown way when watershed characteristics are determined with methods different from those used in the original regression study
3) give only peak flow, not complete hydrograph

4) best suited to undeveloped and unregulated watersheds, though correction for urbanization is available
5) ill-suited to "pre-development/post-development" applications

Therefore, for undeveloped watersheds larger than 1 mi^2 (= 640 ac = 2.6 km^2), USGS regression should be the only method employed when peak flows are needed. For watersheds smaller than 640 ac, Rational and USGS should both be calculated. Instructions for combining estimates are given in a later section. Since the equations calculate only the peak flow, they are inappropriate for storage routing applications and in these situations modeling will be necessary.

Regression Equations and Their Coefficients

Regression equations derive their strength from a large database of annual peak flow records. No other method is so rooted in real Maine data. The current USGS equations are based on records from 70 stations, each with 10 or more years of record. There is just one set of equations for the entire State of Maine, of the form

$$Q_T = b(A)^a 10^{-w(W)}$$

Where T = design flood recurrence interval (years)
b,a,w = coefficients dependent on T value
Q = peak flow (m^3/s)
A = watershed area, km^2
W = percentage (%) wetlands as determined from National Wetlands Inventory (NWI) maps

The USGS equations were developed in the metric system. Therefore, watershed area should be converted to [km^2] before applying the equations:

$$A\ [km^2] = 2.59 \times A\ [mi^2]\ .$$

Similarly, the resulting Q values in [m^3/s] should be converted to [ft^3/s]:

$$Q\ [ft^3/s] = 35.29 \times Q\ [m^3/s]\ .$$

The coefficients and associated estimate error bounds are summarized in Table 12-2.2.

The effect of NWI storage in these equations is to reduce Q_T as compared to the absence of storage. Wetland hydrology is complex and this simplistic accounting may not offer an accurate picture of the behavior of a particular watershed.

Table 12-2.2: Full USGS regression equations[1] and their accuracy for estimating peak flows Q for rural ungaged, unregulated streams watersheds in Maine (from Table 3, Hodgkins, 1999)

T	b	a	w	Standard Error (%)	PRESS Error (%)	Equivalent Years Record
2	1.075	0.848	0.0266	40.6 to -28.9	42.2 to -29.7	1.82
5	1.952	0.820	0.0288	41.9 to -29.5	43.5 to -30.3	2.47
10	2.674	0.806	0.0300	42.9 to -30.0	45.2 to -31.1	3.20
25	3.740	0.790	0.0312	45.2 to -31.1	48.3 to -32.5	4.14
50	4.637	0.780	0.0320	46.9 to -31.9	51.0 to -33.8	4.78
100	5.629	0.771	0.0326	48.6 to -32.7	53.5 to -34.8	5.37
500	8.283	0.754	0.0340	53.5 to -34.8	60.0 to -37.5	6.41

[1]equation of form $Q_T = b(A)^a 10^{-w(W)}$; Q in (m^3/s), A in (km^2), and W as percentage

Area and wetlands are called "predictor variables" because they are used to predict or calculate the "response variable" Q_T. Strictly speaking, regression equations should only be used when the predictor variables (in this case, area and wetlands) of a candidate watershed fall within the range of predictor variables in the underlying database. The limits for the Maine annual maximum database follow in Table 12-2.3.

Table 12.2-3: Limits for Peak Flow Regression Applicability

Minimum	Parameter	Maximum
1 mi^2 = 640 ac = 2.6 km^2	< A <	1660 mi^2 = 4,300 km^2
0.7	< W (%) <	27
----	< Benson Slope <	260 ft/mi = 50 m/km

<u>Determining Watershed Characteristics for Regression Calculation</u>

In general, watershed characteristics should be determined in the same way as they were determined in the original regression study (Hodgkins, 1999). Despite this general guidance, it is recognized that occasionally watershed characteristics may be determined in a different manner. These exceptional cases should be clearly noted, along with the particular methods used to determine watershed variables.

Area and Boundary Delineation: USGS topographic maps (1:24000 scale) were the sources of information for development of the regression equations and therefore these maps are the source of choice of all applications.

Wetlands: It is critical that wetlands percentage be taken from the National Wetlands Inventory (NWI) 1:24000 map series for the State of Maine (U.S. Department of Interior, various years). Wetlands should not be measured from topographic maps, aerial photos, site-specific wetlands aps, or other sources, when NWI maps are available at resolution comparable to watershed scale.

Slope: Slope is not a predictor variable in the USGS (1999) regression equations. However, it is used to determine the applicability of these equations, since they are not applicable to steep catchments (S > 250 ft/mi = 50 m/km). Watershed slope (Benson, 1962) is calculated for the "10-85" flow path. The hydraulically longest path from outlet to divide is identified (longest in the sense of requiring the most time for water to move from divide to outlet). The 10-85 segment starts at the point that is 10% (of total path length) from the outlet and ends at the 85% point.

12-2.05 Rational Method

The Rational Formula has been in use for over 150 years for calculating peak flow. While best suited to small urban drainages, it is also used for small rural catchments. Despite its simplicity, it is essentially a simple deterministic rainfall-runoff model. Pilgrim and Cordery (in Maidment, 1992) and McCuen (1989) give modern presentations of this venerable method. The general formula is

$$Q = \mu C i A$$

where Q = peak runoff (ft^3/s or m^3/s)
μ = unit conversion factor (1 for U.S. Customary; 0.28 for metric)
C = runoff coefficient (dimensionless; $0 < C < 1$)
i = rainfall intensity (in/hr or mm/hr)
A = watershed area (ac or km^2)

Runoff Coefficient

The runoff coefficient is commonly interpreted as the ratio of peak runoff output to (volumetric) rainfall rate. With this understanding, and when the Rational formula is used as a rainfall-runoff model to simulate real events, C is a deterministic watershed parameter that effectively lumps numerous physically meaningful watershed parameters. The runoff coefficient C depends on land use, cover type, slope, and rain event magnitude.

Rainfall Intensity

Rainfall intensity i is assumed uniform over the catchment and constant for the storm duration t_r. For peak flow calculation, the duration t_r in turn is set equal to the catchment time of concentration t_c (time for entire catchment to produce runoff at the outlet) because peak flow is achieved for storm durations $t_r \geq t_c$. Intensity is a random variable and is determined from location-specific intensity-duration-frequency (IDF) curves. For a specified t_c and event return period T, the IDF curve gives the design average storm intensity.

The IDF curves from the January 1994 MDOT Drainage Design Manual have been fit to the functional form

$$i = a/(t_r + c)^b$$

where

i = average rainfall intensity, (in/hr or mm/hr), for duration t_r
t_d = rainfall duration (min)
a,b,c = equation coefficients, specific by location and return period

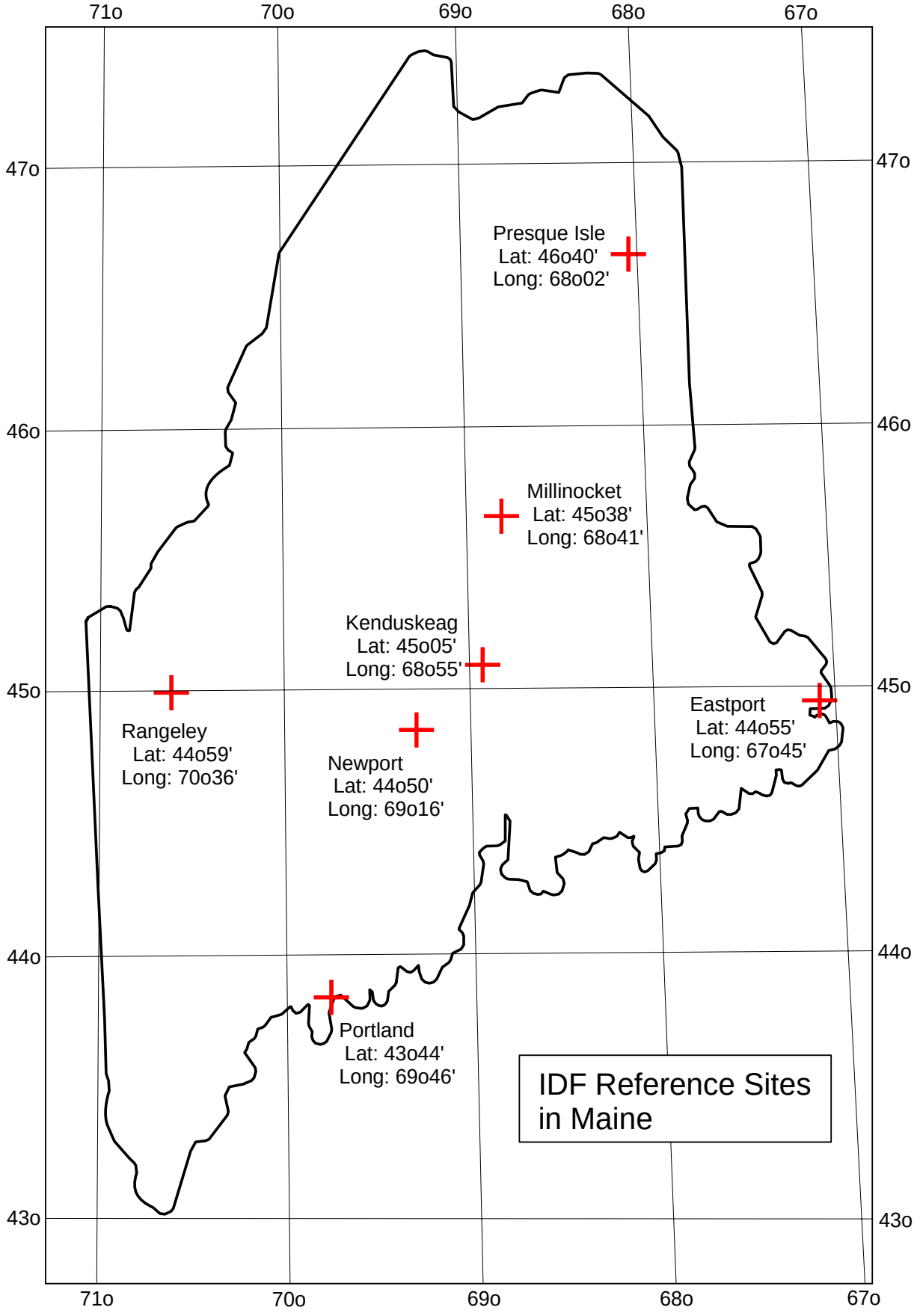
Coefficient values are summarized in Table 12-2.4 for six locations in Maine. The corresponding 10- and 50-year curves are graphed in Figures 12-2.1a 12-2.1f. Note that rainfall intensity decreases as duration increases. To obtain IDF equations in (mm/hr), multiply the "a" coefficients in Table 12-2.4 by 25.4.

Table 12-2.4: Coefficients for Maine Intensity-Duration-Frequency (IDF) Curves (U.S. Customary Units)

	Portland	Eastport	Rangeley	Presque Isle	Newport	Millinocket
2 yr - a	25.76	16.031	21.26	30.91	25.12	28.68
b	0.746	0.683	0.720	0.859	0.767	0.812
c	7.141	4.863	4.224	7.084	6.714	7.724
10 yr - a	30.82	23.77	38.96	40.04	31.66	33.49
b	0.686	0.665	0.754	0.807	0.722	0.744
c	8.133	6.466	8.208	8.357	7.158	8.172
50 yr - a	41.34	36.00	60.78	55.09	45.74	50.41
b	0.691	0.688	0.790	0.809	0.738	0.769
c	8.956	7.744	10.019	9.779	8.803	9.512
100 yr - a	47.59	41.17	82.52	62.59	48.69	44.54
b	0.698	0.691	0.826	0.810	0.726	0.721
c	9.921	8.452	12.368	10.01	8.478	7.727

Note: IDF equation is $i = a/(t_d + c)^b$, i and a in (in/hr), t_d and c in (min); a[mm/hr] = 25.4 x a[in/hr]

Figure 12-2.1a



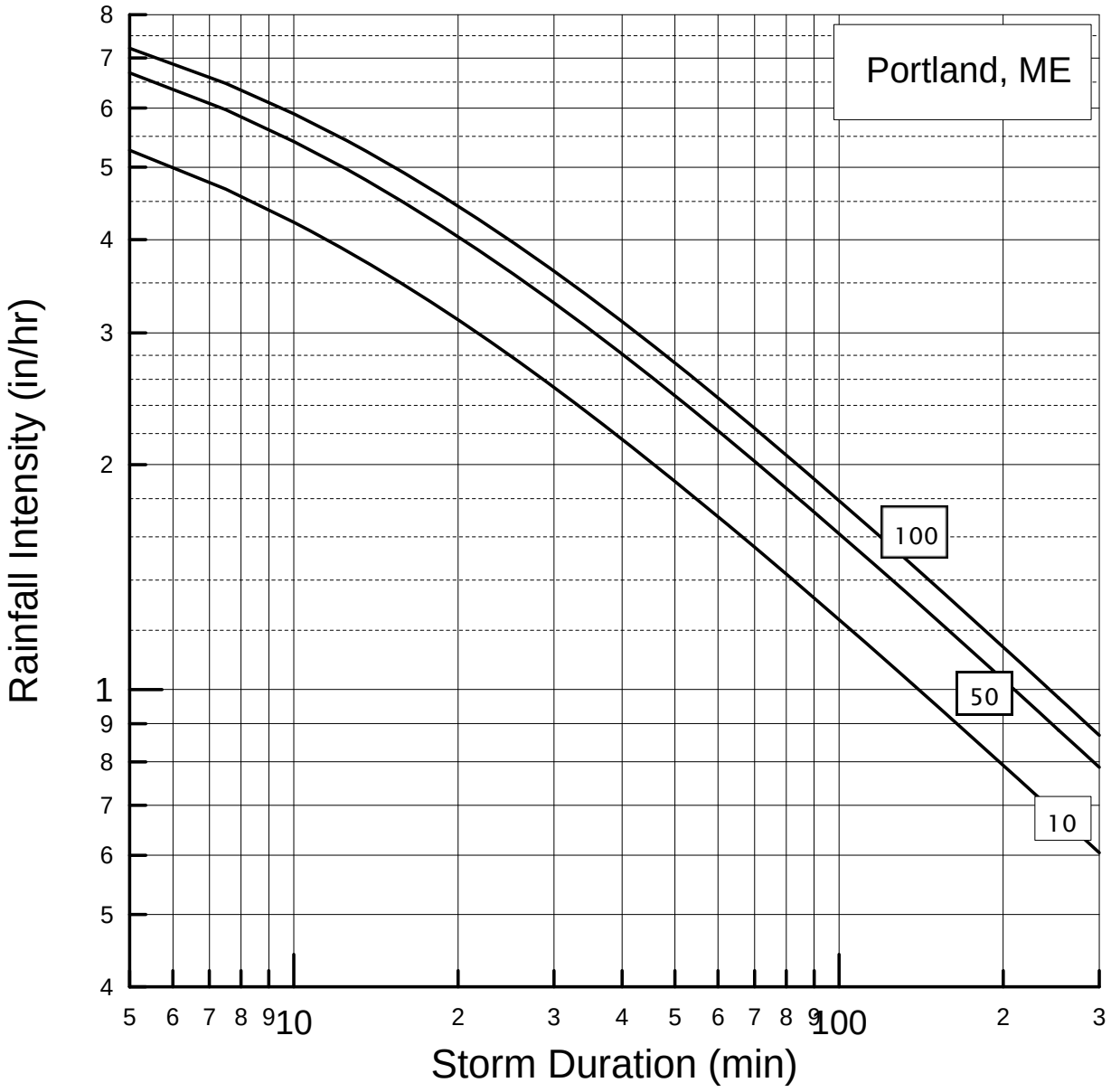


Figure 12-2.1b: Intensity-Duration-Frequency Curve, Portland, Maine (labeled return period in years).

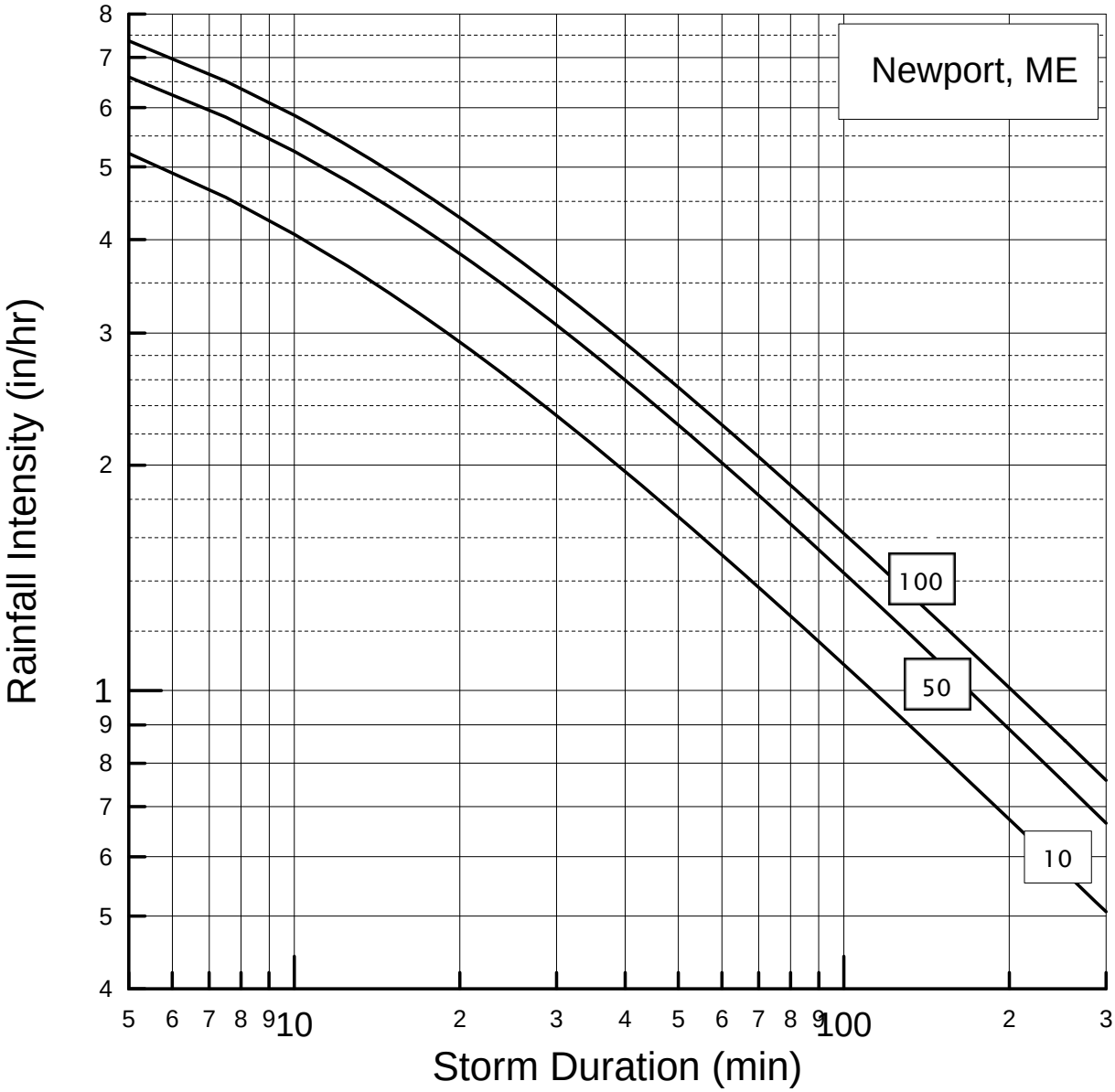


Figure 12-2.1c: Intensity-Duration-Frequency Curve, Newport, Maine (labeled return period in years).

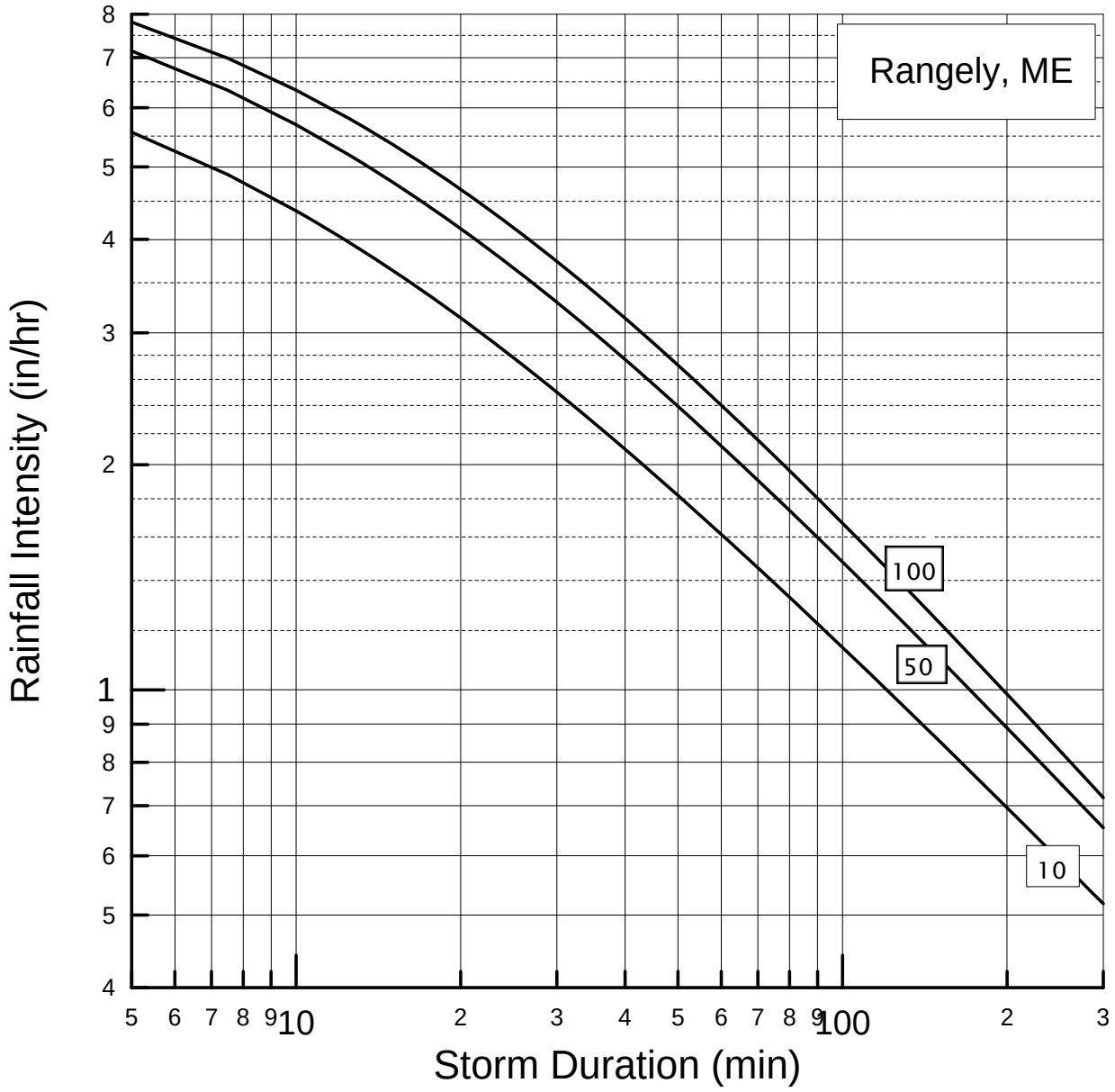


Figure 12-2.1d: Intensity-Duration-Frequency Curve, Rangely, Maine (labeled return period in years).

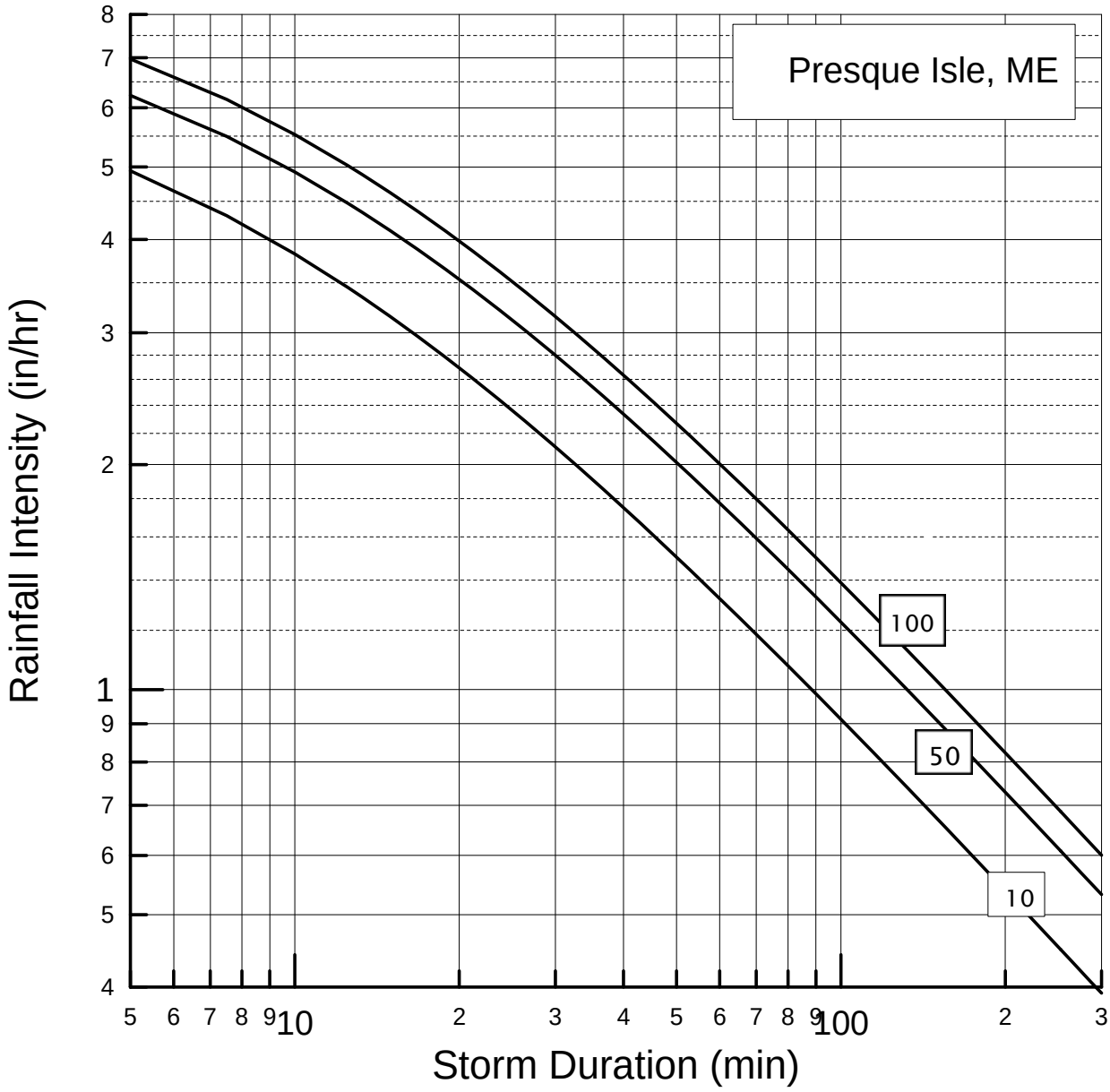


Figure 12-2.1e: Intensity-Duration-Frequency Curve, Presque Isle, Maine (labeled return period in years).

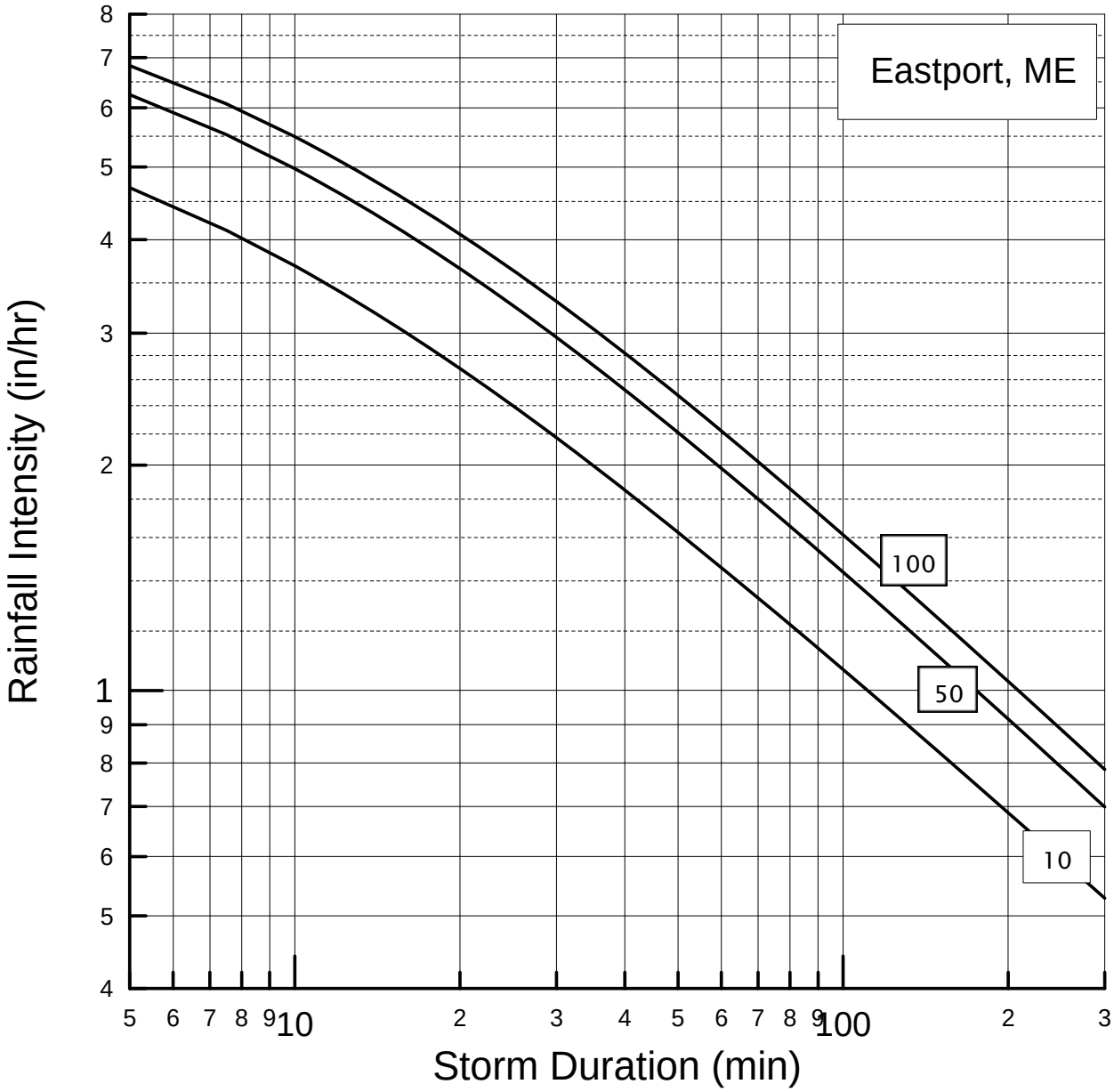


Figure 12-2.1f: Intensity-Duration-Frequency Curve, Eastport, Maine (labeled return period in years).

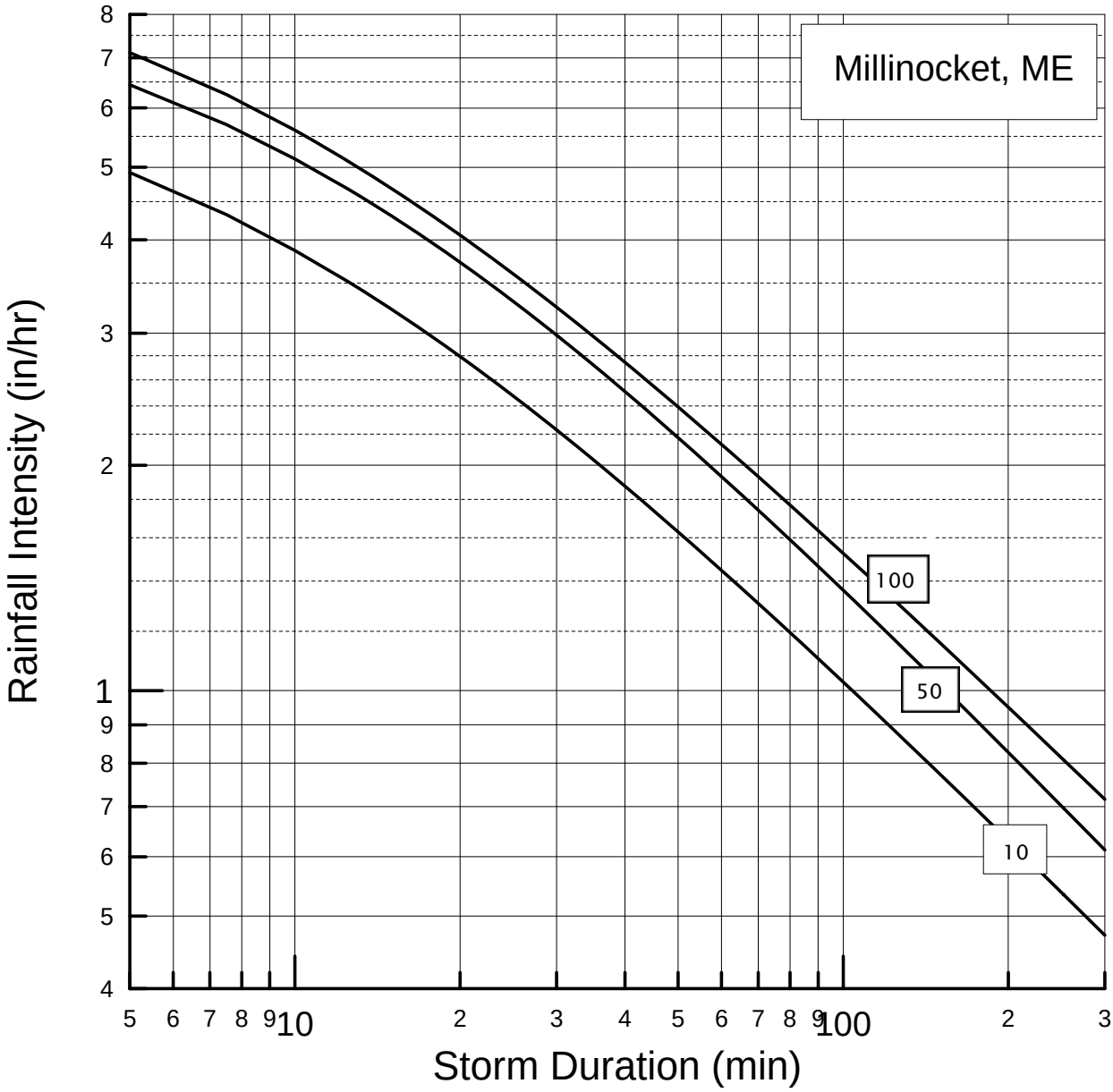


Figure 12-2.1g: Intensity-Duration-Frequency Curve, Millinocket, Maine (labeled return period in years).

Time of Concentration

For a uniform, steady rainfall, time of concentration t_c is the time for the entire watershed to produce runoff at the outlet. A physically based conceptual definition of time of concentration t_c is the time for a drop falling on the *hydraulically* most remote part of a watershed to reach the outlet. By the time this drop reaches the outlet, runoff from every other point in the watershed has also reached the outlet, and thus runoff peaks for a steady rain with duration $t_r = t_c$. Thus, the hydraulically longest flow path is that which has the longest travel time.

According to this definition, a flow path must be delineated before t_c can be calculated. A short distance over a flat, rough area may have a longer travel time than over a longer but steeper and smoother area. As a practical matter, flow paths will be determined on the basis of physical length in most cases. However, the conceptual definition should be kept in mind when unusual situations are encountered. Assumptions can always be checked by calculation.

The essence of the Rational Method (as opposed to the Rational Formula) is the decision to set design storm duration t_r equal to time of concentration t_c. Choosing an appropriate design rainfall is a matter of determining t_c: once t_c is known, intensity i follows directly from the IDF curve, by formula or look-up, with storm duration t_r equal to t_c. In highway hydrology t_c is usually taken as a deterministic watershed parameter. Smaller durations (equal to t_c in Rational Method) correspond to higher intensities for a given return period. Underestimating t_c leads to overestimating Q and thus to overdesign of hydraulic structures. Thus, a reasonable t_c value is critical to calculating reasonable design flows. Conversely, overestimating t_c leads to lower flow estimates and underdesign.

Hydraulics of t_c: The conceptual definition of t_c is at the heart of the "TR-55 approach" (TR-55 (NRCS, 1986) is a simplified version of the TR-20 rainfall-runoff event mode). This approach is recommended for MDOT hydrologic practice. A raindrop starting at the hydraulically most remote point in the watershed is assumed to follow a flow path that consists of some combination of sheet flow, shallow concentrated flow, and channel flow. All three types of flow need not be present. Manning's equation for open channel flow is typically used to calculate the travel time T_t of flow in each flow segment (the travel time is identically the time of concentration of the watershed component). The sum of component travel times gives the watershed time of concentration:

$$t_c = T_{t|sheet} + T_{t|conc} + T_{t|channel}$$

Since the calculations are hydraulics-based, physically meaningful parameters are used: roughness n, slope S, hydraulic radius (surrogate for depth of flow) R_h, and length of flow L. While these parameters are physically meaningful and are not empirical coefficients determined by regression, in practice their determination is somewhat problematic.

Velocity Method for Time of Concentration

The velocity method uses the definition of velocity to calculate travel time:

$$v = L/T_t$$
$$T_t = L/v$$

Manning's equation (in consistent units) is used to calculate velocity:

$$v = \lambda R_h^{2/3} S^{1/2}/n$$

where v = velocity (ft/s or m/s)
λ = unit conversion factor (1.486 for U.S. Customary; 1 for metric)
R_h = hydraulic radius (ft or m) = A_{flow}/P_{wet}
A_{flow} = flow cross-sectional area (ft^2 or m^2)
P_{wet} = wetted flow perimeter (ft or m)
S = slope (dimensionless; ft/ft or m/m)
n = Manning's roughness parameter (value independent of units)

For sheet flow and shallow concentrated flow, the height of roughness elements is on the same order as depth of flow. Furthermore, depth of flow is small and is difficult to measure or calculate reliably. Therefore, the hydraulic radius and roughness are often lumped into a single conveyance parameter k,

$$k = \lambda R_h^{2/3}/n \quad (ft^{2/3} \text{ or } m^{2/3})$$

so that

$$v = kS^{1/2}$$

$$T_t = L/v = \{L/S^{1/2}\}/k$$

(Note that $k\ (m^{2/3}) = k\ (ft^{2/3})/3.28$.)

The "k formulation" is typically limited to sheet flow and shallow concentrated flow; in channel flow, the hydraulic radius R_h can be estimated explicitly.

Sheet Flow: Sheet flow is flow over planar surfaces such as paved areas and fields. The standard assumption is that it occurs primarily in catchment upland areas. With sheet flow, the friction value (Manning's n) is an effective roughness coefficient that includes the effect of raindrop impact, drag over the surface, obstacles to flow (such as litter, crop ridges, rocks, etc), and erosion and transport of sediment. Appropriate n values range from 0.01 for smooth paved surfaces to 0.40 for forest floors. Flow depth (hydraulic radius R_h) should ordinarily not exceed 2 in (50 mm).

Sheet flow is often the "slowest" flow component in the TR-55 conceptual model and thus overall watershed t_c is very sensitive to the sheet flow parameters. Length of sheet flow is particularly problematic. The questionable practice of assuming a "standard" flow length is widespread but cannot be justified. A value of 300 ft (90 m) is cited in TR-55 as being an upper limit; unfortunately this value has crept into practice as a default standard. In fact, this value is probably too large in most situations, possibly justified only on flat, uniform surfaces. Large flow lengths lead to overestimation of t_c, underestimation of rainfall intensity i, underestimation of the design flow, and ultimately underdesign of the hydraulic structure. Assessment of reasonable sheet flow lengths should be part of the hydrologic site inspection.

Exact Kinematic Wave Solution for Sheet Flow: The best physically-based estimate for sheet flow (overland) flow travel time is the Kinematic Wave (KW) approach, which in turn is based on Manning's equation. The method is computationally complex and for manual calculations the approximate solution, also presented here, may be preferred. An advantage of the exact solution is that it captures the dependence of travel time on event return period; more extreme events give a smaller time of concentration.

The KW expression for travel time is derived from the general velocity method by assuming that the depth of sheet flow (essentially the hydraulic radius) is equal to the product of rain intensity and sheet flow travel time $T_{t|sheet}$ for duration $t_r \geq T_{t|sheet}$:

$$R_h = iT_{t|sheet}$$

Substituting into Manning's equation gives

$$v = \lambda(iT_{t|sheet})^{2/3}S^{1/2}/n$$

Continuing with the velocity expression for sheet flow travel time and solving for t_c,

$$T_{t|sheet} = L/v = \{nL/\lambda S^{1/2}\}(iT_{t|sheet})^{2/3}$$

$$T_{t|sheet} = \{nL/\lambda S^{0.5}\}^{0.6}/i^{0.4}$$

where all physical quantities are in consistent units of (ft) or (m) and (s).

For the familiar units of (in/hr) for i and (min) for $T_{t|sheet}$, the equation becomes

$$T_{t|sheet} = 0.94\{nL/S^{0.5}\}^{0.6}/i^{0.4}$$

For the metric units of (mm/hr) for i and (min) for $T_{t|sheet}$, the coefficient 0.94 is replaced by 7.

Note that the factor $0.94\{nL/S^{0.5}\}^{0.6}$ and (and $7\{nL/S^{0.5}\}^{0.6}$ in metric) are characteristic of a specified sheet flow path and as such are watershed parameters.

