DICKINSON BAYOU WATERSHED

REGIONAL DRAINAGE PLAN

DRAINAGE CRITERIA MANUAL

FOR

GALVESTON COUNTY, TEXAS

AND

TEXAS WATER DEVELOPMENT BOARD

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1. Veta Winick, Mayor, City of Dickinson
2. Wayne Johnson III, Galveston County Commissioner
3. Richard Beyer, Galveston County Drainage District No. 1 Commissioner
4. Marcus Junemann, Galveston County Drainage District No. 2 Commissioner
5. Penelope Burke, Galveston County Drainage District No. 3 Commissioner
6. Raymond Williams, Brazoria County Conservation and Reclamation District No. 3 Commissioner
7. Paul Schrader, Mayor, City of Friendswood
8. Joe Lamb, Mayor, City of League City
9. Vince DiPiazza, City Manager, City of Santa Fe
10. Charles T. Doyle, Mayor, City of Texas City

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1. Mike Fitzgerald, Galveston County Engineer
2. Jesse Hegemier, City Engineer / Public Works Director, City of League City
3. Earline Zimmermann, Superintendent, Galveston County Drainage District No. 1 Commissioner (also representing City of Santa Fe)
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- Walsh Engineering, Inc.
- Dodson & Associates, Inc.
- Vazquez Environmental Services, Inc.
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Chapter 1. Purpose and Policies

This drainage criteria manual establishes standard principles and practices for the design and construction of drainage systems in the Dickinson Bayou watershed of Galveston County, Texas. The design factors, formulas, graphs, procedures, tables, and figures presented in the manual are intended to establish guidelines for the solution of drainage problems involving determinations of the quantity of runoff, rate of flow, method of collection, storage, and conveyance of storm water.

Methods of design and analysis other than those indicated herein may be considered in some cases where experience clearly indicates that they are preferable. However, there should be no extensive variations from the practices established within this manual without the express approval of the applicable drainage regulatory authority.

CRITERIA MANUAL CREATION AND AUTHORITY

The creation of this manual was authorized and funded by a cooperative effort of the following government entities:

1. Galveston County
2. Galveston County Drainage District Number 1
3. Galveston County Drainage District Number 2
4. Galveston County Drainage District Number 3
5. The City of Dickinson
6. The City of Friendswood
7. The City of League City
8. The City of Santa Fe
9. The City of Texas City
10. Brazoria County Conservation & Reclamation District No. 3

The Texas Water Development Board also participated in the development and funding of this manual.

This manual is intended to serve as a common statement of policy and criteria for the local entities listed. Any projects falling within the jurisdiction of one or more of these entities should be designed and constructed according to the criteria presented in this manual.

All questions concerning the interpretation or enforcement of these criteria should be directed to the appropriate individual entity or entities having jurisdiction over the project. Chapter 2 of this manual provides further details about the submittal and approval of projects.

DICKINSON BAYOU DRAINAGE CONDITIONS

The Dickinson Bayou watershed is located to the southeast of Houston, Texas, and west of Galveston Bay. Adjoining watersheds include those of Clear Creek, Mustang Bayou, Halls Bayou, Highland Bayou, and Moses Bayou. Dickinson Bayou empties into Dickinson Bay approximately 1.5 miles downstream of State Highway 146. Dickinson Bay lies along the western edge of Galveston Bay.

The Dickinson Bayou watershed covers a total of approximately 63,930 acres, or 100 square miles, upstream of State Highway 146, the downstream terminus of this study. The watershed is elongated in shape, with a length of 22 miles from west to east. The maximum width of the watershed is approximately 7 miles.

Soils in the watershed are typically clayey or loamy in nature. All of the soils are characterized by slow permeability and poor drainage which results in high runoff potential.

Land use in the watershed varies from open farm and rangeland to concentrated development with "subdivision" lots. The areas with the highest percent urban development include the areas in the vicinity of the cities of Dickinson and League City.

Several major roads, railroads and canals cross the watershed. The major roads include Interstate Highway 45, State Highway 146, State Highway 3, State Highway 35, and FM 528. These highways generally run in a north-south direction. Major east-west highways include FM 517, FM 646, and State Highway 6. Three railroads cross the watershed, with two of the three crossing the channel of Dickinson Bayou. Two major irrigation canals, the American Canal and the Galveston County Water Company Canal, also cross the watershed.

Topography and Rainfall Conditions

The topography of the watershed may best be described as gently sloping. Ground elevations vary from 50 feet in the west to mean sea level at the mouth of the Bayou. Ground slope in the watershed varies from about 3 feet per mile to about 13 feet per mile. Areas of consistent ponding or marsh are located near the mouth of the Bayou where the channel begins to meander. No unusual changes in topography occur in the watershed except where canal and irrigation levees are built.

The average yearly rainfall total for the Galveston area is approximately 48 inches. This annual total is more or less evenly distributed throughout the year. However, monthly rainfall totals generally fall below the overall average of four inches per month during the late fall, winter, and spring. Monthly totals for the summer and early fall generally exceed the overall average.

In addition to normal rainfall, the region is subject to intense thunderstorms in the spring and summer months, to hurricanes during late summer and fall, and to extended periods of wet weather during the winter months. Therefore, the potential for floods due to heavy rain or from a combination of rain and tidal surge is always present. While this flooding potential remains fairly constant, the amount of damage resulting from severe storm events increases as development in the area continues.

As an example, during Hurricane Carla in September 1961, 15 inches of rain fell over most of the region, high tides at Galveston measured 9 feet above mean sea level (MSL), and 216 square miles of Galveston County were flooded. Total damages were estimated at $84 million.

Later, in July 1979, Tropical Storm Claudette dropped 10-20 inches of rain over the area, while tides at Galveston were about 5 feet above MSL. The estimated cost of damage was $227.5 million, almost 3 times the cost of damage from Hurricane Carla.

Tropical Storm Claudette was a major storm in the history of the region. The center of the storm crossed the Texas Gulf Coast near Beaumont while moving in a northerly direction. Instead of continuing in this direction, the storm center became erratic and settled over the Houston area for about 30 hours. Rainfall was extremely intense during this time. A record 42 inches of rain fell in less than 24 hours at one location north of Alvin, Texas.

Flooding in the Dickinson Bayou watershed was widespread. Because tides in Galveston Bay were only slightly above normal, the flooding was caused mainly by the extremely intense rainfall. Many of the channels within the Dickinson Bayou watershed were not sufficiently improved or maintained to contain the runoff generated from the heavy rainfall.

Coastal Influences

Dickinson Bayou is a coastal watershed. The main channel and some tributaries are affected by normal tides, and the low-lying portions of the watershed are affected by hurricane storm surge.

WATERSHED GOALS AND OBJECTIVES

The drainage goals and objectives for the Dickinson Bayou watershed include the following (listed roughly in order of priority):

1. **Preservation of Public Safety**: Minimize the loss of life and threats to public safety due to flooding.
2. **Continuity of Critical Services**: Minimize the disruption of critical services due to flooding.
3. **Prevention of Property Damage**: Reduce the level of property damages due to flooding.
4. **Reduction of Public Apprehension**: Reduce the public apprehension due to perceived flooding threats and problems.
5. **Encouraging Future Development**: Provide for the orderly development of the watershed.
6. **Providing Open Space**: Provide for the public need for open space, recreational space, and contact with the natural environment.
7. **Preserving the Natural Environment**: Preserve and enhance the natural environment.

The principles, policies, and criteria contained in this manual are oriented toward meeting these goals and objectives.