This is a non-linear equation and requires iterative solution for $T_{t|sheet}$ because intensity i is a function of $T_{t|sheet}$ (equal to rainfall duration) on the IDF curve. This equation is easily solved using standard root-finding methods in a specially prepared spreadsheet (including MaineDOT hydrology EXCEL applications). Manual calculation by "Picard iteration" (successive approximation, a form of repetitive calculation) is also simple, though tedious for more than a few watersheds. A starting value of travel time is assumed, say $T_{t|sheet}$ = 10 min, along with a corresponding i value by calculation or look-up. Then a new $T_{t|sheet}$ value is calculated using the KW equation above. This $T_{t|sheet}$ value is used in turn to determine (by IDF calculation or chart look-up) a new i value. The process is repeated until the i and $T_{t|sheet}$ values converge to steady values.

Example: Determine sheet flow travel time by Kinematic Wave model for a flow path with L = 100 ft and S = 0.01 on rough grass in the Portland area.

For rough grass, n = 0.4. Employ Picard iteration (successive approximation) to calculate $T_{t|sheet}$. Assume a starting value of $T_{t|sheet;0}$ = 5 min.

Sheet flow segment constant: $0.94\{nL/S^{0.5}\}^{0.6} = 0.94\{0.4 \times 100/0.01^{0.5}\}^{0.6} = 34.23$
Portland 10-year IDF curve: $i = 30.82/(t+8.133)^{0.686}$

Iteration	$T_{t\|sheet} = 34.23/i^{0.4}$ (min)	i (in/hr) by calculation or look-up
0	5	5.27
1	$34.23/5.27^{0.4} = 17.61$	3.32
2	$34.23/3.32^{0.4} = 21.18$	3.04
3	$34.23/3.04^{0.4} = 21.95$	2.98
4	$34.23/2.98^{0.4} = 22.11$	2.97
5	$34.23/2.97^{0.4} = 22.14$	2.97

For this sheet flow path, the final estimate is a travel time of 22.1 min.

Approximate Kinematic Wave Solution for Sheet Flow: The approximate KW solution is

$$T_t = (0.42/P_2^{1/2})(nL/S^{1/2})^{4/5}$$

where P_2 = 2-yr 24-hr rainfall depth (in; see Table 12-2.8).

This equation is suitable for manual calculations. The difference between the exact and approximate solutions becomes more pronounced as the design return period gets larger.

Shallow Concentrated Flow: Shallow concentrated flow commences at the point where sheet flow is too inefficient to transport the volume of water originating upstream of that point. Shallow concentrated flow is visualized as occurring in numerous and closely spaced small channels and rivulets. Depth of flow is still small but Manning's "n" is smaller than in sheet flow, making for faster flow velocity and smaller travel time T_t as compared to sheet flow. In agricultural areas tillage will control the direction of flow, in which case aerial photos and site inspections are essential. Table 12-2.7 can be used to estimate R_h and n; values of 2 in (50 mm) and 0.05, respectively, are reasonable starting estimates.

Channel Flow: Open channels are assumed to begin where surveyed cross section information has been obtained, where channels and singular drainage features are visible on aerial photos, or where blue lines (indicating streams) appear on USGS quadrangle sheets. In general, the use of aerial photos will result in longer channels than USGS topographic maps alone. Both Manning's roughness and hydraulic radius are channel-specific and should be based on actual observation and measurement. Standard practice in hydrologic studies is to base hydraulic radius on bank-full conditions (approximate 2-year event), even though the 50-year design event will be over the bank. Hydraulic analysis can be used to refine R_h is channel and overbank geometry data are available. Roadside ditches should be treated as open channels.

Watershed Area

Area is a straightforward parameter and requires little in the way of interpretation. The biggest complication in highway design work is that the small watersheds suitable for Rational method typically extend beyond the project boundary and thus off the project plans and topography. Yet they can also be too small for reliable delineation from standard USGS 7.5 minute topography, so watershed delineation is often based on aerial photo interpretation. Field checking of delineations is particularly critical for flat watersheds. Also, ditching and crosspipes can significantly effect delineation and flow paths, as compared to simple delineation based on topography alone.

Rational Method Parameter Values

Tables 12-2.5 – 12-2.7 are taken from McCuen (1989) and provide guidance in choosing parameter values for use in Rational Method calculations. Table 12-2.5 correlates C values to land use, slope, and Natural Resources Conservation Service (NRCS; formerly Soil Conservation Service (SCS)) hydrologic soil group. Hydrologic soil group is a soil classification system developed by NRCS based on a soil's long-term infiltration rate, in turn a function of soil composition, depth, and slope. Table 12-2.5 should be used as the primary reference for C values, in order to preserve some consistency between application of Rational Method and NRCS methods (TR-55 and TR-20). A standard source for determining soil types on a project is the series of NRCS county soil surveys; site-specific soil surveys may also be available. Table 12-2.5 is notable for its treatment of C as a random variable, dependent on event return period. This is more realistic than the usual assumption that C (and runoff curve number) is a fixed, deterministic watershed parameter. Table 12-2.6 that follows is a compendium of C values by land use only and is useful as a further check on values used in analysis. NRCS models utilize

an analogous parameter, the runoff curve number (RCN). Values are not tabulated here; the reader is referred to standard references.

Table 12-2.7 lists representative values of hydraulic radius R_h, Manning's roughness n, and composite factor k for land uses and flow regimes encountered in Rational Method application. Flow depth and R_h are essentially identical in sheet flow. This equivalence weakens as flow channelizes, in which case the definition of hydraulic radius ($R_h = A/P$) must be utilized for real flow channels. For shallow concentrated flow it is sufficient to assume R_h on the order of 2 in (50 mm). The travel times for shallow concentrated flow and channel flow are generally much shorter than for sheet flow, so that the overall watershed t_c, intensity i, and resultant Q are relatively insensitive to these components. In cases where all three flow components have roughly equal travel times, sensitivity analysis should be performed on the different parameters.

As noted above, sheet flow travel time is particularly sensitive to length of flow. The associated slope is also important. On typical hillslope profiles, sheet flow originating at the watershed divide will often have a slope less than in the mid-slope region. These distinctions should not be lost. Channel flow length is fairly easy to identify from topographic maps and aerial photos. Shallow concentrated flow is difficult to identify directly and is often treated as what is "left over" after sheet flow and channel flow paths have been identified. Travel times in shallow concentrated flow and channel flow are usually much shorter than sheet flow, so travel times are not very sensitive to channelized and concentrated flow lengths.

Table 12-2.5: Runoff Coefficients for Rational Formula by Soil Type and Slope (McCuen, 1989)

	Hydrologic Soil Group											
	A			B			C			D		
Land Use	0-2%	2-6%	>6%	0-2%	2-6%	>6%	0-2%	2-6%	>6%	0-2%	2-6%	>6%
Cultivated	0.08	0.13	0.16	0.11	.015	0.21	0.14	0.19	0.26	0.18	0.23	0.31
Land	0.14	0.18	0.22	0.16	0.21	0.28	0.20	0.25	0.34	0.24	0.29	0.41
Pasture	0.12	0.20	0.30	0.18	0.28	0.37	0.24	0.34	0.44	0.30	0.40	0.50
	0.15	0.25	0.37	0.23	0.34	0.45	0.30	0.42	0.52	0.37	0.50	0.62
Meadow	0.10	0.16	0.25	0.14	0.22	0.30	0.20	0.28	0.36	0.24	0.30	0.40
	0.14	0.22	0.30	0.20	0.28	0.37	0.26	0.35	0.44	0.30	0.40	0.50
Forest	0.05	0.08	0.11	0.08	0.11	0.14	0.10	0.13	0.13	0.12	0.16	0.20
	0.08	0.11	0.14	0.10	0.14	0.18	0.12	0.16	0.20	0.15	0.20	0.25
Residential	0.25	0.28	0.31	0.27	0.30	0.35	0.30	0.33	0.38	0.33	0.36	0.42
1/8 ac lot	0.33	0.37	0.40	0.35	0.39	0.44	0.38	0.42	0.49	0.41	0.45	0.54
Residential	0.22	0.26	0.29	0.24	0.29	0.33	0.27	0.31	0.36	0.30	0.34	0.40
¼ ac lot	0.30	0.34	0.37	0.33	0.37	0.42	0.36	0.40	0.47	0.38	0.42	0.52
Residential	0.19	0.23	0.26	0.22	0.26	0.30	0.25	0.29	0.34	0.28	0.32	0.29
1/3 ac lot	0.28	0.32	0.35	0.30	0.35	0.39	0.33	0.38	0.45	0.39	0.40	0.50
Residential	0.16	0.20	0.24	0.19	0.23	0.28	0.22	0.27	0.32	0.26	0.30	0.37
½ ac lot	0.25	0.29	0.32	0.28	0.32	0.36	0.31	0.35	0.42	0.34	0.38	0.48
Residential	0.14	0.19	0.22	0.17	0.21	0.26	0.20	0.25	0.31	0.24	0.29	0.35
1 ac lot	0.22	0.26	0.29	0.24	0.28	0.34	0.28	0.32	0.40	0.31	0.35	0.46
Industrial	0.67	0.68	0.68	0.68	.068	0.69	0.68	0.69	0.69	0.69	0.69	0.70
	0.85	0.85	0.86	0.85	0.86	0.86	0.86	0.86	0.87	0.86	0.86	0.88
Commercial	0.71	0.71	0.72	0.71	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72
	0.88	0.88	0.89	0.89	0.89	0.89	0.89	0.89	0.90	0.89	0.89	0.90
Streets	0.70	0.71	0.72	0.71	0.72	0.74	0.72	0.73	0.76	0.73	0.75	0.78
	0.76	0.77	0.79	0.80	0.82	0.84	0.84	0.85	0.89	0.89	0.91	0.95
Open Space	0.05	0.10	0.14	0.08	0.13	0.19	0.12	0.17	0.24	0.16	0.21	0.28
	0.11	0.16	0.20	0.14	0.19	0.26	0.18	0.23	0.32	0.22	0.27	0.39
Parking	0.85	0.86	0.87	0.85	0.86	0.87	0.85	0.86	0.87	0.85	0.86	0.87
	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97

[1] for each land use, first row is for return period T < 25 yrs, second row is for T $\geq$ 25 yrs.

Table 12-2.6: Runoff Coefficients for the Rational Formula by Land Use (McCuen, 1989)

Land Use	Range of Runoff Coefficients "C"	Recommended Value
Business		
Downtown	0.70 - 0.95	0.85
Neighborhood	0.50 - 0.70	0.60
Residential		
Single-Family	0.30 - 0.50	0.40
Multiunits, detached	0.40 - 0.60	0.50
Multiunits, attached	0.60 - 0.75	0.70
Residential (suburban)	0.25 - 0.40	0.35
Apartment	0.50 - 0.70	0.60
Industrial		
Light	0.50 - 0.80	0.65
Heavy	0.60 - 0.90	0.75
Parks, cemeteries	0.10 - 0.25	0.20
Playgrounds	0.20 - 0.35	0.30
Railroad yard	0.20 - 0.35	0.30
Unimproved	0.10 - 0.30	0.20
Pavement		
Asphaltic and Concrete	0.70 - 0.95	0.85
Brick	0.75 - 0.85	0.80
Roofs	0.75 - 0.95	0.85
Lawns, sandy soil		
Flat, < 2%	0.05 - 0.10	0.08
Average, 2-7%	0.10 - 0.15	0.13
Steep, >7%	0.15 - 0.20	0.18
Lawns, heavy soil		
Flat, <2%	0.13 - 0.17	0.15
Average, 2-7%	0.18 - 0.22	0.20
Steep, > 7%	0.25 - 0.35	0.30

Table 12-2.7: Typical Manning's "n" and Hydraulic Radius Values (McCuen, 1989)

Land Use/Flow Regime	Manning's n	Hydraulic Radius R_h (in)	k ($m^{2/3}$) = $R_h^{2/3}/n$	k ($ft^{2/3}$) = $1.486R_h^{2/3}/n$
Forest				
Light underbrush	0.4	2.64	0.41	1.4
Heavy ground litter	0.2	2.40	0.77	2.5
Grass				
Bermudagrass	0.41	1.80	0.31	1.0
Dense (Lawns, Playing fields)	0.24	1.44	0.46	1.5
Short	0.15	1.20	0.65	2.1
Short grass pasture	0.025	0.48	2.12	7.0
Conventional tillage				
With residue	0.19	0.72	0.37	1.2
No residue	0.09	0.60	0.67	2.2
Agricultural				
Cultivated straight row	0.04	1.44	2.75	9.1
Contour or strip crop	0.05	0.72	1.39	4.6
Trash fallow	0.045	0.60	1.36	4.5
Rangeland	0.13	0.48	0.41	1.3
Alluvial fans	0.017	0.48	3.11	10.3
Grassed waterways	0.095	12.00	4.77	15.7
Small upland gullies	0.04	6.00	7.13	23.5
Pavement (smooth; sheet flow)	0.011	0.72	6.30	20.8
Pavement (rubble; sheet flow)	0.025	2.40	6.19	20.4
Paved gutter	0.011	2.40	14.07	46.3

Note: $k(m^{2/3}) = k(ft^{2/3})/3.28$

12-2.06 Rainfall-Runoff Modeling

On occasion the standard methods of Rational Method and Regression Model will be insufficient and rainfall-runoff modeling will be preferred. "Modeling" is a shorthand expression for event-based models such as TR-20, HEC-1, or similar models. HydroCad is a proprietary model based on TR-20 hydrology that is easy to use and is particularly popular within the consulting community. Modeling is commonly used in the consulting community, less so within MDOT. Modeling issues, including review of consultant work and execution of in-house models, should be referred to the MDOT Hydrology Section.

The following situations might warrant the use of modeling:

1) complete hydrograph is needed, as in flow routing, storage modeling, and detention storage design
2) watershed is between area limits for Rational and regression methods, and results from those methods are problematic
3) watershed properties are highly variable in space, thus make choice of Rational "C" coefficient difficult
4) watershed displays significant storage characteristics that cannot be captured by the Rational Method
5) outlet point of interest integrates numerous definable subwatersheds of varying characteristics

Modern models (e.g. HydroCad) are probably as easy to implement as Rational Method. As long as the particular merits and shortcomings of a method are recognized, MDOT will accept drainage studies based on any the commonly recognized models and methods. Internal preference will continue to be given to Rational Method.

TR-20 hydrology does not utilize rainfall intensities from IDF curves. Rather, storms are usually constructed from 24-hour cumulative depths according to four different temporal distributions, or "storm types". The Type III storm should generally be used in coastal Maine areas and Type II elsewhere. Table 12-2.8, taken from the Maine DEP Stormwater BMP Guide (1995), gives accepted 24-hour depths for different return periods along with appropriate storm type.

In addition to the design storm, TR-20 includes another climate-related parameter, the antecedent moisture condition (AMC). AMC relates to soil moisture condition, rainfall preceding the design event, and possibly temperature. Unless documented for a specific reason, "average conditions" (AMC II) should be employed.

Table 12-2.8: 24-Hour Duration Rainfall Depths (inches) for Various Return Periods

	Return Period (years)								Annual	Comments
Location	1	2	5	10	25	50	100	500		
Androscoggin	2.5	3.0	3.9	4.6	5.4	6.0	6.5	7.8	45.3	
Aroostook C	2.1	2.1	3.2	3.6	4.2	4.6	5.0	5.9	36.1	Presque Isle
Aroostook N	2.0	2.3	3.0	3.5	4.0	4.4	4.8	5.7	36.1	Ft Kent
Aroostook S	2.2	2.5	3.3	3.8	4.4	4.9	5.3	6.4	39.0	Houlton
Cumberland NW	2.8	3.3	4.3	5.0	5.8	6.4	6.9	8.3	43.4	NW of Rt 11
Cumberland SE	2.5	3.0	4.0	4.7	5.5	6.1	6.7	8.1	44.4	SE of Rt 11
Franklin	2.4	2.9	3.7	4.2	4.9	5.4	5.9	7.0	45.6	
Hancock	2.4	2.7	3.6	4.2	4.9	5.5	6.0	7.2	45.2	
Kennebec	2.4	3.0	3.8	4.4	5.1	5.6	6.1	7.2	41.7	
Knox-Lincoln	2.5	2.9	3.8	4.4	5.1	5.7	6.2	7.4	46.1	
Oxford E	2.5	3.0	4.0	4.6	5.3	5.9	6.4	7.6	43.0	E of Rt 26
Oxford W	3.0	3.5	4.5	5.2	6.0	6.6	7.1	8.4	43.8	W of Rt 26
Penobscot N	2.2	2.5	3.3	3.8	4.4	4.9	5.4	6.4	41.5	N of Can-Atl RR
Penobscot S	2.4	2.7	3.5	4.1	4.8	5.3	5.8	6.9	39.5	S of Can-Atl RR
Piscataquis N	2.2	2.5	3.3	3.8	4.4	4.9	5.3	6.3	38.5	N of Can-Atl RR
Piscataquis S	2.3	2.6	3.4	4.0	4.6	5.1	5.5	6.6	41.0	S of Can-Atl RR
Sagadahoc	2.5	3.0	3.9	4.6	5.4	5.9	6.5	7.8	45.3	
Somerset N	2.2	2.5	3.3	3.8	4.4	4.9	5.3	6.3	37.3	N of Can-Atl RR
Somerset S	2.4	2.7	3.5	4.1	4.7	5.2	5.7	6.8	39.5	S of Can-Atl RR
Waldo	2.5	2.8	3.7	4.3	4.9	5.5	6.0	7.1	47.2	
Washington	2.4	2.5	3.4	4.0	4.8	5.4	5.9	7.1	44.2	
York	2.5	3.0	4.0	4.6	5.4	6.0	6.6	7.8	46.7	

Source: Maine DEP Stormwater BMP Guide, November, 1995.

Note 1: Use Type II Storm for Oxford and Penobscot Counties, excepting towns listed below.

Note 2: Use Type III Storm for all other counties and the following towns in Oxford County (Porter, Brownfield, Hiram, Denmark, Oxford, Hebron, Buckfield, Hartford) and Penobscot County (Dixmont, Newburgh, Hampden, Bangor, Veazie, Orono, Bradley, Clifton, Eddington, Holden, Brewer, Orrington, Plymouth, Etna, Carmel, Hermon, Glenburn, Old Town, Milford, Greenfield).

Note 3: 50-yr depths approximated as mid-point between 25- and 100-yr depths based on log-Normal probability plots.

12-2.07 Calibration and Observation-Based Estimates

Rainfall and runoff data are rarely available for calibration of highway drainage calculations and simulations. Very often the best available site-specific data consists of anecdotal information about high water marks, inundation levels, and their relative frequency. Such information is not suitable for numerical calibration of calculations, but it can be used to judge the reasonableness of the calculations. Every effort should be made to identify specific problems areas prone to flooding and to collect anecdotal information about that flooding. This information can be used with backwater analysis and culvert analysis to independently estimate extreme flows. In the case of flood estimates at locations with existing culverts, the hydrologist should compare calculations against the existing pipe capacity in the context of the pipe history. If calculations indicate flooding while experience suggests acceptable pipe performance, the hydrologist should exercise and document professional judgment in recommending a final design peak flow.

12-2.08 Combining Estimates from Different Methods

Rational and USGS Estimates: Area Weighting

In many projects estimates will be determined by these two methods only. The problem arises for areas in the intermediate range 320 ac < A < 640 ac (0.5 mi^2 < A < 1 mi^2; 1.3 km^2 < A < 2.6 km^2). Barring unusual circumstances, Rational estimates will be used for A < 320 ac and regression estimates will be used for A > 1 mi^2.

Experience and training should provide the basis for determining a single final design value. However, simple area weighting can be used to point towards a final value.

$$Q_T = wQ_{T,U} + (1-w)Q_{T,R} \quad , \quad 320 \text{ ac} < A < 640 \text{ ac}$$

Where $w = (A-320)/320$ (area weight; A = watershed area in ac)

$Q_{T,U}$ = estimate by USGS regression

$Q_{T,R}$ = estimate by Rational method

By this equation, the estimates for smaller watersheds (closer to 320 ac) will be weighted towards the Rational method while the larger watershed (closer to 640 ac) estimates will be weighted towards the regression estimate.

12-2.09 PC-Based Computer Applications

It is expected that most highway drainage calculations will be executed on a PC, using either

- MDOT-developed spreadsheet applications for standard calculations
- dedicated modeling software such as TR-20, HydroCAD, HEC-RAS, HEC-HMS, and HY8
- Computational tools developed independently

HEC-HMS and TR-20 both contain a full implementation of NRCS TR-20 hydrology. However, they are less easy to use in engineering applications than HydroCAD because they do not contain a full range of the engineered hydraulic structures (e.g., culverts, weirs, orifices, etc) commonly utilized in engineering hydrologic design, whereas HydroCAD implements a wide variety of structures directly in the model. Instead, with HEC-HMS and TR-20, structure-specific stage-discharge relations must prepared outside the models and imported.

MDOT will accept analyses completed with recognized hydrology and hydraulic software, though the use of freely available public domain software is encouraged. All data files and spreadsheet files should be submitted in digital format with a hydrology report. While independently developed tools may be accepted, MDOT reserves the right to require use of standard models and MDOT spreadsheets.

12-2.10 Documentation of Hydrologic Studies

This guidance is directed towards documentation of TR20-type rainfall-runoff modeling studies. It is also generally applicable to other methods of hydrologic analysis, though it may differ in details. Analysis by Rational Method or regression equations may require less detail. The intent of this guidance is to encourage hydrology reports that can be efficiently reviewed for compliance with MDOT drainage and design policy. Expected benefits of this effort are a more uniform standard of reporting and a higher standard of self-review by consultants prior to submittal to MDOT. MDOT understands that individual reports may vary somewhat from this suggested outline. The following describes the format and minimum information acceptable to MDOT for review and action.

Supervision: Hydrologic studies should be executed under the direct supervision and involvement of a Maine Professional Engineer with demonstrated education and competence in hydrology and hydraulics. The engineer or hydrologist in responsible charge should sign the report cover letter, thereby declaring mastery of and responsibility for the contents of the report.

Report Checklist: A checklist of report format and contents follows this guidance as Table 12-2.9. This checklist should be completed and inserted immediately following the report title page.

Executive Summary: A one-page executive summary of the analysis, major results, and design implications should be prepared.

Introduction: The report proper should begin with a brief description of the project and an explanation of why a hydrologic analysis is being performed. Significant hydrologic features of the project should be noted. If site hydrology is to be changed in any way, these changes should be summarized. All individuals contributing to the analysis should be identified by name and title, along with their roles in the analysis.

Hydrologic Site Description: The site should be described from a hydrologic perspective. Drainage features, topography, soils, geology, surface water, and ground water should be noted as appropriate to the analysis. Existing hydrologic conditions and expected changes, in the absence of any mitigation measures, should be discussed. The need for and effects of mitigation should also be described. The narrative description should be supported by clear and informative figures and tables.

Figures are intended to bring out the important hydrologic features only and therefore should not contain extraneous design information pertinent only to other aspects of the project. Figures should be 8.5” x 11” or 11” x 17”, unless otherwise instructed. At a minimum, figures should consist of

- site located on section from USGS quad map. PC-based mapping software is acceptable for this purpose.
- site map showing soils distribution, watershed delineation, drainage features, and topography
- site map showing land use (including wetlands), watershed delineation, and drainage features.
- site map showing watershed delineation and principal flow paths for time of concentration determination

All site maps should be to the same scale. In the event that site hydrology is to be changed (e.g., paving, grading, etc), these changes should be described fully and separate site maps should be prepared for pre- and post-development hydrologic configurations.

Aerial photos should be included if they are used to develop the hydrologic model. They should be presented as photocopies to the same scale as the site maps, with watersheds clearly delineated.

Other photos can be included if they contribute to understanding the site hydrology and model development.

Tables should summarize all quantitative information derived from maps, as well as time of concentration calculations and rainfall design event data.

Map data to be listed should include at least the following:

- watershed areas
- land uses, soil types, and curve numbers/rational coefficients by area within watersheds
- land use and soil type total areas.

A separate table summarizing time of concentration calculations should also be included. This table should include all parameters used in the calculations and the method of calculation.

Rainfall design event data can be included in tabular form or as a probability plot. The data source should be identified.

Conceptual Model: The conceptual hydrologic model should be described in enough detail so that understanding the model implementation follows directly. Features in the conceptual model should be related clearly to the physical representation in the previous section. This documentation will consist of narrative, tables, and figures. The narrative should justify the proposed conceptual model for the physical watershed described previously. Significant approximations, assumptions, and weaknesses in the conceptual model should be clearly noted.

Figures should be based on the site map and clearly show model components such as watersheds, channels, and ponds and other storage features. A model process flow chart should also be included, showing all model elements and the manner in which they are connected.

Tables should summarize all model parameters associated with individual model elements.

Model Calibration: The manner in which the model is calibrated should be described. Calibration results should be summarized in tables, as appropriate. Sensitivity to model parameter values should be discussed. It is recognized that in most hydrology studies, calibration data are unavailable. At a minimum, anecdotal information regarding historical experience should be gathered from MDOT maintenance personnel, local officials, and area residents. This information can be used to at least establish the reasonableness of simulation results.

Model Results and Interpretation: A brief narrative of model results should be given. Quantitative results should be communicated primarily by tables and figures. A more fully developed narrative should be devoted to interpretation of the results. This interpretation should clearly identify implications for project design and compliance with MDOT drainage policy.

Appendices: Model input files and spreadsheet files (e.g., Excel) should be supplied on CD-ROM. Input file hard (paper) copy should also be included in the appendices. Additional

information (e.g., large format figures) may also be included if it contributes to the completeness and usefulness of the report.

Submittal, Review and Acceptance: Hydrology studies should be submitted directly to the designated MDOT project contact person. The report should be submitted in paper hardcopy, original software native format (e.g., MS-Word [DOC] file) and Adobe [PDF] digital file formats. Hand-written papers should be scanned and included in the electronic submittal. Large format figures should be submitted as paper copy and Adobe [PDF] files for half-size (11” x 17”) printing. Project contact will forward the report for review as needed. MDOT may contact the engineer or hydrologist for further clarification of hydrology and drainage issues before the report is finally accepted. Consultants are encouraged to contact MDOT Hydrology Section with questions regarding MDOT hydrology practices.

Table 12-2.9: Checklist for Hydrology Reports

Project PIN: _____
Project Location: _____

Check	Item	Comments
	Checklist	Completed
	Statement of Supervision	In cover letter
	Executive Summary	
	Introduction	
	Hydrologic Site Description	
	Supporting Documentation	
	Figures for Site Description	8.5x11 or 11x17; to same scale
	Site location	On USGS quad map
	Site map	Soils, watershed bounds, drainage features, topo
	Site map	Land use, watershed bounds, drainage features
	Site map	Watershed bounds, principal flow paths, t_c info
	Aerial photos	If used in analysis; marked appropriately
	Other photos	If informative; annotate or describe
	Tables for Site Description	
	Land uses, soil types, C/RCN by area within watersheds	
	Land use, soil types total areas	
	Time of concentration calcs	
	Model Description	
	Supporting Documentation	
	Figures for Model	Show model components with basic site info
	Tables for Model	Summarize model parameters
	Model Calibration	Sensitivity analysis, anecdotal info
	Results &Interpretation	Identify implications for design & stormwater control
	Appendices	Model input files printout; all computer files on digital media
	Submittal	Paper report; all computer files in native formats and as [PDF] files on CD

12-3 OPEN CHANNELS AND DITCHES

12-3.01 Types

The Department uses the following types of open channels and ditches for drainage:

1) Roadside Ditches: These are used adjacent to and parallel with the highway. They remove the storm runoff from the highway section and drain the subbase material.
2) Median Ditches: These are relatively shallow, depressed areas in medians of multilane, divided highways.
3) Berm Ditches: These are provided longitudinally at the top of a cut to intercept runoff from the hillside.
4) Channels: In general, these refer to naturally occurring pathways for water (e.g., stream channels). Hydraulic design of channels will likely require environmental permitting. Therefore, work in natural channels should always be coordinated with the Environmental Office. The Hydrology and Water Resources Units should be consulted early in the project, before design work begins. Channel design should be executed by a Professional Engineer and/or qualified geomorphologist.

12-3.01 Hydraulic Design

Unless there is a reason to suspect a problem, the designer will not ordinarily perform a hydraulic analysis of ditches. However, channel re-routing should always trigger an analysis. See item (4) above. The following FHWA references may prove useful if detailed analysis and design are required:

Reference: *FHWA Hydraulic Design Series (HDS) # 3*
Design Charts for Open Channel Flow
http://www.fhwa.dot.gov/bridge/hydpub.htm#hds

FHWA Hydraulic Design Series (HDS) #4
Introduction to Highway Hydraulics
http://www.fhwa.dot.gov/bridge/hydpub.htm#hds

12-3.02 Design Criteria

Ditches are employed to provide both stormwater and subbase (shallow ground water) drainage. Ditches and channels will be designed according to the following criteria.

Cross-Section

Chapters 7, 8 and 11 provide the Department's criteria for the shape and dimensions of roadside and median ditches. This includes the front slope, backslope, ditch width and ditch depth. Experience has shown that the standard dimensions generally provide protection against extreme events and therefore ditches are not usually sized explicitly for project-specific water quantity considerations (hydrology and hydraulics). The cross-section of the ditch will be based on the functional classification of the highway and project scope of work. The cross-section of a re-routed channel will be based on hydraulic analysis and geomorphologic considerations. See item (4) above.

Subbase drainage can only be achieved if the ditch is of adequate depth. Occasionally, side ditches cannot be constructed to design standards because the required backslope is too long and will impact abutting property owners. In such cases, a shallow ditch may employed in conjunction with underdrain. The capacity of the shallow ditch should be checked against the design event by hydraulic analysis.

Minimum Gradients

The desirable minimum gradient is 1.0%. The grade should not be less than 0.5%.

Channel/Ditch Lining

While ditches and channels are generally not subject to technical analysis, they should always be reviewed for performance in protecting against erosion and sedimentation (water quality). Designers are referred to the Maine DOT Best Management Practices (BMP) Guide for design guidance. Several FHWA documents provide useful technical back-up to the BMP guide.

References: *Maine DOT Best Management Practices Guide*
http://www.maine.gov/mdot/environmental-office-homepage/surface-water-resources.php

FHWA Hydraulic Engineering Circular (HEC) #11
Design of Riprap Revetment
http://www.fhwa.dot.gov/bridge/hydpub.htm#hec

FHWA Hydraulic Engineering Circular (HEC) #15
Drainage of Roadside Channels with Flexible Linings
http://www.fhwa.dot.gov/bridge/hydpub.htm#hec

12-4 CULVERTS

12-4.01 Physical Characteristics

Culvert Materials

For pipe culverts, there are two options:

1) Option I: Any one of the following may be used:
   a) corrugated steel, metallic (zinc or aluminum) coated pipe;
   b) reinforced concrete pipe; or
   c) any metal pipe allowed under Option III.

2) Option III: Any one of the following may be used:
   a) fiber-bonded corrugated steel pipe;
   b) corrugated aluminum alloy pipe;
   c) polyvinylchloride (PVC) pipe (12 in (300 mm) and 15 in (375 mm) diameters only);
   d) polymer-precoated galvanized corrugated steel pipe; or
   e) reinforced concrete pipe.

The types of culvert material recommended for various site locations are:

1) Reinforced Concrete Pipe (RCP): Use at guardrail locations and other locations at the designer's discretion. Deep fills and high-use roads may be candidates for concrete pipe.
2) Aluminum Alloy Pipe: Use in salt water areas.
3) Flexible Pipe: use where soils are susceptible to settlement. Flexible pipe is everything except reinforced concrete pipe.
4) Option I: use under driveways.
5) Option III: Use unless the criteria in Nos. 1, 2, 3 or 4 apply.

Culvert Shapes

Use circular pipes wherever possible. Pipe arches or elliptical pipes may be used where there are clearance problems, restrictive room for headwater, etc. As discussed in Section 12-1.02, the Bridge Design Division is responsible for the design of all box culverts.

Minimum Size

The minimum sizes of culvert for maintenance purposes are as follows:

1) Driveways. Use minimum 15 in (375 mm) diameter pipe.

2) Cross Culverts. Use minimum 18 in (450 mm) diameter pipe.

Minimum Cover (Design)

Desirably, the pipe will be placed with a cover of at least 2 ft (0.6 m) from the subgrade. The minimum cover for any type of pipe is 1 ft (0.3 m) below the subgrade.

Spacing Between Multiple Pipes

The minimum spacing allowed between multiple pipes is as follows:

1. Up to 36 in (900 mm) pipe: minimum 18 in (450 mm) spacing recommended.
2. 36 in – 72 in (900 mm – 1800 mm) pipe: spacing will be equal to the radius of the larger pipe.
3. Larger than 72 in (1800 mm) pipe: 36 in (900 mm) spacing is recommended.

Minimum Slope

Except where the pipe is used as an equalizer, a minimum slope of 0.5% (0.005 ft/ft) should be used for any type of culvert at any site.

Special Design Considerations

For pipe sizes greater than 48 in (1200 mm), the following should be considered:

1. Foundation. The type of soil at the site may dictate the type of culvert which should be used. For example, in soils susceptible to settlement (e.g., marshes/boggy areas), the culvert may settle more than the area around its entrances. Therefore, the designer should use a flexible pipe or a reinforced concrete pipe with an imperfect trench or similar treatment.

2. Step-Beveled Ends. Where used, step-beveled ends should be well-anchored with a toe wall, or other approved methods, especially at the culvert entrance. Otherwise, an uplift may occur at the opening. This problem may be especially dramatic for metal culverts.

3. Camber: For some culverts under high fills, it may be appropriate to provide a camber for the culvert which would be upbending arc for the culvert gradient. This allows the culvert to attain the desired gradient at the ultimate settlement position. Culvert manufacturer's handbooks and construction manuals provide criteria for the camber technique. Staff geotechnical engineers will assist in determining whether the native soils are susceptible to settlement and will assist in calculating the amount of camber in the culvert.

4. Increased Structural Support. For extremely high fills, certain gages of metal pipe and rigid pipes may need to be center strutted and fitted with compressible wooden caps. This would be necessary during the placement of the fill, and the increased support would be removed after placement. Concrete pipe should also be considered in cases of deep fill.

5. Allowable Headwater. Highway culverts should ordinarily be designed to a nominal headwater-to-diameter ratio (H_w/D) of 1.5 for the 50-year design flow. The actual design ratio will usually be something smaller or larger than 1.5 (but no larger than 2) since pipes come in incremental sizes. The overriding concern should be to keep the design event below the base and subbase. In cases of shallow cover, this may result in (H_w/D) values less than 1.5.

Wall Thicknesses

The following tables present the Department's criteria for the minimum wall thicknesses of pipe culverts:

Table 12-4.1 "Circular Culvert Pipes"
Table 12-4.2 "Pipe Arch Culverts"
Table 12-4.3 "Coupling Band Width Requirements", and
Table 12-4.4 "Types B & C Underdrain Pipes"

Table 12-4.1
CIRCULAR CULVERT PIPES – WALL THICKNESSES

	Nominal Pipe Wall Thickness in (in)							
	Corrugated Metal				Plastic	Reinforced Concrete		
	Option I		Option I/III		Option III	Option I/III		
Diameter (in)	M 218	M 274	M-246 & Fiber Bonded	M 197	M 278	M 170 Class III Wall A	M 170 Class III Wall B	M 170 Class III Wall C
12	0.079	0.064	0.064	0.075	0.358	1 ¾	2	
15	0.079	0.064	0.064	0.075	0.438	1 7/8	2 ¼	
18	0.109	0.079	0.079	0.075		2	2 ½	
21	0.109	0.079	0.079	0.075		2 ¼	2 ¾	
24	0.109	0.079	0.079	0.075		2 ½	3	3 ¾
27	0.109	0.079	0.079	0.105		2 5/8	3 ¼	4
30	0.109	0.079	0.079	0.105		2 ¾	3 ½	4 ¼
33	0.109	0.079	0.079	0.105		2 7/8	3 ¼	4 ½
36	0.109	0.079	0.079			3	4	4 ¾
36 (1)			0.079	0.75				
42	0.138		0.109			3 ½	4 ½	5 ¼
42 (1)		0.109	0.079	0.105				
48	0.138		0.109			4	5	5 ¾
48 (1)		0.109	0.079	0.105				
54	0.168		0.138			4 ½	5 ½	6 ¼
54 (1)		0.138	0.079	0.105				
60	0.168		0.138			5	6	6 ¾
60 (1)		0.138	0.079	0.105				
66 (1)			0.079	0.135		5 ½	6 ½	7 ¼
72 (1)			0.109	0.135		6	7	7 ¾
78 (1)			0.109	0.164			7 ½	8 ¼
84 (1)			0.109	0.164			8	8 ¾

Metal Pipe values are for 2 2/3” x ½” corrugations unless diameter is followed by (1), which requires 3 in x 1 in corrugations for aluminum pipes and 3” x 1” or 5” x 1” corrugations for steel pipes; corrugations in (in).

Option I pipes shall only be used for entrances. Fill heights over 15 ft (4.5 m) may require larger metal gages.

M 170 = Reinforced Concrete Pipe
M 218 = Zinc-coated (galvanized) corrugated steel pipe
M 278 = Polyvinyl Chloride Pipe
M 274 = Aluminum-coated (Type 2) corrugated steel pipe
M 197 = Corrugated Aluminum Alloy Pipe
M 246 = Polymer precoated galvanized corrugated steel pipe
Fiber Bonded = MDPT Spec. 707.04

Table 12-4.2
PIPE ARCH CULVERTS – WALL THICKNESSES

Nominal Wall Thickness in (in)			
	Corrugated Metal Pipe Arch Option III		
Nominal Size in (in Span x Rise	**M 246 & Fiber Bonded**	**M 197**	**Coated Steel Pipe Equivalents (in)**
21 x 15	0.079	0.075	18 gage = 0.052
24 x 18	0.079	0.075	16 gage = 0.064
28 x 20	0.079	0.105	14 gage = 0.079
35 x 24	0.109	0.105	12 gage = 0.109
40 x 31 (1)	0.079	0.075	10 gage = 0.138
42 x 29 (2)	0.109		8 gage = 0.168
46 x 36 (1)	0.079	0.105	
49 x 33 (2)	0.138		**Aluminum Pipe Equivalents (in)**
53 x 41 (1)	0.079	0.105	
57 x 38 (2)	0.138		18 gage = 0.048
60 x 46 (1)	0.109	0.135	16 gage = 0.06
64 x 43 (2)	0.168		14 gage = 0.075
66 x 51 (1)	0.109	0.135	12 gage = 0.105
73 x 55 (1)	0.109	0.164	10 gage = 0.135
81 x 59 (1)	0.109	0.164	8 gage = 0.164

Metal pipe values are for 2 2/3” x ½” corrugations unless size is followed by a (1), which denotes 3” x 1” corrugations.

M 246 = Polymer pre-coated galvanized corrugated steel pipe
M 197 = Corrugated Aluminum Alloy Pipe
Fiber Bonded = MDOT Spec. 707.04

Minimum Cover is 3 Feet (1 m)
(2) = Either size is acceptable

Table 12-4.3
COUPLING BAND WIDTH REQUIREMENTS

Nominal Corrugation (in)	Nominal Pipe Inside Diameter	Coupling Band Width (in)			
		Annular Corrugated Bands		Helically Corrugated Bands	
		M 196	M 36	M 196	M 36
1 ½ x ¼	6	10 ½	10 ½	7	7
2 2/3 x ½	12 – 84	10 ½	10 ½		
3 x 1	30 - 84	12	12		
5 x 1	36 x 84		20		

Helically corrugated pipe 12” diameter and larger shall have the ends rerolled to provide at least two annular corrugations.

Pipe with spiral corrugations shall have continuous helical lock seams.

M 196 = Corrugated Aluminum Alloy Pipe
M 36 = Corrugated Steel Pipe

Table 12-4.4
TYPES B & C UNDERDRAIN PIPE

Metal Pipe				Plastic Pipe Stiffness @ 5% Deflection			
Nominal Wall Thickness (in)				PVC Pipe		Polyethylene Pipe	
Diameter	M 218	M 274 M 246	M 197	M 278	ASTM F 949	M 294 SP Dual-Wall Unanchored	M 252 SP Dual-Wall Unanchored
Type “B” 6	0.064	0.052	0.048	46	50		60
Type “C” 12	0.079	0.064	0.075	46		50	
15	0.079	0.064	0.075	46		42	
18	0.079	0.064	0.075			40	
21	0.079	0.064	0.075				
24	0.079	0.064	0.075			40	
30	0.109	0.079	0.105				
36	0.109	0.079	0.105				

12-4.02 Maximum Cover/Height of Fill

The maximum allowable cover (or height of fill) over a culvert will depend on several factors, including the size of the culvert, its material and shape, and the type of bedding it is placed on. The following presents the Department's criteria for various culvert types.

Reinforced Concrete Pipe

The following procedure has been developed to assist the designer in determining the type of installation and class of pipe for the fill height encountered at any particular culvert location using a Class "C" bedding. It is intended that the designer consider the culvert strength design in the following order of construction procedures:

1. Positive projecting conduit
2. Zero projecting conduit
3. Negative projecting conduit or imperfect (induced) trench

Positive Projecting Conduit

This method assumes that the conduit is installed in the specified bedding with the top of the conduit projecting above the surface of the natural ground or compacted fill, at the time of installation, and then covered with earth fill. The use of this method by installation of field personnel will be assumed unless another method is specified. The maximum allowable fill heights for this method are shown in Table 12-4.5.

TABLE 12-4.5
MAXIMUM ALLOWABLE HEIGHT OF FILL IN FEET
(Positive Projecting Conduit)

Pipe Diameter (in)	Class III	Class IV	Class V
12	11.40	16.83	21.01
15	11.71	17.28	21.57
18	11.94	17.61	21.97
24	12.25	18.04	22.50
30	12.46	18.33	22.85
36	12.62	18.55	23.11
42	12.75	18.72	23.31
48	12.86	18.86	23.47
54	12.96	18.98	23.61
60	13.05	19.09	23.73
66	13.13	19.18	23.84
72	13.21	19.27	23.94

Note: These fill heights have been derived assuming a soil weight of 125 lbs per cubic foot and a safety factor of 1.5 times the ultimate pipe strength.

Zero Projecting Conduit

This method assumes that the conduit is installed, with the specified bedding, in shallow trenches of such depth that the top of the conduit is even with the surface of the natural ground or compacted fill and then covered with and embankment which extends above this ground level. The maximum allowable fill heights for this method are shown in Table 12-4.6. It should be noted that the trench width used in the development of this table was the culvert outside diameter plus 30 in (750 mm).

TABLE 12-4.6
MAXIMUM ALLOWABLE HEIGHT OF FILL IN FEET
(Zero Projecting Conduit)

Pipe Diameter (in)	Class III	Class IV	Class V
12	15.66	23.16	28.93
15	16.08	23.77	29.68
18	16.37	24.20	30.22
24	16.77	24.77	30.92
30	17.03	25.14	31.38
36	17.22	25.40	31.70
42	17.37	25.61	31.94
48	17.50	25.77	32.13
54	17.60	25.90	32.29
60	17.69	26.02	32.43
66	17.77	26.12	32.55
72	17.84	26.21	32.65

Note: These fill heights have been derived assuming a soil weight of 125 lbs per cubic foot and a safety factor of 1.5 times the ultimate pipe strength.

Negative Projecting Conduit of Imperfect Trench

This method assumed that the conduit is installed, with the specified bedding, in shallow trenches of such depth that the top of the conduit is below the surface of the natural ground or compacted fill and then covered with an embankment which extends above this ground level.

The imperfect (induced) trench method assumes that the conduit is installed in the same manner as a positive projecting conduit with the desired class of bedding. After the embankment has been constructed to some predetermined elevation, a trench is excavated in the compacted fill directly over the pipe, the trench backfilled with highly compressible material and the balance of the fill completed by normal construction methods.

When the conduits are encountered with fill heights in excess of the maximums shown in Tables 12-4.7 and 12-4.8, they will be designed on an individual basis using the procedures shown in "Loads and Supporting Strengths for Precast Concrete Pipe", a copy of which is available from the Design Engineer. Both of the above methods will be considered and a cost comparison

made. It should be noted that a minimum soil density of 125 lbs/ft^3 (= 2000 kg/m^3 = 2 g/cm^3) should be used and a minimum safety factor of 1.5 times the ultimate strength must be attained.

Corrugated Metal Pipe

Table 12-4.7 presents the maximum heights of fill for the following corrugated metal pipes:

1. corrugations of 2-2/3 in x ½ in (67.8 mm x 6.4 mm)
2. smoothlined corrugations

TABLE 12-4.7
MAXIMUM HEIGHTS OF FILL (Corrugated Metal Pipe)

Pipe Diameter (in)	Standard Thick (in)/ Height of Fill (ft)	Non-Standard Thick. /Height of Fill	Non-Standard Thick. /Height of Fill	Non-Standard Thick. /Height of Fill
12 & 15	0.064/1.5 – 45			
18	0.064/1.5 – 35	0.079/35 – 55		
21	0.064/1.5 – 35	0.079/35 – 50	0.109/50 – 55	
24	0.064/1.5 – 20	0.079/20 – 40	0.109/40 – 50	0.138/50 – 60
30	0.079/1.5 – 25	0.109/25 – 40	0.138/25 – 45	0.168/55 – 60
36	0.079/1.5 – 15	0.109/15 – 25	0.138/25 – 45	0.168/45 – 60
42	0.109/1.5 – 20	0.138/20 – 35	0.168/35 – 60	
48	0.109/1.5 – 25	0.138/20 – 50	0.168/50 – 60	
54	0.109/1.5 – 20	0.138/20 – 40	0.168/40 – 50	
60	0.138/1.5 – 25	0.168/25 – 45		
66	0.138/1.5 – 20	0.168/20 – 40		
72	0.168/1.5 – 30			

Notes:

1) This table applies to metal pipe with smoothlined corrugations and 2 2/3” x ½” corrugations.
2) Shop strut for pipe diameters of 48”and larger.

Corrugated Steel Pipe Arches

Table 12-4.8 presents the maximum heights of fill for steel pipe arches with corrugations of 2-2/3 in x ½ in (67.8 mm x 6.4 mm).

TABLE 12-4.8
MAXIMUM HEIGHTS OF FILL - CORRUGATED STEEL PIPE ARCHES
(Corrugations of 2 2/3" x ½")

Equivalent Pipe Diameter (in.)	Span (in.)	Rise (in.)	Height of Fill Above Top of Arch			
			18"-3'	4'-5'	6'-10'	11'-15'
			Wall Thickness (in.)			
15	18	11	.060	.060	.060	.060
18	22	13	.060	.060	.060	.060
24	29	18	.075	.075	.075	.075
30	36	22	.075	.075	.075	.075
36	43	27	.105	.105	.105	.105
42	50	31	.105	.105	.105	.105
48	58	36	.135	.105	.105	.135
54	65	40	.135	.135	.135	.164
60	72	44	.164	.164	.164	

Note: minimum cover is 18 in (450 mm).

Structural Steel Plate

Table 12-4.9 presents the maximum heights of fill for structural plate steel circular pipes with corrugations of 6 in x 2 in (150 mm x 50 mm). Table 12-4.10 presents the maximum heights of fill for structural fill for structural plate steel pipe arches with corrugations of 6 in x 2 in. Table 12-4.11 presents the maximum heights of fill for structural plate steel arches with corrugations of 6 in x 2 in.

Table 12-4.9
STRUCTURAL PLATE STEEL CIRCULAR PIPE

(corrugations of 6” x 2”)

			Height of Fill Above Steel Pipe (ft)																	
Dia	Area	Min	Min	5	10	15	20	25	30	35	40	45	50	55	60	70	80	90	100	110
(ft)	(ft^2)	Fill (ft)	5	10	15	20	25	30	35	40	45	50	55	60	70	80	90	100	110	120
60	20.6	1.5	0.11	0.11	0.11	0.11	0.11	0.11	0.14	0.14	0.14	0.14	0.17	0.17	0.19	0.22	0.22	0.25	0.28	●
66	23.8	1.5	0.11	0.11	0.11	0.11	0.11	0.14	0.14	0.14	0.14	0.14	0.17	0.19	0.19	0.22	0.25	0.28	●	●
72	28.3	1.5	0.11	0.11	0.11	0.11	0.11	0.14	0.14	0.14	0.17	0.17	0.19	0.19	0.22	0.25	0.25	0.28	●	♦
78	33.2	1.5	0.11	0.11	0.11	0.11	0.11	0.14	0.14	0.17	0.17	0.19	0.19	0.22	0.25	0.25	0.28	●	●	♦
84	38.5	1.5	0.11	0.11	0.11	0.11	0.14	0.14	0.14	0.17	0.17	0.19	0.19	0.22	0.25	0.28	●	●	♦	
90	44.2	1.5	0.14	0.14	0.14	0.14	0.17	0.14	0.17	0.17	0.17	0.19	0.22	0.22	0.25	0.28	●	♦	♦	
96	50.3	1.5	0.14	0.14	0.14	0.14	0.17	0.17	0.17	0.17	0.19	0.22	0.22	0.25	0.28	●	●	♦		
102	56.7	2	0.14	0.14	0.14	0.14	0.17	0.17	0.17	0.19	0.19	0.22	0.25	0.25	0.28	●	♦	♦		
108	63.6	2	0.14	0.14	0.14	0.14	0.17	0.17	0.17	0.19	0.22	0.22	0.25	0.28	●	●	♦			
114	70.9	2	0.14	0.14	0.14	0.14	0.19	0.17	0.19	0.19	0.22	0.22	0.25	0.28	●	♦	♦			
120	78.5	2	0.14	0.14	0.14	0.14	0.19	0.17	0.19	0.22	0.22	0.25	0.28	●	●	♦				
126	86.6	2	0.14	0.14	0.14	0.14	0.19	0.17	0.19	0.22	0.22	0.25	0.28	●	●	♦				
132	95.0	2	0.17	0.14	0.14	0.14	0.19	0.19	0.22	0.22	0.25	0.28	●	●	♦	♦				
138	103.9	2	0.17	0.14	0.14	0.17	0.19	0.19	0.22	0.22	0.25	0.28	●	●	♦					
144	113.1	2	0.17	0.17	0.17	0.17	0.19	0.19	0.22	0.25	0.25	0.28	●	●	♦					
150	122.7	2	0.17	0.17	0.17	0.17	0.19	0.19	0.22	0.25	0.28	●	●	♦						
156	132.7	2	0.17	0.17	0.17	0.17	0.19	0.22	0.25	0.25	0.28	●	●	♦						
162	143.1	2	0.19	0.17	0.17	0.17	0.19	0.22	0.25	0.25	0.28	●	♦	♦						
168	153.9	2	0.19	0.17	0.17	0.19	0.19	0.22	0.25	0.28	●	●	♦	♦						
174	165.1	2	0.19	0.19	0.19	0.19	0.22	0.22	0.25	0.28	●	♦	♦							
180	176.7	2	0.19	0.19	0.19	0.22	0.22	0.25	0.25	0.28	●	♦	♦							

● 0.28 in thick; 6 bolts per 1 ft long seam

♦ 0.28 in thick; 8 bolts per 1 ft long seam

All other thicknesses: 4 bolts per 1 ft long seam

Notes: As design requires for added resistance to abrasion and/or corrosion, use next heavier thickness (maximum 0.28 in) for bottom plates.
All structural plate pipes must be 5% elliptical.
4- and 6-plate pipes should have 1 bottom plate; 8- and 10-plate pipes should have 3 bottom plates.

Table 12-4.10
STEEL STRUCTURAL PLATE PIPE ARCHES
(Corrugations of 6” x 2”)

Span (ft-in)	Rise (ft-in)	Area (ft^2)	Corner Plate Radius (in)	Min Fill ft-in	Height of Fill Above Top of Pipe Arches (ft)											
					2	3	4	5-7	8	9	10	11	12	13	14	15
					For Steel Thickness (Inches)											
6-1	4-7	22	18	2-0	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109
6-4	4-9	24	18	2-0	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.138
6-9	4-11	26	18	2-0	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.138
7-0	5-1	28	18	2-0	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.138
7-3	5-3	31	18	2-0	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.138	0.138
7-8	5-5	33	18	2-0	0.138	0.138	0.138	0.109	0.109	0.109	0.109	0.109	0.109	0.109	0.138	0.138
7-11	5-7	35	18	2-0	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138
8-2	5-9	38	18	2-6		0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138
8-7	5-11	40	18	2-6		0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138
8-10	6-1	43	18	2-6		0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138
9-4	6-3	46	18	2-6		0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138
9-6	6-5	49	18	2-6		0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138
9-9	6-7	52	18	2-6		0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.168
10-3	6-9	55	18	2-6		0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.168	0.168
10-8	6-11	58	18	2-6		0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.138	0.168	0.168	0.168
10-11	7-1	61	18	2-6		0.168	0.168	0.138	0.138	0.138	0.138	0.138	0.138	0.168	0.168	0.188
11-5	7-3	64	18	2-6		0.168	0.168	0.168	0.138	0.138	0.138	0.138	0.168	0.168	0.168	0.188
11-7	7-5	67	18	2-6		0.168	0.168	0.168	0.138	0.138	0.138	0.138	0.168	0.168	0.188	0.188
11-10	7-7	71	18	2-6		0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.188	0.188
12-4	7-9	74	18	2-6		0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.188	0.188
12-6	7-11	78	18	2-6		0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.188	0.188
12-8	8-1	81	18	2-6		0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.188	0.188	
12-10	8-4	85	18	2-6		0.168	0.168	0.168	0.168	0.168	0.168	0.188	0.188	0.188	0.188	
13-5	8-5	89	18	2-6		0.188	0.168	0.168	0.168	0.168	0.168	0.188	0.188	0.188	0.188	
13-11	8-7	93	18	2-6		0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188		
14-1	8-9	97	18	2-6		0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188		
14-3	8-11	101	18	2-6		0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.218			
14-10	9-1	105	18	2-6		0.188	0.188	0.188	0.188	0.188	0.218	0.218	0.218			
15-4	9-3	109	18	2-6		0.218	0.188	0.188	0.188	0.188	0.218	0.218	0.218			
15-6	9-5	113	18	2-6		0.218	0.218	0.188	0.188	0.188	0.218	0.218				
15-8	9-7	118	18	2-6		0.218	0.218	0.188	0.188	0.188	0.218	0.218				
15-10	9-10	122	18	2-6		0.218	0.218	0.188	0.188	0.188	0.218	0.218				
16-5	9-11	126	18	2-6		0.218	0.218	0.218	0.218	0.218	0.218					
16-7	10-1	131	18	2-6		0.218	0.218	0.218	0.218	0.218	0.218					
13-3	9-4	98	31	2-6		0.188	0.168	0.168	0.168	0.168	0.168	0.188	0.188	0.188	0.188	0.188
13-6	9-6	102	31	2-6		0.188	0.168	0.168	0.168	0.168	0.168	0.188	0.188	0.188	0.188	0.188
14-0	9-8	106	31	2-6		0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188
14-2	9-10	110	31	2-6		0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188
14-5	10-0	115	31	2-6		0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.218	0.218
14-11	10-2	119	31	2-6		0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.218	0.218
15-4	10-4	124	31	2-6		0.218	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.218	0.218	0.218
15-7	10-6	129	31	2-6		0.218	0.218	0.188	0.188	0.188	0.188	0.188	0.188	0.218	0.218	0.218
15-10	10-8	133	31	2-6		0.218	0.218	0.188	0.188	0.188	0.188	0.188	0.218	0.218	0.218	0.218
16-3	10-10	138	31	2-6		0.218	0.218	0.218	0.188	0.188	0.188	0.188	0.218	0.218	0.218	0.218
16-6	11-0	143	31	2-6		0.218	0.218	0.218	0.218	0.218	0.218	0.218	0.218	0.218	0.218	
17-0	11-2	148	31	3-6			0.218	0.218	0.218	0.218	0.218	0.218	0.218	0.218	0.218	
17-2	11-4	153	31	3-6			0.218	0.218	0.218	0.218	0.218	0.218	0.218	0.218	0.218	
17-5	11-6	158	31	3-6			0.218	0.218	0.218	0.218	0.218	0.218	0.218	0.218		
17-11	11-8	163	31	3-6			0.218	0.218	0.218	0.218	0.218	0.218	0.249	0.249		
18-1	11-10	168	31	3-6			0.218	0.218	0.218	0.218	0.218	0.249	0.249	0.249		
18-7	12-0	174	31	3-6			0.218	0.218	0.218	0.218	0.218	0.249	0.249	0.249		
18-9	12-2	179	31	3-6			0.218	0.218	0.218	0.218	0.249	0.249	0.249			
19-3	12-4	185	31	3-6			0.218	0.218	0.218	0.218	0.249	0.249	0.249			
19-6	12-6	190	31	3-6			0.218	0.249	0.249	0.249	0.249	0.249	0.249			
19-8	12-8	196	31	3-6			0.218	0.249	0.249	0.249	0.249	0.249				
19-11	12-10	202	31	3-6			0.218	0.249	0.249	0.249	0.249	0.249				
20-5	13-0	208	31	3-6			0.249	0.249	0.249	0.249	0.249	0.249				
20-7	13-2	214	31	3-6			0.249	0.249	0.249	0.249	0.249	0.249				

USE 31 in RADIUS STRUCTURES

Note: for abrasion/corrosion resistance, use next heavier thickness (max. .280” thickness) for bottom and corner plates.

Table 12-4.11
FILL HEIGHT for STEEL STRUCTURAL PLATE ARCHES
(Corrugations of 6” x 2”)

Span (ft)	Min Cover (in)	Height of Fill Above Top of Steel Structural Plate Arch (ft)								
		2	3	4	5	6	7	8	9	10
		For Steel Thickness (in)								
4-10	24						0.109	0.109	0.109	0.109
11	24	0.138	0.138	0.109	0.109		0.109	0.109	0.109	0.109
12	24	0.138	0.138	0.109	0.109		0.109	0.109	0.138	0.138
13	24	0.168	0.138	0.109	0.109		0.109	0.138	0.138	0.168
14	24	0.168	0.168	0.138	0.138		0.138	0.168	0.168	0.168
15	24	0.188	0.168	0.138	0.138		0.138	0.168	0.168	0.188
16	24	0.218	0.188	0.168	0.168		0.168	0.188	0.188	0.218
17	24	0.218	0.218	0.188	0.168	0.168	0.168	0.188	0.218	0.249
18	24	0.249	0.218	0.188	0.188	0.168	0.188	0.218	0.249	0.249
19	24	0.280*	0.249	0.218	0.218	0.188	0.218	0.249	0.249	0.280
20	24	0.280*	0.280*	0.249	0.218	0.218	0.218	0.249	0.280	
21	24		0.280*	0.249	0.249	0.218	0.249	0.280		
22	24			0.280	0.249	0.249	0.249	0.280		
23	24				0.280	0.249	0.280			
24	24					0.280	0.280			
25	24					0.280				

Note: * Not to be used when rise to span ratio is 0.3 or less.

Corrugated Aluminum

Table 12-3.12 presents the maximum heights of fill for corrugated aluminum alloy circular pipe culverts. Table 12-3.13 presents the maximum heights of fill for corrugated aluminum pipe arches.

Table 12-4.12
FILL HEIGHT for ALUMINUM ALLOY CORRUGATED CULVERT

Culvert Diameter (Inches)	Type of Shape	Minimum (a) Recommended Cover (Inches)	Maximum Height of Fill (ft) for Pipe Thickness (in)				
			.060	.075	.105	.135	.164
12	Full Circle	8	35	40	50		
15	Full Circle	8	32	35	40		
18	Full Circle	8	26	30	35		
21	Full Circle	9	21	25	30		
24	Full Circle	9	13	21	30		
30	Full Circle	9		19	25	30	
	5% Vertically Elongated	9		24	30	35	
36	Full Circle	10		10	18	25	30
	5% Vertically Elongated	10			21	30	35
42	Full Circle	12			16	20	25
	5% Vertically Elongated	12			20	25	30
	5% Field Strutted (b)	12			30	35	40
48	Full Circle	15			15	20	25
	5% Vertically Elongated	15			18	25	30
	5% Field Strutted (b)	15			30	35	40
54	Full Circle	15			15	20	25
	5% Vertically Elongated	15			18	22	30
	5% Field Strutted (b)	15			25	30	35
60	Full Circle	18				14	18
	5% Vertically Elongated	18				17	25
	5% Field Strutted (b)	18				25	30
66	Full Circle	21				13	17
	5% Vertically Elongated	21				15	20
	5% Field Strutted (b)	21				25	30
72	Full Circle	24				12	15
	5% Field Strutted (b)	24				20	25
78	5% Field Strutted (b)	24				16	20
84	5% Field Strutted (b)	24					15
96	5% Field Strutted (b)	24					10

(a) For the special case of heavy construction wheeled vehicles, use 2 feet cover to 36” diameter and 2/3 of the diameter at greater than 36” diameter.

(b) Field strutting is defined as shaping pipe elliptically by wire or timber strutting or careful, thorough compaction of backfill around pipe during installation.

The following apply: Loading: AASHTO – H2O Highway Shape: 2 2/3” x ½”

Table values are for 85% or greater compaction.

Table 12-4.13
COVER HEIGHT for CORRUGATED ALUMINUM PIPE ARCHES
(Corrugations of 2 2/3" x ½")

Arch Span and Rise (in) (Helical or Annular)	Minimum and Maximum Height of Cover, in feet, For Various Metal Thicknesses (in)				
	.060	.075	.105	.135	.165
17 x 13	1-15				
21 x 15	1-15				
24 x 18	1-14	1-16			
28 x 20		1-14			
35 x 24		1-13	1-16		
42 x 29			1 ¼-13	1 ¼-16	
49 x 33			1 ¼-12	1 ¼-16	
57 x 38			1 ¼-9	1 ¼-12	
64 x 43				1 ¼-11	1 ½-14
71 x 47				2-9	2-11
77 x 52				3-9	2-10
83 x 57					2-10

12-4.03 Hydraulic Design of Culverts

A complete treatment of hydraulic design of culverts is beyond the scope of the manual. General guidelines are given, as well as methods for design of simple projecting culverts under inlet control. It is recommended that designers limit their work to cross-pipes that convey only storm water. Perennial streams, outlet control, sophisticated inlet treatments for improved efficiency, and fish passage should be referred to hydraulic engineering staff.

Basic Design Controls

Certain criteria will control the hydraulic design of culverts:

1. Allowable Headwater (Design Storm). For inlet control, the headwater depth above inlet at the upstream end of the culvert should be 1.5 (desirable) times the diameter of the culvert ($H_w/D = 1.5$) during the design discharge. The actual ratio may be somewhat smaller or larger (up to a maximum of 2), since pipes come in incremental sizes and the exact size needed to deliver $H_w/D = 1.5$ may not be available. Headwater at the inlet should not rise above the subgrade during the design discharge. In addition, the designer should consider:

   a. Existing and future land use in the watershed
   b. Impacts on the surrounding land

c. Potential pavement damage when the water rises above the subbase elevation
d. 100-year flood requirements of the Federal Emergency Management Agency (FEMA). When the 100-year flood stage is above the culvert outlet, design should be referred to an experienced hydraulic engineer.
e. debris
f. the need to create lower-than-existing headwater ponding in the flood-prone or sensitive areas upstream from the culvert.

2. Allowable Headwater (Check Storm). For both inlet and outlet control, the water level at the inlet should not be higher than the edge of the shoulder berm.

3. Perennial Streams and Outlet to Adjacent Streams. Flood stage in adjacent streams and perennial streams crossing under roads will back water up through the culvert. Each cross-pipe should be identified as to whether or not it passes a perennial stream or outlets to an adjacent perennial stream. Design should be referred to experienced hydraulic engineers so that flood stage tailwater conditions can be properly accounted for.

4. Multiple Pipes. The design discharge is assumed evenly divided in each barrel.

5. Maximum Outlet Velocity. Where high outlet velocities are unavoidable because of steep slopes, erosion control measures and/or energy dissipators should be considered at the downstream end of the culvert.

6. Fish Passage: The designer should confirm with the Environmental Office whether the culvert must be designed for fish passage. If fish passage is an issue, the designer should consult with the Environmental Office for further guidance.

7. Reference. The designer is encouraged to review the following Federal Highway Administration (FHWA) references for a more complete discussion of hydrology and hydraulic design of culverts:

HDS # 2 (Highway Hydrology)
HDS # 5 (Hydraulic Design of Highway Culverts)
HEC #14 (Energy Dissipators for Culverts and Channels)

These documents are available as hard copy, on the FHWA web page, and on CD-ROM distributed by FHWA.

8. Microcomputer Programs. Readily available public domain (HY8 from Federal Highway Administration, available from FHWA and MDOT web pages) and proprietary software (e.g., CulvertMaster by Haested Methods, Waterbury, Connecticut) may be used culvert analysis.

Types of Culvert Flow

There are two major types of culvert flow:

Inlet Control: Inlet control occurs when the discharge through the culvert is controlled by inlet conditions: the pipe itself is capable of passing more water than the inlet. This occurs when the culvert is on a steep slope or when there is too much constriction of the flood plain. Critical depth occurs at the inlet and the flow in the culvert is mostly supercritical. The pipe inlet is modeled as an orifice when inlet control prevails. A useful indication of inlet control is free outlet flow with little or no tailwater. Outlet conditions (including tailwater) and culvert barrel roughness and length are not factors in determining culvert capacity.

Outlet Control: Outlet control occurs when the discharge through the culvert is controlled by outlet or pipe conditions. The flow in an outlet control culvert may be subcritical, full, or pressure flow. In outlet control, the hydraulic and geometric characteristics of the culvert (type, slope, length, roughness "n", etc) and the tailwater elevation play an important role in determining the discharge capacity of the culvert. When a culvert conveys a perennial stream or discharges to an adjacent stream, tailwater effects in the culvert are likely under peak flow conditions. Design for tailwater (outlet control) is beyond the scope of this manual.

Design Procedure

This manual is limited to stormwater culverts under inlet control; perennial streams and tailwater are beyond the scope of this manual. However, the simple methods presented here are useful for developing preliminary estimates for these more complicated situations.

Headwall inlets will give the smallest H_w/D for a given flow, while mitred inlets give ratios between headwall and simple projecting treatments. Routine design should be limited to simple projecting inlets; other inlet treatments should be referred to hydraulic engineering staff.

Culverts under inlet control and with fully submerged inlets function hydraulically as orifices. The equation for simple orifice flow and adapted for culverts is

$$Q = C_d A\{2g(H_w - 0.5D)\}^{0.5}$$

where C_d is the dimensionless discharge coefficient (usually 0.6; can range between 0.5 and 1) and g is acceleration due to gravity (32.2 ft/s^2 = 9.81 m/s^2). Combining coefficients and solving for the headwater-diameter ratio gives

$$H_w/D = c\{Q/AD^{0.5}\}^2 + Y$$

where $c = 1/(2gC_d^2)$ and Y = 0.5.

The value Y = 0.5 is for a simple generic orifice. Empirical results yield other Y values for real culverts operating under inlet control.

Pipes are sized to achieve a specified (H_w/D) value. Larger allowable (H_w/D) creates more flow capacity in a pipe of given size. For a specified (H_w/D) ratio and circular flow area, pipe size D is calculated as

$$D = \{[\{H_w/D - Y\}/c]^{-0.5}4Q/\pi\}^{0.4} = a_cQ^{0.4}$$

where $a_c = \{4[\{H_w/D - Y\}/c]^{-0.5}/\pi\}^{0.4}$.

Table 12-4.14 shows orifice equation coefficients for commonly used culvert inlet treatments. The simple projecting inlet is probably used most often in highway stormwater cross-pipe applications. Headwall inlets are generally discouraged because the massive concrete structure presents a potentially dangerous obstacle to motorists. Mitred inlets may be employed on larger perennial streams but typically are not used for smaller stormwater flows.

Table 12-4.14
Orifice Equations for CMP Culvert under Inlet Control

	Coefficient					
	C_d	c		Y	a_c (H_w/D = 1.5)	
Inlet Type		U.S.	metric		U.S.	metric
Projecting	0.53	0.055	0.181	0.54	0.622	0.789
Mitred	0.58	0.046	0.152	0.75	0.631	0.800
Headwall	0.64	0.038	0.124	0.69	0.597	0.757
Generic	0.60	0.043	0.142	0.50	0.587	0.745
$Q = C_dA\{2g(H_w - 0.5D)\}^{0.5}$		$H_w/D = c\{Q/AD^{0.5}\}^2 + Y$			$c = 1/(2gC_d^2)$	

Table 12-4.15 shows the size equation coefficients for circular CMP culvert and different H_w/D values. While culverts are generally sized to $H_w/D = 1.5$, H_w/D as large as 2 may be permissible under high fills where the high water can be kept out of the subbase. Also, strict design for $H_w/D < 1.5$ may occasionally lead to using pipes larger than necessary. Using a smaller pipe may give H_w/D just slightly larger than 1.5, or in the acceptable rang of 1.5 to 2.

Culvert size can also be checked against just-full ($H_w/D = 1$) open-channel Manning's Equation flow. For circular pipes Manning's Equation is

$$Q = Av = (\pi D^2/4)\{\lambda R_h^{2/3}S^{1/2}/n\} = (\pi D^2/4)\{\lambda(D/4)^{2/3}S^{1/2}/n\}$$
$$D = \{4^{5/3}n/\lambda\pi S^{1/2}\}^{3/8}Q^{3/8} = a_MQ^{0.375}$$

where $a_M = \{4^{5/3}n/\lambda\pi S^{1/2}\}^{3/8}$

λ = Manning unit conversion factor (1.486 for US Customary; 1 for metric).

Table 12-4.15
Coefficients for CMP Culvert Sizing Functions

Inlet Control Equation			Manning's Equation		
H_w/D	a_C		S	a_M	
	U.S.	metric		U.S.	metric
1.0	0.721	0.914	0.001	1.222	1.418
1.5	0.622	0.789	0.005	0.904	1.049
2.0	0.572	0.726	0.010	0.794	0.921
2.5	0.540	0.684	0.020	0.697	0.809
3.0	0.516	0.654	0.050	0.587	0.681
$D = a_c Q^{0.4}$			$D = a_M Q^{0.375}$		

Notes: values are for simple projecting, non-embedded, circular CMP culvert, n = 0.025
Inlet control (slope correction not included)
Manning's Equation just-full free-surface (non-pressurized) flow

These equations for pipe size D are graphed in Figure 12-4.1; the "diameter" axis is in standard pipe size increments of 6 in (150 mm). For specified Q and (H_w/D) values, one simply looks up the appropriate pipe size. The final design size should be rounded to the next available size that offers the best compromise between cost, performance, and physical configuration, H_w/D preferably less than 1.5 and nevcr exceeding $H_w/D = 2$. The actual (H_w/D) value for the chosen size should be included in the design report.

The basic steps for designing simple circular culverts are:

1) Select the design frequency return period T, ordinarily 50 years.
2) Determine the design discharge Q_T, using hydrologic methods presented elsewhere in this manual.
3) Size the culvert using the equations above or look up on chart. Report the actual H_w/D for the final size.

Comprehensive culvert design should be performed using FHWA computer program HY8 or equivalent. Routine design for highway projects should be limited to simple projecting, circular CMP culverts. Other shapes, inlet types, and sophisticated treatments for improved inlet efficiencies, and all pipes D = 10 ft (3 m) and larger, should be referred to staff experienced in hydraulics and culvert design. Routine design can be performed using the equations and charts in this manual, as well as accompanying spreadsheet tools.

Example: Peak flow Q_{50} has been estimated to be 66 ft^3/s (1.87 m^3/s). Assuming a simple projecting inlet under inlet control, size the culvert.

Standard policy is to design for $H_w/D = 1.5$, with H_w/D as large as 2 acceptable provided high water is below the subbase. By calculation,

$$D = a_c Q^{0.4} = 0.622(66)^{0.4} = 3.32 \text{ ft} = 1.01 \text{ m}$$
$$= 40 \text{ in} = 1010 \text{ mm for } H_w/D = 1.5$$

In order to maintain $H_w/D \leq 1.5$, the next standard pipe size (D = 42 in or 1050 mm; A = 9.62 ft^2 = 0.866 m^2) should be used. The next lower pipe size (D = 36 in = 900 mm; A = 7.07 ft^2 = 0.636 m^2) should be checked to see what the actual H_w/D would be with the smaller pipe. The smaller pipe may be acceptable from a hydraulic perspective, and it may be necessary for other reasons.

$$H_w/D = c\{Q/AD^{0.5}\}^2 + Y$$
$$\{Q/AD^{0.5}\}^2 = \{66/(7.07 \times 3^{.5})\}^2 = 29.05$$
$$H_w/D = 0.055(29.05) + 0.54 = 2.14 > 2, \text{ unacceptable}$$

Since $H_w/D > 2$, a 36 in pipe with simple projecting inlet should not be used. If the smaller size is still needed, alternative inlet configurations could be investigated.

Headwall: $H_w/D = 0.038(29.05) + 0.69 = 1.79 < 2$, marginally acceptable
Mitred: $H_w/D = 0.046(29.05) + 0.75 = 2.09 > 2$, marginally unacceptable

Thus, depending on base and subbase elevations, headwall and mitred inlets may permit use of a 36 in pipe. Other sophisticated inlet treatments can also be used to improve inlet efficiency. Their evaluation should be referred to an experienced hydraulic engineer.

This same problem can be evaluated by chart lookup. In Figure 12-4.1 (for simple projecting CMP culverts), locate Q = 66 ft^3/s on the horizontal axis and then draw a vertical line to the curve for $H_w/D = 1.5$. The required diameter is read from the vertical axis as approximately 1000 mm. The next size that keeps $H_w/D < 1.5$ is 1050 mm. Note that the curve for $H_w/D = 2$ is also above the 900 mm grid line at Q = 66, indicating that a 900 mm is inadequate even at the higher depth ratio.

Alternatively, the traditional design nomograph in Figure 12-4.2 can be used. Draw a line connecting D = 900 mm and Q = 1.87 m^3/s to the first (headwall) H_w/D axis. Note that the scales for mitred and projecting inlets are projected to the headwall scale before reading. This gives ratio values of 1.8, 2.1, and 2.2 for headwall, mitred, and projecting inlets, respectively.

Using Figure 12-4.2, draw a line connecting $H_w/D = 1.5$ and $Q = 1.87\ m^3/s$ to the diameter axis, giving D = 1000 mm; the next largest stock sizes are 1050 and 1200 mm. The H_w/D ratios for the 900 mm pipe can be checked by drawing a line through D = 900 mm and $Q = 1.87\ m^3/s$. This gives ratios of 1.8 (headwall) and 2.2 (mitred and projecting). Note that the mitred and projecting values are projected onto the headwall axis for reading.

Figure 12-4.1
Design Chart for Sizing Simple CMP Culverts Under Inlet Control

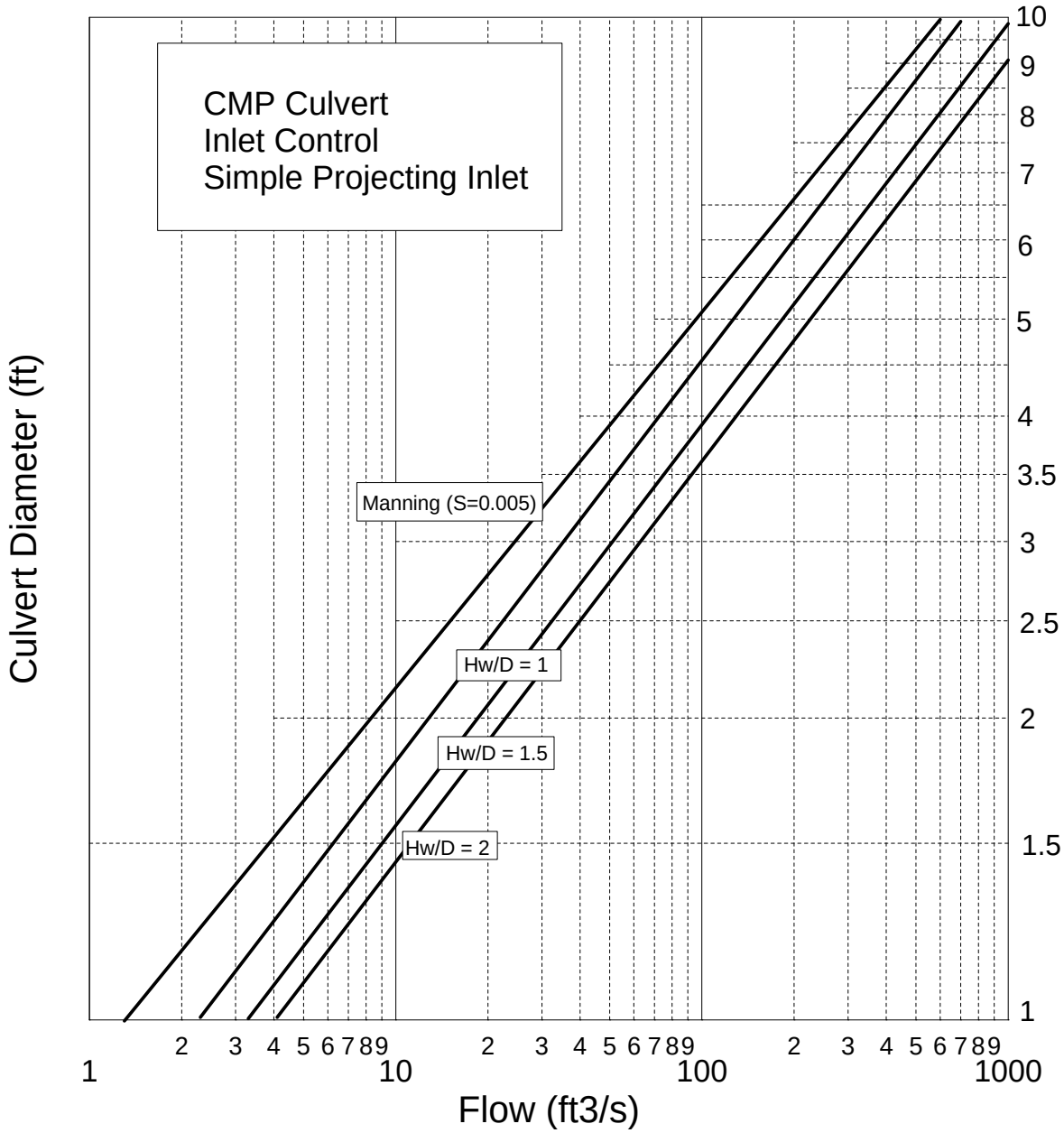
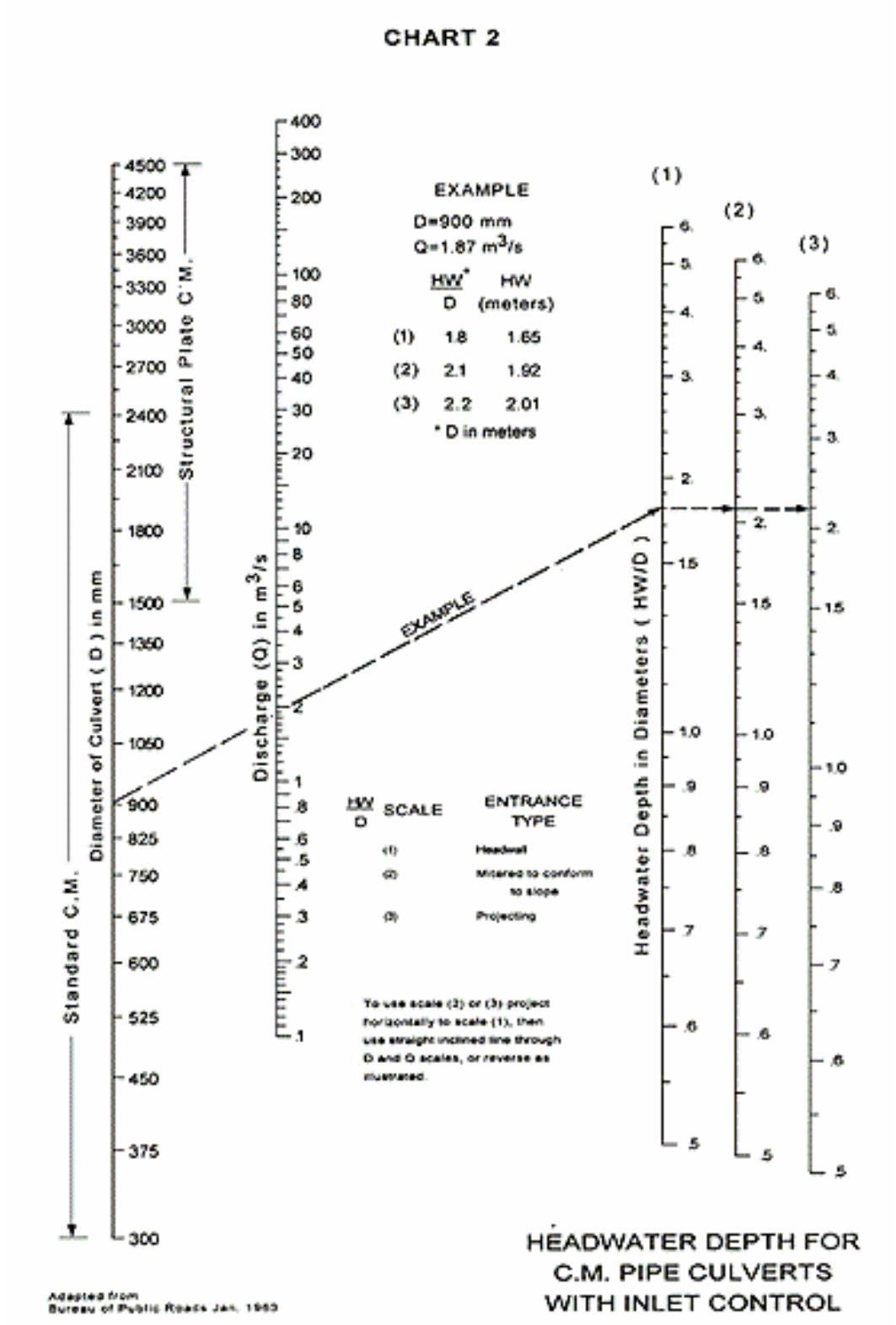


Figure 12-4.2
Design Nomograph for Sizing CMP Culverts Under Inlet Control



12-4.04 Tailwater

The simplified procedure recommended here is only suitable for cross-pipes that carry stormwater and not perennial streams. Hydraulic engineers in the Hydrology Section or Bridge Program should be consulted for perennial streams and situations where backwater may be expected.

12-4.05 Pipe Equivalences and End Areas

Culvert end-area information is summarized in Table 12-4.16 for both round and arch culverts. Table 12-4.17 gives multi-pipe equivalences as the number of smaller diameter pipes equal in hydraulic capacity to that of one larger size culvert. The table assumes that all other hydraulic factors are constant (e.g., material, slope, entrance type).