Planning Principles and Policies

The basic principles which apply to drainage planning are as follows:

1. **Development is Inevitable**: Further development of the Dickinson Bayou watershed is practically inevitable, as is the development of any attractive and economical block of real estate held by many private owners in a free market economy.
2. **Development Affects Drainage**: Development of the watershed has many effects on drainage, the most important of which are an increase in the volume of runoff, an acceleration in the timing of runoff, and an increase in the maximum rate of runoff.
3. **Effects are Cumulative**: The effects of many small developments and related changes, which are difficult or impossible to fully predict or measure individually, become significant, measurable, and often predictable, when considered together.
4. **Effects Demand Action**: It follows from the previous principles that some action is necessary in order to achieve, or come closer to achieving, any or all of the watershed goals and objectives listed in the previous section, in view of the inevitability of development and its related drainage effects. In fact, some actions are necessary simply in order to keep the drainage system functioning at its existing level under the increasing demands of development.
5. **Actions Have Effects**: Most of the actions which may be taken to address or provide for the effects of development within the watershed have effects of their own which must be considered.
6. **Effects are Watershed-Wide**: The cumulative effects of developments in upstream areas are present at all locations along the drainage channels downstream. In addition, the drainage effects of any local actions taken to address the effects of development also have watershed-wide effects on drainage.
7. **Actions Must Be Regional**: In order to meet the watershed-wide nature of the effects of development, actions must be carried out on a watershed-wide or regional basis, which requires advance planning, coordination among regulatory agencies, and regional sources of funding.

Interlocal Cooperation Policies

The local governmental entities sponsoring this drainage criteria manual have entered into an interlocal agreement which provides for basic common drainage policies. It is the intention of all of the participating entities to work collectively and individually to maintain and apply a common set of design objectives and technical criteria for all drainage projects within the Dickinson Bayou watershed, and to develop a master plan which will be used to determine specific policies and actions for each portion of the watershed.

Through interlocal cooperation, the individual entities are addressing the need for regional actions to address the regional effects of existing and possible future flooding problems.

Level of Protection

The desired level of protection for all facilities in the Dickinson Bayou watershed is the 100-year storm event. This level of protection is consistent with the National Flood Insurance Storm, and with current standard engineering practice within the United States.

As detailed in later chapters of this manual, local storm sewers and roadside ditches are not designed to provide 100-year flow capacity. However, they should be designed as a part of a system which accommodates a 100-year storm event without flooding of permanent structures. The overall system for accommodating the 100-year storm event may include not only the capacity of the storm sewer or roadside ditch, but overland flow capacity and street ponding capacity, as required, to convey flows to a point in the drainage system which does have 100-year capacity, or to temporarily store flows until the system has sufficient capacity to receive the stored volume of storm water.

Responsibility for Drainage

This section describes the policies of the local entities supporting this manual toward the expenditure of public and private funds toward the prevention and mitigation of flooding damages in the Dickinson Bayou watershed.

Public Responsibility for Drainage

Several of the watershed goals and objectives identified earlier in this chapter are clearly appropriate areas of responsibility for public agencies, including the following:

1. **Preservation of Public Safety**
2. **Continuity of Critical Services**
3. **Prevention of Property Damage**
4. **Calming Public Apprehension**
5. **Provide Open Space**
6. **Preserve Natural Environment**

In general, public money should be used to identify and eliminate existing flooding problems, and to maintain flood control improvements.

The goals listed above, particularly the goals of preventing property damage, cannot be absolute goals to be achieved without regard to cost. It is not generally possible to eliminate all flooding damages, and it is often not possible to meet the objective of eliminating all flooding damages resulting from the 100-year storm event. Some private property owners will continue to suffer some damages, because of the physical location or characteristics of their property. The overriding criterion for the public agency must be the well-being of the community as a whole.

Some may argue that the preservation of the natural environment is not an appropriate area of responsibility for local public agencies. However, the environmental policies of the federal government have made the preservation of the natural environment a major consideration in any drainage project.

Some might also argue that the encouragement of development is an appropriate area of responsibility for local public agencies.

Land Development Responsibility for Drainage

Land development projects place additional demands on the drainage system within the watershed. According to the principles stated earlier in this chapter, land development (along with existing flooding problems) is therefore the driving force which requires regional planning, coordination, and action on drainage issues.

Because of the effects of development on drainage, land development projects should provide storm water runoff systems compatible with the publicly-developed basin master plan.

Land development projects should also consider the potential effects of upstream development in planning local drainage systems, to assure that adequate flow-through capacity and right-of-way is provided.

National Flood Insurance Program

The local entities supporting this manual intend to maintain flood protection policies and criteria which meet or exceed the requirements of the National Flood Insurance Program. The municipal and county governments which utilize this manual are participants in the National Flood Insurance Program.

GENERAL POLICY PROVISIONS

This section sets forth the general policy provisions which apply to all entities within the Dickinson Bayou Watershed.

These policies will be developed through the work performed in subsequent phases of the Dickinson Bayou Regional Drainage Plan.

SPECIFIC POLICIES

Individual governmental entities may attach a preface to this manual stating individual or unique drainage policies, including policies applicable only to particular watersheds.

Chapter 2. Submittal and Approval Process

The purpose of the chapter is to summarize the types of projects requiring approval or response and the requirements for approval of each type of project. The interaction between local drainage approval and the National Flood Insurance Program is described.

Types of Submittals

This section introduces the 3 typical types of engineering submittals:

- **Engineering Reports and Letter Reports** – Analyses of proposed or existing flooding conditions or flood control facilities. These may be submitted as a basis for providing a better understanding of existing conditions (such as a flood plain revision report), to support the approval of construction documents for a proposed facility (such as a Preliminary Engineering Report), or to serve as a plan for ultimate conditions (such as a Master Drainage Plan report).
- **Construction Documents** – Engineering drawings and specifications for a proposed facility which affects drainage or flood protection.
- **Permit Application Referrals** – Applications for building permits, flood plain fill permits, and other permits issued by agencies such as the Galveston County Engineering Department or municipalities in Galveston County.

Review and Approval of Engineering Reports and Letter Reports

This section describes the various types of engineering reports, including letter reports, preliminary engineering reports, new studies, update studies, impact studies, mitigation studies, and master plans.

Basic data required for all engineering reports:

- **Cover letter** containing basic information. This would typically include the engineer's seal, with signature and date.
- **Report Text** containing detailed information and conclusions.
- **Tables** presenting numeric results.
- **Exhibits** of existing conditions and proposed changes.
- **Computations** for hydrology & hydraulics, including the various types of computer models required for certain types of reports. If HEC-1 or HEC-2 computer models are used, copies of the input data should be submitted on diskette.

Review and Approval of Open Channel Designs

The following information must be submitted for the design of open channels:

1. **Vicinity Map**: A vicinity map of the site and subject reach. The **subject reach** is the stretch of channel necessary for any altered flow profile to match the upstream and downstream existing profiles.