Table 12-4.16
CROSS-SECTIONAL END AREAS

Round Pipe			Pipe Arch	
Normal Diameter (in)	**End Area (ft^2)**	**Thickness (in)**	**End Area (ft^2)**	**Formed Diameter (in)**
12	0.790	0.060	0.72	14 x 10
15	1.227	0.060	1.1	17 x 13
18	1.767	0.060	1.5	21 x 15
21	2.400	0.060	2.2	24 x 18
24	3.142	0.075	2.8	28 x 20
30	4.909	0.075	4.4	35 x 24
36	7.068	0.105	6.4	42 x 29
42	9.621	0.105	8.7	49 x 33
48	12.566	0.105	11.4	57 x 38
54	16.000	0.105	14.3	64 x 43
60	19.635	0.135	17.6	71 x 47
66	23.758	0.135	21.4	77 x 52
72	28.274	0.164	25.5	83 x 57

Note: circular pipe $A = \pi D^2/4$ (D, A in consistent units)
pipe arch $A = 0.71(w \times h)^{1.023}$ (w & h in ft, A in ft^2)

It is almost always preferable to use a single culvert rather than multiple pipes with the same capacity. When a single circular pipe cannot be used (for example, due to headroom limitations) a low-profile alternative can often be substituted. However, there will always be the occasional circumstance where multiple pipes are the best choice. When choosing a number of identical smaller pipes with the capacity as a single larger pipe, there are two key mistakes to avoid:

- the number of pipes is *not* the ratio of larger diameter to smaller diameter, (D_L/D_S)
- the number of pipes is *not* the ratio of larger end area to smaller area, (A_L/A_S)

Rather, the number N of equivalent smaller pipes is given by the equation

$$N = (D_L/D_S)^{\beta}$$

where β is an exponent depending on the flow equation (Manning, $\beta = 8/3$; inlet control, $\beta = 5/2$)

Pipe equivalences by Manning's equation for fully flowing pipes are tabulated in Table 12-4.17. These give conservative results when applied to culverts under inlet control. Alternatively, the above equation can be used.

TABLE 12-4.17
MULTIPLE-PIPE EQUIVALENCES

Diameter of Larger Pipe (in)	Diameter of Smaller Pipe (in)									
	12	**18**	**24**	**30**	**36**	**42**	**48**	**54**	**60**	**66**
12	1									
18	3	1								
24	6	2	1							
30	12	4	2	1						
36		6	3	2	1					
42		10	4	2	2	1				
48			6	4	2	2	1			
54			9	5	3	2	2	1		
60			12	6	4	3	2	2	1	
66				8	5	3	2	2	2	1
72				10	6	4	3	2	2	2
78					8	5	4	3	2	2
84					10	6	4	3	3	2
90					12	8	5	4	3	2

Assumes fully flowing pipe by Manning's equation, $N = (D_L/D_S)^{8/3}$

Example: How many 12 in culverts provide the same hydraulic capacity as one 24 in culvert?

- *By look-up in Table 12-4.17, six (6) 12 in pipes are equivalent to one (1) 24 in pipe*
- *By calculation and assuming Manning's flow (same as Table 12-4.17)*
  $N = (24/12)^{8/3} = 6.35$
- *By calculation and assuming inlet control with $H_w/D = 1.5$*
  $N = (24/12)^{5/2} = 5.66$

12-4.06 Embedded Culverts and Design for Fish Passage Through Culverts

All new and replacement culverts on perennial streams should be designed so as to not present obvious obstacles to fish passage, even when the stream has not been identified as a specific fishery. Additional steps shall be taken when the culvert is in an identified fishery habitat. The document *Fish Passage Policy and Design Guide* (Maine DOT, 1/2005) presents policy and technical details. The Environmental Office should be consulted as to fish passage requirements for specific streams.

It is fairly common procedure to embed culverts in the natural stream substrate. Embedding a pipe reduces the available cross-section flow area as compared to the nominal end area. This area reduction must be accounted for in pipe sizing and may necessitate some degree of up-sizing. A simple correction can be applied by first sizing the pipe as if it were not embedded. Then choose an embedded pipe with the same open end area. Table 12-4.18 gives the equations for calculating open end areas of embedded circular pipes; Table 12-4.19 gives results for commonly used circular pipe sizes. Table 12-4.20 gives open areas for embedded elliptical pipe arches.

Table 12-4.18: Equations for Embedded Circular Pipe Geometry

Pipe Parameters	radius R, diam D, embed depth d_b
Embedded Area	$A_b = R^2\cos^{-1}[(R-d_b)/R] - (R-d_b)\{2Rd_b-d_b^2\}^{0.5}$
Open Area	$A_o = \pi R^2 - A_b$
Embedded Perimeter	$P_b = D\cos^{-1}[(R-d_b)/R]$
Open Perimeter	$P_o = \pi D - P_b$
Bottom Width	$w = \{4d_b(D - d_b)\}^{0.5}$
Distance from bottom to center	$d = R - d_b$

*Use consistent units

Table 12-4.19: Open End Areas (ft^2) for Embedded Circular Pipes

	Embedded Depth (in)					Embedded Depth (in)			
Dia (in)	3	6	9	12	Dia (in)	3	6	9	12
12	0.415	0.393	0.370	0.000	66	17.645	15.568	14.236	13.383
18	1.015	0.898	0.884	0.869	72	21.277	18.857	17.305	16.233
24	1.923	1.661	1.581	1.571	78	25.263	22.483	20.670	19.392
30	3.153	2.710	2.525	2.463	84	29.605	26.448	24.360	22.865
36	4.714	4.058	3.738	3.592	90	34.303	30.753	28.379	26.655
42	6.612	5.716	5.237	4.981	96	39.359	35.402	32.729	30.764
48	8.850	7.691	7.032	6.646	102	44.774	40.395	37.412	35.197
54	11.434	9.989	9.131	8.596	108	50.550	45.734	42.429	39.954
60	14.364	12.613	11.539	10.839	114	56.686	51.421	47.783	45.039
					120	63.185	57.457	53.475	50.452

(These equations and tables for embedded pipes can also be used to evaluate pipes flowing partially full. Then the depth of embedment is analogous to the depth of flow. This is discussed in further detail in Section 12-5.03.)

<u>*Example*</u>: *Hydrologic analysis indicates that a nominal 36 in pipe is needed to convey the 50-yr event. It is required that the pipe be embedded by 6 inches. What size pipe should be used?*

A 36 in pipe has an end area of $\pi(1.5\ ft)^2 = 7.07\ ft^2$. *Use Table 12-4.19 under the column for 6 in embedding to find the size with open end area closest to* $7.07\ ft^2$. *The needed size is 48 in.*

There are two situations where embedding is not generally recommended. In the case of an equalizer pipe where sluggish standing water is observed under low water conditions, the pipe need only be placed so that the invert is at the natural stream bottom elevation. In the case of steep streams, embedding may propagate a head upstream of the culvert. Therefore, simply matching the pipe invert to natural stream bottom is suggested. Staff with special expertise should be consulted in those instances where significant outlet drops have developed.

Table 12-4.20
OPEN AREA IN EMBEDDED ELLIPTICAL PIPE

Corner Radius = 18 in

Span (ft)	Rise (ft)	Open Area (ft^2) Depth of Embedding (in)			
		0 in	6 in	9 in	12 in
6.08	4.58	22.03	19.95	18.64	17.24
6.33	4.75	24.00	22.17	20.83	19.37
6.75	4.92	26.17	24.47	23.06	21.54
7.00	5.08	28.29	26.36	24.88	23.29
7.25	5.25	30.53	28.38	26.82	25.15
7.67	5.42	32.94	30.94	29.34	27.60
7.92	5.58	35.23	33.01	31.32	29.51
8.17	5.75	37.70	35.20	33.41	31.51
8.58	5.92	40.27	38.01	36.27	34.27
8.83	6.08	42.87	40.34	38.44	36.40
9.33	6.25	45.78	43.48	41.59	39.50
9.50	6.42	48.44	46.02	43.89	41.72
9.75	6.58	51.29	48.42	46.29	44.02
10.25	6.75	54.32	51.82	49.74	47.43
10.67	6.92	57.48	55.11	52.96	51.00
10.92	7.08	60.61	58.04	55.90	53.49
11.42	7.25	64.01	61.61	59.61	57.25
11.58	7.42	67.08	64.49	62.24	59.83
11.83	7.58	70.40	67.59	65.24	62.61
12.33	7.75	74.09	71.47	69.30	66.73
12.50	7.92	77.40	74.58	72.15	69.51
12.67	8.08	80.93	77.85	75.59	72.39
12.83	8.33	85.48	82.07	79.33	76.38
13.42	8.42	88.44	85.39	82.84	79.89
13.92	8.58	92.52	89.67	87.30	84.50
14.08	8.75	96.25	93.19	90.55	87.65
14.25	8.92	100.07	96.76	84.16	90.84
14.83	9.08	104.57	101.50	98.95	96.21
15.33	9.25	108.90	106.02	103.61	100.77

Corner Radius = 31 in

Span (ft)	Rise (ft)	Open Area (ft^2) Depth of Embedding (in)			
		0 in	6 in	9 in	12 in
15.50	9.42	112.93	109.86	107.30	104.28
15.67	9.58	117.09	113.81	111.08	105.54
15.83	9.83	122.64	119.11	116.17	112.73
16.42	9.92	126.19	122.91	120.18	116.96
16.58	10.08	130.55	127.05	124.13	120.68
13.25	9.33	97.69	95.03	92.68	90.27
13.50	9.50	101.79	98.94	96.58	93.90
14.00	9.67	106.29	103.59	101.34	98.70
14.17	9.83	110.24	107.38	104.96	102.24
14.42	10.00	114.53	111.46	108.91	106.01
14.92	10.17	119.28	116.39	113.98	111.14
15.33	10.33	123.84	121.07	118.76	116.05
15.58	10.50	128.39	125.47	123.03	120.17
15.83	10.67	133.08	129.89	127.23	124.10
16.25	10.83	137.80	134.85	132.39	129.51
16.50	11.00	142.60	139.49	136.89	133.86
17.00	11.17	147.81	144.67	142.06	138.99
17.17	11.33	150.80	147.65	145.03	141.94
17.42	11.50	157.56	154.24	151.47	148.22
17.92	11.67	163.02	159.86	157.23	154.12
18.08	11.83	167.92	164.60	161.83	158.56
18.58	12.00	173.54	170.36	167.71	164.58
18.75	12.17	178.64	175.30	172.52	169.23
19.25	12.33	184.47	181.25	178.57	175.42
19.50	12.50	190.01	186.63	183.83	180.52
19.67	12.67	195.37	191.82	188.91	185.44
19.92	12.83	201.11	197.39	194.29	190.63
20.42	13.00	207.17	203.64	200.69	197.21
20.58	13.17	212.72	209.00	205.91	202.25

12-5 PAVEMENT DRAINAGE

12-5.01 Drainage Appurtenances

The purpose of pavement drainage is to remove the storm water from the pavement through a combination of drainage features and then discharge it into a receiving watercourse, reservoir, or conduit system. The pavement drainage system includes curb and gutter, grate inlets, catch basins, manholes, and the underground pipe conduit system ("closed system"). The Department's Standard Detail Sheets provide the geometry and structural details for those drainage appurtenances approved for use by the Department.

By their nature, closed systems have limited capacity and therefore are designed for just the 10-year event instead of the much larger 50-year event. Standard gutters and inlets can only accommodate relatively small flows; pipe capacity may be limited by physical constraints. Every effort should be made to limit the introduction of off-pavement runoff into the closed system and to divert that runoff into alternative drainage paths.

Catch Basins and Manholes

A catch basin conveys stormwater into the subsurface storm drain system. It typically includes a grate or curb inlet at ground surface where stormwater enters the catch basin and a cylindrical subsurface structure that provides connections to the underground drain system. This structure is often called the "catch basin", separate from the surface inlet. The lower volume of the catch basin may be below the outlet pipe invert, in which case that volume acts as a sump to capture sediment, debris and associated pollutants. Catch basins act as pretreatment for other treatment practices by capturing large sediments. The performance of catch basins at removing sediment and other pollutants depends on the design of the catch basin (e.g., the size of the sump), and routine maintenance to retain the storage available in the sump to capture sediment.

A manhole provides access to the underground pipe system for inspection and cleaning. They are located where system parameters change, including direction, pipe size, grade, and at pipe junctions. Manholes are also placed at regular intervals (300 ft – 500 ft; 90 – 150 m) on long uniform straight segments in order to provide general access.

The following provides selection criteria for the types of catch basins presented in the Standard Details:

1) Type 1 and Type 2: use along roadside curbs, median curbs, and in parking lots.
2) Type 5: use to avoid a utility.
3) Type E: use off the highway (e.g., lawns)
4) Type F: use on lawns and for shallow highway drainage systems

The following provides selection criteria for catch basin tops:

1) Type A: use with granite curb.
2) Type B: use with granite edging and bituminous curbs.
3) Type C: use in areas protected from vehicular travel.
4) Type D: use at manholes.
5) Cascade Type: use in urban areas.

Manholes are required where there is

1) a change in direction;
2) a change in pipe size; or
3) a junction of two or more lines.

Catch Basins and Inlets

The following guidance applies to the location of catch basins:

1) Regardless of the results of the pavement drainage analysis, catch basins and inlets should be spaced no more than 300 ft (90 m) apart.
2) Catch basins should be placed on the high side of bridge approaches.
3) If the location, according to the hydraulic analysis, falls within an intersection, driveway entrance area, curb-cut ramp, or pedestrian crosswalk, the catch basin should be placed on the high side of the feature.
4) Catch basins should be placed to capture the side street flow before it reaches the major highway.
5) On superelevated curves, catch basins should be placed to prevent water from sheeting across the highway.
6) In sag locations where the catch basins will be the only outlet for stormwater (i.e., no overflow path is available), additional catch basins should be installed flanking the low point. Flank locations should be determined using methods in this manual, with the flanking inlets a minimum of 4 in (100 mm) above the elevation of the sag inlet.
7) In sag locations on freeways, four catch basins (two on each side of the roadway) are typical – one on each side of the roadway usually on the flatter of the two longitudinal upgrades.
8) Where granite curb is proposed, the catch basin must be located in a full-height curb section and not within a terminal curb section.

Closed Systems

1) Do not use pipe sizes less than 12 in (300 mm) (although 6 in underdrain is acceptable as the first section of pipe upgradient of the first inlet)

2) It is desirable to have a 3 in (75 mm) difference in elevation between the inlet pipe and the outlet pipe in a catch basin
3) Desirably, the pipe will have a cover of at least 2 ft (0.6 m) below the subgrade. The minimum cover for any pipe is 1 ft (0.3 m) below subgrade.
4) Pipes that run transversely from catch basin to catch basin will be non-perforated.
5) Pipes that run longitudinally from catch basin to catch basin may be perforated, with perforations up.
6) Wherever possible, system pipes that cross under a road shall have a minimum size of 18 in (450 mm) in order to facilitate regular maintenance and anticipate future drainage growth.
7) Closed system pipes that abut an upgradient combined (sanitary and storm) system shall be sized to accept the existing and projected storm flow components of the combined system for the eventuality of future separation of the combined system.
8) Catch basins should have a sump of at least 2 ft (0.6 m) beneath the lowest pipe invert.
9) Catch basin and manhole outlet pipes should be at least as large as the largest inlet pipe.

12-5.02 Hydraulic Analysis of Gutters and Inlets

This section gives design calculation procedures for the most commonly encountered gutter and inlet configurations. A complete development of gutter flow equations is given in the appendix. Designers should refer to the FHWA documents HEC #12 and HEC #22 for general background and techniques for additional configurations. It is recommended that MDOT staff use spreadsheets prepared by the Hydrology Section for routine gutter and inlet design.

Basic Design Controls

1) Frequency: the design frequency recommended for closed systems is a 10-year return period with storm duration no less than 5 minutes.
2) Hydrologic Method: the Rational Method will typically be used for pavement drainage; other methods may be used in consultation with MDOT Hydrology Unit.
3) Allowable Water Spread "T": Table 12-5.1 presents the maximum allowable water spread width "T" on the highway pavement during the design flood frequency.
4) Minimum Grade: the centerline profile on highways and streets with curb should have a minimum gradient of 0.25% (0.0025 m/m). Desirably, the minimum grade will be 0.5% (0.005 ft/ft).
5) Cross-section Slope: slope from centerline to curb shall be at least 2% (0.02 ft/ft)
6) Inlet Spacing: inlets should be spaced no more than 300 ft (90 m) apart

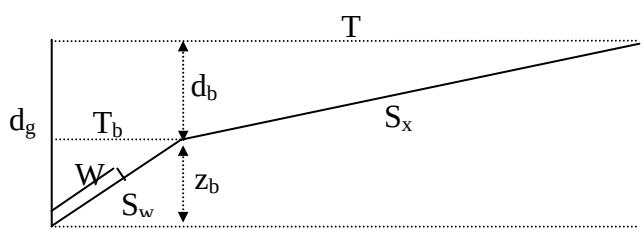
Table 12-5.1
ALLOWABLE WATER SPREAD "T"

Type of Facility	Allowable Water Spread T
Freeways & Other Multilane Highways	Shoulder width or curb offset
Two-lane arterials	Shoulder width or curb offset plus 2 ft (0.6 m) of travelway (typical T = 10 ft = 3 m)
Two-lane Collectors & Locals	Shoulder width or curb offset plus ½ of travelway
Ramps	Shoulder width or curb offset

Gutter Hydraulics

A composite triangular gutter typically terminates at a vertically-walled curb. Flow is controlled by surface roughness n, slopes S_x and S_w in cross-section, longitudinal slope S along the curb length, allowable water spread width T, and distance T_b from curb to break in cross-slope. At locations to be determined, inlets of width W intercept and convey flow to the subsurface closed drainage system. A schematic of gutter flow geometry is shown in Figure 12-5.1.

Figure 12-5.1: General Triangular Gutter Geometry Schematic



The depressed section in a composite shape ($S_w > S_x$) creates additional gutter flow capacity and thereby provides several advantages, including increased catch basin spacing (and fewer catch basins) for a given flow. Alternatively, a composite section can be employed to reduce spread onto the road. The standard MDOT design cross-section utilizes $S_x = 0.02$ (2%).

A simple uniform right-triangular section is obtained by setting $S_w = S_x$. Basic geometry and Manning's hydraulics of a right-triangle gutter channel (uniform S_x) are summarized in Table 12-5.2.

The equations for a composite section are more complicated, but with the ready availability of dedicated computer spreadsheets and programs, design of composite sections is straightforward. The equations are summarized in Table 15-5.3 below. These equations are amenable to manual calculation in a paper-and-pencil worksheet in the order shown, composite sections are best analyzed by computer.

Table 12-5.2: Simple Right-Triangle Channel Geometry and Hydraulics

T, S_x, W		Specified design parameters
Flow depth at curb	d_g	S_xT
Flow depth at inlet edge	d_w	$S_x\{W/(1+S_x^2)^{1/2}\}$
Gutter capacity	Q_g	$(3/8)(\lambda S^{1/2}/n)S_x^{5/3}T^{8/3} = (3/8)(\lambda S^{1/2}/n)d_g^{8/3}/S_x$
Intercepted flow	Q_w	$(3/8)(\lambda S^{1/2}/n)S_x^{5/3}\{T^{8/3}-(T-W)^{8/3}\}=(3/8)(\lambda S^{1/2}/n)\{d_g^{8/3}-d_w^{8/3}\}/S_x$
Interception efficiency	E_o	$Q_w/Q_g = 1 - \{1-W/T\}^{8/3}$
Manning's unit conversion	λ	1.486, U.S. Customary; 1, metric
Manning's roughness	n	asphalt, 0.013; concrete, 0.016

Table 12-5.3: Composite Triangle Channel Geometry and Hydraulics

T, T_b, S_x, S_w, W		Specified design parameters
Projected inlet width	W_x	$W/(1+S_w^2)^{1/2}$
Flow depth at curb	d_g	$S_x(T-T_b)+S_wT_b$
Flow depth at slope break	d_b	S_wT_b
Flow depth at inlet edge	d_w	$S_x(T-T_b)+S_w(T_b-W_x)$
Gutter capacity	Q_g	$(3/8)(\lambda S^{1/2}/n)\{(d_g^{8/3} - d_b^{8/3})/S_w + d_b^{8/3}/S_x\}$
Intercepted flow	Q_w	$(3/8)(\lambda S^{1/2}/n)\{d_g^{8/3} - d_w^{8/3}\}/S_w$
Interception efficiency	E_o	Q_w/Q_g
Manning's unit conversion	λ	1.486, U.S. Customary; 1, metric
Manning's roughness	N	asphalt, 0.013; concrete, 0.016

<u>Inlets</u>

Inlets are structures placed in the gutter in order to convey water via a catch basin into the closed drainage system buried in the ground. Standard inlets are 2 ft x 2 ft (0.6 m x 0.6 m) square. Two types are commonly used, parallel bar and cascade. Cascade inlets are preferred in urban areas, since they are safe for bicycle traffic. Cascade inlets may have slightly smaller hydraulic capacity.

The problem in gutter and inlet design is to space the inlets so that gutter runoff does not extend beyond the design spread T value. Mathematically, inlets are spaced so that design runoff spread exactly equals the specified design T value at each inlet.

It is inefficient to space inlets so that the entire gutter flow is captured. Instead, a certain amount of flow is intentionally allowed to bypass an inlet and flow to the next inlet. Based just on simple geometry, the "frontal flow" ratio of captured flow Q_w to total gutter flow Q_g is

$$E_o = Q_w/Q_g$$

Efficiency equations for uniform and composite sections are in the tables above.
For example, an inlet that is just 20% of the spread T captures 45% of the frontal flow in a uniform section.

Some the approaching flow within the capture width W may bypass the inlet by "splash over" when velocities are excessive. A conservative assumption that no more than 5% (equivalently, interception efficiency R_f = 95%) splashes over simplifies the analysis; this can be addressed in more detail if necessary. Then the overall capture efficiency E is

$$E = R_f E_o = 0.95 E_o$$

It follows that the inlet and bypass flows, Q_i and Q_b, are given by

$$Q_g = Q_i + Q_b \quad = EQ_g + Q_b$$
$$Q_b = (1-E)Q_g \quad \text{and} \quad Q_i = EQ_g$$

These equations pertain to the hydraulic capacities of gutter and inlet; they do not refer to the actual amount of runoff (the hydrology) in the system.

<u>Gutter Hydrology</u>
Ideally, the only runoff entering a closed system originates on the paved surface. Runoff Q_p generated on the pavement is calculated by the Rational Method, modified for dimension units (ft or m) convenient to pavement sections:

$$Q_p = \mu^* C i w_R L_b$$

Where Q_p = pavement runoff (ft^3/s or m^3/s)
μ^* = unit conversion coefficient (1/43560 for U.S. Customary; 0.28×10^{-6} for metric)
C = runoff coefficient (1 for pavement)
i = 10-year, 5-minute rainfall intensity
(5.55 in/hr = 141 mm/hr for suitable for most locations inMaine)
w_R = roadway runoff-generating width (ft or m)
L_b = distance between inlets (ft or m)

Note that Rational Equation has been rewritten in units (ft and ft^2 instead of ac) more appropriate to the relatively small runoff-generating pavement segments. All terms on the righthand side except L_b are known. When additional off-project runoff Q_o must be accounted for, total runoff Q_r in the gutter is given as

$$Q_r = Q_p + Q_o$$

where Q_o is assumed to known or estimated.

Determination of Inlet Spacing

Inlet spacing is determined by combining the hydraulics and hydrology of the system. Configurations of arbitrary complexity can be accommodated, though most projects will be straight-forward. Systems subject to significant offsite runoff should be referred to hydraulic engineering staff or the Hydrology Section. The remainder of this section is limited to runoff generated on simple rectangular pavement areas, suitable for Rational Method analysis.

In all cases, inlets are located so that spread at an inlet just equals the maximum allowable design value T. Between inlets, the actual spread increases to the design limit as the gutter captures an increasing area of runoff-generating pavement. By setting the gutter flow capacity Q_g (from hydraulics) equal to total predicted gutter flow (from hydrology), the distance between inlets can be calculated. The predicted gutter flow consists of two components, bypass Q_b from the previous inlet and pavement runoff Q_p generated since the previous inlet. Since spread at an inlet is set equal to the specified T, the generated pavement runoff Q_p must equal the flow Q_i entering the inlet. Equating gutter capacity (hydraulics) to gutter flow (hydrology) at an inlet gives

$$Q_g = Q_b + Q_i$$
$$= (1-E)Q_g + Q_p$$
$$Q_p = EQ_g = Q_i \qquad \text{(flow into the inlet)}$$

The runoff quantity Q_p is determined by hydrologic calculation, e.g. Rational Method. Substituting for Q_p with the Rational Formula and solving for inlet spacing L_b gives

$$L_b = \{1/\mu^{*}Ciw_R\}Q_i = 7850Q_i/w_R \text{ (U.S. Customary)}$$

Inlet spacing on roads of uniform grade and no offsite runoff can be read from Figures 12-5.2 that follow, provided the road satisfies the default conditions in Table 12-5.3. These figures allow determination of inlet spacing for various combinations of profile slope S and cross-section slope S_x, subject to the assumed parameter values appropriate for arterial roads in Table 12-5.3. These figures have a maximum vertical scale of 400 ft (125 m); maximum allowable spacing between inlets is 300 ft (90 m).

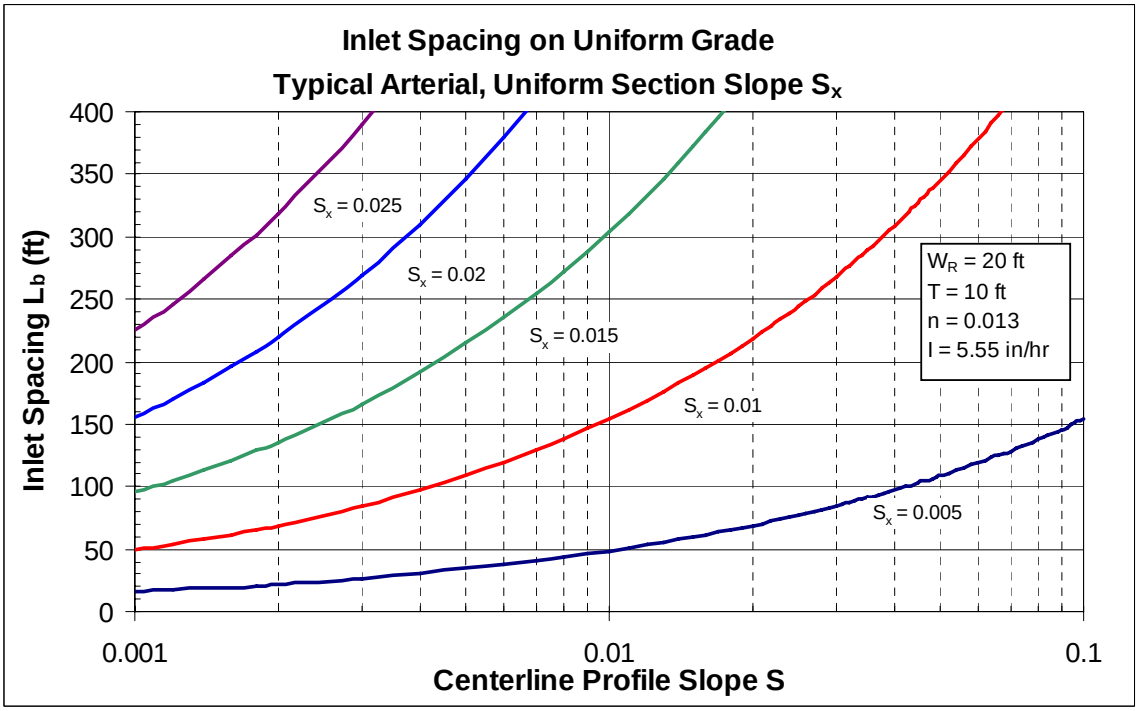
Note that, in Figures 12-5.3, for the standard design value $S_x = 0.02$ the maximum spacing of 300 ft (90 m) is acceptable over most of the range of centerline profiles. In such cases, other design considerations (e.g., intersections, side streets) may require spacing less than this maximum. In marginal situations, additional gutter capacity, reduced spread, and increased spacing can be achieved by using a composite gutter cross-section.

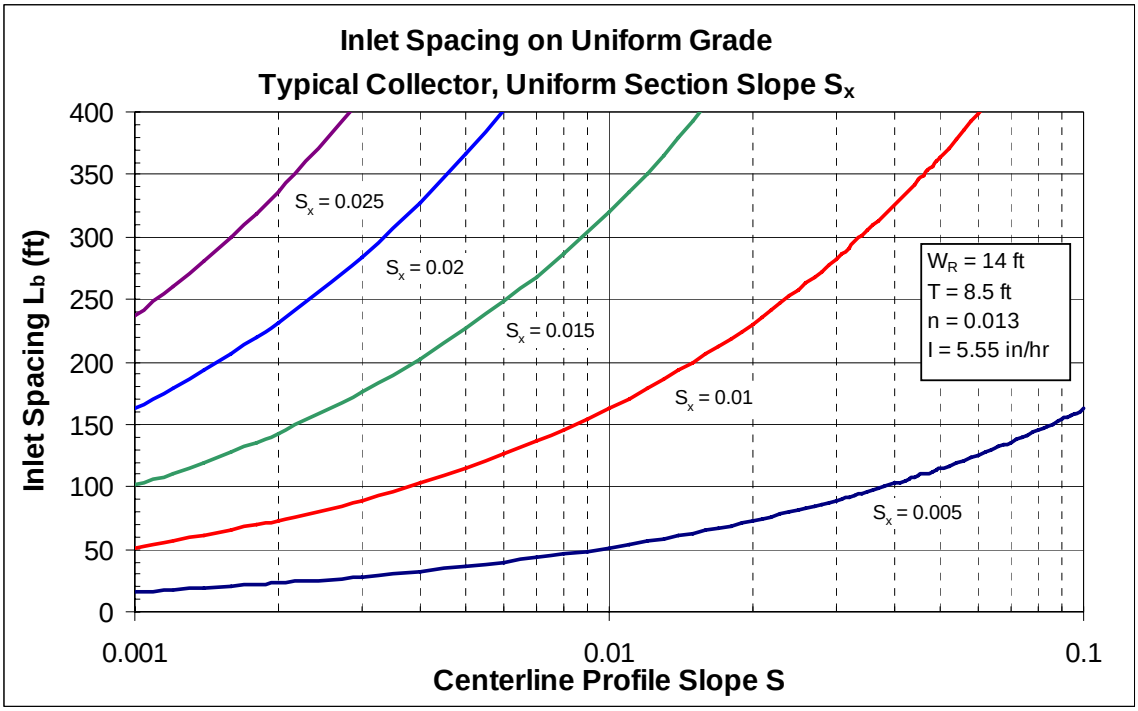
Table 12-5.4: Parameters for Simplified Inlet Spacing Analysis on Arterial Roads

Parameter	Symbol	Value
Road half-width	w_R	20 ft (6 m)
Allowable water spread on road	T	10 ft (3 m)
Time of concentration	t_c	5 min
Return Period	T	10 yr
Design Rainfall Intensity	i_{10}	5.5 in/hr (140 mm/hr)
Runoff Coefficient	C	1
Off-pavement runoff	Q_o	0 ft^3/s

Significant departures from the assumptions in Table 12-5.3 warrant a careful engineering analysis. In particular, inlet systems are easily overwhelmed by the introduction of off-pavement runoff. At the very least, it is desirable to introduce concentrated off-pavement runoff into the closed system via stub connections into manholes or catch basins, rather accept the runoff into the gutter and inlet system. In the event that off-pavement contributions cannot be re-directed, it is imperative that a complete analysis be performed.

Figure 12-5.2: Inlet Spacing for Simplified Design Scenarios





Example: Inlet Spacing

What is the maximum spacing between inlets on an asphalt arterial with centerline slope S = 0.008? Site constraints limit the cross-sectional slope to $S_x = 0.01$. There is no off-pavement runoff. Assume a simple uniform section and standard inlet W = 2 ft.

By Chart Look-up: Since "arterial" is specified, the assumptions of half-width = 20 ft and spread T = 10 ft are acceptable. The standard design storm for closed systems is the 10-year storm. Thus, the conditions in Table 12-5.2 are satisfied and it is appropriate to use Figures 12-5.2 to estimate inlet spacing.

Use the figure for asphalt paving (n = 0.013). Locate S = 0.008 on the horizontal axis. Draw a vertical line from the horizontal axis to intercept the curve for $S_x = 0.01$. Draw a horizontal line from this intersection point to the vertical axis. This is the required inlet spacing: 140 ft.

By calculation:
Gutter Capacity $Q_g = \lambda(0.375/n)S_x^{1.67}S^{0.5}T^{2.67} = (0.376/0.013)0.01^{1.67}0.008^{0.5}10^{2.67} = 0.822\ ft^3/s$
Frontal Flow Capture Ratio $E_o = Q_w/Q_g = 1 – (1\text{-}W/T)^{2.67} = 1 – (1 – 2/10)^{2.67} = 0.45$
Overall Capture Efficiency $E = R_fE_o = (0.95)(0.45) = 0.43$
Inlet Flow $Q_i = EQ_g = (0.43)(0.822) = 0.353\ ft^3/s$
Bypass Flow $Q_b = Q_g – Q_i = 0.469\ ft^3/s$
Inlet spacing: $L_b = 7850Q_i/w_R = 7850(0.353)/20 = 139$ ft

Example: Inlet Spacing

What is the maximum spacing between inlets on an asphalt road with centerline slope S = 0.01? The road is through a residential area; half-width is 13 ft with a 3.3 ft shoulder; allowable spread T has been set at 6.6 ft. Site constraints limit the cross-sectional slope to $S_x = 0.01$. The road is a critical route to the local hospital for which no detour is available. Homeowners have traditionally discharged their stormwater and cellar drains to the roadway.

The assumptions in Table 12-5.2 are violated are numerous counts:

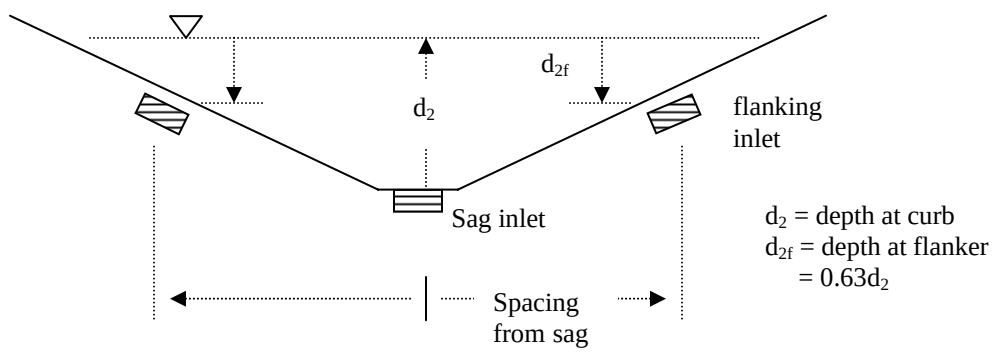
- the road is significantly narrower than an arterial
- allowable spread T is smaller
- critical route suggests that higher return period be considered
- there may be significant off-pavement runoff entering system

This problem is beyond the simplified methodology using the design charts in Figure 12-5.2. The designer should consult with staff hydraulic engineers or the Hydrology Section to be sure that the critical hydraulic and hydrologic issues are addressed in the design.

Grate Inlet Capacity – Inlet at Sag Low Point

Runoff is often conveyed in closed systems to the low (sag) point between two grades. From there the runoff is routed into a natural or constructed drainage feature and ultimately to a natural discharge. Inlets must be placed in the sag to prevent the accumulation of standing water on the road. Unlike inlets on a grade, sag inlets receive water from both directions and there is no bypass flow. The sag inlet must accept the entire design flow while not exceeding the acceptable design spread T. Figure 12-5.3 shows a curb line profile schematic of a sag inlet with adjacent flanking inlets.

Figure 12-5.3: Schematic Profile for Design of Sag and Flanking Inlets



As with inlets on a continuous grade, the design objective for sag inlets is to prevent gutter flow spread from exceeding the maximum allowable T. Analysis is somewhat more complicated, since flow into the inlet can take one of two forms, weir or orifice. In either case, flow is driven by average depth d of water over the grate. This in turn is a function of the design spread T and cross-section slope S_x. Grates are generally sized to operate as weirs, as this is a conservative design and assures additional capacity under orifice conditions at higher flows. In cases of both weir and orifice flow, the flow rate is determined by the average depth of water at the inlet.

Simplified Procedure for Sizing Sag Inlets and Calculating Inlet Capacity

The following procedure can be used to calculate the size of sag inlets. This procedure is straightforward and can be executed manually. However, specially prepared computer spreadsheets are recommended. This procedure should only be used in simple situations where off-pavement runoff is excluded from the road and the gutter section is simple triangular. Hydraulic engineering staff or the Hydrology Section should be consulted in cases where excessive inlet capacity is indicated, as subtle differences in weir and orifice flow are not included in this method.

1) Estimate design flow: the design flow at the sag consists of 4 components

   a. Bypass from upgradient right inlet

b. Bypass from other upgradient left inlet

Quantities for (a) and (b) should be available from the calculations for the last grade inlets immediately upgradient of the sag using methods of the previous section.

c. Pavement runoff between sag and right upgradent inlet
d. Pavement runoff between sag and left upgradient inlet

Quantities (c) and (d) are calculated using the Rational Method. For i_{10} = 5.55 in/hr and road half-width w_R = 20 ft, the Rational formula is

$$Q = CiA(ft^2)/43560 = 2.30x10^{-5}L\ ft^3/s$$

where L = distance (in ft) from sag to nearest upgradient inlet (calculate for right and left sides of sag).

These four components are summed to get the total design flow Q_i at the sag inlet.

2) Calculate average water depth d_{av} over the sag inlet

For a simple triangular gutter,

Depth at curb: $d_2 = S_xT$
Average depth over inlet: $d_{av} = S_xT - \{0.5S_x/(1+S_x^2)^{0.5}\}W$

3) Specify an inlet width W (usually 2 ft (0.6 m))

4) Calculate inlet length L_i assuming weir flow into inlet

Weir flow equation: $Q_i = C_wPd_{av}^{1.5}$
where C_w = weir coefficient = $(8g/27)^{1/2}$
= 3.0 $ft^{1/2}/s$ U.S. Customary; 1.66 $m^{1/2}/s$ metric
P = flow perimeter ($2L_iW$, free-standing; $2W+L_i$, against curb)

$L_i = Q_i/(C_wd_{av}^{1.5}) - 2W$ (inlet against curb)
$L_i = Q_i/(C_wd_{av}^{1.5}) - W$ (free-standing inlet)

Ordinarily, sag inlets are sized to function as weirs as this produces conservative results. If the resulting size is too big, proceed with step 5; otherwise, skip to step 6.

5) If L_i is unacceptably large, calculate L_i assuming orifice flow into inlet

Orifice flow equation: $Q_i = 0.67A_g(2gd_{av})^{0.5}$

$$L_i = Q_i/\{0.67(2gd_{av})^{0.5}a_gW\}$$

Where g = acceleration due to gravity = 32.2 ft/s^2 (9.81 m/s^2)
a_g = grate relative open area as a decimal fraction (0.35, cascade; 0.6, parallel bar)
A_g = inlet clear area = a_gL_iW

If L_i by orifice flow is still unacceptably large, then additional upgradient inlets should be inserted between the sag and the inlets immediately upgradient. This approach can also be combined with using a larger, non-standard inlet width W. Repeat the process until an acceptable sag inlet length is achieved.

6) Locate upgradient flanking inlets:

Flanking inlets are intended to relieve ponding of water on grade if the sag inlet becomes clogged. A design procedure for locating flanking inlets is given in the next section.

This procedure can be modified to calculate the capacity of a specified inlet size. Then the hydrologic calculations are omitted and the hydraulic capacity ("Q") equations are used. Two capacities are calculated, for weir and orifice conditions. The lesser of the two results is used to give a conservative estimate of inlet capacity.

<u>Example: sag inlet capacity</u>

Problem: Determine the capacity of a 2 ft x 2 ft against-curb parallel bar grate in a sag

Given: $S_x = 0.042$, T = 8 ft, curbed roadway, parallel bar grate ($a_g = 0.6$)

Preliminary calculations:

d_{av} = $S_xT - \{0.5S_x/(1+S_x^2)^{0.5}\}W = (0.042)(2.44) - \{0.5 \times 0.042/(1 + 0.042^2)^{0.5}\}(2)$
= 0.29 ft (average depth over inlet)
P = 2W + L (curbed) = 2(2) + 2 = 6 ft
A_g = $a_gA = 0.6(LW) = 0.6(2 \times 2) = 2.4$ ft^2 (grate clear area)

Calculate capacity for weir and orifice flows:

$$Q_{weir} = C_wPd^{1.5} = 3.0(6)(0.29^{1.5}) = 2.8 \text{ ft}^3/\text{s}$$

$$Q_{or} = 0.67(2gd_{av})^{0.5}A_g = 0.67(2 \times 32.2 \times 0.29)^{0.5}(2.4) = 7.0 \text{ ft}^3/\text{s}$$

The conservative estimate of inlet capacity is the smaller of the two results, 2.8 ft^3/s.

Example: sag inlet sizing

Problem: Determine the parallel bar inlet size and curb water depth for an against-curb sag inlet. Standard inlet width is W = 2 ft.

Given: $Q_i = 6.7\ ft^3/s$; $S_x = 0.05$ m/m; n = 0.016; T = 10 ft; W = 2 ft; $a_g = 0.6$

Calculations:

Curb water depth: $d_2 = S_xT = (0.05)(10) = 0.5$ ft

Average depth over grate: $d = S_xT - \{0.5S_x/(1+S_x^2)^{0.5}\}W$

$= (0.05)(10) - \{0.5x0.05/(1 + 0.05^2)^{0.5}\}(2) = 0.45$ ft

Design for weir flow: $L = Q_i/(3.0d^{1.5}) - 2W = (6.7)/(3.0 \times 0.45^{1.5}) - 2(2)$

$= 3.4$ ft (P = L+2W = 7.4 ft)

Design for orifice flow: $L = Q_i/\{0.67(2gd_{av})^{0.5}a_gw_i\} = (6.7)/\{0.67(2x32.2x0.45)^{0.5}(0.6x2)$

$= 1.55$ ft

Final design: use weir flow results as they give larger inlet, hence more conservative combination of standard inlets that give P ≥ 7.4 ft
use a 2 ft x 4 ft inlet or two 2 ft x 2 ft inlets (perimeter P = 8 ft)

Example: sag inlet sizing with blockage

Problem: Re-size the inlet in the previous example, assuming 50% of open area blocked.

Design Objective: In weir flow, the objective in allowing for blockage is to preserve the same effective perimeter that accepts the design flow.

Assumptions: design for blockage requires the designer to make assumptions about the nature of the blockage. For example, 50% blockage along the width w_i, 50% blockage along the length L, and 30% blockage along L and w_i all produce the same 50% reduction in area. Assume 50% reduction in w_i.

Preliminary calculations:

Perimeter adjustment for blockage:

from previous example, effective perimeter P = 8 ft is required to accept Q_i

$P_{eff} = (0.5)2W + L = 8$ ft (0.5 factor is for assumed model of inlet width blockage)

$L = 8 - W$

Try reasonable L and W combinations to preserve $P_{eff} \geq 8$ ft:

W = 2 ft: $L \geq 6$ ft

W = 3 ft: $L \geq 5$ ft

The key parameter for maintaining capacity is the effective perimeter. The following configurations all preserve P_{eff} = 8 ft:

- two 2 ft x 3 ft grates
- three standard 2 ft x 2 ft grates
- one 3 ft x 5 ft grate

Design will ordinarily use a standard dimension such as W = 2 ft. Note the increase when blockage is anticipated.

Flanking Inlets: Mitigation Against Clogging and Blockage

Sags are natural accumulation points for debris, and therefore blockage should be accounted for. Blockage reduces inlet capacity by reducing flow perimeter under weir conditions and open area under orifice conditions. In such cases, flanking inlets short distances away from the sag grate or curb inlets should be considered as mitigation against blockage. The purpose of the flanking inlets is to act in relief of the inlet at the low point if it should become clogged or if the design spread is exceeded. While they may also provide additional system capacity when the sag is not blocked, this is not the design intent.

Flanking inlets can be located so they will function before water spread exceeds the allowable spread T at the sag location. They should be located to that they receive all of the flow when the primary sag inlet is clogged, without exceeding the design spread in the sag. If the flanking inlets are the same dimensions as the primary (sag) inlet, they will each intercept one-half the design flow when they are located such that the ponding depth d_{2f} at the flanking inlets is 63% of the ponding depth d_2 at the low point (i.e., when the flanking inlets are $0.37d_2$ higher than the sag). Regardless of the calculation results, the flanking inlets should be at least 4 in (100 mm) higher in elevation than the sag low point. If the flanking inlets are not the same size as the primary inlet, it will be necessary to either develop a new factor or do a trial-and-error solution using assumed depths with the weir equation to determine the capacity of the flanking inlet at the given depths.

Vertical curves are characterized by the curvature rate factor K:

$$K = L/(S_2 - S_1)$$

where K = vertical curve rate, ≤ 165 ft/% (50 m/%) for drainage evaluation
L = length of vertical curve (ft)
S = approach grades in percent

Spacing from the sag is calculated as

$$x = \{200(\Delta d)K\}^{0.5}$$

where Δd = difference between ponding depths at sag and flanking inlets = $d_2 - d_{2f} = 0.37d_2$, x and Δd in consistent units.

AASHTO geometrics policy specifies maximum K values for various design speeds and a maximum K = 165 ft/% (50 m/%) considering drainage.

<u>Example: Locate Flanking Inlets</u>

A 490 ft sag vertical curve has beginning and ending slopes of –2.5% and +2.5%, respectively. Maximum allowable spread T is 10 ft. Locate flanking inlets to accommodate a completely clogged sag inlet. Assume a constant cross-section slope, $S_x = 0.02$.

Rate of curvature: $K = L/(S_{end} - S_{beg}) = 490 \text{ ft}/(2.5 - {}^{-}2.5) = 98 \text{ ft}/\%$

Water depth at curb in sag: $d_2 = S_x T = (0.02)(10) = 0.2$ ft

Depth at flanking locations (assume flanking inlets same size as primary):

d_{2f} = 63% of sag depth = (0.63)(0.2) = 0.126 ft

Calculate spacing: $x = \{200(\Delta d)K\}^{0.5} = \{200 \times (0.2 - 0.126) \times 98\}^{0.5} = 38$ ft

Flanking inlet dimensions: same as sag inlet (assumed; and required for 0.63 factor)

12-5.03 Hydraulic Analysis of Closed Systems

12-5.03.1 Introduction

A closed drainage system is one in which storm water runoff is conveyed by underground pipes as opposed to open roadside ditches. Pavement runoff is introduced to the system via catch basins and inlets. Off-pavement runoff might enter through stub connections to pipes and catch basins or as flow onto the pavement. The runoff is ultimately discharged to a natural drainage course or stream, or a swale in the adjacent fields or woods. The system pipes generally run laterally along both sides of the road. Depending on the particular site conditions, cross-pipes may carry water from one side of the road to the other. The lateral pipes are commonly "type C underdrain", perforated pipe with the perforations up. These pipes, in addition to carrying stormwater, also drain the subbase. Closed system cross-pipes are non-perforated.

Basic Design Controls:

1) size pipe for just-full free-surface flow (i.e., not pressurized)
2) average velocity in pipe should be between 2 ft/s (0.6 m/s) and 15 ft/s (4.5 m/s)
3) pipe should follow profile grade, maintaining a slope of at least 0.003 ft/ft
4) system should ultimately discharge to a free outfall
5) maintain a 3 in (75 mm) difference in elevation between inlet pipes and outlet pipe in a catch basin
6) in the presence of other underground utilities, potential conflicts should be assessed on the basis of pipe outside diameter
7) calculations should be performed for both smooth and corrugated pipe

In addition to these considerations, the designer should also refer to guidelines under "Drainage Appurtenances".

In the sections that follow, a simplified procedure is given here that accomodates backwater effects and minor losses at pipe junctions and other structures. Controls (3) and (4) above (slope and free outfall) are intended to prevent backwater; control (5) above (invert differentials) is intended to account for minor losses due to changes in size and direction at manholes and catch basins. Hydraulic engineering staff or the Hydrology Unit should be consulted when these design controls cannot be maintained and surcharging under design conditions is suspected.

12-5.03.2 Hydraulics of Fully Flowing Circular Pipes

Manning's equation for open channel flow is

$$v = \lambda R_h^{2/3} S^{1/2}/n$$

where v = velocity (m/s)
λ = unit conversion factor (1.486 for U.S. Customary; 1 for metric)
R_h = hydraulic radius (ft or m) = A/P; A = flow area and P = wetted perimeter
S = slope (dimensionless; ft/ft or m/m)
n = Manning's roughness

For just-full flow (i.e., depth of flow equals pipe diameter) in a circular pipe:

$A = \pi D^2/4$ (circular flow area)
$P = \pi D$ (wetted circular perimeter)

and

$$v = \lambda\{1/(4^{2/3}n)\}D^{2/3}S^{1/2} = \lambda(0.397/n)D^{2/3}S^{1/2} \quad \text{(ft/s or m/s)}$$
$$Q = Av = \lambda(\pi/4^{5/3})D^{8/3}S^{1/2}/n = \lambda(0.312/n)\, D^{8/3}S^{1/2} = k(D)S^{1/2} \quad (\text{ft}^3\text{/s or m}^3\text{/s})$$

The quantity k(D) (= $Q/S^{1/2} = \lambda(0.312/n)\, D^{8/3}$) is known as the conveyance function and depends on pipe characteristics (diameter and roughness) only.

For a known flow Q, the pipe diameter that just accommodates this flow under non-pressurized (fee-surface) conditions can be calculated:

$$D = \{4^{5/3}nQ/\lambda\pi S^{1/2}\}^{3/8} = (1.55/\lambda^{3/8})\{nQ/S^{1/2}\}^{3/8} \quad \text{(ft or m)}$$

These various forms of Manning's equation for flow in circular pipe are summarized in Figures 12-5.4 and 12-5.5 in dimensionless form. The geometric and conveyance functions are tabulated in Table 12-5.3.

Example: Determine the discharge through a fully flowing 18 in (450 mm) corrugated metal pipe on a slope of 0.02.

Since corrugated metal is specified, n = 0.025 is an acceptably conservative estimate. From Table 12-5.3 or Figure 12-5.5, conveyance k = 55. The discharge $Q = kS^{1/2} = (55)(0.02)^{1/2} = 7.8$ ft^3/s.

Table 12-5.5
Circular Pipe Geometric and Conveyance Functions

				Conveyance k (ft^3/s)			
Diam D (in)	**Area A (ft^2)**	**Perim P (ft)**	**Hydraulic Radius R_h (ft)**	**n = 0.010**	**n = 0.015**	**n = 0.020**	**n = 0.025**
6	0.196	1.571	0.125	7.3	4.9	3.7	2.9
12	0.785	3.142	0.250	46.4	30.9	23.2	18.5
18	1.767	4.712	0.375	136.7	91.1	68.4	54.7
24	3.142	6.283	0.500	295	196	147	118
30	4.909	7.854	0.625	534	356	267	214
36	7.069	9.425	0.750	868	579	434	347
42	9.621	10.996	0.875	1310	873	655	524
48	12.566	12.566	1.000	1870	1247	935	748
54	15.904	14.137	1.125	2560	1707	1280	1024
60	19.635	15.708	1.250	3391	2261	1696	1356
66	23.758	17.279	1.375	4372	2915	2186	1749
72	28.274	18.850	1.500	5515	3676	2757	2206
78	33.183	20.420	1.625	6827	4551	3413	2731
84	38.485	21.991	1.750	8319	5546	4159	3327
90	44.179	23.562	1.875	9999	6666	5000	4000
96	50.265	25.133	2.000	11877	7918	5939	4751
102	56.745	26.704	2.125	13962	9308	6981	5585
108	63.617	28.274	2.250	16261	10840	8130	6504
114	70.882	29.845	2.375	18783	12522	9391	7513
120	78.540	31.416	2.500	21536	14358	10768	8615

Notes: for Manning's equation in fully flowing circular pipe (ft or m and sec):

$v = \lambda R_h^{2/3} S^{1/2}/n$

$A = \pi D^2/4$

$P = \pi D$

$R_h = D/4$

$k = \lambda(\pi/4^{5/3})D^{8/3}/n = (0.312\lambda/n)\, D^{8/3}$; $\lambda = 1$, metric; 1.486, U.S. Customary

$Q = kS^{1/2}$

Figure 12-5.4: Circular Pipe Full Flow Geometric Functions

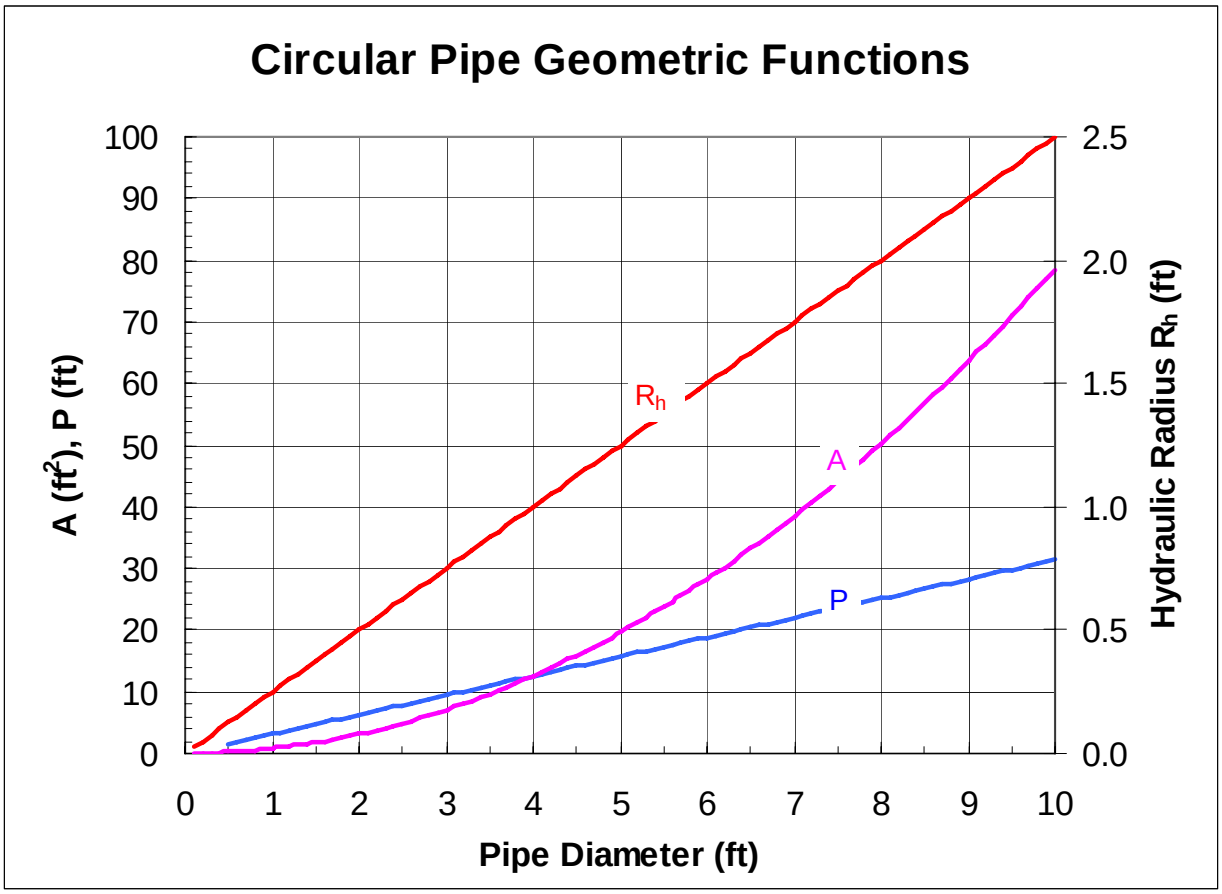
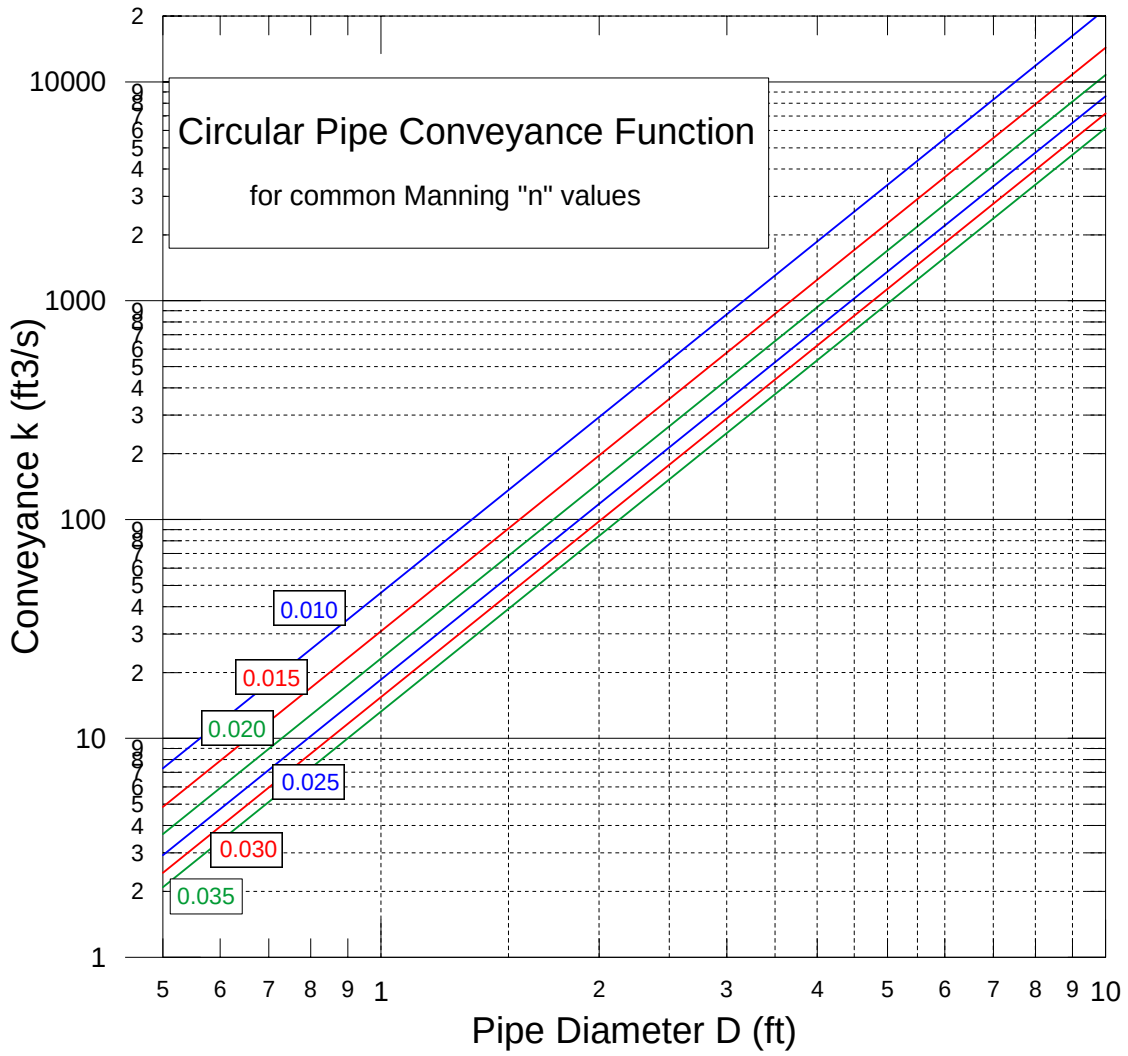


Figure 12-5.5: Circular Pipe Conveyance Function



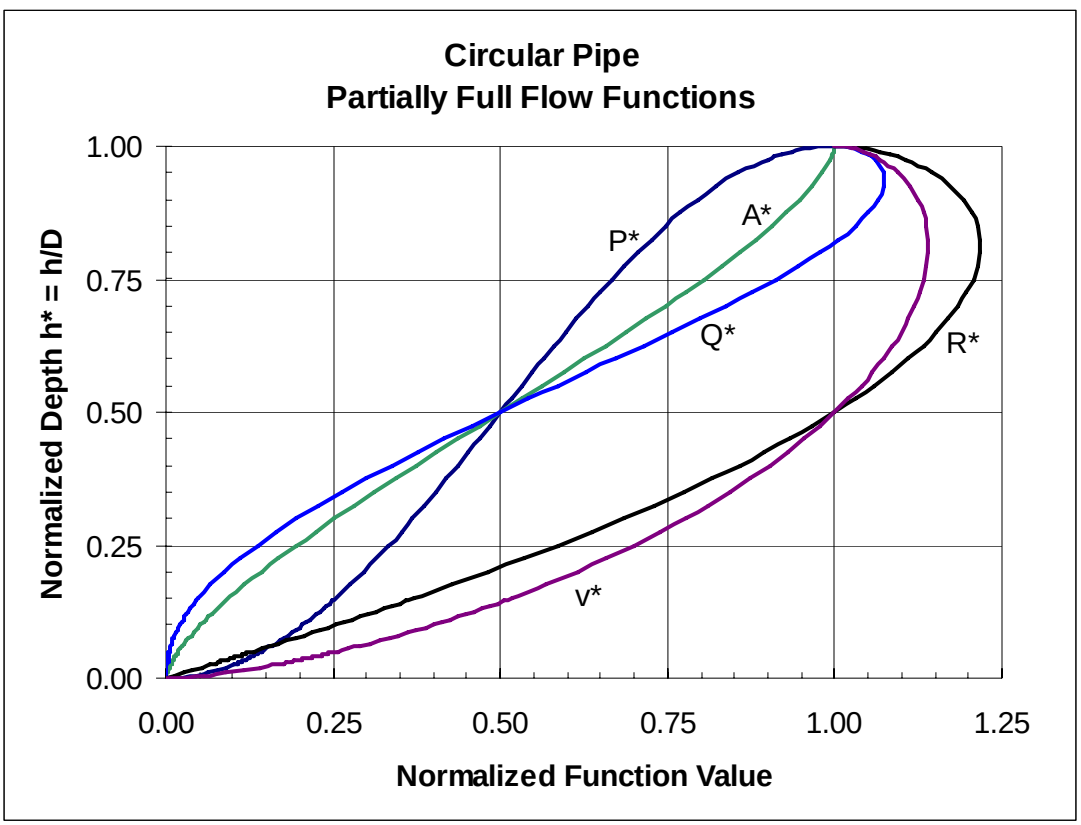
12-5.03.3 Hydraulics of Partially Flowing Circular Pipes

Occasionally it is necessary to consider pipes flowing partially full. Partial flow is best approached through scaled, dimensionless quantities as defined in Table 12-5.4. Figure 12-5.6 shows the dimensionless partial flow functions; analysis can also be done using the equations in Table 12-4.18. These values can be converted to values for pipes of specific size by simple scaling. A dimensionless quantity is the partial flow quantity divided by the corresponding quantity of a full-flowing pipe of specified size.

Table 12-5.6: Dimensionless Quantities for Partially Flowing Pipes

Dimensionless Quantity	Symbol	Definition	Equation for Partial Quantity
Depth	h^*	h/D	$H = Dh^*$
Flow Area	A^*	A/A_f	$A = A_f A^*$
Wetted Perimeter	P^*	P/P_f	$P = P_f P^*$
Hydraulic Radius	R_h^*	$R_h/R_{h\,f}$	$R_h = R_{h\,f} R_h^*$
Flow	Q^*	Q/Q_f	$Q = Q_f Q^*$
Notes: "*" = dimensionless quantity; "f" = full flowing quantity			

Figure 12-5.6: Functions for Partially Flowing Pipes



<u>Example: Flow in Partially Full Pipe</u> A 24 in (600 mm) pipe, S = 0.001 and n = 0.021, flows 75% full (by depth). Determine the geometric quantities and the flow in the partially full pipe.

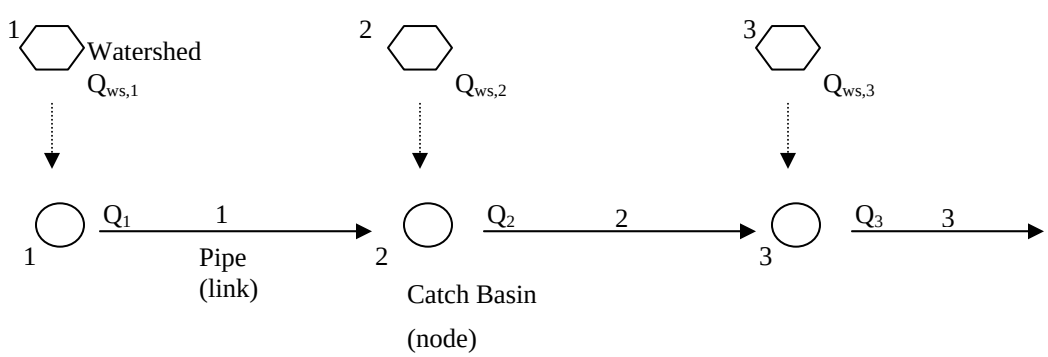
$h^* = h/D = 0.75$ => $h = (.75)(24\text{ in}) = 18\text{ in}$ = depth of flow
$A_f = \pi(2)^2/4 = 3.142\text{ ft}^2$; $A^* = 0.805$ => $A = A_f A^* = (3.142)(0.805) = 2.53\text{ ft}^2$
$P_f = \pi(2) = 6.283\text{ ft}$; $P^* = 0.667$ => $P = P_f P^* = (6.283)(0.667) = 4.19\text{ ft}$
$R_{hf} = (2)/4 = 0.5\text{ ft}$; $R_h^* = 1.207$ => $R_h = R_{hf} R_h^* = (0.5)(1.207) = 0.60\text{ ft}$
$Q_f = kS^{1/2} = (147)(0.001)^{1/2} = 4.65\text{ ft}^3/\text{s}$; $Q^* = 0.912$ => $Q = Q_f Q^* = (4.65)(0.912) = 4.25\text{ ft}^3/\text{s}$
$v = Q/A = 1.68\text{ ft/s}$

12-5.03.4 <u>Conceptual Model for Closed System Design</u>

The usual objective in closed system design is to determine pipe sizes that convey specified design flows, subject to a variety of system and design policy constraints. The general procedure is to estimate the peak flow Q (typically the ten-year flow, Q_{10}) using hydrologic methods already described. Closed systems usually drain small areas and so the Rational Method will most often be the hydrologic tool of choice. The pipe size is calculated using Manning's equation and the peak flow estimate from hydrologic analysis.

Closed drainage systems for road projects are generally simpler than municipal storm water systems, in that branching is minimal. A closed system consists of small runoff-generating areas (watersheds) connected by pipes. The system is modeled as an assemblage of links (the pipes) and nodes (junctions, primarily catch basins), as shown in Figure 12-5.7.

Figure 12-5.7: Schematic of Closed System



Water is only allowed to enter the system at nodes, usually via inlets or stub pipes to catch basins. Water moves through the system in links (the pipes). The system is assumed to start at the upgradient end with a node and terminate at the downgradient end with a pipe. Every node has an associated composite watershed and a single associated outlet pipe; the node constitutes the outlet point of the composite watershed. This composite consists of two kinds of subwatersheds:

1) Those connected to a node via inlet pipes from upgradient nodes
2) Those that drain directly to a catch basin, e.g., by an inlet or stub connection

The total flow into a node determines the size of the outlet pipe. Every pipe in the system is sized to pass flows with the same return period (usually 10 years).

For example, in the schematic above, the total composite watershed at node 2 consists of subwatershed 1 and subwatershed 2. Watershed area and flow path length increase as one moves downgradient through the system. Thus, time of concentration t_c increases as one moves down through the system. When the Rational Method is used to estimate design flows, this means that the design storm duration increases and the design storm intensity decreases from the first to last node.

Pipe sizes are determined by combining peak flow estimates with equations of mass (water) balance and simple Manning's equation hydraulics. In applying the equations, all quantities are assumed known except for the pipe size. Analysis always proceeds sequentially from the uppermost initial node to the final system outlet pipe.

Mass balance requires that the total water entering a node must equal (balance) the water leaving. In most cases there is a single entering pipe, a single subwatershed direct contribution through an inlet, and a single outlet pipe. For a simple non-branching system, the nodes, pipes and subwatersheds are organized sequentially as 1, 2, … N. Node 1 is the uppermost node; pipe N is the system outlet. The flow Q_i is the flow exiting node i in pipe i; this flow Q_i enters node i+1 as the inlet pipe. The subwatershed runoff $Q_{ws,i}$ enters node i directly through the catch basin inlet. The total flow $Q_{in,i}$ entering node entering node i is the sum of all pipes entering node i and all subwatersheds draining directly to node i. The mass balance equation for the i-th node is

$$\text{Flow Out} = \text{Flow In}$$
$$Q_i = Q_{i-1} + Q_{ws,i} = Q_{in,i}$$

For example, referring to the figure above, $Q_2 = Q_1 + Q_{ws,1}$. The flow terms Q_{i-1} and $Q_{ws,i}$ are known from the hydrology. The outlet flow Q_i (known, equal to total flow in) is written in terms of the unknown pipe size D_i (pipe leaving node i) using Manning's equation:

$$Q_i = (\lambda\pi/4^{5/3})D_i^{8/3}S^{1/2}/n = Q_{in,i} \quad (\text{ft}^3/\text{s or m}^3/\text{s})$$

$$D_i = 16\{nQ_i/S^{1/2}\}^{3/8} \quad (\text{inches})$$

One the pipe size is calculated, the velocity should be calculated using Manning's equation and checked against the allowable range of values (2 – 15 ft/s; 0.6 – 4.5 m/s) in order to protect against excessive abrasion. Velocity can be increased by increasing the slope or decreasing the pipe size. Increasing the slope will necessitate reducing the elevation of all downstream structures and/or raising the elevation of all upgradient structures, options often not available due

to site constraints. Hydraulic engineering staff or the Hydrology Unit should be consulted if acceptable velocities cannot be obtained using the simplified method outlined here.

12-5.03.5 Rational Method for Closed System Design

Using the Rational Method to estimate design flows to a catch basin (node), the design storm duration t_r is set equal to the watershed time of concentration. For any steady rainfall intensity i, peak flow will occur when the entire watershed is contributing runoff at the outlet; rain must persist for a duration at least equal to the time of concentration t_c for this to occur. The time of concentration t_c is the time for a raindrop falling on the hydraulically most distant point in the watershed to reach the outlet. However, according to the rainfall IDF curve (Figure 12-2.1), average intensity decreases as duration increases. Thus, design peak flow is obtained when storm duration is just equal to the time of concentration. When $t_r < t_c$, the entire watershed is not contributing; when $t_r > t_c$, the entire watershed is contributing but at reduced intensity. In a closed system, the time of concentration must be determined independently at each node for the total composite watershed draining to that node.

The Rational Method assumption that peak flow is obtained when storm duration just equals time of concentration is not always justified. In particular, watersheds with a significant impervious fraction near the outlet may peak at a time less than the total watershed time of concentration. This is particularly problematic when the remainder of the watershed has a natural land cover, or generally when the watershed displays runoff coefficient variability. This may indicate that rainfall-runoff modeling is the more appropriate method for determining peak flows. Staff hydraulic engineers or the Hydrology Unit should be consulted in such instances.

At each node (catch basin) in a closed system, there is a finite number of flow paths that might qualify as hydraulically longest. Each pipe entering a node marks the path draining from an upgradient node and associated watershed; the other possibilities are the hydraulically longest flow paths in subwatersheds draining directly to the node. For a pipe entering node i, the corresponding time of concentration is the time of concentration $t_{c,i-1}$ to the node (i-1) immediately upgradient plus the time in pipe $t_{t,i-1}$ between nodes (i-1) and i. For a subwatershed draining directly to node i, t_c (= $t_{ws,i}$ below) is calculated using the TR-55 conceptual model already presented. The travel time in pipe (t_t) is calculated using Manning's equation:

$$t_t \text{ (min)} = L/60v \text{ , where L = pipe length (ft) and v = velocity (ft/s)}$$

As a practical matter, the pipe travel time t_t is usually much smaller than the subwatershed t_c. Among the candidate times of concentration, the maximum is chosen. For example, at node 3 in the schematic above

$$t_{c3} = \text{maximum } \{ t_{c2} + t_{t2} \text{ ; } t_{ws,3} \}$$

The maximum value is chosen to insure that the entire composite watershed is contributing. This equation is easily generalized to multiple direct-draining watersheds and multiple inlet pipes and their associated upgradient watersheds.

12-5.03.6 Computational Procedure

Calculations for closed system design are best performed using dedicated software or specially prepared spreadsheets. The general computational sequence will be outlined here for the occasional time when manual calculation might be preferred. Included in this sequence are the basic steps of the Rational Method for closed system design. Note that diameter D, hydraulic radius R_h, and flow depth h are calculated in (in), and intensity i in (in/hr). Conversion factors are built into the calculations.

The basic computational algorithm assumes no more than two watersheds draining to a catch basin. When two watersheds drain to a catch basin, one watershed is assumed to drain via an inlet pipe and the other enters directly without specifying a particular inlet device. The objective is to size the basin outlet pipe. Cases of more than two watersheds entering a basin can be addressed by a simple extension of the basic algorithm. Detailed step-by-step calculations are presented in Table 12-5.5, the manual calculation analog to the Excel worksheet template in Figure 12-5.8.

1) Gather all watershed and system data and make initial assumptions regarding pipe invert elevations, lengths, slopes, roughnesses, etc.
2) Create a link-node diagram of the system. Number the diagram components and label the diagram with pertinent data.

Starting at node (catch basin) 1 and working sequentially downgradient towards the outlet, for the i-th node

3) Determine $t_{c,i}$ at node i using the TR-55 method and Manning's equation for pipe travel time $t_{t,i-1}$

   $$t_{c,i} = \max\{ t_{c,i-1} + t_{t,i-1} ; t_{ws,i} \}$$

4) Determine design storm intensity i for duration $t_r = t_c$ (Figure 12-2.1)
5) Calculate weighted runoff coefficient C for composite watershed at node i
6) Calculate peak flow using Rational Equation: $Q = \mu CiA$ (ft^3/s)
7) Calculate pipe size to accommodate design flow Q:
   $D = 16\{nQ/S^{1/2}\}^{3/8}$ (in) $D = 1.34\{nQ/S^{1/2}\}^{3/8}$ (ft)
8) Round D (in) to next available size and check for feasibility
9) Check velocity against acceptable limits: v (ft/s) $= 0.59D^{2/3}S^{1/2}/n$ (D in ft)

These steps are formalized in Figure 12-5.8 (a MS-Excel worksheet; calculations proceed from bottom to top), with instructions for each calculation in the corresponding computation template in Table 12-5.6. While these calculations can be performed manually, it is recommended that the worksheet be used, in which case the calculations are performed automatically after basic data has been input. This manual does not give detailed instructions for using the worksheet, since future modifications are likely. Interested users should contact the Hydrology Section.

Table 12-5.6 provides additional instructions for manual calculations. The numbered rows in the table correspond to the rows in the worksheet. For computational and organizational reasons, the order of the rows in the worksheet and table are slightly different near the end of the calculations. Referring to Table 12-5.6, items labeled “D” are data entry items; some items that are calculated (“C”) in the worksheet would probably be evaluated by look-up (“L”) in the appropriate charts (Figures 12-2.1 and 12-5.6).

This algorithm applies directly to non-branching networks that proceed from the most upgradient catch basin through a sequence of catch basins to the outlet. Branching networks require a small adjustment. Again, start with the most upgradient node, but pause the analysis at the first branch point encountered. On the other branching stem above this node, proceed from the uppermost node to the branch point. Repeat for any other stems that outlet to the branch node. These composite watersheds that outlet to branch node are then treated as single incremental watersheds draining directly to the branch node. Analysis then proceeds in the usual manner until another branch node is encountered and the process is repeated. This is illustrated in the example that follows. Branching to three or more stems requires a further modification. This is discussed following the example.

Table 12-5.7a: Template for Closed System Design by Manual Calculation

	Item	Source	Variable	Notes
1	Watershed ID	D	Identifier	From layout
2	From station	D	x_0 (ft or m)	From layout
3	To station	D	x_1 (ft or m)	From layout
4	Length	C	$x_1 - x_0$ (ft or m)	
5	Invert Elev Upper end	D	z_0 (ft or m)	From layout
6	Invert Elev Lower End	D	z_1 (ft or m)	From layout
7	Slope	C	$(x_1-x_0)/(z_1-z_0)$ (ft/ft or m/m)	
8	Time of Conc	D	T_c (min) for incremental area	From hydrology
9	Incremental Area	D	A_{inc} (ac or km^2)	From hydrology
10	Total Time of Concentration	C	T_c (min)	Max T_c {all subwatersheds draining into catch basin}
11	Total area A_{tot}	C	$A_{tot} = A_{prev} + A_{inc}$ (ac or km^2)	Sum previous and incremental areas
12	Incremental Rational C	D	C	From hydrology
13	Cumulative Weighted C	C	$\Sigma A_i C_i / A_{tot}$	For total area draining to CB
14	Rainfall Rate	C/L	i_{10} (in/hr or mm/hr)	10-yr intensity; Fig 12-2.1
15	Runoff Direct	C	$Q = \mu CiA$ (ft^3/s or m^3/s)	Rational Equation
16	Runoff Offsite	D	Q_{off} (usually 0)	Additional runoff source
17	Runoff total	C	$Q_{tot} = Q + Q_{off}$	Total runoff to catch basin
18	Manning roughness n	D	N	CMP 0.025; plastic 0.012
19	Flow depth fraction	D	Target $h/D \leq 1$	Usually = 1
20	Pipe Diam	C	$D = (1.55\alpha/\lambda^{3/8})\{nQ/S^{1/2}\}^{3/8}$ (in or mm)	Exact solution
21	Pipe Diam (design)	C	D (in or mm) & (ft or m)	Design: round to next size
22	Pipe Area (full)	C	$A_f = \pi(D/4)^2$ (ft^2 or m^2)	
23	Hydraulic Radius (full)	C	$R_{h,f} = D/4$ (ft or m)	
24	Pipe Full Velocity	C	$v_f = \lambda(R_h)^{2/3}S^{1/2}/n$ (ft/s or m/s)	Manning's equation
25	Pipe Full Capacity	C	$Q_f = A_f v_f$ (ft^3/s or m^3/s)	
26	Percent of Full Capacity	C	$Q^* = Q_{tot}/Q_f$	
27	Flow Depth Fraction	C/L	h^* (look-up against Q^* value)	Fig 12-5.6
28	Flow depth	C	$h = h^*D$ (in or mm)	
29	Velocity	C/L	$v^* \Rightarrow v = v^* v_f$ (look up v^* against h^*)	Fig 12-5.6
30	Travel time in pipe	C	$T_t = L/60v$ (min)	

Note: U.S. Customary Units: $\alpha = 12$, $\lambda = 1.486$, $\mu = 1$; metric: $\alpha = 1000$, $\lambda = 1$, $\mu = 0.28$

Source: D = specified data; C = calculation; L = look-up

Table 12-5.7b: Template for Closed System Design by Manual Calculation

	Item	Source	Variable	Value
1	Watershed ID	D	Identifier	
2	From station	D	x_0 (ft or m)	
3	To station	D	x_1 (ft or m)	
4	Length	C	$x_1 - x_0$ (ft or m)	
5	Invert Elev Upper end	D	z_0 (ft or m)	
6	Invert Elev Lower End	D	z_1 (ft or m)	
7	Slope	C	$(x_1-x_0)/(z_1-z_0)$ (ft/ft or m/m)	
8	Time of Conc	D	T_c (min) for incremental area	
9	Incremental Area	D	A_{inc} (ac or km^2)	
10	Total Time of Concentration	C	T_c (min)	
11	Total area A_{tot}	C	$A_{tot} = A_{prev} + A_{inc}$ (ac or km^2)	
12	Incremental Rational C	D	C	
13	Cumulative Weighted C	C	$\Sigma A_i C_i / A_{tot}$	
14	Rainfall Rate	C/L	i_{10} (in/hr or mm/hr)	
15	Runoff Direct	C	$Q = \mu CiA$ (ft^3/s or m^3/s)	
16	Runoff Offsite	D	Q_{off} (usually 0)	
17	Runoff total	C	$Q_{tot} = Q + Q_{off}$	
18	Manning roughness n	D	N	
19	Flow depth fraction	D	Target $h/D \leq 1$	
20	Pipe Diam	C	$D = (1.55\alpha/\lambda^{3/8})\{nQ/S^{1/2}\}^{3/8}$ (in or mm)	
21	Pipe Diam (design)	C	D (in or mm) & (ft or m)	
22	Pipe Area (full)	C	$A_f = \pi(D/4)^2$ (ft^2 or m^2)	
23	Hydraulic Radius (full)	C	$R_{h,f} = D/4$ (ft or m)	
24	Pipe Full Velocity	C	$v_f = \lambda(R_h)^{2/3}S^{1/2}/n$ (ft/s or m/s)	
25	Pipe Full Capacity	C	$Q_f = A_f v_f$ (ft^3/s or m^3/s)	
26	Percent of Full Capacity	C	$Q^* = Q_{tot}/Q_f$	
27	Flow Depth Fraction	C/L	h^* (look-up against Q^* value)	
28	Flow depth	C	$h = h^*D$ (in or mm)	
29	Velocity	C/L	$v^* \Rightarrow v = v^* v_f$ (look up v^* against h^*)	
30	Travel time in pipe	C	$T_t = L/60v$ (min)	

Note: U.S. Customary Units: $\alpha = 12$, $\lambda = 1.486$, $\mu = 1$; metric: $\alpha = 1000$, $\lambda = 1$, $\mu = 0.28$

Source: D = specified data; C = calculation; L = look-up

Figure 12-5.8: Worksheet Template for Design of Closed Systems

Closed System Worksheet - Preliminary Design of Simple Systems

Start data entry from bottom; enter data in blue cells only.