2. **Site Map**: A detailed map of the area and subject reach with all pertinent physiographic information.
3. **Watershed Map**: A watershed map showing existing and proposed drainage area boundaries along with all sub-area delineations and all areas of existing or proposed development.
4. **Discharge Calculations**: Discharge calculations specifying computed discharges at key locations, with comparisons to discharges existing without the proposed project. The report text should describe the source and methodology for determining discharges. All assumptions made during the analysis should be identified and discussed, including percent development and impervious cover, changes in drainage area, rainfall loss rates, computation of unitgraph coefficients, and method of flood routing. A table of computed HEC-1 parameters should be included.
5. **Hydraulic Calculations**: Hydraulic calculations specifying the methodology used, and summarizing the results as a table of cross-sections and computed water surface elevations. All assumptions and values of design parameters must be clearly stated.
6. **Benchmark Information**: A description of the benchmark used in obtaining field survey data, including the location, elevation, datum, and year of adjustment.
7. **Right-of-way Map**: A right-of-way map illustrating all existing and proposed channel rights-of-way.
8. **Soils Report**: A soils report which addresses erosion and slope stability.
9. **Plotted Stream Profile**: A stream profile of the subject reach which includes the following:
   a) All pertinent water surface profiles. This will minimally include the 25-year and 100-year frequency floods for both existing and proposed conditions.
   b) All existing and proposed bridge, culvert and pipeline crossings.
   c) The locations of all tributary and drainage confluences.
   d) The locations of all hydraulic structures (e.g. dams, weirs, drop structures, etc.).
10. **Plotted Cross-Sections**: Typical existing and proposed cross-sections.
11. **Flood Plain Map**: A map showing the effects of the project on 100-year flood plain boundaries.

Review and Approval of Detention Facilities

The following information must be submitted for the design of detention facilities:

1. **Vicinity Map**: A vicinity map which illustrates the location of the proposed development and detention site.
2. **Site Map**: A detailed map of the proposed development and detention site with all pertinent physiographic information.
3. **Watershed Map**: A watershed map showing existing and proposed drainage area boundaries along with all sub-area delineations and all areas of existing or proposed development.
4. **Discharge Calculations**: Discharge calculations specifying computed discharges at key locations, with comparisons to discharges existing without the proposed project. The report text should describe the source and methodology for determining discharges. All assumptions made during the analysis should be identified and discussed, including percent development and impervious cover, changes in drainage area, rainfall loss rates, computation of unitgraph coefficients, and method of flood routing. A table of computed HEC-1 parameters should be included.
5. **Hydraulic Calculations**: Hydraulic calculations for outlet structure design, specifying the methodology used. All assumptions and values of design parameters must be clearly stated. The detention basin volume/elevation table and basin outflow curve should be included.
6. **Right-of-Way Map**: A map illustrating all existing and proposed rights-of-way.
7. **Benchmark Information**: A description of the benchmark used in obtaining field survey data, including the location, elevation, datum, and year of adjustment.
8. **Facility Layouts**: Plan view and typical cross-section(s) of proposed detention facility.
9. **Soils Report**: A soils report which addresses erosion and slope stability, if required.

Chapter 3. Hydrologic Analysis

The purpose of this section is to establish standard procedures and criteria for the performance of hydrologic analyses.

INTRODUCTION

The planning, design, and construction of drainage facilities are based on the determination of one or more aspects of storm runoff. If the estimate of storm runoff is incorrect, the constructed facilities may be undersized, oversized, or otherwise inadequate. An improperly designed drainage system can be uneconomical, cause flooding, interfere with traffic, disrupt commercial and other activities, and be a general nuisance in the affected area. However, the peak flow rate, volume and time-sequence of storm runoff related to a certain recurrence interval (frequency) can only be approximated because of the many physical and climatic factors involved.

Continuous long-term records of rainfall and resulting storm runoff in an area provide the best data source on which to base the design of storm drainage and flood control systems in that area. However, it is not possible to obtain such records in sufficient quantities for all locations requiring storm runoff computations. Therefore, the accepted practice is to relate storm runoff to rainfall, thereby providing a means of estimating the rates, timing, and volume of runoff expected within local watersheds at various recurrence intervals.

EFFECTS OF URBANIZATION

It is generally accepted that urban development has a pronounced effect on the rate and volume of runoff from a given rainfall. Urbanization generally alters the hydrology of a watershed by improving hydraulic efficiency, reducing surface infiltration and reducing storage capacity. The reduction of a watershed's storage capacity and surface infiltration is a result of the elimination of porous surfaces and ponding areas by grading and paving building sites, streets, drives, parking lots, and sidewalks and by constructing buildings and other facilities characteristic of urban development.

Zoning maps, future land use maps, and watershed master plans should be used as aids in establishing the anticipated surface character following development. The selection of design runoff coefficients and impervious cover factors, which are explained in the following discussions of runoff calculation, must be based upon the appropriate degree of urbanization.

SELECTION OF METHODS FOR HYDROLOGIC ANALYSIS

Because of its versatility and accuracy, the widely used HEC-1 computer program, which was developed by the U.S. Army Corps of Engineers Hydrologic Engineering Center (HEC) in Davis, California, is recommended as the primary tool for modeling storm runoff. Accordingly, the hydrologic design techniques described in this manual incorporate many of the routines contained in HEC-1. The principal routines used for computing runoff as presented in this chapter are based on the Clark unit hydrograph technique, design rainfall events, and empirical rainfall loss functions.

A methodology for deriving the parameters used to compute the Clark unit hydrograph was developed for Harris County, Texas by the Harris County Flood Control District. It has been revised slightly by Dodson & Associates, Inc. for the Dickinson Bayou Watershed Study [1992]. This methodology was developed from optimization studies utilizing U.S. Geological Survey regional rainfall-runoff data and standard unit hydrograph techniques. The Revised HCFCD Methodology is appropriate for a wide range of watershed sizes and is the recommended method for Galveston in all but certain small areas in which only peak discharge determinations are required.

Within this manual, the use of the HEC-1 computer program with the Revised HCFCD Methodology is called ***Hydrologic Method III***.

For areas less than 500 acres (one square mile) and greater than 50 acres, drainage area-discharge curves have been developed as a means to determine peak discharge. This is ***Hydrologic Method II***.

For drainage areas of less than 50 acres, the Rational Method may be used to determine peak discharges. This is ***Hydrologic Method I***.

Hydrologic Method III is always required when a project or analysis affects an existing Dickinson Bayou watershed model.

METHOD I HYDROLOGIC ANALYSIS

For small drainage areas (less than 50 acres in size), the widely used Rational Method provides a useful means of determining peak discharges. In situations requiring determination of a complete flood hydrograph, and not just a peak discharge, a method developed by H.R. Malcom [Malcom, Undated] should be utilized. The Malcom method is described later in this manual. Engineers wishing to use an alternative design technique should consult the appropriate drainage regulatory agency prior to design.

Background of Hydrologic Method I

Hydrologic Method I(the Rational Method) represents an accepted method for determining peak storm runoff rates for small watersheds that have a drainage system unaffected by complex hydrologic situations such as ponding areas, storage basins and watershed transfers (overflows) of storm runoff. This widely used method provides satisfactory results if understood and applied correctly.

The Rational Method is based on a direct relationship between rainfall intensity and runoff, and is expressed by the following equation:

Equation 3.1

$$Q = CiA$$

in which:

Q = the peak rate of runoff in cubic feet per second (cfs). Actually, Q is in units of inches per hour per acre. Since this rate of in-ac/hr differs from cubic feet per second by less than one percent, the more convenient units of cfs are used.

C = the dimensionless coefficient of runoff representing the ratio of peak discharge per acre to rainfall intensity (i).

i = the average intensity of rainfall in inches per hour for a period of time equal to the time of concentration for the drainage area at the point of interest.

A = the area in acres contributing runoff to the point of interest during the critical storm duration.

Assumptions and Limitations of Hydrologic Method I

Basic assumptions associated with the Hydrologic Method I (the Rational Method) are:

1. **Runoff**: The peak rate of runoff at the point of interest is a function of the average rainfall intensity during a period of time equal to the time of concentration at that point.
2. **Storm Frequency**: The frequency or recurrence interval of the peak discharge equals the frequency of the average (uniform) rainfall intensity associated with the critical storm duration.
3. **Critical Storm Duration**: The critical storm duration equals the time of concentration, except where the total time of concentration includes watershed areas which do not contribute significantly to the peak flow rate at the point of interest.
4. **Uniform Runoff Coefficient**: The ratio of runoff rate to rainfall intensity, C, is uniform during the storm duration.
5. **Uniform Rainfall Intensity**: Rainfall intensity is uniform during the storm duration.
6. **Contributing Area**: The contributing area is the area that drains to the point of interest within the critical time of concentration.

These assumptions are generally valid only for relatively small drainage areas; this is why Hydrologic Method I is restricted to drainage areas of 50 acres or less.

Rational Method Runoff Coefficient C

In relating peak rainfall rates to peak discharges, the runoff coefficient C in the Rational Formula is dependent on the character of the surface of the drainage area. The rate and volume of runoff that reaches a storm drainage system depends on the relative porosity (***imperviousness***), ponding character, slope, and conveyance properties of the surface. Soil type, vegetative condition, and the presence of impervious surfaces, such as asphalt pavements and the roofs of buildings, are the major determining factors in selecting an area's C factor. The type and condition of the surface determines its ability to absorb precipitation and transport runoff.

The rate at which a soil absorbs precipitation generally decreases as rainfall continues for an extended period of time. The soil absorption or infiltration rate is also influenced by the presence of soil moisture before a rain (***antecedent precipitation***), the rainfall intensity, the depth of the ground water table, the degree of soil compaction, the porosity of the subsoil, vegetation, ground slopes, depressions, and storage. On-site inspections and aerial photographs may prove valuable in evaluating the nature of the surface within the drainage area.

Recommended C Values

Table 3.1 presents recommended values for the runoff coefficient *C* for various residential districts and specific surface types for 5 to 10 year frequency storms.

TABLE 3.1 Rational Method Runoff Coefficients for 5-10 Year Frequency Storms

Description of Area	Basin Slope < 1%	Basin Slope 1%-3.5%	Basin Slope 3.5%-5.5%
Single Family Residential Districts			
Lots greater than 1/2 acre	0.30	0.35	0.40
Lots 1/4 - 1/2 acre	0.40	0.45	0.50
Lots less than 1/4 acre	0.50	0.55	0.60
Multi-Family Residential Districts	0.60	0.65	0.70
Apartment Dwelling Areas	0.75	0.80	0.85
Business Districts			
Downtown	0.85	0.87	0.90
Neighborhood	0.75	0.80	0.85
Industrial Districts			
Light	0.50	0.65	0.80
Heavy	0.60	0.75	0.90
Railroad Yard Areas	0.20	0.30	0.40
Cemeteries	0.10	0.18	0.25
Playgrounds	0.20	0.28	0.35
Streets			
Asphalt	0.80	0.80	0.80
Concrete	0.85	0.85	0.85
Concrete Drives and Walks	0.85	0.85	0.85
Roofs	0.85	0.85	0.85
Lawn Areas			
Sandy Soil	0.05	0.08	0.12
Clay Soil	0.15	0.18	0.22
Woodlands			
Sandy Soil	0.15	0.18	0.25
Clay Soil	0.18	0.20	0.30
Pasture			
Sandy Soil	0.25	0.35	0.40
Clay Soil	0.30	0.40	0.50
Cultivated			
Sandy Soil	0.30	0.55	0.70
Clay Soil	0.35	0.60	0.80

The runoff coefficient *C* is difficult to precisely determine. Its use in the Rational Method implies a fixed ratio of runoff rate to rainfall intensity for any given drainage area, which in reality is not the case. A reasonable coefficient must be chosen to represent the integrated effects of infiltration, detention storage, evaporation, retention, flow routing, and interception, all of which affect the time distribution and peak rate of runoff. Proper use of the Rational Method requires judgement and experience on the part of the engineer, especially in the selection of the runoff coefficient.

Composite C Values

Coefficients for specific surface types can be used to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage area. This procedure is often applied to typical "sample" blocks as a guide to the selection of reasonable values of the coefficient for an entire area.

Adjusted C Values

Adjustment of the *C* value for use with more severe (less frequent) storms can be made by multiplying the runoff coefficient by a ***frequency factor*** C_f, which is used to account for antecedent precipitation conditions. The Rational Formula now becomes:

$$Q = C_f \times CiA$$

Equation 3.2

Table 3.2 presents recommended values of C_f. ***The product of C and C_f should not exceed 1.0.***

TABLE 3.2 Rational Method Frequency Factor Adjustment

Source: "Urban Storm Drainage Criteria Manual," 1969.

Frequency of Storm	Frequency Factor (C_f)
≤ 10	1.00
25	1.10
50	1.20
100	1.25

Rational Method Rainfall Intensity (*i*)

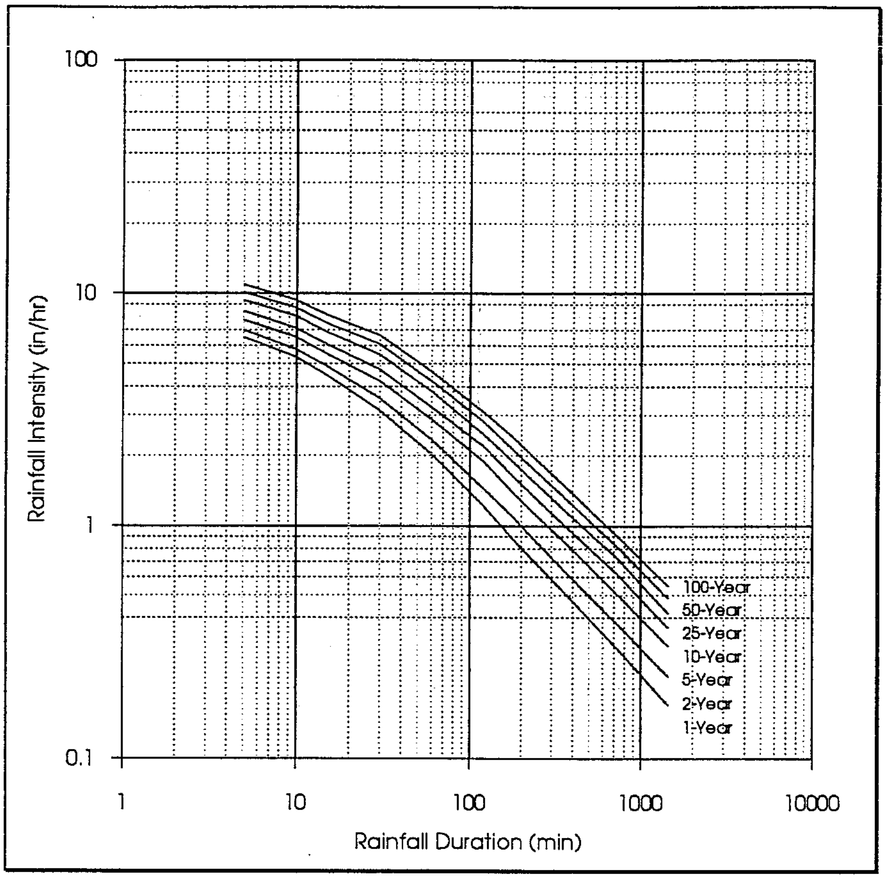
Rainfall intensity (*i*) is the average rainfall rate in inches per hour which is considered for a particular basin or sub-basin. The rainfall intensity is determined on the basis of design rainfall duration and design frequency of occurrence. The ***design rainfall duration*** is equal to the critical time of concentration for all portions of the drainage area under consideration that contribute flow to the point of interest. The ***frequency of occurrence*** used in design computations is a statistical variable which is established by design standards or chosen by the engineer as a design parameter. It is usually expressed in terms of the average storm recurrence interval in years.