Units: **US** (US or metric)
Design Event: **10** -Year

Section	Subsection	No.	Item	Units											
Design of Closed System	Partial Flow	30	Time in Pipe Section	min											
		29	Velocity (design) =V	ft/s											
		28	Flow Depth (design)	in											
		27	Flow Depth Fraction (design)												
		26	Percent of Capcity Q_F												
	Full Flow	25	Pipe-full Capacity Q_F	ft3/s											
		24	Pipe-Full Velocity V_F	ft/s											
		23	Hydraulic Radius	ft											
		22	Pipe Area	ft2											
	Pipe Design	21	***Pipe Diam (design)***	***in***											
		20	Pipe Diam (exact)	in											
		19	Flow Depth Fraction (nom)		1										
		18	Manning Roughness Coeff = n		0.013										
Runoff Calculation		17	Runoff Total (design) Q_T	ft3/s											
		16	Runoff Offsite = Q_{off}	ft3/s											
		15	Runoff Direct Q = µCiA	ft3/s											
		14	Rainfall Rate I	in/hr											
		13	Cum Wt'ed C												
		12	Total Area =A_T	ac											
		11	Total T_c (min)	min											
		10	Incremental Rational Coeff=C												
		9	Incremental Area A_I	ac											
		8	Time of Conc T_c	min											
System	Topography	7	slope S	ft/ft											
		6	Inv Elev Lower End	ft											
		5	Inv Elev Upper End	ft											
		4	Length (m)	ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	Location	3	To Station (lower)	ft											
		2	From Station (upper)	ft											
		1	CB/Node												

Pin #

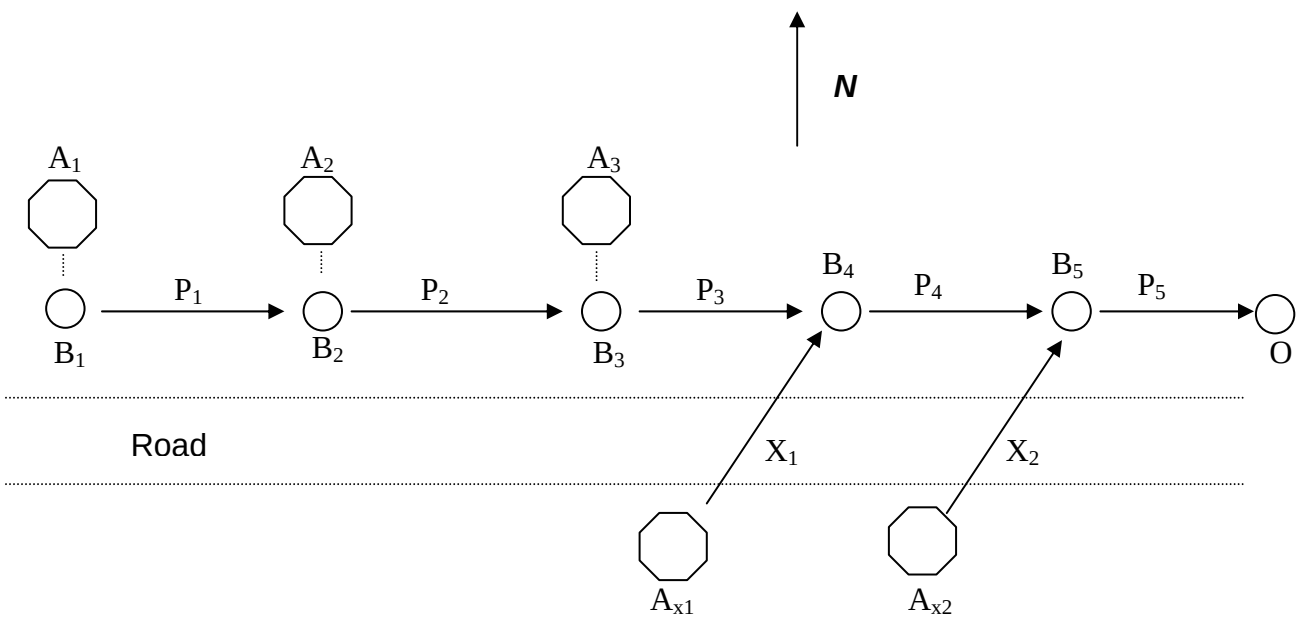
Project:

Precip Station	1	(Portland-1/Eastport-2/Rangely-3/ Presque Isle-4/Newport-5/Millinocket-6)
Event Return Period (yrs)	10	

Units	US	(US or metric)

Example: Closed System Design

The watershed system schematic was constructed from the watershed delineation and proposed pipe layout. Along the north side of the road there are 3 subwatersheds in series from east to west, A_1, A_2, and A_3. A_1 directly feeds pipe P_1; A_2 and pipe P_1 (watershed A_1) feed catch basin B_2 from which exits pipe P_2; etc. On the south side of the road are two individual subwatersheds (A_{x1} and A_{x2}) that cross the road through cross-pipes X_1 and X_2. The catch basins may be real physical entities or may just serve as conceptual nodes in the system model. The watershed area draining to a catch basin (node) increases as one moves toward the eventual outlet O (not a real catch basin). This system is not just a simple serial configuration, though, as it branches at basins B_5 and B_4 (branching is best seen by starting at the outlet and working upgradient).



Analysis always proceeds from the most upgradient subwatershed towards the outlet. When a branch node is reached (e.g., B_4), all subwatersheds and pipes upgradient of the node have to be analyzed before proceeding.

In the attached Excel worksheet in Figure 12-5.9, data entry starts in the bottom row and works upwards. Immediately below the system design area, the representative IDF curve (Figure 12-2.1) is specified once. The entries and calculations for catch basin B_2 and pipe P_2 proceed as follows. Note that the composite watershed draining to B_2 consists of two subwatersheds, A_1 and A_2. A_1 is assumed to enter B_2 through inlet pipe P_1 that has been previously sized; A_2 is assumed to drain directly to B_2.

Figure 12-5.9: Worksheet Template, Closed System Design Example

Closed System Worksheet - Preliminary Design of Simple Systems

Start data entry from bottom; enter data in blue cells only.

Units: US (US or metric)
Design Event: 10 -Year

1/6/2004

Pin # 9999.99

MDOT Drainage Manual Example

Project:

Design of Closed System	Partial Flow	30	Time in Pipe Section	min	1.1	0.3	0.5		0.1		0.6		0.1		0.5
		29	Velocity (design) =V	ft/s	3.9	10.9	10.0		7.1		6.8		7.8		8.0
		28	Flow Depth (design)	in	18	19	24		8		29		8		29
		27	Flow Depth Fraction (design)		0.67	0.72	0.71		0.68		0.70		0.57		0.73
		26	Percent of Capcity Q_F		79	87	86		80		84		61		89
	Full Flow	25	Pipe-full Capacity Q_F	ft3/s	13.9	38.6	52.89		5.04		58.09		9.14		58.38
		24	Pipe-Full Velocity V_F	ft/s	3.5	9.7	8.9		6.4		6.0		7.4		7.0
		23	Hydraulic Radius	ft	0.56	0.56	0.69		0.25		0.88		0.31		0.81
		22	Pipe Area	ft2	3.98	3.98	5.94		0.79		9.62		1.23		8.30
	Pipe Design	21	***Pipe Diam (design)***	***in***	***27***	***27***	***33***		***12***		***42***		***15***		***39***
		20	Pipe Diam (exact)	in	25	26	31		11		39		12		37
		19	Flow Depth Fraction (nom)		1	1	1	1	1	1	1	1	1	1	1
		18	Manning Roughness Coeff = n		0.013	0.013	0.013	0	0.013	0	0.013	0.013	0.013	0	0.013
Runoff Calculation		17	Runoff Total (design) Q_T	ft3/s	11.0	33.4	45.4		4.051		48.725		5.597		51.879
		16	Runoff Offsite = Q_{off}	ft3/s	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
		15	Runoff Direct Q = μCiA	ft3/s	11.0	33.4	45.4		4.051		48.725		5.597		51.879
		14	Rainfall Rate I	in/hr	3.9	3.7	3.5		3.5		3.4		4.8		3.4
		13	Cum Wt'ed C		0.40	0.58	0.51		0.44		0.50		0.33		0.48
		12	Total Area =A_T	ac	7.0	15.8	25.70		2.6000		28.3000		3.5000		31.8000
		11	Total T_c (min)	min	12.00	14.20	16.00		15.30		16.50		6.70		17.05
		10	Incremental Rational Coeff=C		0.4	0.72	0.40		0.44		0.50		0.33		0.48
		9	Incremental Area A_i	ac	7.0	8.8	9.90		2.6000		28.3000		3.5000		31.8000
		8	Time of Conc T_c	min	12.0	14.2	16.0		15.3		16.5		6.7		17.1
System	Topography	7	slope S	ft/ft	0.0020	0.0156	0.0100		0.0200		0.0033		0.0200		0.0050
		6	Inv Elev Lower End	ft	107.50	103.50	100.00		100.00		99.00		99.00		97.50
		5	Inv Elev Upper End	ft	108.00	107.00	103.00		100.50		99.75		99.50		98.75
		4	Length (m)	ft	250.00	225.00	300.00	##	25.00	0.00	225.00	0.00	25.00	##	250.00
	Location	3	To Station (lower)	ft	250	475	775		25		1000		25		1250
		2	From Station (upper)	ft	0	250	475		0		775		0		1000
		1	CB/Node		B1	B2	B3		Ax1		B4		Ax2		B5

Precip Station	1
Event Return Period (yrs)	10

(Portland-1/Eastport-2/Rangely-3/ Presque Isle-4/Newport-5/Millinocket-6)

Units	US

(US or metric)

Design of pipe P_2 by manual calculation is illustrated in Table 12-5.6. This discussion will also assist in using the worksheet template; blank forms (Figure 12-5.8) of the Excel worksheet template should be used if manual computation is contemplated.

Data ("D") in lines 1 – 3 and 5- 6 are obtained from the preliminary design layout. Lines 8, 9 and 10 contain the basic hydrologic information for the incremental additional area draining to B_2. The "Total Time of Concentration" T_c (line 11) is calculated for the entire composite watershed draining to B_2. It is the maximum of times of concentration of all upgradient ("previous") watersheds entering B_2 as well as the incremental area A_2. In this case, there is just one "previous" watershed (T_c = 12 + 0.8 = 12.8 min); do not forget to add the inlet pipe travel time, if appropriate. Watershed A_2 has T_c = 14.2 min, so T_c of the composite watershed draining to B_2 is Max{12.8, 14.2} = 14.2 min. Total area (line 12) is the calculated sum of all areas draining to B_2; cumulated weighted runoff coefficient C (line 13) is calculated for the total area in line 12. The design rain intensity (line 14) is calculated by the equations in Table 12-2.4 or look-up in Figures 12-2.1. Runoff Direct (line 15) is runoff entering node B_2 calculated by the Rational Method. Runoff Offsite (line 16) is an optional data entry item to include additional runoff entering the system that is not included in the Rational calculation; in most cases this entry is 0. The direct and offset runoffs are summed to get the total runoff Q_{tot} (line 17).

Pipe design information is entered in lines 18 (Manning's roughness n) and 19 (target flow depth fraction). Ordinarily, the nominal flow depth fraction (h^* = h/D) is entered as 1, i.e., a full flowing pipe. Since pipes come in incremental sizes, the final design size will be something larger than the exact solution and the flow depth fraction will be something less than 1. A smaller target fraction can be specified if a greater degree of conservatism is desired; the Hydrology Section should be consulted in such cases so that artifacts in the mathematical solution can be accounted for. The remaining lines in the worksheet are completed by automatic calculation in the worksheet or by a combination of calculation ("C") and look-up ("L") in manual calculations.

The exact solution for a fully flowing pipe with capacity equal to the runoff in line 17 is reported in line 20. The next available pipe size (in 150 mm increments) is calculated in the worksheet; in manual calculations the actual size must be specified. Basic geometric and hydraulic quantities are calculated in lines 22 – 26. Line 27 calculates the partial flow depth fraction. This is a complicated calculation and for manual calculation Figure 12-5.6 should be used to look up the depth fraction h^* (= h/D) for a specified flow fraction Q^* ($=Q_{tot}/Q_f$). Once h^* has been calculated or looked up, the actual partial flow depth is calculated in line 28. And for the h^* value, the corresponding dimensionless partial velocity v^* is either calculated automatically or looked up, from which the partial flow velocity is calculated ($v = v^* v_f$) in line 29. Finally, knowing the partial flow velocity, the travel time in pipe is calculated in line 30.

For simple series configurations, this process is repeated until the outlet is reached.

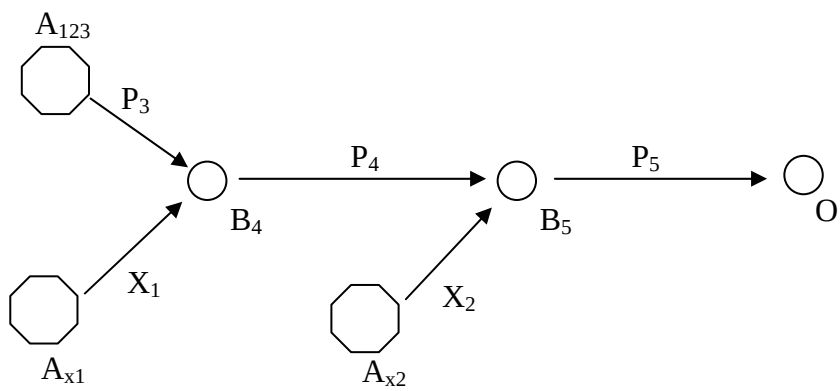
Table 12-5.8: Example of Closed System Design – Sizing Pipe P_2 by Manual Calculation

	Item	Source	Variable	Notes
1	Route/Street/Watershed ID	D	B_2	
2	From station	D	x_0 = 75 ft	From layout
3	To station	D	x_1 = 475 ft	From layout
4	Length	C	$x_1 - x_0$ = 225 ft	
5	Invert Elev Upper end	D	z_0 = 107 ft	From layout
6	Invert Elev Lower End	D	z_1 = 103.5 ft	From layout
7	Slope	C	$(x_1-x_0)/(z_1-z_0)$ = 0.016 ft/ft	
8	Time of Conc T_c	D	14.2 min	From hydrology
9	Incremental Area	D	A_{inc} = 8.8 ft^2	From hydrology
10	Total T_c	C	Max{14.2, 13.1} = 14.2 min	Max{incremental area T_c OR T_c of area entering catch basin by inlet pipe)
11	Total area A_{tot}	C	$A_{tot} = A_{prev} + A_{inc}$ = 7 + 8.8 = 15.8 ft^2	
12	Incremental Rational C	D	C = 0.72	From hydrology
13	Cumulative Weighted C	C	{7x.4 + 8.8x.72}/15.8 = 0.58	For total area draining to CB
14	Rainfall Rate	C/L	3.7 in/hr	10-yr intensity for specified region
15	Runoff Direct	C	$Q = \mu CiA$ = 33.4 ft^3/s	Rational Equation
16	Runoff offsite	D	0.0	Additional runoff source
17	Runoff total	C	$Q_{tot} = Q + Q_{off}$ = 33.4 ft^3/s	Total runoff to catch basin
18	Manning roughness n	D	0.013	
19	Flow depth fraction	D	1	Usually = 1
20	Pipe Diam (in)	C	$D = (1.55\alpha/\lambda^{3/8})\{nQ/S^{1/2}\}^{3/8}$ = 25.5 in	Exact solution
21	Pipe Diam (design;in)	C	D = 27 in = 2.25 ft	Rounded to next size
22	Pipe Area (full)	C	$A_f = \pi D^2/4$ = 3.97 ft^2	
23	Hydraulic Radius (full)	C	$R_{h,f}$ = D/4 = 0.5625 ft	
24	Pipe Full Velocity	C	$v_f = \lambda R_h^{2/3} S^{1/2}/n$ = 9.9 ft/s	Manning's equation
25	Pipe Full Capacity	C	$Q_f = Av$ = 39.1 ft^3/s	
26	Percent of Full Capacity	C	$Q^* = Q/Q_f$ = 100x33.4/39.1 = 85%	
27	Flow Depth Fraction	C/L	H^* = 0.70	Look up for Q^* value
28	Flow depth	C/L	$h = h^*D$= (0.70)(27) = 18.9 in	
29	Velocity	C/L	V* = 1.10 => $v = v^* v_f$ = 10.9 ft/s	Look up v^* for h^* value
30	Travel time in pipe	C	T_t = L/60v = 225/60/10.9 = 0.34 min	

Note: U.S. Customary, λ = 1.486, μ = 1, α = 12; metric, λ = 1, μ = 0.28, α = 1000

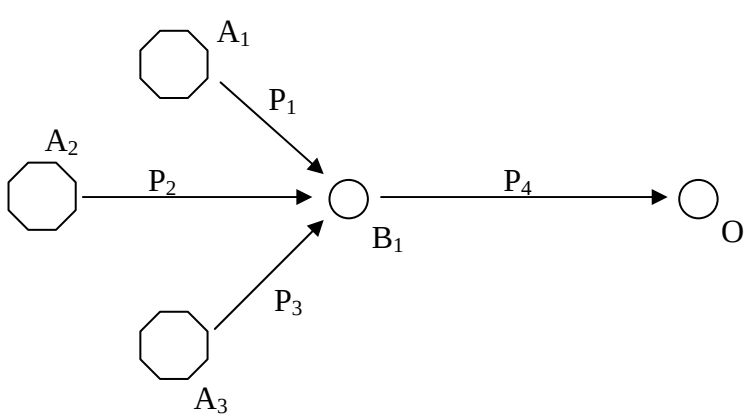
Source: D = specified data; C = calculation; L = look-up

As noted, this is an example of a simple branching network. Basins 1, 2, and 3 are in series; the system branches at B_4 and B_5. Looking upgradient from B_4, the network branches across the street to subwatershed A_{x1} and to the series of basins $B_3 – B_1$. The network can be redrawn with a single composite watershed A_{123} replacing A_1, A_2 and A_3. Pipe P_3 is already sized; cross-pipe X_1 is also sized in the usual manner. The composite watershed A_{123} is characterized by the sum of the component areas, the maximum of the component times of concentration (including pipe travel times), and the area-weighted average of the component runoff coefficients. Now pipe P_4 can also be sized in the usual manner, with two watersheds(A_{B3} and A_{x1}) draining to node B_4. Similarly, there are two watersheds draining to B_5, the composite A_{B4} (A_{B3} and A_{x1}) and A_{x2}. This treatment is illustrated in the Excel worksheet in Figure 12-5.9; manual calculations would proceed in the same manner. The compositing procedure is described in the next section for a triple branch.



<u>Branching to Three or More Stems</u>

A branch node with 3 or more stems might look like the following:



The single pipes $P_1 – P_3$ draining the individual watersheds are sized in the usual manner. The problem is to size the outlet pipe P_4 in the framework of the algorithm presented earlier. There are several approaches, probably the simplest being to combine the individual watersheds into a single composite. Following that, the regular algorithm can be applied. The Excel worksheet for the above schematic is shown in Figure 12-5.10. The individual inlet pipes $P_1 – P_3$ are sized in

the first three columns. In the fourth column, the outlet pipe P_4 is sized for the single composite watershed. Note that the area is sum of the individual areas, T_c is the maximum of the three individual T_c values, and the runoff coefficient C is the weighted average of the individual values. The procedure can be used for any number of pipes (and watersheds) entering a catch basin.

Figure 12-5.10: Closed System with Triple Branching

Closed System Worksheet - Preliminary Design of Simple Systems **Units: US**

Start data entry from bottom; enter data in blue cells only. **Design Event: 10**

1/10/2005	Design of Closed System	Partial Flow	30	Time in Pipe Section	min	0.6		0.6		0.7		0.6	
			29	Velocity (design) =V	ft/s	5.3		8.2		5.6		7.1	
			28	Flow Depth (design)	in	17		20		21		29	
			27	Flow Depth Fraction (design)		0.80		0.81		0.78		0.73	
			26	Percent of Capcity Q_F		98		99		96		89	
Pin # 9999.99		Full Flow	25	Pipe-full Capacity Q_F	ft3/s	11.2		22.62		19.59		52.22	
			24	Pipe-Full Velocity V_F	ft/s	4.7		7.2		4.9		6.3	
			23	Hydraulic Radius	ft	0.44		0.50		0.56		0.81	
			22	Pipe Area	ft2	2.41		3.14		3.98		8.30	
		Pipe Design	21	***Pipe Diam (design)***	***in***	***21***		***24***		***27***		***39***	
			20	Pipe Diam (exact)	in	21		24		27		37	
			19	Flow Depth Fraction (nom)		1		1		1		1	
			18	Manning Roughness Coeff = n		0.013		0.013		0.013		0.013	
	Runoff Calculation		17	Runoff Total (design) Q_T	ft3/s	11.0		22.5		18.742		46.331	
			16	Runoff Offsite = Q_{off}	ft3/s	0.0		0.0		0.0		0.0	
			15	Runoff Direct Q = µCiA	ft3/s	11.0		22.5		18.742		46.331	
			14	Rainfall Rate I	in/hr	3.9		3.6		3.1		3.1	
			13	Cum Wt'ed C		0.40		0.70		0.60		0.58	
Triple Branch Example			12	Total Area =A_T	ac	7.0		9.00		10.0000		26.0000	
			11	Total T_c (min)	min	12.00		15.00		20.00		20.74	
			10	Incremental Rational Coeff=C		0.4		0.70		0.60		0.58	
			9	Incremental Area A_i	ac	7.0		9.0		10.0		26.0	
			8	Time of Conc T_c	min	12.0		15.0		20.0		20.7	
	System	Topography	7	slope S	ft/ft	0.0050		0.0100		0.0040		0.0040	
			6	Inv Elev Lower End	ft	100.00		100.00		100.00		100.00	
			5	Inv Elev Upper End	ft	101.00		103.00		101.00		101.00	
			4	Length (m)	ft	200.00	0.00	300.00	0.00	250.00	0.00	250.00	0.00
Project:	Location		3	To Station (lower)	ft	200		300		250		250	
			2	From Station (upper)	ft	0		0		0		0	
			1	CB/Node		A_1		A_2		A_3		B_1	

Precip Station	1	(Portland-1/Eastport-2/Rangely-3/
Event Return Period (yrs)	10	Presque Isle-4/Newport-5/Millinocket-6)

Units

12-6 REFERENCES

Applied Microcomputer Systems, Inc., 2003. *HydroCad*, Version 7, Chocorua, NH.

Benson, M.A., 1962. Factors influencing the occurrence of floods in a humid region of diverse terrain. *Water-Supply Paper 1580-B*, U.S. Geological Survey, Lakewood, CO.

Dudley, R., G. Hodgkins, A. Mann, and J. Chisholm, 2001. Evaluation of the effects of suburban development on peak-flow hydrographs for Collyer Brook, Maine, *Water Resources Investigations Report 01-4156*, U.S Geological Survey, Augusta, ME.

Federal Highway Administration, *Hydraulic Design Series #5*, Hydraulic Design of Culverts, September 1985.

Federal Highway Administration, *Hydraulic Design Series #2*, Highway Hydrology, September 1997.

Federal Highway Administration, *Hydraulic Engineering Circular #12*, Drainage of Highway Pavement, March, 1984.

Federal Highway Administration, *Hydraulic Engineering Circular #14*, Energy Dissipators for Culverts and Channels, 2000.

Federal Highway Administration, *Hydraulic Engineering Circular #22 (2nd Ed.)*, Urban Drainage Manual, August, 2001.

Hodgkins, 1999. Estimating the Magnitude of Peak Flows for Streams in Maine for Selected Recurrence Intervals, *Water-Resources Investigations Report 99-4008*, U.S. Geological Survey, Augusta, ME.

Izzard, C.F., 1946. Hydraulics of runoff from developed surfaces. *Proc. Highway Research Board*, 26: 129-150, Highway Research Board, Washington, D.C.

Maidment, D.R., 1992. *Handbook of Hydrology*, McGraw-Hill, NY.

Maine DEP, 1995. Stormwater Management for Maine: Best Management Practices, Augusta, Maine, November.

Maine DOT, 1/2005. Fish Passage Policy and Design Guide, Augusta, Maine.

Maine DOT, 9/2002. Maine DOT Best Management Practices Guide, Augusta, Maine.

McCuen, R.H., 1989. *Hydrologic Analysis and Design*, Prentice Hall, NY.

Natural Resources Conservation Service, 1986. Urban hydrology for small watersheds. *Technical Release 55*, Washinton, DC.

Natural Resources Conservation Service, 1986. Computer program for project formulation, *Technical Release 20*, Washinton, DC.

Sauer, V.B., W.O. Thomas, V.A. Stricker, and K.V. Wilson, 1983. Flood characteristics of urban watersheds in the United States, *Water-Supply Paper 2207*, U.S. Geological Survey, Lakewood, CO.

Viessman, W., J.W. Knapp, G.L. Lewis, and T.E. Harbaugh, 1977. *Introduction to Hydrology*, IEP (Harper & Row), New York, pp. 507-514.

Washington State DOT, 1997. *Hydraulics Manual (M23-03)*, Environmental & Engineering Service Center/Hydraulics Branch, Olympia, WA, available on-line, pp. 2-3 – 2-9.

Appendix 12-A

Hydraulics of Flow in Gutters

HYDRAULICS OF FLOW IN GUTTERS

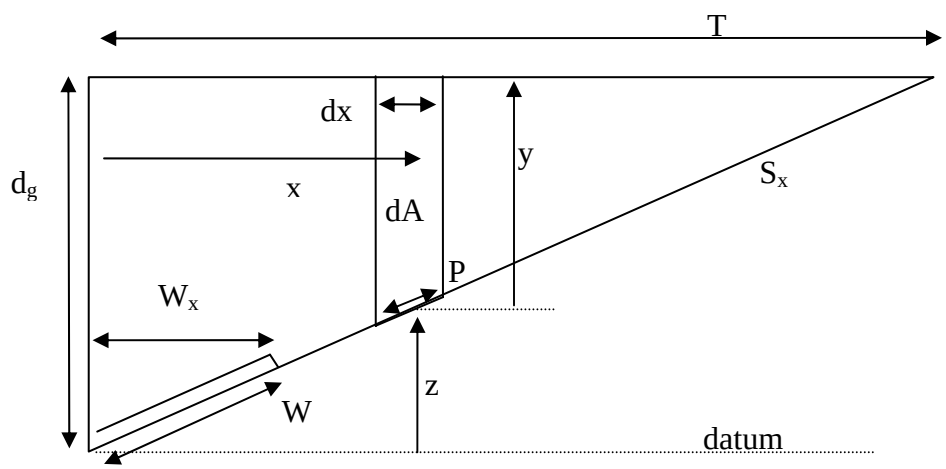
Equations for gutter flow hydraulics are developed using Manning's equation and gutter section geometry. Two section types are included in this manual, simple right-triangular and composite triangular. The equations developed here can be used to determine inlet catch basin spacing. The composite section is more efficient and has higher capacity than the simple triangular section. Often maximum spacing cannot be exploited because of other system constraints and requirements. In an urban setting, intersections usually require inlets and the intersections may be spaced closer than the hydrology and hydraulics would otherwise allow. However, on long straight runs, or on flat sections, composite gutter sections may allow for longer spacing between inlets, or simply provide the capacity unavailable with a uniform section.

Simple Right-Triangular Flow Section

The simplest gutter section is just a simple right triangle. In many applications this shape is adequate for the transmission of water closed drainage system inlets. The basic design parameters to be specified are spread T, cross-slope S_x, centerline slope S, and Manning's roughness n. Combined with hydrologic analysis, the hydraulic equation developed here can be used to calculate spacing between inlets.

Section Geometry

A simple right-triangular gutter flow section is depicted in the following schematic:



The section is defined by the following terms:

T - spread (in ft or m) of flow on the pavement
S_x - cross-section slope (ft/ft or m/m), uniform from curb to T
d_g - depth of flow at curb = S_xT
W - width of grated inlet structure

Not shown in the figure is S, the longitudinal (centerline) slope of the roadway. The other terms shown are utilized in the hydraulic analysis. Note that the reference elevation datum is the toe of curb. The depth of flow d_g at the curb is

$$d_g = S_x T$$

The final curb height should be higher than the depth of flow. The elevation of the pavement is

$$z = S_x x$$

Section Hydraulics

The flows of interest are the flow through the section and the flow directly intercepted by the inlet structure. Manning's equation for uniform flow is the basic tool of the hydraulic analysis:

$$v = \lambda(S^{1/2}/n)R_h^{2/3}$$

Where v = velocity
S = longitudinal (centerline) slope of roadway
n = Manning's roughness
λ = unit conversion factor (1.486 in U.S. Customary; 1 in metric)
R_h = hydraulic radius = A/P
A = cross-sectional flow area
P = wetted perimeter

Traditionally the flow Q (ft^3/s or m^3/s) is calculated by continuity as the simple product of sectional area and the flow velocity through the section. However, because the width of flow is much greater than the flow depth, flow is calculated here by integrating Manning's velocity equation over the sectional area (Izzard, 1946, p. 149; FHWA, 2001, p. 4-9) from the curb ($x_l = 0$) to the spread limit ($x_u = T$):

$$Q = \int v(x)dA(x) \qquad \textit{from } 0 \textit{ to } T$$
$$Q = \lambda(S^{1/2}/n) \int R_h(x)^{2/3}dA(x)$$

where x = distance from curb
v(x) = velocity at location x from curb
dA(x) = differential flow area at location x

From the section geometry,

$$
\begin{aligned}
dA(x) &= y(x) \times dx \\
y(x) &= \text{depth of flow at location } x \\
&= d_g - z = S_xT - S_xx = S_x(T-x) \\
z &= \text{elevation of pavement at location } x = S_xx \\
dx &= \text{differential flow width}
\end{aligned}
$$

It follows that

$$dA = S_x(T-x)dx$$

The wetted perimeter P of the incremental flow section dA is just the incremental bottom pavement dx; the vertical curb wall is ignored as it is so much smaller than the perimeter length across the section . By the Pythagorean Theorem

$$
\begin{aligned}
P^2 &= (dx)^2 + (dz)^2 = (dx)^2 + (S_xdx)^2 \\
P &= (1+S_x^2)^{1/2}dx
\end{aligned}
$$

Substituting into the hydraulic radius R_h yields

$$
\begin{aligned}
R_h &= dA/P \\
&= \{S_x(T-x)dx\}/\{(1+S_x^2)^{1/2}dx\} = \{S_x/(1+S_x^2)^{1/2}\}(T-x)
\end{aligned}
$$

.

Substituting for R_h into the integration for Q gives

$$
\begin{aligned}
Q &= \lambda(S^{1/2}/n)\ \{S_x/(1+S_x^2)^{1/2}\}^{2/3} \int (T-x)^{2/3}\{S_x(T-x)dx\} \\
&= \lambda(S^{1/2}/n)\ S_x\{S_x/(1+S_x^2)^{1/2}\}^{2/3} \int (T-x)^{5/3}dx \qquad \textit{from } x = 0 \text{ to } T
\end{aligned}
$$

The integration can be completed by substitution:

$$
\begin{aligned}
u &= T - x \quad \rightarrow \quad dx = -du \\
x_l &= 0 \quad \rightarrow \quad u_l = T \\
x_u &= T \quad \rightarrow \quad u_u = 0
\end{aligned}
$$

$$
\begin{aligned}
Q &= -\lambda(S^{1/2}/n)\ S_x\{S_x/(1+S_x^2)^{1/2}\}^{2/3} \int u^{5/3}du \ \textit{from } u = T \text{ to } 0 \\
&= \lambda(S^{1/2}/n)\ S_x\{S_x/\{(1+S_x^2)^{1/2}\}^{2/3} \int u^{5/3}du \textit{from } u = 0 \text{ to } T \\
&= (3/8)\lambda(S^{1/2}/n)\ S_x\{S_x/(1+S_x^2)^{1/2}\}^{2/3}u^{8/3} \qquad \textit{from } u = 0 \text{ to } T
\end{aligned}
$$

$$Q = (3/8)\lambda(S^{1/2}/n)\ S_x\{S_x/(1+S_x^2)^{1/2}\}^{2/3}T^{8/3}$$

Note how the leading negative is removed by switching the lower and upper integration limits. This operation will be performed in all subsequent integrations without explicit comment.

The cross-slope S_x is a very small number (typically $S_x = 0.02$) in uniform sections, so

$$(1+S_x^2)^{1/2} \cong 1$$

and

$$Q \cong (3/8)\lambda(S^{1/2}/n)S_x^{5/3}T^{8/3}$$

In terms of flow depth at the gutter,

$$Q \cong (3/8)\{\lambda S^{1/2}/nS_x\}d_g^{8/3}$$

These last two are the forms commonly cited in the standard references (FHWA, 2001, p. 4-9).

<u>Intercepted Flow and Interception Efficiency</u>

The flow intercepted directly by the grated inlet is calculated by integrating the flow equation over the interval $[0 \leq x \leq W_x]$, where W_x is the inlet width projected in the x-direction:

$$W_x = W/(1 + S_x^2)^{1/2}$$

As above, but with a different upper limit of integration ($x_u = W_x$ instead of T; $u_u = T-W_x$ instead of 0), the intercepted flow is

$$Q_w = (3/8)\lambda(S^{1/2}/n)\, S_x\{S_x/(1+S_x^2)^{1/2}\}^{2/3}u^{8/3} \qquad \textit{from } u = (T-W_x) \text{ to } T$$

$$= (3/8)\lambda(S^{1/2}/n)\, S_x\{S_x/(1+S_x^2)^{1/2}\}^{2/3}\{T^{8/3} - (T-W_x)^{8/3}\}$$

Employing the approximation $(1+S_x^2)^{1/2} \cong 1$,

$$Q_w \cong (3/8)\lambda(S^{1/2}/n)\, S_x^{5/3}\{T^{8/3} - (T-W_x)^{8/3}\}$$

The (geometric) interception efficiency E_o is the ratio of intercepted flow to total flow:

$$E_o = Q_w/Q = \{T^{8/3} - (T-W_x)^{8/3}\}/T^{8/3}$$

$$= 1 - \{1 - W_x/T\}^{8/3}$$

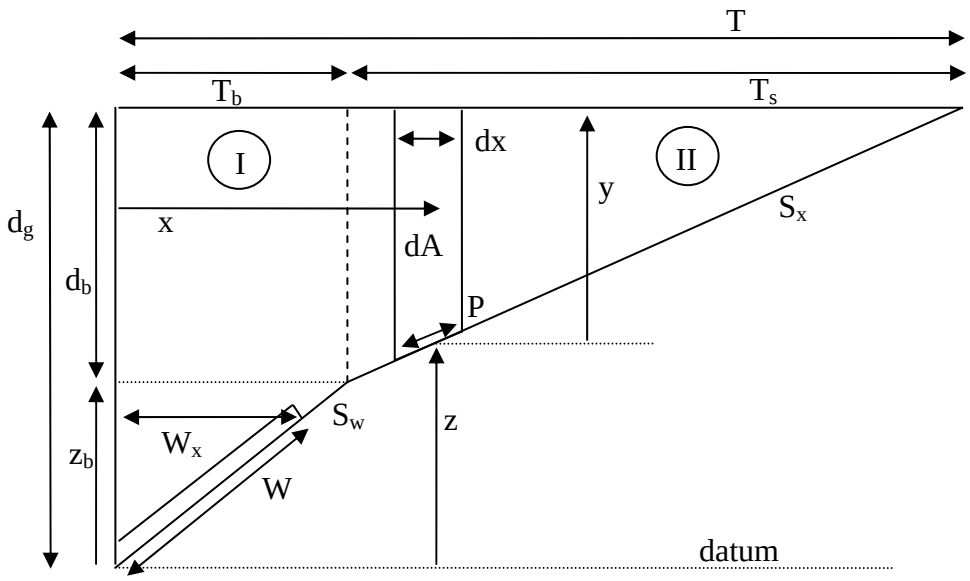
As before, W_x is closely approximated by the nominal grate width W for the small S_x values typical of gutters. Then

$$E_o \cong 1 - \{1 - W/T\}^{8/3}$$

This is the form commonly cited in standard references (FHWA, 2001, p. 4-9).

Composite Triangular Flow Section

The hydraulic capacity of a triangular section can be significantly increased by introducing a depressed triangular section in the shoulder area and outside the travelway. The composite section consists of a steeper cross-slope S_w at the curb and the regular travelway cross-slope S_x away from the curb. This increased capacity allows for greater spacing between inlets for a specified spread T, or a reduced spread T for a specified inter-inlet spacing.



The general approach is the same as taken for a simple triangular section, i.e., integrating Manning's equation over the flow section. However, since the section is a composite of two triangular sections, the integration is done in parts corresponding to Regions I and II in the figure above. The total flow is the sum of the two component flows:

$$Q = Q_I + Q_{II}$$

The basic design parameters to be specified are spread T and distance T_b to cross-slope break, cross-slopes S_x and S_w, inlet width W ($W \le T_b$) , centerline slope S, and Manning's roughness n. Combined with hydrologic analysis, the hydraulic equation developed here can be used to calculate spacing between inlets.

<u>Region I: $0 \le x \le T_b$</u>

Fixed Elevations and Flow Depths

Pavement elevation at break in cross-slope:

$$z_b = S_w T_b$$

Depth at break in cross-slope:

$d_b = S_x(T-T_b)$

Depth at edge of inlet in shoulder:

$d_w = d_b + z_b - z_w = S_x(T-T_b) + S_wT_b - S_wW_x = S_x(T-T_b) + S_w(T_b - W_x)$

Depth at gutter curb:

$$\begin{aligned} d_g &= d_b + z_b \\ &= S_x(T-T_b) + S_wT_b \\ &= S_w\{(S_x/S_w)(T-T_b) + T_b\} \end{aligned}$$

The final design curb height should be higher than the calculated curb flow depth.

Depth of Flow y as function of position x:

$$\begin{aligned} y &= d_g - z \\ &= d_g - S_wx = S_w\{(S_x/S_w)(T-T_b) + T_b\} - S_wx \\ &= S_w\{(S_x/S_w)(T-T_b) + T_b - x\} \end{aligned}$$

Differential Flow Area as function of position x:

$$\begin{aligned} dA &= y \times dx = (d_g-S_wx)dx \\ &= \{S_w\{(S_x/S_w)T_s + T_b\} - S_wx\}dx \\ &= S_w\{(S_x/S_w)(T-T_b) + T_b - x\}dx \end{aligned}$$

Wetted Perimeter of differential flow area:

$$P = (1+S_w^2)^{1/2}dx$$

Hydraulic Radius R_h :

$$\begin{aligned} R_h &= dA/P = \{(d_g-S_wx)dx\}/\{(1+S_w^2)^{1/2}dx\} \\ &= S_w\{(S_x/S_w)(T-T_b) + T_b - x\}/(1+S_w^2)^{1/2} \\ &= \{S_w/(1+S_w^2)^{1/2}\}\{(S_x/S_w)(T-T_b) + T_b - x\} \end{aligned}$$

Manning's Equation

$$\begin{aligned} v &= \lambda(S^{1/2}/n)R_h^{2/3} \\ &= \lambda(S^{1/2}/n)[\{S_w/(1+S_w^2)^{1/2}\}\{(S_x/S_w)(T-T_b) + T_b - x\}]^{2/3} \\ &= \lambda(S^{1/2}/n)\{S_w/(1+S_w^2)^{1/2}\}^{2/3}\{(S_x/S_w)(T-T_b) + T_b - x\}^{2/3} \end{aligned}$$

Flow Equation

$$Q_I = \int v(x)dA(x) \qquad \textit{from } x = 0 \text{ to } T_b$$
$$= \lambda(S^{1/2}/n)\{S_w/(1+S_w^2)^{1/2}\}^{2/3}$$
$$\times \int\{(S_x/S_w)(T-T_b)+T_b-x\}^{2/3}\, S_w\{(S_x/S_w)(T-T_b)+T_b-x\}dx$$
$$= \lambda(S^{1/2}/n)S_w\{S_w/(1+S_w^2)^{1/2}\}^{2/3} \int\{(S_x/S_w)(T-T_b)+T_b-x\}^{5/3}\, dx$$

Integrate by substitution:

Let $u = (S_x/S_w)(T-T_b)+T_b-x \rightarrow -du = dx$
$x_l = 0 \rightarrow u_l = (S_x/S_w)(T-T_b)+T_b$
$x_u = T_b \rightarrow u_u = (S_x/S_w)(T-T_b)$

$$Q_I = \lambda(S^{1/2}/n)S_w\{S_w/(1+S_w^2)^{1/2}\}^{2/3} \int u^{5/3}\, du$$
$$\textit{from } u = (S_x/S_w)(T-T_b) \textit{ to } (S_x/S_w)(T-T_b)+T_b$$

$$Q_I = \lambda(S^{1/2}/n)S_w\{S_w/(1+S_w^2)^{1/2}\}^{2/3}(3/8)u^{8/3}$$

$$Q_I = (3/8)\lambda(S^{1/2}/n)S_w\{S_w/(1+S_w^2)^{1/2}\}^{2/3}$$
$$\times [\{(S_x/S_w)(T-T_b)+T_b\}^{8/3} - \{(S_x/S_w)(T-T_b)\}^{8/3}]$$

Employing the approximation $(1+S_w^2)^{1/2} \cong 1$ and substituting with flow depths yields:

$$Q_I \cong (3/8)\, \lambda(S^{1/2}/n)S_w^{5/3}[\{(S_x/S_w)(T-T_b)+T_b\}^{8/3} - \{(S_x/S_w)(T-T_b)\}^{8/3}]$$

$$Q_I \cong (3/8)(\lambda S^{1/2}/n)\{d_g^{8/3} - d_b^{8/3}\}/S_w$$

<u>Region II: $T_b \le x \le T$</u>

Elevation of Pavement z:

$$S_x = (z - z_b)/(x - T_b)$$
$$z = S_x(x-T_b) + z_b = S_x(x-T_b) + S_wT_b = (S_w - S_x)T_b + S_xx$$

Depth of Flow y:

$$y = d_g - z = S_x(T - x)$$

Area:

$$dA = y \times dx = S_x(T - x)\, dx$$

Wetted Perimeter:

$$P = (1+S_x^2)^{1/2}dx$$

Hydraulic Radius R_h :

$$R_h = dA/P = \{S_x(T - x)dx\}/\{(1+S_x^2)^{1/2}dx\}$$
$$= \{S_x/(1+S_x^2)^{1/2}\}(T-x)$$

Manning's Equation

$$v = \lambda(S^{1/2}/n)R_h^{2/3}$$
$$= \lambda(S^{1/2}/n)[\{S_x/(1+S_x^2)^{1/2}\}(T-x)]^{2/3}$$

Flow Equation

$$Q_{II} = \int v(x)dA(x) \qquad \textit{from } x = T_b \text{ to } T$$
$$= \lambda(S^{1/2}/n)\int [\{S_x/(1+S_x^2)^{1/2}\}(T-x)]^{2/3}\, S_x(T-x\}dx$$
$$= \lambda(S^{1/2}/n)S_x\{S_x/(1+S_x^2)^{1/2}\}^{2/3} \int(T-x)^{5/3}\, dx$$

Integrate by substitution:

Let $u = T - x \rightarrow -du = dx$
$x_l = T_b \rightarrow u_l = T - T_b$
$x_u = T \rightarrow u_u = 0$

$$Q_{II} = \lambda(S^{1/2}/n)S_x\{S_x/(1+S_x^2)^{1/2}\}^{2/3} \int u^{5/3}\, du \qquad \textit{from } u = 0 \textit{ to } T - T_b$$
$$Q_{II} = \lambda(S^{1/2}/n)S_x\{S_x/(1+S_x^2)^{1/2}\}^{2/3}(3/8)u^{8/3}$$

$$Q_{II} = (3/8)\lambda(S^{1/2}/n)S_x\{S_x/(1+S_x^2)^{1/2}\}^{2/3}(T-T_b)^{8/3}$$

Employing the approximation $(1+S_x^2)^{1/2} \cong 1$ and substituting with flow depth:

$$Q_{II} \cong (3/8)\lambda(S^{1/2}/n)S_x^{5/3}(T-T_b)^{8/3}$$
$$Q_{II} \cong (3/8)(\lambda S^{1/2}/n)d_b^{8/3}/S_x$$

<u>Total Gutter Flow</u>

$$Q = Q_I + Q_{II}$$

$$Q = (3/8)\lambda(S^{1/2}/n)S_w\{S_w/(1+S_w^2)^{1/2}\}^{2/3}$$
$$\times [\{(S_x/S_w)(T-T_b)+T_b\}^{8/3} - \{(S_x/S_w)(T-T_b)\}^{8/3}]$$

$$+ (3/8)\lambda(S^{1/2}/n)S_x\{S_x/(1+S_x^2)^{1/2}\}^{2/3}(T-T_b)^{8/3}$$

This form, while complicated, is recommended for use in computer-based application. For manual calculations, the approximation in terms of flow depths is easier to use:

$$Q \cong (3/8)(\lambda S^{1/2}/n)\{d_g^{8/3} - d_b^{8/3}\}/S_w + (3/8)(\lambda S^{1/2}/n)d_b^{8/3}/S_x$$

$$Q \cong (3/8)(\lambda S^{1/2}/n)\{(d_g^{8/3} - d_b^{8/3})/S_w + d_b^{8/3}/S_x\}$$

where $d_b = S_x(T-T_b)$ and $d_g = S_wT_b + S_x(T-T_b)$.