The statistical relationship between the rainfall intensity and duration for the 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year frequency storms are shown in Figure 3.1. These curves are presented for durations from 5 minutes to 24 hours. Table 3.3 presents rainfall depths for a variety of durations and frequencies. The rainfall intensities plotted in Figure 3.1 are computed by dividing the point rainfall amounts listed in Table 3.3 by the corresponding duration of rainfall.

TABLE 3.3 Point Rainfall Depths for Varying Durations and Frequencies

Storm Duration	1-yr.	2-yr.	5-yr.	10-yr.	25-yr.	50-yr.	100-yr.	Source
5 min.	0.54	0.58	0.64	0.70	0.78	0.84	0.91	HY-35
10 min.	0.89	0.96	1.08	1.18	1.33	1.45	1.56	HY-35
15 min	1.12	1.22	1.38	1.51	1.71	1.86	2.02	HY-35
30 min	1.58	1.78	2.12	2.37	2.74	3.03	3.31	HY-35
60 min.	2.05	2.37	2.89	3.27	3.81	4.24	4.66	HY-35
2 hr	2.45	2.91	3.82	4.44	5.00	5.68	6.23	TP-40
3 hr	2.64	3.28	4.18	4.92	5.69	6.40	7.27	TP-40
6 hr.	3.08	3.83	5.11	6.00	7.00	7.95	8.92	TP-40
12 hr.	3.56	4.56	6.11	7.50	8.75	9.82	10.95	TP-40
24 hr.	4.05	5.36	7.25	8.73	10.10	11.71	13.18	TP-40

FIGURE 3.1 Rainfall Intensity-Duration-Frequency Curves for Galveston County



Critical Storm Duration

The ***critical storm duration*** is the rainfall duration (and related rainfall intensity) which results in the maximum peak runoff rate from all or part of the upstream drainage area at the point of interest. This may be equal to or less than the ***time of concentration*** for the entire drainage area upstream of the point of interest. Runoff from a watershed usually reaches a peak at the time when the entire drainage area is contributing. However, the runoff rate may reach a peak prior to the time when the entire upstream drainage area is contributing. In such instances, only the portions of the drainage area able to contribute flow at the point of interest during the critical time of concentration should be used in determining the peak discharge.

A trial and error procedure can be used to determine the critical storm duration. The following steps are involved:

1. **Time of Concentration**: Compute the time of concentration for the entire upstream drainage area as the time required for water to flow from the most remote point in the watershed to the point of interest. The next section of this manual describes how to compute the time of concentration.
2. **Peak Flow Rate**: Using the computed time of concentration as the rainfall duration, use the Rational Method to compute a peak flow rate at the point of interest.

3. **Non-Contributing Areas**: Inspect the drainage area map and the computations for the time of concentration to determine if any of the upstream or outer portions of the drainage area are contributing more to the computed time of concentration than to the drainage area of the watershed. For example, a poorly-drained area at the upstream end of the watershed may be contributing 20% of the time of concentration, but may constitute only 5% of the total watershed area.
4. **Revised Peak Flow Rate**: Re-compute the time of concentration and resulting peak flow rate for the watershed without the area(s) identified in step 3). Repeat steps 1) through 4) until the highest peak flow rate results.

Time of Concentration

The time of concentration at any point in a storm drainage system is a combination of the following:

- ***Inlet Time***: the time for water to flow over the watershed surface to the storm sewer inlet or channel.
- ***Travel Time***: the time for water to flow through the conduit or channel from the inlet to the point of interest.

Inlet Time

Inlet time decreases as the slope and the hydraulic efficiency of the surface increase. It increases as the distance over which the water has to travel increases and as retention by the contact surfaces increases.

The total inlet time may be computed as the sum of the travel times for overland sheet flow and shallow concentrated flow. If the inlet time is calculated to be in excess of 20 minutes, the designer should verify that the time is reasonable.

Overland Flow

Average velocities for estimating travel time for overland flow can be calculated using methods outlined in SCS TR-55 [SCS, 1986].

Overton & Meadows (1976) developed the following equation for time of travel for overland sheet flow over distances of 300 feet or less:

$$T_t = \frac{0.007(n \times L)^{0.8}}{\sqrt{P_2} \times S^{0.4}}$$

Equation 3.3

in which:

T_t = overland travel time (hours)

n = Manning's roughness coefficient

L = overland flow distance (feet)

P_2 = 2-year, 24-hour rainfall depth (inches)

S = land slope (feet per foot)

This equation is based on the following assumptions:

1. Shallow, steady uniform flow
2. Constant intensity of rainfall excess
3. Rainfall duration equal to 24 hours
4. Infiltration has a minor effect on travel time.

Table 3.4 presents representative values of Manning's roughness coefficient for a variety of flow surfaces.

TABLE 3.4 Manning's Roughness Coefficients for Overland Sheet Flow

Source: SCS TR-55 (SCS, 1986)

Surface	n
Smooth Surfaces (concrete, asphalt, gravel, bare soil)	0.011
Fallow (no residue)	0.05
Cultivated Soils: Residue Cover ≤ 20%	0.06
Cultivated Soils: Residue Cover > 20%	0.17
Grass: Short Grass Prairie	0.15
Grass: Dense Grasses	0.24
Grass: Bermuda Grass	0.41
Range (natural)	0.13
Woods: Light Underbrush	0.40
Woods: Dense Underbrush	0.80

Shallow Concentrated Flow

The following flow velocity equations are presented in SCS TR-55 for shallow concentrated flow:

Equation 3.4

$$V = 16.1345\sqrt{S} \qquad \text{(unpaved areas)}$$

Equation 3.5

$$V = 20.3282\sqrt{S} \qquad \text{(paved areas)}$$

in which:

V = flow velocity (feet per second)

S = overland slope (feet per foot)

Using flow velocities computed from these equations, overland travel times may be computed using the following equation:

Equation 3.6

$$T = \frac{D_F}{60V}$$

in which:

T = overland flow time (minutes)

D_F = flow distance (feet)

V = average velocity of runoff flow (ft/sec)

Travel Time

The ***travel time*** in the conduit or channel is the quotient of the length of the conduit or channel and the velocity of flow as computed using the hydraulic characteristics of the conduit or channel. Usually, the actual time for the flood crest to reach a given point includes not only the travel time, but also the ***time of storage***, which is the time required to fill the conduit or channel. However, the time of storage is usually small compared with the travel time, and it may be neglected in the design of storm runoff conduits.

Rational Method Drainage Area, A

As mentioned previously, the drainage area used in determining peak discharges is the portion of the area that contributes flow to the point of interest within the critical time of concentration. The boundaries of the drainage area may be determined through the use of topographic maps, supplemented by field surveys where topographic data has changed or where the contour interval is too great to distinguish the direction of flow. A drainage area map shall be provided for each project. The drainage area contributing to the system being designed and the drainage sub area contributing to each inlet point shall be identified. The boundaries of each drainage area must follow actual drainage divides rather than artificial land divisions as used in the design of sanitary sewers. The drainage divide lines are determined by pavement slopes, downspout locations, grading of lawns, and many other features that are introduced by the urbanization process.