<u>Intercepted Flow and Interception Efficiency</u>

The flow intercepted directly by the grated inlet is calculated by integrating the flow equation over the interval $[0 \le x \le W_x]$, where W_x is the inlet width projected in the x-direction ($W \le T_b$ always).

$$W_x = W/(1 + S_w^2)^{1/2}$$

As above for the flow in Region I, but with a different upper limit of integration ($x_u = W_x$ instead of T_b; $u_u = (S_x/S_w)(T-T_b)+T_b-W_x$ instead of $(S_x/S_w)(T-T_b)$), the intercepted flow is

$$Q_w = (3/8)\lambda(S^{1/2}/n)\, S_w\{S_w/(1+S_w^2)^{1/2}\}^{2/3}u^{8/3}$$

from $u = (S_x/S_w)(T-T_b)+T_b-W_x$ *to* $(S_x/S_w)(T-T_b)+T_b$

$$= (3/8)\lambda(S^{1/2}/n)\, S_w\{S_w/(1+S_w^2)^{1/2}\}^{2/3} \times [\{(S_x/S_w)(T-T_b)+T_b\}^{8/3} - \{(S_x/S_w)(T-T_b)+T_b-W_x\}^{8/3}]$$

Employing the approximation $(1+S_w^2)^{1/2} \cong 1$,

$$Q_w \cong (3/8)\lambda(S^{1/2}/n)\, S_w^{5/3}[\{(S_x/S_w)(T-T_b)+T_b\}^{8/3} - \{(S_x/S_w)(T-T_b)+T_b-W_x\}^{8/3}]$$

Substituting with flow depths gives

$$Q_w \cong (3/8)(\lambda S^{1/2}/n)\{d_g^{8/3} - d_w^{8/3}\}/S_w$$

The (geometric) interception efficiency E_o is the ratio of intercepted flow to total flow:

$$E_o = Q_w/Q = Q_w/(Q_I + Q_{II})$$

As before, W_x is closely approximated by the nominal grate width W for the small S_w values typical of gutters.

Appendix B

Tables in Metric Units

Table 12-1.2: Preferred Units for Hydrologic and Hydraulic Analysis and Design

Quantity	Symbol	Units
Flow Path Length	L	m or km
Area	A	km^2
Flow Rate	Q	m^3/s
Rainfall Intensity	I	mm/hr
Pipe Diameter	D	mm
Time of Concentration	t_c	min
Slope	S	m/m or m/km

Table 12-2.1: Methods[1] for Peak Flow Calculation for Culverts & Ditches

Watershed Area	Rural	Urban
< 1.25 km^2 (= 320 ac)	Rational and Modeling	Rational and Urban USGS[3]
1.25 – 2.5 km^2 (= 640 ac = 1 mi^2)	Rational, USGS[2,5] and Modeling[4]	Rational, Urban USGS and Modeling
> 2.5 km^2	USGS	Urban USGS

Notes:

1) At the discretion of MDOT, other methods may be used on a project-specific basis
2) USGS indicates USGS (Hodgkins, 1999) regression equations (Water-Resources Investigations Report 99-4008, USGS, Augusta, ME, 1999)
3) Urban USGS indicates regression equations with Sauer correction for urbanization as documented in Hodgkins (1999)
4) Modeling will generally be performed with TR-20 or equivalent. MDOT may approve alternative models on a project-specific basis. See further comments on use of modeling.
5) USGS is only a secondary method for small watersheds (<2.5 km^2)

Table 12.2-3: Limits for Peak Flow Regression Applicability

Minimum	Parameter	Maximum
2.5	< A (km^2) <	4,300
0.7	< W (%) <	27
----	< Benson Slope (m/km) <	50

Table 12-2.4: Coefficients for Maine Intensity-Duration-Frequency Curves

	Portland	Eastport	Rangeley	Presque Isle	Newport	Millinocket
10 yr - a	782.8	603.7	989.6	1017.2	804.1	850.7
b	0.686	0.665	0.754	0.807	0.722	0.744
c	8.133	6.466	8.208	8.357	7.158	8.172
50 yr - a	1050.1	914.6	1543.7	1399.4	1161.9	1280.4
b	0.691	0.688	0.790	0.809	0.738	0.769
c	8.956	7.744	10.019	9.779	8.803	9.512
100 yr - a	1208.9	1045.8	2096.0	1589.7	1236.8	1131.3
b	0.698	0.691	0.826	0.810	0.726	0.721
c	9.921	8.452	12.368	10.01	8.478	7.727

Note: IDF equation is $i = a/(t_d + c)^b$, i in(mm/hr), t_d in (min)

Table 12-2.7: Typical Manning's "n" and Hydraulic Radius Values (McCuen, 1989)

Land Use/Flow Regime	Manning's n	Hydraulic Radius R_h (mm)	k ($m^{2/3}$) = $R_h^{2/3}/n$	k ($ft^{2/3}$) = $1.486R_h^{2/3}/n$
Forest				
Light underbrush	0.4	67.0	0.41	1.4
Heavy ground litter	0.2	61.0	0.77	2.5
Grass				
Bermudagrass	0.41	45.7	0.31	1.0
Dense (Lawns, Playing fields)	0.24	36.6	0.46	1.5
Short	0.15	30.5	0.65	2.1
Short grass pasture	0.025	12.2	2.12	7.0
Conventional tillage				
With residue	0.19	18.3	0.37	1.2
No residue	0.09	15.2	0.67	2.2
Agricultural				
Cultivated straight row	0.04	36.6	2.75	9.1
Contour or strip crop	0.05	18.3	1.39	4.6
Trash fallow	0.045	15.2	1.36	4.5
Rangeland	0.13	12.2	0.41	1.3
Alluvial fans	0.017	12.2	3.11	10.3
Grassed waterways	0.095	305.0	4.77	15.7
Small upland gullies	0.04	152.5	7.13	23.5
Pavement (smooth; sheet flow)	0.011	18.3	6.30	20.8
Pavement (rubble; sheet flow)	0.025	61.0	6.19	20.4
Paved gutter	0.011	61.0	14.07	46.3

Note: $k(m^{2/3}) = k(ft^{2/3})/3.28$

Table 12-4.1

CIRCULAR CULVERT PIPES – WALL THICKNESSES

	Nominal Pipe Wall Thickness in (mm)							
	Corrugated Metal				Plastic	Reinforced Concrete		
	Option I		Option I/III		Option III	Option I/III		
Diameter (mm)	M 218	M 274	M-246 & Fiber Bonded	M 197	M 278	M 170 Class III Wall A	M 170 Class III Wall B	M 170 Class III Wall C
300	2.0	1.63	1.63	1.91	9.09	44.5	50.8	
375	2.0	1.63	1.63	1.91	11.1	47.6	57.2	
450	2.77	2.01	2.01	1.91		50.8	63.5	
525	2.77	2.01	2.01	1.91		57.2	69.9	
600	2.77	2.01	2.01	2.67		63.5	76.2	95.3
675	2.77	2.01	2.01	2.67		66.7	82.3	101.6
750	2.77	2.01	2.01	2.67		69.9	88.9	108
825	2.77	2.01	2.01			73.0	95.3	114.3
900	2.77	2.01	2.01	1.91		76.2	101.6	120.7
900[1]			2.01					
1050	3.51	2.77	2.77	2.67		88.9	114.3	133.4
1050[1]			2.01					
1200	3.51	2.77	3.51	2.67		101.6	127	146.1
1200[1]			2.01					
1350	4.27	3.51	3.51	2.67		114.3	139.7	158.8
1350[1]			2.01					
1500	4.27	3.51	3.51	2.67		127	152.4	171.5
1500[1]			2.01	3.43				
1650[1]			2.01	3.43		139.7	165.1	184
1800[1]			2.01	4.17		152.4	177.8	196.9
1950[1]			2.01	4.17			190.5	209.6
2100[1]			2.77	4.17			203	222.3

Metal Pipe values are for 67.8 mm x 12.7 mm corrugations unless diameter is followed by (1), which requires 76.2 mm x 25.4 mm corrugations for aluminum pipes and 76.2 mm x 25.4 mm or 127 mm x 25.4 mm corrugations for steel pipes; corrugations in (mm).

Option I pipes shall only be used for entrances. Fill heights over 4.5 m may require larger metal gages.

M 170 = Reinforced Concrete Pipe M 218 = Zinc-coated (galvanized) corrugated steel pipe

M 278 = Polyvinyl Chloride Pipe M 274 = Aluminum-coated (Type 2) corrugated steel pipe

M 197 = Corrugated Aluminum Alloy Pipe M 246 = Polymer precoated galvanized corrugated steel pipe

Fiber Bonded = MDPT Spec. 707.04

Table 12-4.2
PIPE ARCH CULVERTS – WALL THICKNESSES

Nominal Wall Thickness in (mm)			
	Corrugated Metal Pipe Arch Option III		
Nominal Size in (mm) Span x Rise	**M 246 & Fiber Bonded**	**M 197**	**Coated Steel Pipe Equivalents (mm)**
533 x 381	2.00	1.91	18 gage = 1.32
610 x 457	2.00	1.91	16 gage = 1.63
711 x 508	2.00	2.67	14 gage = 2.00
889 x 610	2.77	2.67	12 gage = 2.77
1016 x 787 [1]	2.00	1.91	10 gage = 3.51
1067 x 737 [2]	2.77	1.91	8 gage = 4.27
1168 x 914 [1]	2.00	2.67	
1245 x 838 [2]	3.51	2.67	**Aluminum Pipe Equivalents (mm)**
1346 x 1041 [1]	2.00	2.67	
1448 x 965 [2]	3.51	2.67	18 gage = 1.22
1524 x 1168 [1]	2.77	3.43	16 gage = 1.52
1626 x 1092 [2]	4.27	3.43	14 gage = 1.91
1676 x 1295 [1]	2.77	3.43	12 gage = 2.67
1854 x 1397 [1]	2.77	4.17	10 gage = 3.43
2057 x 1499 [1]	2.77	4.17	8 gage = 4.17

Metal pipe values are for 67.8 mm x 12.7 mm corrugations unless size is followed by a (1), which denotes 76.2 mm x 25.4 mm corrugations.

M 246 = Polymer pre-coated galvanized corrugated steel pipe
M 197 = Corrugated Aluminum Alloy Pipe
Fiber Bonded = MDOT Spec. 707.04

Minimum Cover is 1 m
(2) = Either size is acceptable

Table 12-4.3
COUPLING BAND WIDTH REQUIREMENTS

Nominal Corrugation (mm)	Nominal Pipe Inside Diam (mm)	Coupling Band Width			
		Annular Corrugated Bands		Helically Corrugated Bands	
		M 196	M 36	M 196	M 36
38 x 6.4	150	267	267	178	178
68 x 12.8	300 - 2100	267	267		
76.2 x 25.4	750 - 2100	305	305		
127 x 25.4	900 - 2100		508		

Helically corrugated pipe 300 mm diameter and larger shall have the ends rerolled to provide at least two annular corrugations.

Pipe with spiral corrugations shall have continuous helical lock seams.

M 196 = Corrugated Aluminum Alloy Pipe
M 36 = Corrugated Steel Pipe

Table 12-4.4
TYPES B & C UNDERDRAIN PIPE

Metal Pipe				Plastic Pipe Stiffness @ 5% Deflection			
Nominal Wall Thickness (mm)				PVC Pipe		Polyethylene Pipe	
Diameter (mm)	M 218	M 274 M 246	M 197	M 278	ASTM F 949	M 294 SP Dual-Wall Unanchored	M 252 SP Dual-Wall Unanchored
Type "B" 150	1.63	1.32	1.22	46	50		60
Type "C" 300	2.00	1.63	1.91	46		50	
375	2.00	1.63	1.91	46		42	
450	2.00	1.63	1.91			40	
525	2.00	1.63	1.91				
600	2.00	1.63	1.91			40	
750	2.77	2.00	2.67				
900	2.77	2.00	2.67				

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TABLE 12-4.5
MAXIMUM ALLOWABLE HEIGHT OF FILL IN METERS

Diameter (mm)	Class III	Class IV	Class V
300	3.48	4.99	6.41
375	3.57	5.27	6.58
450	3.64	5.37	6.70
600	3.73	5.50	6.86
750	3.80	5.59	6.97
900	3.85	5.66	7.05
1050	3.89	5.71	7.11
1200	3.92	5.75	7.16
1350	3.95	5.79	7.20
1500	3.98	5.82	7.23
1650	4.00	5.85	7.27
1800	4.03	5.88	7.30

Note: These fill heights have been derived assuming a soil mass of 2000 kg/m^3 (2 g/cm^3 = 125 lbs/ft^3) and a safety factor of 1.5 times the ultimate pipe strength.

TABLE 12-4.6
MAXIMUM ALLOWABLE HEIGHT OF FILL IN METERS
(Zero Projecting Conduit)

Diameter (mm)	Class III	Class IV	Class V
300	4.77	7.06	8.82
375	4.90	7.25	9.05
450	4.99	7.38	9.21
600	5.11	7.55	9.43
750	5.19	7.66	9.57
900	5.25	7.74	9.66
1050	5.30	7.81	9.74
1200	5.34	7.86	9.80
1350	5.37	7.90	9.84
1500	5.39	7.93	9.89
1650	5.42	7.96	9.92
1800	5.44	7.99	9.95

Note: These fill heights have been derived assuming a soil mass of 2000 kg/m^3 (2 g/cm^3 = 125 lbs/ft^3) and a safety factor of 1.5 times the ultimate pipe strength.

TABLE 12-4.7
MAXIMUM HEIGHTS OF FILL (Corrugated Metal Pipe)

Diameter	Std Thk (mm) /	Non-Std Thk (mm)/	Non-Std Thk (mm)/	Non-Std Thk

(mm)	Height of Fill (m)	Height of Fill (m)	Height of Fill (m)	(mm)/ Height of Fill (m)
300 & 375	1.63 / 0.5 – 13.7			
450	1.63 / 0.5 – 10.7	2.00 / 10.7 – 16.8		
525	1.63 / 0.5 – 10.7	2.00 / 10.7 – 15.2	2.77 / 15.2 – 16.8	
600	1.63 / 0.5 – 6.1	2.00 / 6.1 – 12.2	2.77 / 12.2 – 15.2	3.51 / 15.2 – 18.3
750	2.00 / 0.5 – 7.6	2.77 / 7.6 – 12.2	3.51 / 7.6 – 13.7	4.27 / 16.8 – 18.3
900	2.00 / 0.5 – 4.6	2.77 / 4.6 – 7.6	3.51 / 7.6 – 13.7	4.27 / 13.7 – 18.3
1050	2.77 / 0.5 – 6.1	3.51 / 6.1 – 10.7	4.27 / 10.7 – 18.3	
1200	2.77 / 0.5 – 7.6	3.51 / 6.1 – 15.2	4.27 / 15.2 – 18.3	
1350	2.77 / 0.5 – 6.1	3.51 / 6.1 – 12.2	4.27 / 12.2 – 15.2	
1500	3.51 / 0.5 – 7.6	4.27 / 7.6 – 13.7		
1650	3.51 / 0.5 – 6.1	4.27 / 6.1 – 12.2		
1800	4.27 / 0.5 – 9.1			

Notes:

1) This table applies to metal pipe with smoothlined corrugations and 67.8 mm x 12.7 mm corrugations.
2) Shop strut for pipe diameters of 1200 mm and larger.

TABLE 12-4.8
CORRUGATED STEEL PIPE ARCHES
(Corrugations of 67.8 mm x 12.7 mm)

Equivalent Pipe Diam (mm)	Span (mm)	Rise (mm)	Height of Fill Above Top of Arch (m)			
			0.5 – 1	1.2 – 1.5	1.8 – 3	3.4 – 4.6
			Wall Thickness (mm)			
375	450	279	1.52	1.52	1.52	1.52
450	559	330	1.52	1.52	1.52	1.52
600	737	450	1.91	1.91	1.91	1.91
750	900	559	1.91	1.91	1.91	1.91
900	1092	686	2.67	2.67	2.67	2.67
1050	1270	787	2.67	2.67	2.67	2.67
1200	1473	900	3.43	2.67	2.67	3.43
1350	1651	1016	3.43	3.43	3.43	4.17
1500	1823	1118	4.17	4.17	4.17	

Note: minimum cover is 450 mm (18 in).

Table 12-3.9

STRUCTURAL PLATE STEEL CIRCULAR PIPE

(corrugations of 150 mm x 50 mm)

			Height of Fill Above Top of Steel Pipe (meters)																	
Dia	**Area**	**Min**	Min	1.5 -	3 -	4.5 -	6 -	7.5 -	9 -	1.05	12 -	13.5	15 -	16.5	18 -	21 -	24 -	27 -	30 -	33 -
(mm)	**(m^2)**	**Filll (m)**	- 1.5	3	4.5	6	7.5	9	10.5	- 12	13.5	- 15	16.5	- 18	21	24	27	30	33	36
			For Steel Thicknesses (mm)																	
1500	1.77	0.45	2.77	2.77	2.77	2.77	2.77	2.77	3.51	3.51	3.51	3.51	4.27	4.27	4.78	5.54	5.54	6.32	7.11	●
1650	2.14	0.45	2.77	2.77	2.77	2.77	2.77	3.51	3.51	3.51	3.51	3.51	4.27	4.78	4.78	5.54	6.32	7.11	●	●
1800	2.54	0.45	2.77	2.77	2.77	2.77	2.77	3.51	3.51	3.51	4.27	4.27	4.78	4.78	5.54	6.32	6.32	7.11	●	♦
1950	2.99	0.45	2.77	2.77	2.77	2.77	2.77	3.51	3.51	4.27	4.27	4.78	4.78	5.54	6.32	6.32	7.11	●	●	♦
2100	3.46	0.45	2.77	2.77	2.77	2.77	3.51	3.51	3.51	4.27	4.27	4.78	4.78	5.54	6.32	7.11	●	●	♦	
2250	3.98	0.45	3.51	3.51	3.51	3.51	4.27	3.51	4.27	4.27	4.27	4.78	5.54	5.54	6.32	7.11	●	♦	♦	
2400	4.52	0.45	3.51	3.51	3.51	3.51	4.27	4.27	4.27	4.27	4.78	5.54	5.54	6.32	7.11	●	●	♦		
2550	5.11	0.6	3.51	3.51	3.51	3.51	4.27	4.27	4.27	4.78	4.78	5.54	6.32	6.32	7.11	●	♦	♦		
2700	5.73	0.6	3.51	3.51	3.51	3.51	4.27	4.27	4.27	4.78	5.54	5.54	6.32	7.11	●	●	♦			
2850	6.38	0.6	3.51	3.51	3.51	3.51	4.78	4.27	4.78	4.78	5.54	5.54	6.32	7.11	●	♦	♦			
3000	7.07	0.6	3.51	3.51	3.51	3.51	4.78	4.27	4.78	5.54	5.54	6.32	7.11	●	●	♦				
3150	7.79	0.6	3.51	3.51	3.51	3.51	4.78	4.27	4.78	5.54	5.54	6.32	7.11	●	●	♦				
3300	8.55	0.6	4.27	3.51	3.51	3.51	4.78	4.78	5.54	5.54	6.32	7.11	●	●	♦	♦				
3450	9.35	0.6	4.27	3.51	3.51	4.27	4.78	4.78	5.54	5.54	6.32	7.11	●	●	♦					
3600	10.17	0.6	4.27	4.27	4.27	4.27	4.78	4.78	5.54	6.32	6.32	7.11	●	●	♦					
3750	11.04	0.6	4.27	4.27	4.27	4.27	4.78	4.78	5.54	6.32	7.11	●	●	♦						
3900	11.95	0.6	4.27	4.27	4.27	4.27	4.78	5.54	6.32	6.32	7.11	●	●	♦						
4050	12.88	0.6	4.78	4.27	4.27	4.27	4.78	5.54	6.32	6.32	7.11	●	♦	♦						
4200	13.85	0.6	4.78	4.27	4.27	4.78	4.78	5.54	6.32	7.11	●	●	♦	♦						
4350	14.86	0.6	4.78	4.78	4.78	4.78	5.54	5.54	6.32	7.11	●	♦	♦							
4500	15.90	0.6	4.78	4.78	4.78	5.54	5.54	6.32	6.32	7.11	●	♦	♦							

● 7.11 mm thick: 6 bolts per 0.30 m long seam

♦ 7.11 mm thick: 8 bolts per 0.30 m long seam

All other thicknesses: 4 bolts Per 0.30 m long seam

Notes:
1. As design requires for added resistance to abrasion and/or corrosion, use next heavier thickness (maximum 7.11 mm) for bottom plate(s).
2. All structural plate pipes must be 5% elliptical.
3. 4- and 6-plate pipes should have 1 bottom plate; 8- and 10-plate pipes should have 3 bottom plates.

Table 12-4.10

STEEL STRUCTURAL PLATE PIPE ARCHES

(Corrugations of 150 mm x 50 mm)

Span (m)	Rise (m)	Area (m^2)	Corner Plate Radius (mm)	Min Fill (m)	Height of Fill Above Top of Pipe Arches (m)											
					0.6	0.9	1.2	1.5–2.1	2.4	2.7	3.0	3.3	3.6	3.9	4.3	4.6
					For Steel Thickness (mm)											
1.85	1.40	2.04	457	0.60	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	3.51
1.93	1.45	2.23	457	0.60	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	3.51
2.06	1.50	2.42	457	0.60	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	3.51
2.13	1.55	2.60	457	0.60	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	3.51
2.21	1.60	2.88	457	0.60	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	2.77	3.51	3.51
2.34	1.65	3.07	457	0.60	3.51	3.51	3.51	2.77	2.77	2.77	2.77	2.77	2.77	2.77	3.51	3.51
2.41	1.70	3.25	457	0.60	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51
2.49	1.75	3.53	457	0.75		3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51
2.62	1.80	3.72	457	0.75		3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51
2.69	1.85	4.00	457	0.75		3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51
2.84	1.91	4.28	457	0.75		3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51
2.90	1.96	4.55	457	0.75		3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51
2.97	2.01	4.83	457	0.75		3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	4.27
3.13	2.06	5.11	457	0.75		3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	4.27	4.27
3.25	2.11	5.39	457	0.75		3.51	3.51	3.51	3.51	3.51	3.51	3.51	3.51	4.27	4.27	4.27
3.33	2.16	5.67	457	0.75		4.27	4.27	3.51	3.51	3.51	3.51	3.51	3.51	4.27	4.27	4.77
3.48	2.21	5.95	457	0.75		4.27	4.27	4.27	3.51	3.51	3.51	3.51	4.27	4.27	4.27	4.77
3.53	2.26	6.23	457	0.75		4.27	4.27	4.27	3.51	3.51	3.51	3.51	4.27	4.27	4.77	4.77
3.63	2.31	6.60	457	0.75		4.27	4.27	4.27	4.27	4.27	4.27	4.27	4.27	4.27	4.77	4.77
3.76	2.36	6.88	457	0.75		4.27	4.27	4.27	4.27	4.27	4.27	4.27	4.27	4.27	4.77	4.77
3.81	2.41	7.25	457	0.75		4.27	4.27	4.27	4.27	4.27	4.27	4.27	4.27	4.27	4.77	4.77
3.86	2.46	7.53	457	0.75		4.27	4.27	4.27	4.27	4.27	4.27	4.27	4.27	4.77	4.77	
3.91	2.54	7.90	457	0.75		4.27	4.27	4.27	4.27	4.27	4.27	4.27	4.27	4.77	4.77	
4.09	2.57	8.27	457	0.75		4.77	4.27	4.27	4.27	4.27	4.27	4.77	4.77	4.77	4.77	
4.24	2.62	8.64	457	0.75		4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77		
4.29	2.67	9.01	457	0.75		4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77		
4.34	2.72	9.39	457	0.75		4.77	4.77	4.77	4.77	4.77	4.77	4.77	5.54			
4.52	2.77	9.76	457	0.75		4.77	4.77	4.77	4.77	4.77	5.54	5.54	5.54			
4.67	2.82	10.13	457	0.75		5.54	4.77	4.77	4.77	4.77	5.54	5.54	5.54			USE
4.73	2.87	10.50	457	0.75		5.54	5.54	4.77	4.77	4.77	5.54	5.54				
4.78	2.92	10.97	457	0.75		5.54	5.54	4.77	4.77	4.77	5.54	5.54				787 mm RADIUS
4.83	3.00	11.34	457	0.75		5.54	5.54	4.77	4.77	4.77	5.54	5.54				
5.01	3.02	11.71	457	0.75		5.54	5.54	5.54	5.54	5.54	5.54					STRUCTURES
5.06	3.07	12.17	457	0.75		5.54	5.54	5.54	5.54	5.54	5.54					
4.04	2.84	9.12	787	0.75		4.77	4.27	4.27	4.27	4.27	4.27	4.77	4.77	4.77	4.77	4.77
4.12	2.90	9.48	787	0.75		4.77	4.27	4.27	4.27	4.27	4.27	4.77	4.77	4.77	4.77	4.77
4.27	2.95	9.85	787	0.75		4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77
4.32	3.00	10.22	787	0.75		4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77
4.40	3.05	10.69	787	0.75		4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77	5.54	5.54
4.55	3.10	11.06	787	0.75		4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77	5.54	5.54
4.67	3.15	11.52	787	0.75		5.54	4.77	4.77	4.77	4.77	4.77	4.77	4.77	5.54	5.54	5.54
4.75	3.20	11.99	787	0.75		5.54	5.54	4.77	4.77	4.77	4.77	4.77	4.77	5.54	5.54	5.54
4.83	3.25	12.36	787	0.75		5.54	5.54	4.77	4.77	4.77	4.77	4.77	5.54	5.54	5.54	5.54
4.95	3.30	12.83	787	0.75		5.54	5.54	5.54	4.77	4.77	4.77	4.77	5.54	5.54	5.54	5.54
5.03	3.35	13.29	787	0.75		5.54	5.54	5.54	5.54	5.54	5.54	5.54	5.54	5.54	5.54	
5.18	3.40	13.75	787	1.07			5.54	5.54	5.54	5.54	5.54	5.54	5.54	5.54	5.54	
5.23	3.45	14.22	787	1.07			5.54	5.54	5.54	5.54	5.54	5.54	5.54	5.54	5.54	
5.31	3.51	14.68	787	1.07			5.54	5.54	5.54	5.54	5.54	5.54	5.54	5.54		
5.46	3.56	15.15	787	1.07			5.54	5.54	5.54	5.54	5.54	5.54	6.32	6.32		
5.51	3.61	15.61	787	1.07			5.54	5.54	5.54	5.54	5.54	6.32	6.32	6.32		
5.67	3.66	16.17	787	1.07			5.54	5.54	5.54	5.54	5.54	6.32	6.32	6.32		
5.72	3.71	16.64	787	1.07			5.54	5.54	5.54	5.54	6.32	6.32	6.32			
5.89	3.76	17.19	787	1.07			5.54	5.54	5.54	5.54	6.32	6.32	6.32			
5.95	3.81	17.66	787	1.07			5.54	6.32	6.32	6.32	6.32	6.32	6.32			
6.00	3.86	18.22	787	1.07			5.54	6.32	6.32	6.32	6.32	6.32	6.32			
6.07	3.91	18.77	787	1.07			5.54	6.32	6.32	6.32	6.32	6.32	6.32			
6.22	3.96	19.33	787	1.07			6.32	6.32	6.32	6.32	6.32	6.32				
6.28	4.01	19.89	787	1.07			6.32	6.32	6.32	6.32	6.32	6.32				

Note: for abrasion/corrosion resistance, use next heavier thickness (max 7.11 mm) for bottom and corner plates.

Table 12-4.11
STEEL STRUCTURAL PLATE ARCHES
(Corrugations of 150 mm x 50 mm)

Span (m)	Min Cover (m)	Height of Fill Above Top of Steel Structural Plate Arch (m)								
		0.6	0.9	1.2	1.5	1.8	2.1	2.4	2.7	3.0
		For Steel Thickness (mm)								
1.2 - 3	0.6						2.77	2.77	2.77	2.77
3.3	0.6	3.51	3.51	2.77	2.77		2.77	2.77	2.77	2.77
3.6	0.6	3.51	3.51	2.77	2.77		2.77	2.77	3.51	3.51
3.9	0.6	4.27	3.51	2.77	2.77		2.77	3.51	3.51	4.27
4.2	0.6	4.27	4.27	3.51	3.51		3.51	4.27	4.27	4.27
4.5	0.6	4.78	4.27	3.51	3.51		3.51	4.27	4.27	4.78
4.8	0.6	5.54	4.78	4.27	4.27		4.27	4.78	4.78	5.54
5.1	0.6	5.54	5.54	4.78	4.27	4.27	4.27	4.78	5.54	6.32
5.4	0.6	6.32	5.54	4.78	4.78	4.27	4.78	5.54	6.32	6.32
5.7	0.6	7.1 *	6.32	5.54	5.54	4.78	5.54	6.32	6.32	7.1
6.0	0.6	7.1 *	7.1 *	6.32	5.54	5.54	5.54	6.32	7.1	
6.3	0.6		7.1 *	6.32	6.32	5.54	6.32	7.1		
6.6	0.6			7.1	6.32	6.32	6.32	7.1		
6.9	0.6				7.1	6.32	7.1			
7.2	0.6					7.1	7.1			
7.5	0.6					7.1				

Note: * Not to be used when rise to span ratio is 0.3 or less.

Table 12-4.12
ALUMINUM ALLOY CORRUGATED CULVERT

Culvert Diameter (mm)	Type of Shape	Minimum Recommended Cover (mm) [a]	Maximum Height of Fill (m) for Pipe Thicknesses (mm)				
			1.5	1.9	2.7	3.4	4.2
300	Full Circle	200	10.7	12.2	15.2		
375	Full Circle	200	9.8	10.7	12.2		
450	Full Circle	200	7.9	9.1	10.7		
525	Full Circle	225	6.4	7.6	9.1		
600	Full Circle	225	4.0	6.4	9.1		
750	Full Circle	225		5.8	7.6	9.1	
	5% Vertically Elongated	225		7.3	9.1	10.7	
900	Full Circle	250		3.0	5.5	7.6	9.1
	5% Vertically Elongated	250			6.4	9.1	10.7
1050	Full Circle	305			4.9	6.1	7.6
	5% Vertically Elongated	305			6.1	7.6	9.1
	5% Field Strutted (b)	305			9.2	10.7	12.2
1200	Full Circle	380			4.6	6.1	7.6
	5% Vertically Elongated	380			9.1	7.6	9.1
	5% Field Strutted (b)	380			4.6	10.7	12.2
1350	Full Circle	380			5.5	6.1	7.6
	5% Vertically Elongated	380			7.6	6.7	9.1
	5% Field Strutted (b)	380				9.1	10.7
1500	Full Circle	460				4.3	5.5
	5% Vertically Elongated	460				5.1	7.6
	5% Field Strutted (b)	460				7.6	9.1
1650	Full Circle	535				4.0	5.1
	5% Vertically Elongated	535				4.6	6.1
	5% Field Strutted (b)	535				7.6	9.2
1800	Full Circle	610				3.7	4.6
	5% Field Strutted (b)	610				6.1	7.6
1950	5% Field Strutted (b)	610				4.9	6.1
2100	5% Field Strutted (b)	610					4.6
2400	5% Field Strutted (b)	610					3.0

a: For the special case of heavy construction wheeled vehicles, use 0.6 m (2 ft) cover on 900 mm (36 in) pipe and 2/3 of the diameter at greater than 900 mm diameter.

b: Field strutting is defined as shaping pipe elliptically by wire or timber strutting or careful, thorough compaction of backfill around pipe during installation.

The following apply: Loading: AASHTO – H2O Highway Shape: 67.8 mm x 12.7 mm (2-2/3 in x ½ in)
Table values are for 85% or greater compaction.

Table 12-4.13
CORRUGATED ALUMINUM PIPE ARCHES
(Corrugations of 67.8 mm x 12.7 mm)

Arch Span and Rise (mm) Helical or Annular	Minimum and Maximum Height of Cover (m) For Various Metal Thicknesses (mm)				
	1.52	1.91	2.67	3.43	4.19
432 x 330	0.3 – 4.6				
533 x 381	0.3 – 4.6				
610 x 457	0.3 – 4.3	0.3 – 4.9			
711 x 508		0.3 – 4.3			
889 x 610		0.3 – 4.0			
1067 x 737			0.4 – 4.0	0.4 – 4.9	
1245 x 838			0.4 – 3.7	0.4 – 4.9	
1448 x 965			0.4 – 2.7	.04 – 3.7	
1626 x 1092				0.45 – 3.4	0.45 – 4.3
1803 x 1194				0.6 – 2.7	0.6 – 3.4
1956 x 1321				0.9 – 2.7	0.6 – 3
2108 x 1448					0.6 – 3

Table 12-4.16
CROSS-SECTIONAL END AREAS

Round Pipe			Pipe Arch	
Normal Diameter	End Area (m^2)	Thickness (mm)	End Area (m^2)	Formed Diameter (mm)
300	0.071	1.52	0.067	356 x 254
375	0.110	1.52	0.102	432 x 330
450	0.159	1.52	0.139	533 x 381
525	0.216	1.52	0.204	610 x 457
600	0.283	1.91	0.260	711 x 508
750	0.442	1.91	0.409	889 x 610
900	0.636	2.67	0.595	1067 x 737
1050	0.866	2.67	0.809	1247 x 838
1200	1.131	2.67	1.059	1448 x 965
1350	1.431	2.67	1.329	1626 x 1092
1500	1.767	3.43	1.636	1803 x 1194
1650	2.138	3.43	1.988	1956 x 1321
1800	2.545	4.17	2.365	2108 x 1448

Note: circular pipe $A = \pi D^2/4$ (D in meters)

pipe arch $A = 0.752(w \times h)^{1.023}$ (approximate; w & h in meters)

TABLE 12-4.17
MULTIPLE-PIPE EQUIVALENCES

Diameter of Larger Pipe (mm)	**Diameter of Smaller Pipe (mm)**									
	300	**375**	**450**	**525**	**600**	**750**	**900**	**1050**	**1200**	**1350**
300	1									
375	1.7	1								
450	2.5	1.5	1							
525	3.6	2.2	1.4	1						
600	5	3	2	1.4	1					
750	8	5	3	2.3	1.7	1				
900	12	8	5	3.5	2.5	1.5	1			
1050	18	11	7	5	3.6	2.2	1.4	1		
1200	24	15	10	7	5	3	1.9	1.4	1	
1350	32	19	13	9	6.5	4	2.6	1.8	1.3	1
1500	41	25	16	11	8	5	3.3	2.3	1.7	1.3
1650	51	29	20	14	10	6	4	2.8	2	1.6
1800	63	37	25	17	12	7.5	5	3.5	2.5	1.9
2100	90	53	35	25	18	11	7	5	3.6	2.8

Example: One 600 mm diameter culvert is equivalent to five 300 mm culverts or two 450 mm culverts in hydraulic capacity.

Table 12-5.4
Circular Pipe Geometric and Conveyance Functions

				Conveyance C (m^3/s)	
Diameter D (mm)	**Area A (m^2)**	**Perimeter P (m)**	**Hydraulic Radius R_h (m)**	**N = 0.012**	**n = 0.021**
150	0.018	0.471	0.038	0.165	0.094
300	0.071	0.942	0.075	1.047	0.598
450	0.159	1.414	0.113	3.086	1.764
600	0.283	1.885	0.150	6.648	3.799
750	0.442	2.356	0.188	12.054	6.888
900	0.636	2.827	0.225	19.602	11.201
1050	0.866	3.299	0.263	29.570	16.897
1200	1.131	3.770	0.300	42.219	24.125
1350	1.431	4.241	0.338	57.801	33.029
1500	1.767	4.712	0.375	76.554	43.745
1650	2.138	5.184	0.413	98.711	56.406
1800	2.545	5.655	0.450	124.493	71.139
1950	2.986	6.126	0.488	154.119	88.068
2100	3.464	6.597	0.525	187.799	107.314
2250	3.976	7.069	0.563	225.738	128.993
2400	4.524	7.540	0.600	268.138	153.222
2550	5.107	8.011	0.638	315.194	180.111
2700	5.726	8.482	0.675	367.098	209.770
2850	6.379	8.954	0.713	424.039	242.308
3000	7.069	9.425	0.750	486.202	277.830

Notes: for Manning's equation in fully flowing circular pipe (units of meters & sec):

$v = R_h^{2/3} S^{1/2}/n$

$A = \pi D^2/4$

$P = \pi D$

$R_h = D/4$

$C = (\pi/4^{5/3}) D^{8/3}/n = (0.312/n)\, D^{8/3}$

$Q = CS^{1/2}$

Table 3a
OPEN AREA IN EMBEDDED ELLIPTICAL PIPE (metric)

	Span (m)	Rise (m)	Open Area (m^2) Depth of Embedding (mm)			
			0 mm	150	225	300
Corner Radius = 457 mm	1.855	1.397	2.048	1.854	1.733	1.602
	1.931	1.448	2.231	2.061	1.936	1.800
	2.058	1.499	2.433	2.275	2.143	2.002
	2.134	1.550	2.630	2.450	2.313	2.165
	2.210	1.601	2.838	2.638	2.493	2.338
	2.337	1.651	3.062	2.876	2.727	2.565
	2.414	1.702	3.275	3.068	2.911	2.743
	2.490	1.753	3.504	3.272	3.105	2.929
	2.617	1.804	3.743	3.533	3.371	3.185
	2.693	1.855	3.985	3.750	3.573	3.383
	2.846	1.905	4.255	4.041	3.866	3.672
	2.896	1.956	4.503	4.278	4.080	3.878
	2.973	2.007	4.767	4.501	4.303	4.092
	3.125	2.058	5.049	4.817	4.623	4.409
	3.252	2.109	5.343	5.123	4.923	4.740
	3.328	2.160	5.634	5.395	5.196	4.972
	3.481	2.210	5.950	5.727	5.541	5.321
	3.532	2.261	6.235	5.994	5.785	5.561
	3.608	2.312	6.544	6.283	6.064	5.820
	3.760	2.363	6.887	6.643	6.441	6.203
	3.811	2.414	7.194	6.932	6.706	6.461
	3.862	2.464	7.522	7.236	7.026	6.729
	3.913	2.541	7.945	7.628	7.374	7.100
	4.090	2.566	8.221	7.937	7.700	7.426
	4.243	2.617	8.600	8.335	8.115	7.854
	4.294	2.668	8.946	8.662	8.417	8.147
	4.345	2.718	9.302	8.994	7.823	8.444
	4.522	2.769	9.720	9.434	9.197	8.943
	4.675	2.820	10.122	9.855	9.631	9.367

	Span (m)	Rise (m)	Open Area (m^2) Depth of Embedding (mm)			
			0 mm	150	225	300
Corner Radius = 787 mm	4.726	2.871	10.497	10.212	9.974	9.693
	4.776	2.922	10.884	10.579	10.325	9.810
	4.827	2.998	11.399	11.071	10.798	10.478
	5.005	3.023	11.729	11.425	11.171	10.872
	5.056	3.074	12.135	11.809	11.538	11.217
	4.040	2.846	9.080	8.833	8.615	8.391
	4.116	2.896	9.461	9.197	8.977	8.728
	4.268	2.947	9.880	9.629	9.420	9.174
	4.319	2.998	10.247	9.981	9.756	9.503
	4.395	3.049	10.646	10.360	10.123	9.854
	4.548	3.100	11.087	10.819	10.595	10.331
	4.675	3.150	11.511	11.254	11.039	10.787
	4.751	3.201	11.934	11.663	11.436	11.170
	4.827	3.252	12.370	12.073	11.826	11.535
	4.954	3.303	12.809	12.534	12.306	12.038
	5.030	3.354	13.255	12.966	12.724	12.442
	5.183	3.404	13.739	13.447	13.205	12.919
	5.234	3.455	14.017	13.724	13.481	13.193
	5.310	3.506	14.645	14.337	14.079	13.777
	5.462	3.557	15.153	14.859	14.615	14.326
	5.513	3.608	15.608	15.300	15.042	14.738
	5.666	3.659	16.131	15.835	15.589	15.298
	5.716	3.709	16.605	16.294	16.036	15.730
	5.869	3.760	17.147	16.847	16.598	16.305
	5.945	3.811	17.662	17.347	17.087	16.779
	5.996	3.862	18.160	17.830	17.559	17.237
	6.072	3.913	18.693	18.348	18.059	17.719
	6.225	3.963	19.257	18.928	18.654	18.331
	6.275	4.014	19.772	19.427	19.139	18.799

Appendix C

Figures in Metric Units

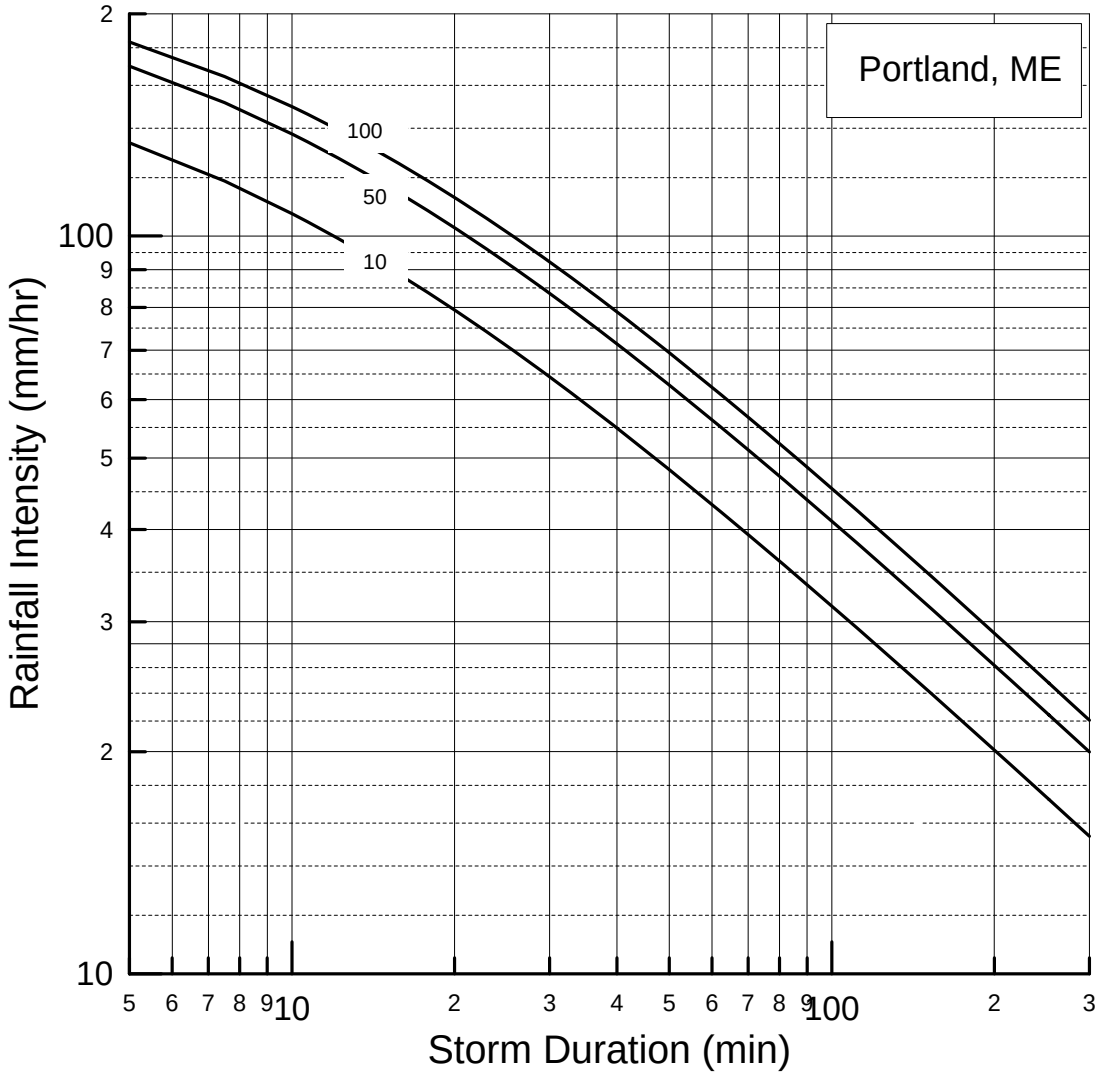

Figure 12-2.1b: Intensity-Duration-Frequency Curve, Portland, Maine (labeled return period in years).

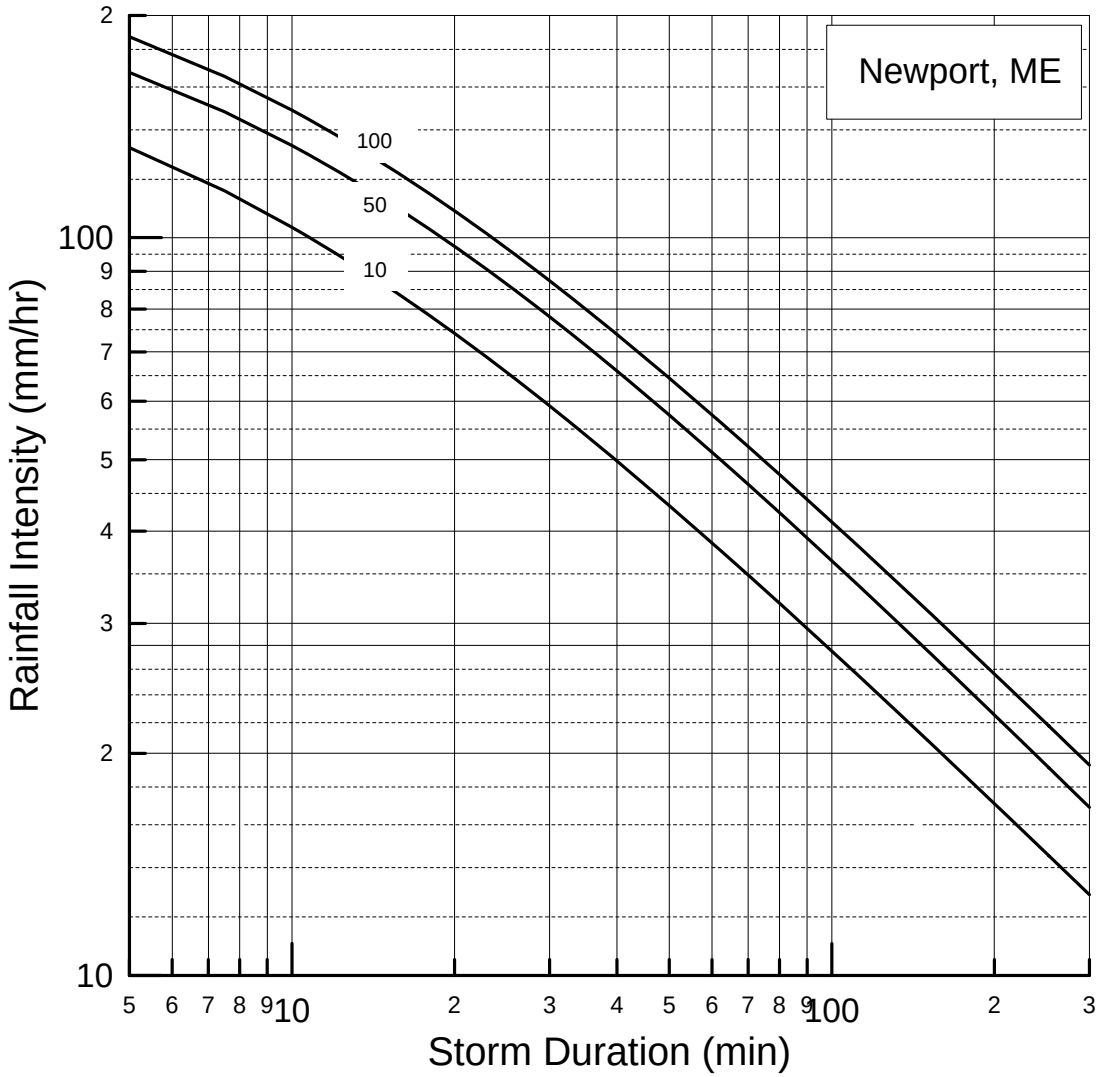

Figure 12-2.1c: Intensity-Duration-Frequency Curve, Newport, Maine (labeled return period in years).

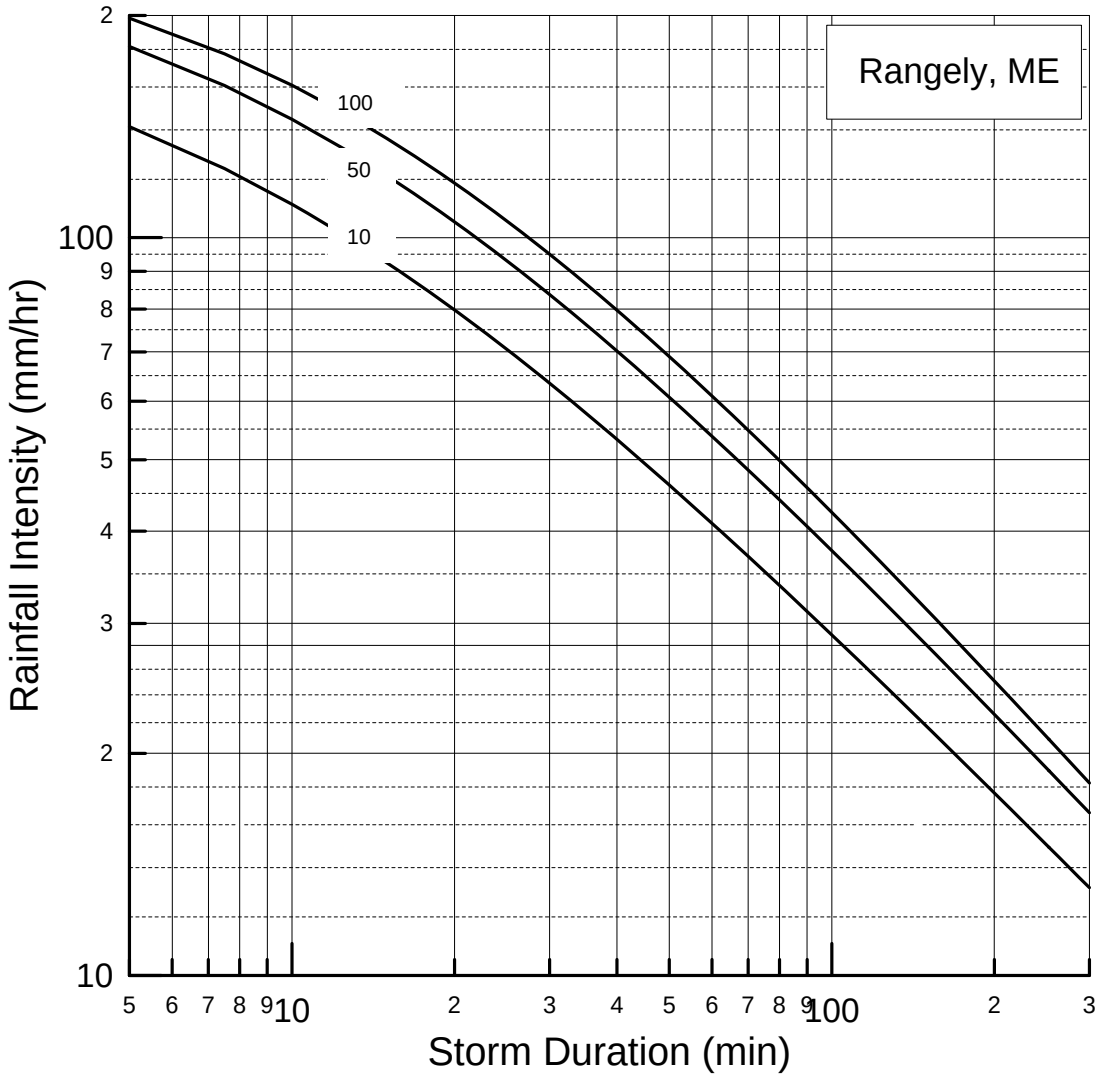

Figure 12-2.1d: Intensity-Duration-Frequency Curve, Rangely, Maine (labeled return period in years).

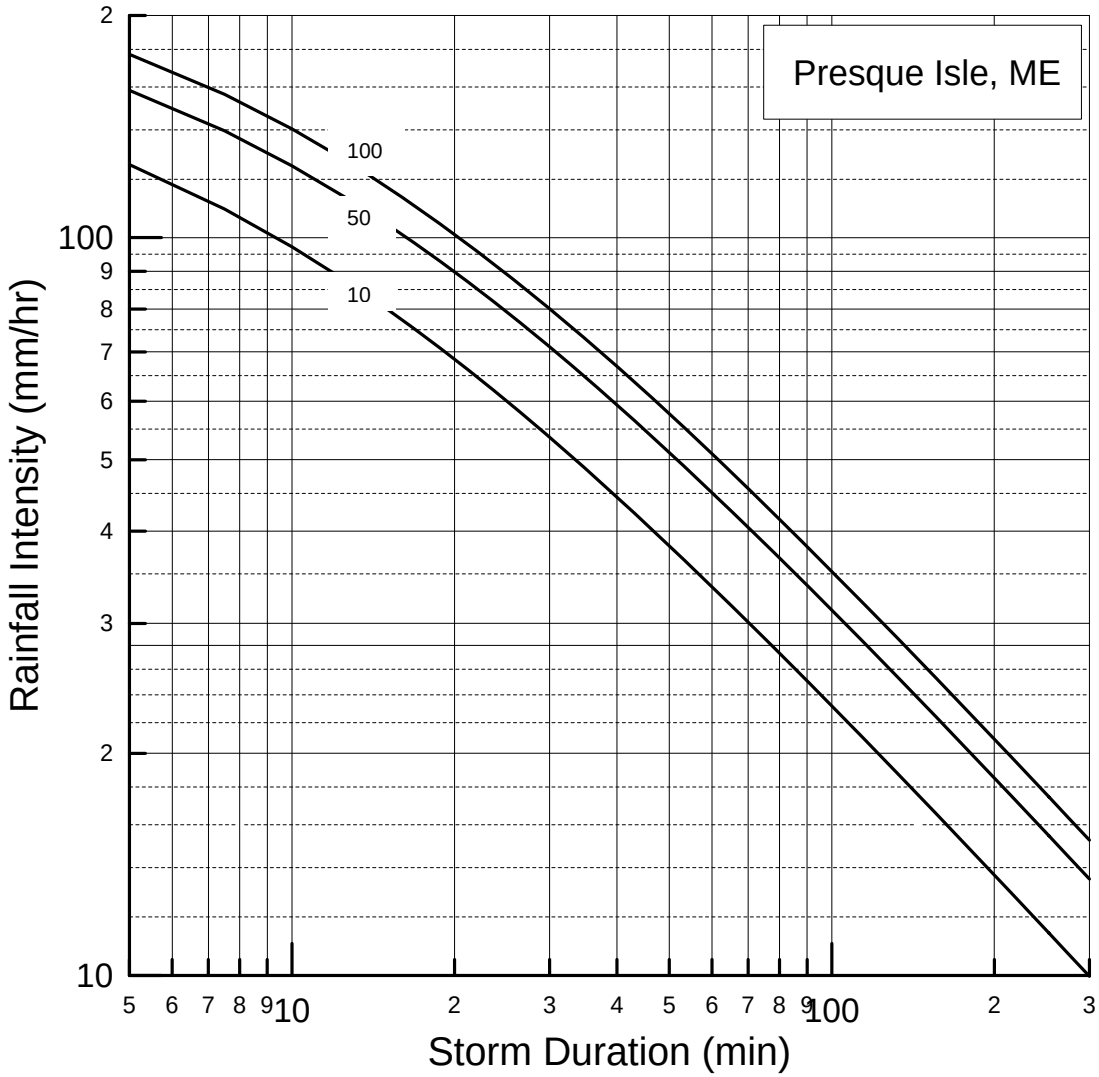

Figure 12-2.1e: Intensity-Duration-Frequency Curve, Presque Isle, Maine (labeled return period in years).

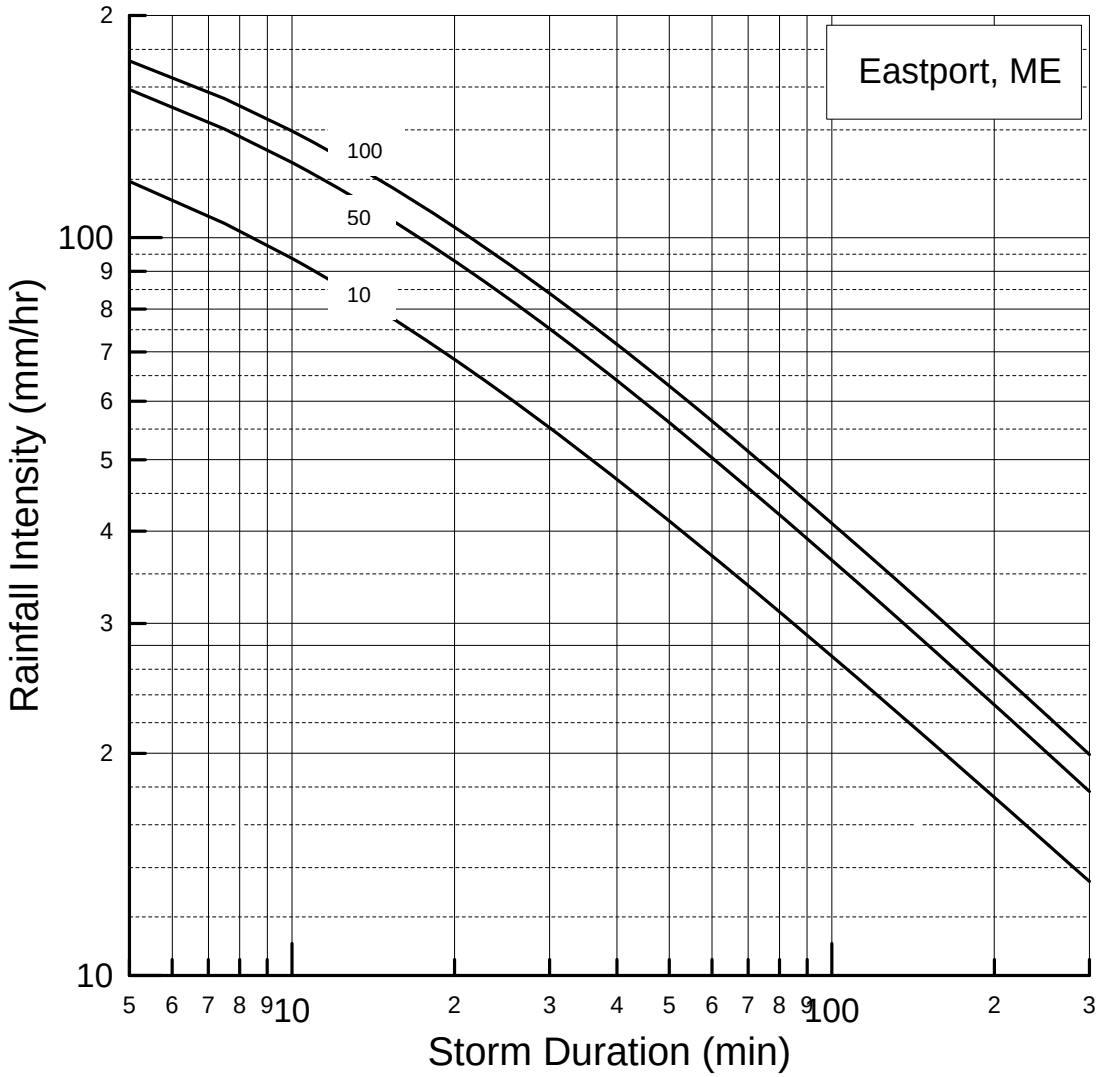

Figure 12-2.1f: Intensity-Duration-Frequency Curve, Eastport, Maine (labeled return period in years).

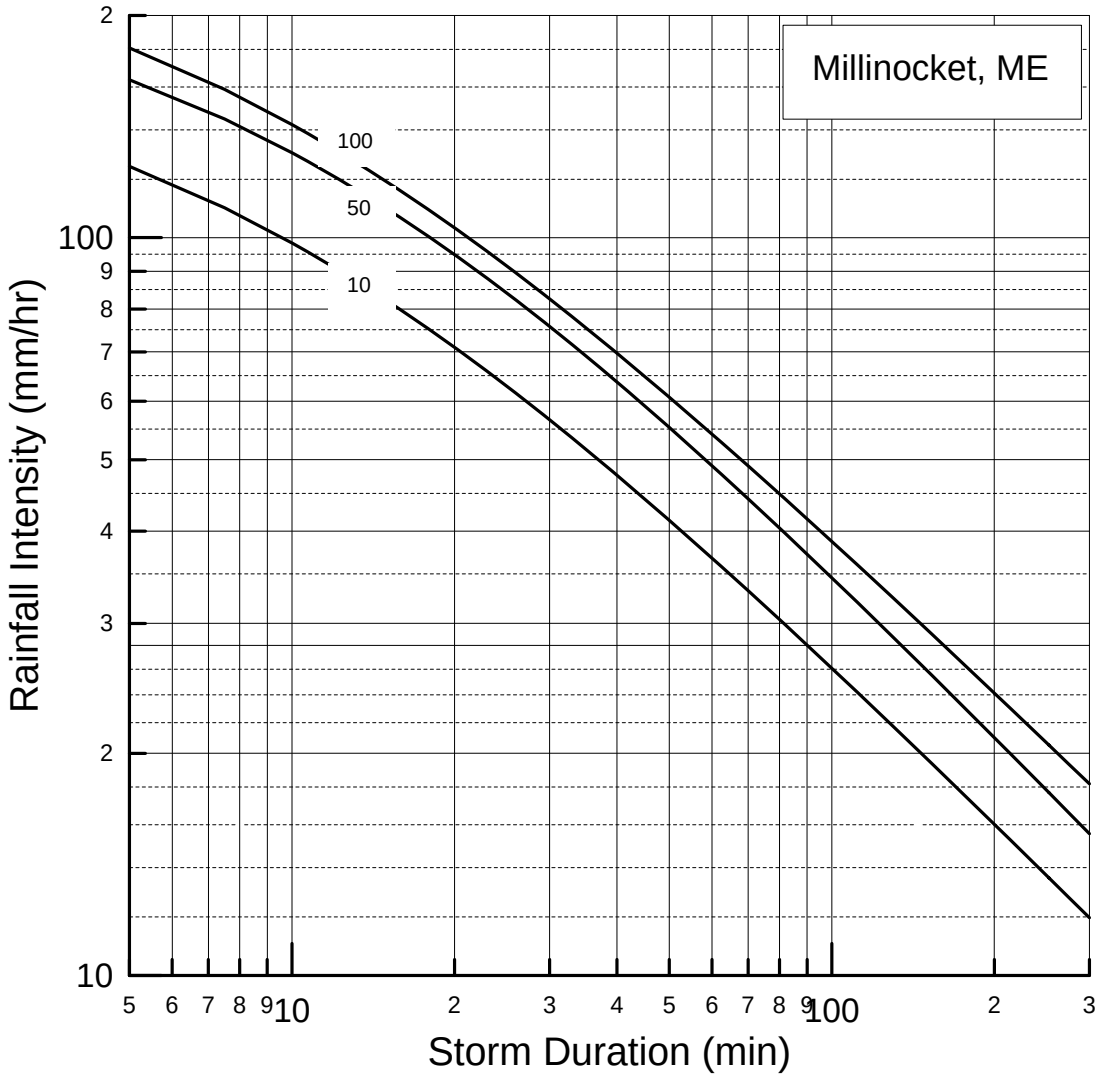

Figure 12-2.1g: Intensity-Duration-Frequency Curve, Millinocket, Maine (labeled return period in years).

Figure 12-4.1
Design Chart for Sizing Simple CMP Culverts Under Inlet Control

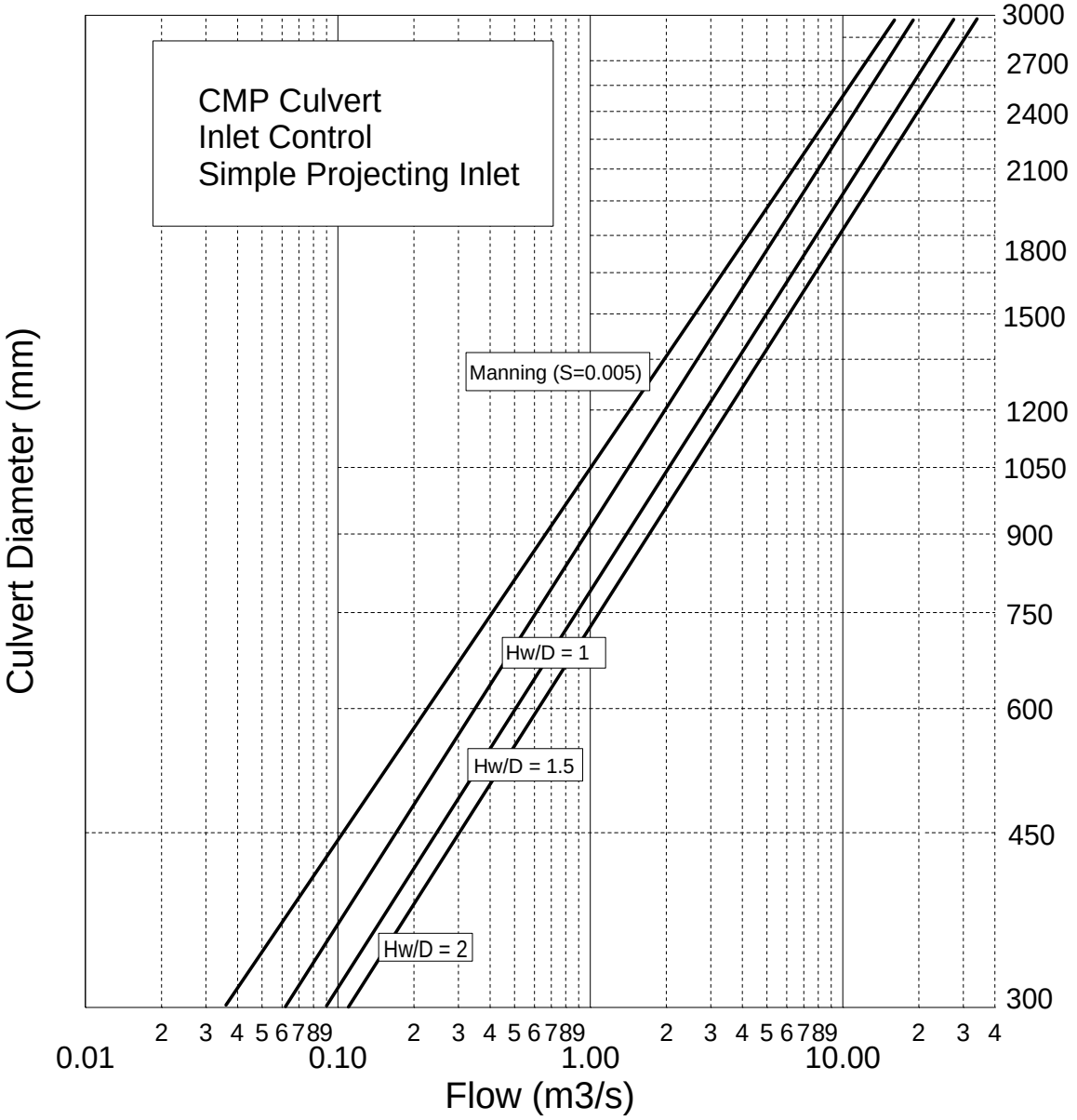

Figure 12-4.2
Design Chart for Sizing CMP Culverts Under Inlet Control

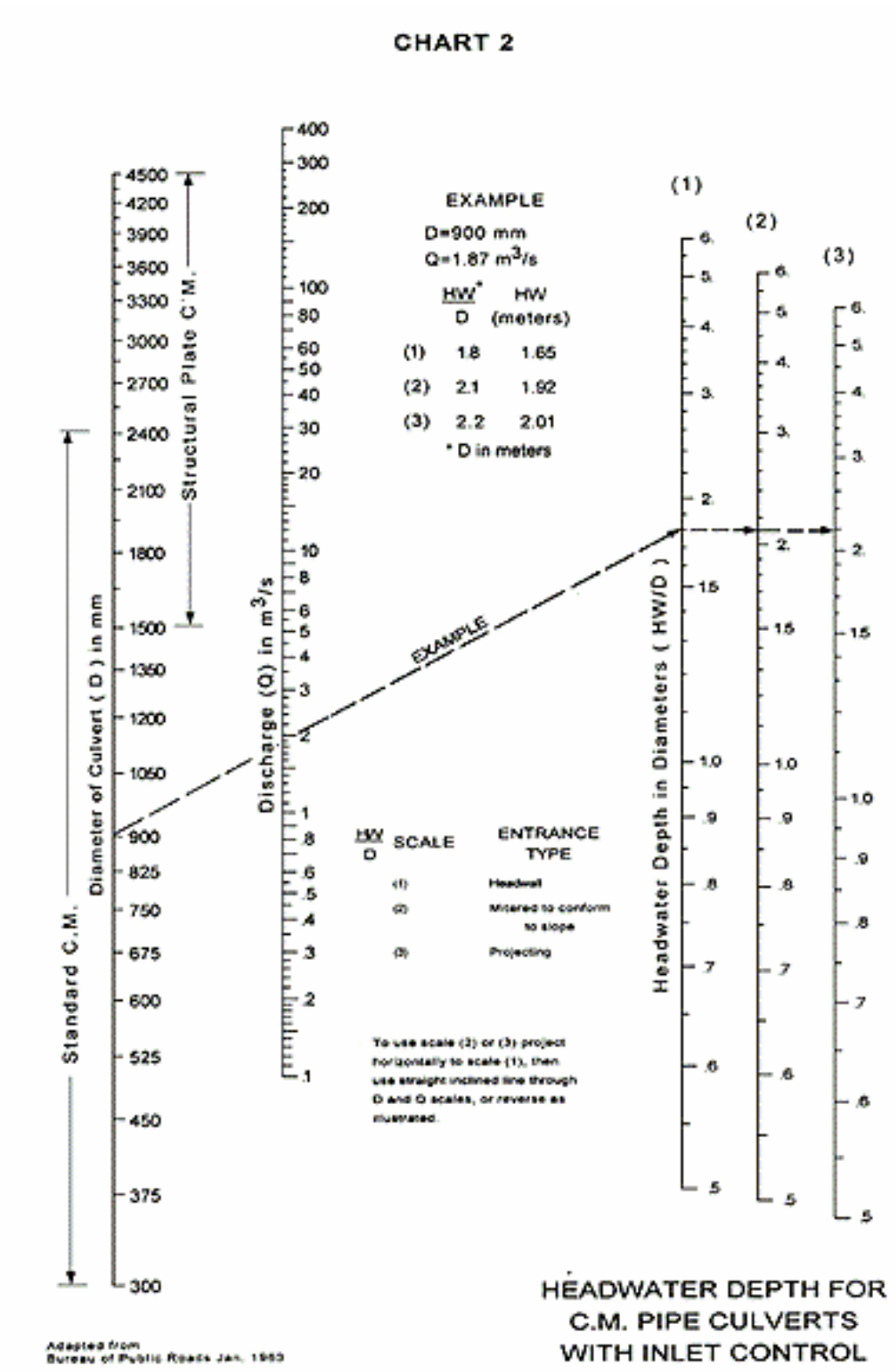

Figure 12-5.2a: Inlet Spacing for Simplified Design Scenarios (asphalt, no offsite runoff)

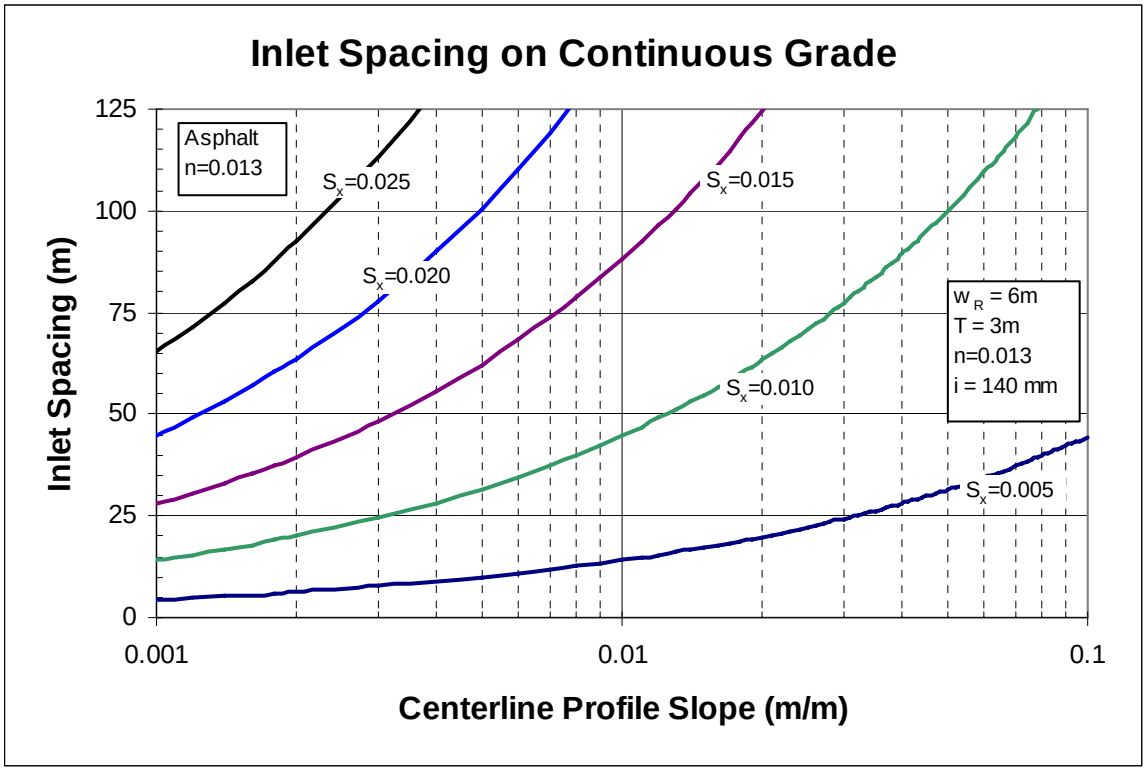

Figure 12-5.2b: Inlet Spacing for Simplified Design Scenarios (concrete, no offsite runoff)

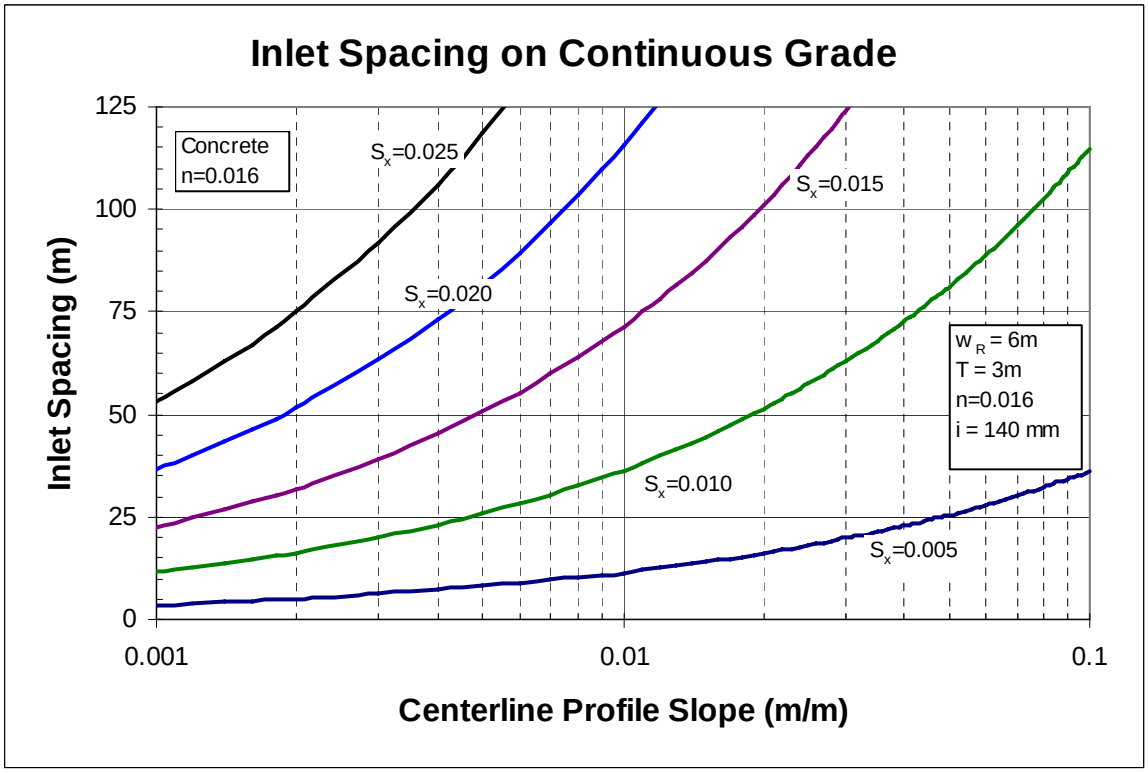

Figure 12-5.4: Circular Pipe Full Flow Geometric Functions

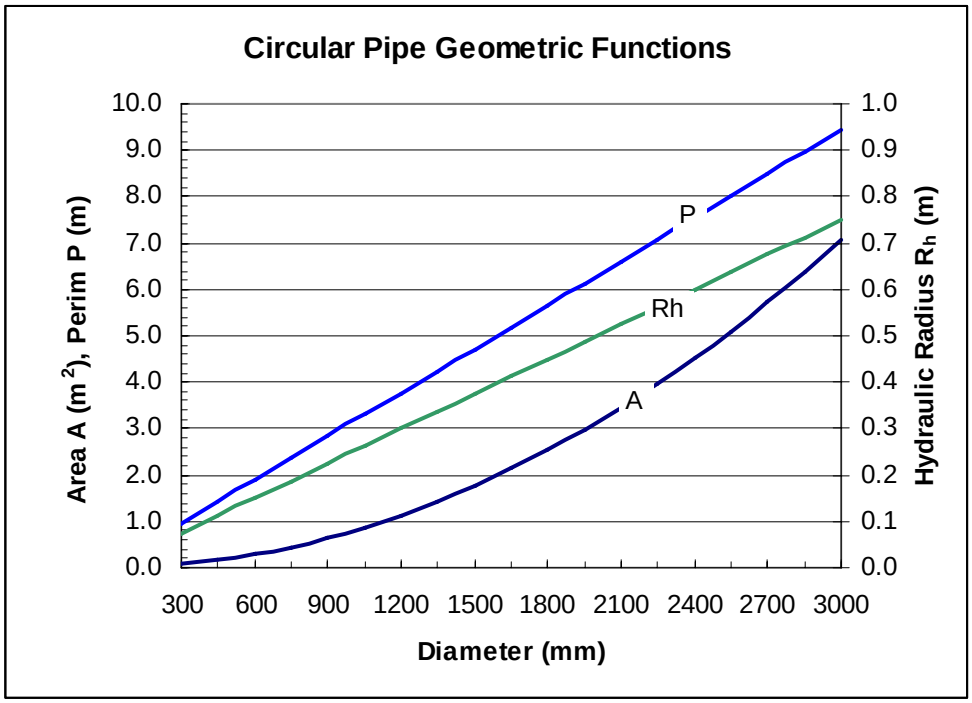
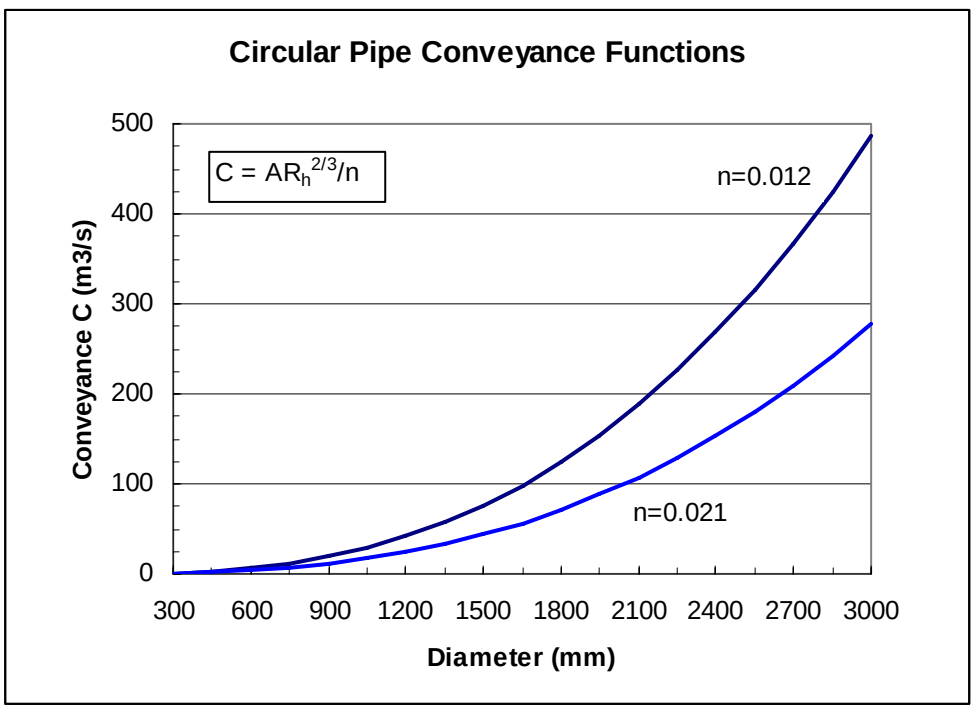
Figure 12-5.5: Circular Pipe Conveyance Function