Example of Hydrologic Method I

A storm drainage system includes the four areas shown in Figure 3.2.

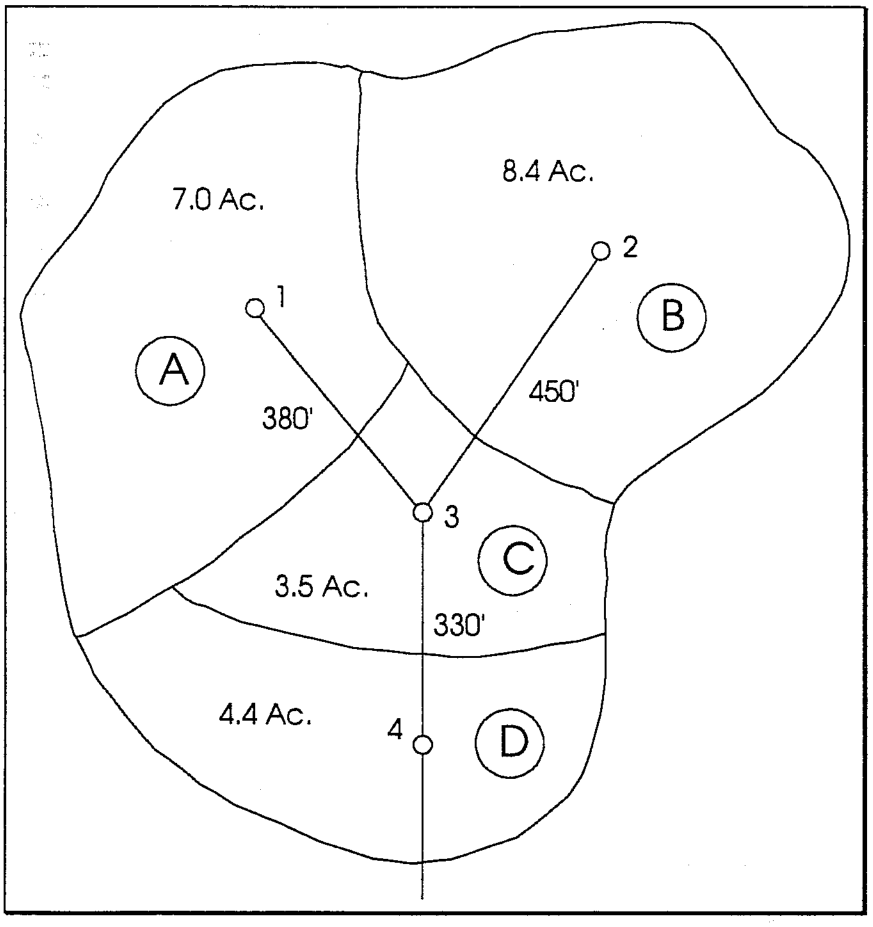


FIGURE 3.2 Watershed for Rational Method Example

Table 3.5 lists the drainage area, runoff coefficient, flow distance to the inlet, average overland slope, and development condition for each sub-area. Table 3.6 illustrates the computation of the inlet time using Equation 3.6.

TABLE 3.5 Description Data for Sub-Areas of Example Watershed

Sub-Area	Area (ac)	Runoff Coefficient	Flow Distance (ft)	Overland Slope (ft/ft)	Development Condition
A	7.0	0.5	550	0.001	Unpaved
B	8.4	0.5	600	0.001	Unpaved
C	3.5	0.6	420	0.002	Unpaved
D	4.4	0.8	600	0.01	Paved

TABLE 3.6 Computed Concentration Times for Sub-Areas of Example Watershed

Note: Inlet Time computed using Equation 3.6. Flow Velocity computed using Equations 3.4 - 3.5.

Sub-Area	Overland Slope (ft/ft)	Velocity Equation	Flow Velocity (fps)	Flow Distance (ft)	Inlet Time (min)
A	0.001	3.4	0.5	550	18
B	0.001	3.4	0.5	600	20
C	0.002	3.4	0.7	420	10
D	0.01	3.5	2.0	600	5

Table 3.7 lists the data for each pipe segment in the storm drainage system, including the computed full flow velocity and travel time for each pipe.

TABLE 3.7 Pipe Segments in Example Watershed

Note: Full Flow Velocity computed using Manning Equation.

Pipe Segment	Length (ft)	Diameter (in)	Slope (ft/ft)	Full Flow Velocity (fps)	Travel Time (min)
1-3	380	30	0.003	5.0	1.3
2-3	450	30	0.003	5.0	1.5
3-4	330	42	0.002	5.0	1.1

Table 3.8 lists the results of the flow computations for the 5-year storm event. The sequence of computations is as follows:

1. **Sub-Areas**: Compute the product of the drainage area and runoff coefficient for each sub-area, $a \times C$. Since this is a 5-year storm event, the frequency factor C_f equals 1.0.
2. **Analysis Points**: Compute the total $a \times C$ value at each analysis point, considering all drainage areas contributing to flow at that location.
3. **Times of Concentration**: Compute the total time of concentration at each analysis point, considering inlet time as well as travel time. For points where two or more storm sewer branches come together, such as Point No. 3, the total time of concentration should be computed for each possible flow path. The longest time of concentration is used for flow computations. ***Never add peak flow rates at junctions.***
4. **Rainfall Intensities**: Determine the 5-year rainfall intensity from the Intensity-Duration-Frequency curves illustrated in Figure 3.1 for each time of concentration.
5. **Peak Flow Rates**: Compute the peak flow rate with Equation 3.1, using the total $a \times C$ value and the computed rainfall intensity for each analysis point.

Note: The time of concentration from MH1 to MH4 is not considered at MH4 because the flow path from MH2 to MH4 represents the longest time of concentration for the total system. This may be seen by comparing the time of concentrations at MH3 for flow path 1-3 and flow path 2-3.

Man-hole	Area, a (ac)	Coefficient C	$a \times C$	$\sum a \times C$	Flow Path	Inlet Time (min)	Travel Time (min)	Total Time (min)	Intensity (in/hr	Q (cfs)
1	7.0	0.5	3.5	3.5	A	18.0	0.0	18.0	5.1	17.9
2	8.4	0.5	4.2	4.2	B	20.0	0.0	20.0	4.9	20.6
3	3.5	0.6	2.1	9.8	A.1-3	18.0	1.3	19.3		
	3.5	0.6	2.1	9.8	B.2-3	20.0	1.5	21.5	4.7	46.1
4	4.4	0.8	3.5	13.3	B.2-4	20.0	2.6	22.6	4.6	61.2

TABLE 3.8 Example Rational Method Calculations

METHOD II HYDROLOGIC ANALYSIS

Hydrologic analyses involving watersheds of greater than or equal to 50 acres and less than 500 acres must be completed using one of two approaches:

1. **Runoff Rate Curves**: The use of runoff rate curves to determine peak flow rates and the Malcom Method to develop runoff hydrographs.
2. **HEC-1 Computer Program**: The second approach is the use of the HEC-1 computer program to compute complete runoff hydrographs, applied as described for Hydrologic Method III later in this chapter. The HEC-1 method will be required whenever it is necessary to perform detailed analyses of watersheds with multiple sub-areas.

Background of Hydrologic Method II

The Runoff Rate curves represent a simplified method for the determination of the peak discharge in a relatively small watershed. The use of this type of analysis requires that the watershed and its physical characteristics be relatively uniform and not contain complex hydrologic features such as ponding areas, storage basins, or watershed overflows. HEC-1 should be used instead of the Runoff Rate curves if channel routing or hydrograph combination steps are required.

The curves developed for this manual for the 10-year, 50-year and 100-year rainfall events, respectively, are shown in Figures 3.3, 3.4 and 3.5. The curves are applicable to drainage areas between 50 and 500 acres. The curves may also be useful in providing preliminary estimates of flow rates for larger areas.

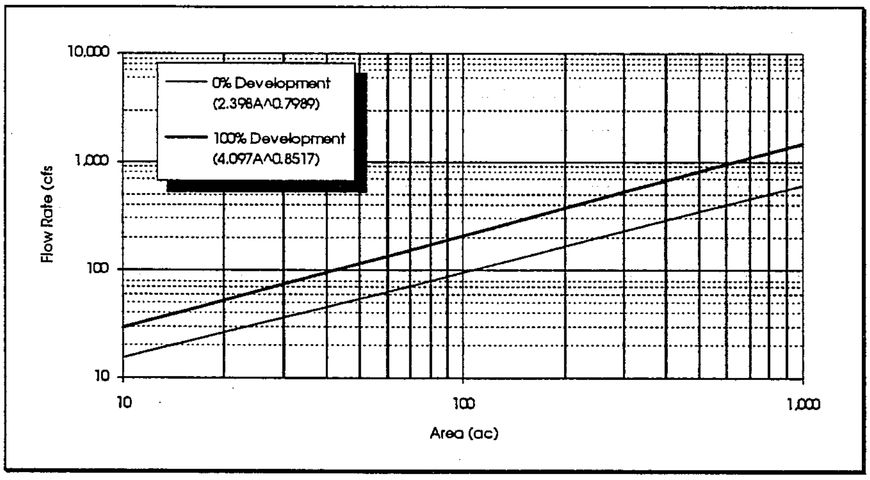


FIGURE 3.3 Runoff Rate Curves for 10-Year Storm

FIGURE 3.4 Runoff Rate Curves for 50-Year Storm

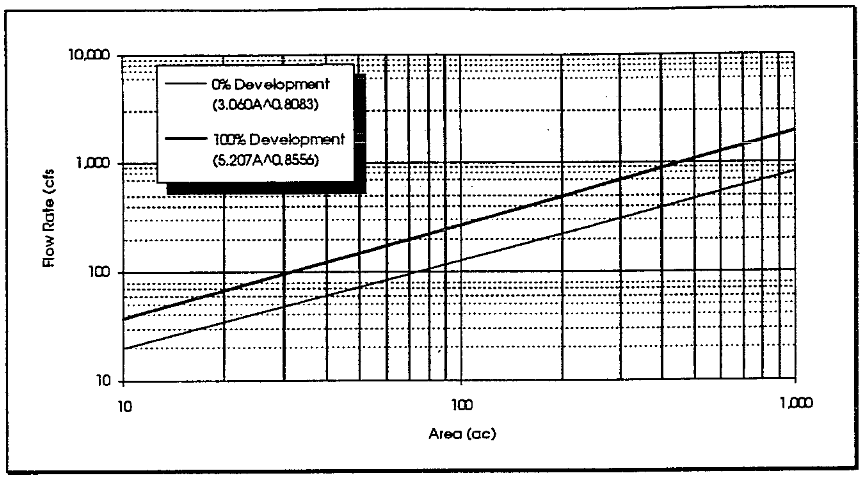
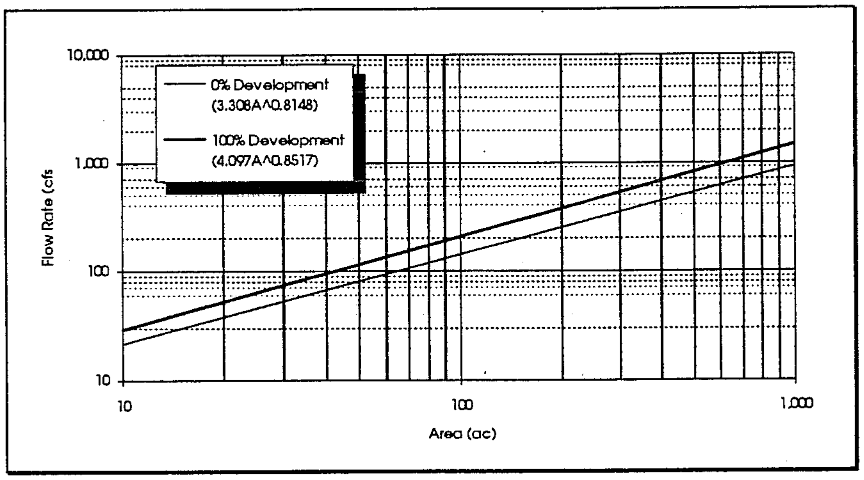


FIGURE 3.5 Runoff Rate Curves for 100-Year Storm



When flow rates for storm events other than the 10-year, 50-year and 100-year are required, plot the 10-year and 100-year flow rates on log-probability paper and connect them with a straight line. Interpolate or extrapolate as needed to determine the peak flow rate for the required storm frequency.

Applicable flow rates for existing conditions in the design of detention facilities should be determined on a case-by-case basis working closely with the appropriate drainage regulatory agency (See Chapter 7).

Hydrograph Computations Using Hydrologic Method I or II

A technique for hydrograph development which is useful in the design of detention facilities serving relatively small watersheds has been presented by H.R. Malcom [Malcom, Undated]. This procedure can be used in conjunction with the Rational Method or the Runoff Rate Curves. The methodology utilizes a pattern hydrograph to obtain a curvilinear design hydrograph which peaks at the design flow rate and which contains a runoff volume consistent with the design rainfall. The pattern hydrograph is a step function approximation to the dimensionless hydrograph proposed by the Bureau of Reclamation and the Soil Conservation Service [SCS, 1972].

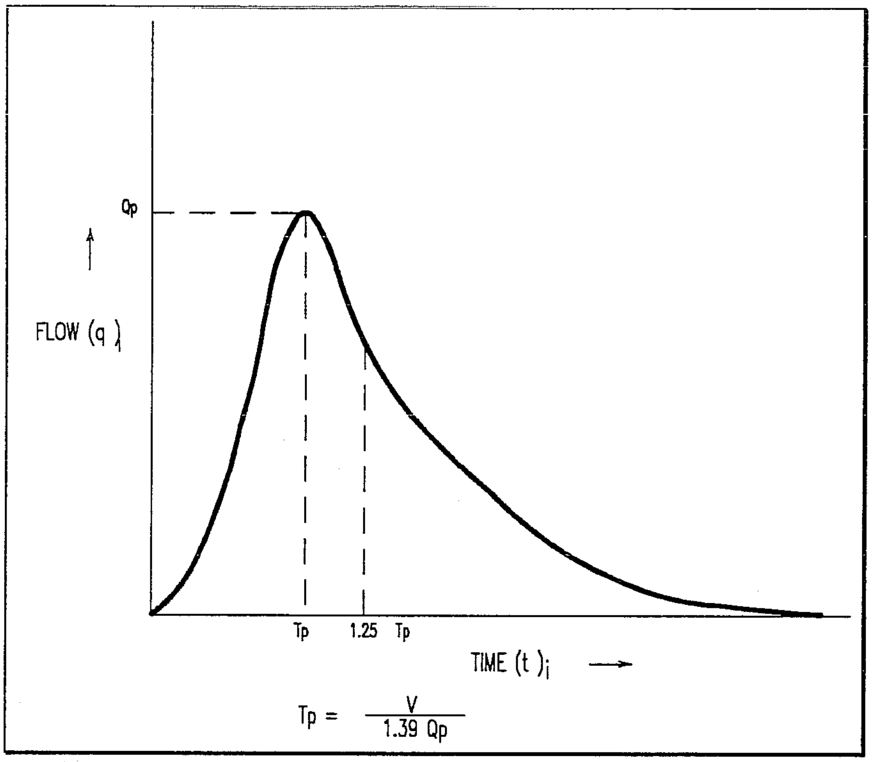


FIGURE 3.6 Malcom Method Hydrograph

Hydrograph Computation Equations

Malcom's Method consists of the following equations:

$$T_P = \frac{V}{1.39 Q_P}$$ Equation 3.7

$$q_i = \left(\frac{Q_P}{2}\right)\left[1 - \cos\left(\frac{\pi t_i}{T_P}\right)\right] \qquad t_i \le 1.25 T_P$$ Equation 3.8

$$q_i = 4.34 Q_P e^{\left(-1.3 t_i / T_P\right)} \qquad t_i > 1.25 T_P$$ Equation 3.9

in which:

Q_P = peak design flow rate in cfs

T_P = time to Q_P in seconds

V = total volume of runoff for the design storm in cubic feet

The variables t_i and q_i are the respective time and flow rates which determine the shape of the hydrograph. **In Equation 3.8, the argument of the COS function must be expressed in radians, not degrees.**

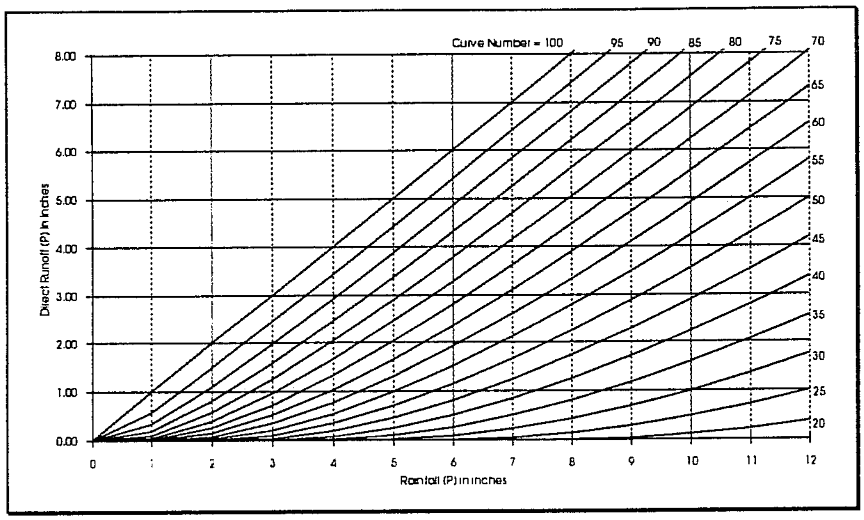
Procedures for Applying Hydrograph Computations for Hydrologic Methods I and II

The peak design flow rate can be calculated directly, using Hydrologic Method I or Hydrologic Method II, depending upon the size of the area considered.

The total volume of runoff is dependent on the characteristics of the soil and the degree of urbanization of the area (i.e. percent of impervious cover). Loss rate totals may be estimated using the **SCS Curve Number** methodology developed by the Soil Conservation Service [SCS, 1972].

Figure 3.7 provides graphs for the determination of runoff volume for a given rainfall depth and SCS Curve Number.

FIGURE 3.7 Determination of Runoff Volume Using SCS Curve Number



The SCS provides information on relating soil group type to curve number as a function of soil cover, land use type and antecedent moisture conditions. The SCS soil classification system uses four groups, as follows:

- **Group A**: deep sand, deep loess, aggregated silts
- **Group B**: shallow loess, sandy loam
- **Group C**: clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay.
- **Group D**: soils that swell significantly when wet, heavy plastic clays, and certain saline soils.

All other factors being equal, Group A soils have the lowest runoff potential and Group D soils have the highest runoff potential.

Tables 3.9 and 3.10 of this manual lists appropriate values for the SCS Curve Number for each of the four SCS soil groups. The tables are also organized according to the SCS cover complex, which consists of three factors: land use, land treatment or practice, and hydrologic condition. For example, the land use for a particular area may be "Row crops". If the land treatment or practice is "Straight row" and the hydrologic condition is "Good", then the SCS Curve Number would range from 67 to 89, depending on the soil group.

TABLE 3.9 Values of SCS Curve Number for Rural Areas

Source: (McCuen, 1982)

Land Use Description	Hydrologic Soil Group			
	A	B	C	D
Fallow				
Straight Row	77	86	91	94
Row Crops				
Straight Row, Poor Condition	72	81	88	91
Straight Row, Good Condition	67	78	85	89
Contoured, Poor Condition	70	79	84	88
Contoured, Good Condition	65	75	82	86
Contoured and Terraced, Poor Condition	66	74	80	82
Contoured and Terraced, Good Condition	62	71	78	81
Small Grain				
Straight Row, Poor Condition	65	76	84	88
Straight Row, Good Condition	63	75	83	87
Contoured, Poor Condition	63	74	82	85
Contoured, Good Condition	61	73	81	84
Contoured and Terraced, Poor Condition	61	72	79	82
Contoured and Terraced, Good Condition	59	70	78	81
Close-Seeded Legumes or Rotation Meadow				
Straight Row, Poor Condition	66	77	85	89
Straight Row, Good Condition	58	72	81	85
Contoured, Poor Condition	64	75	83	85
Contoured, Good Condition	55	69	78	83
Contoured and Terraced, Poor Condition	63	73	80	83
Contoured and Terraced, Good Condition	51	67	76	80
Pasture or Range				
Poor Condition	68	79	86	89
Fair Condition	49	69	79	84
Good Condition	39	61	74	80
Contoured, Poor Condition	47	67	81	88
Contoured, Fair Condition	25	59	75	83
Contoured, Good Condition	6	35	70	79
Meadow, Good Condition	30	58	71	78
Woods or Forest Land				
Poor Condition	45	66	77	83
Fair Condition	36	60	73	79
Good Condition	25	55	70	77
Farmsteads	59	74	82	86

TABLE 3.10 Values of SCS Curve Number for Urban and Suburban Areas

Source: (McCuen, 1982)

Land Use Description	Hydrologic Soil Group			
	A	B	C	D
Residential				
1/8 acre or less average lots (65% impervious)	77	85	90	92
1/4 acre average lots (38% impervious)	61	75	83	87
1/3 acre average lots (35% impervious)	57	72	81	86
1/2 acre average lots (25% impervious)	54	70	80	85
1 acre average lots (20% impervious)	51	68	79	84
Paved parking lots, roofs, driveways, etc.	98	98	98	98
Streets and Roads				
Paved with curbs and storm sewers	98	98	98	98
gravel	76	85	89	91
dirt	72	82	87	89
Commercial & Business Areas (85% Impervious)	89	92	94	95
Industrial Districts (72% Impervious)	81	88	91	93
Open Spaces, Lawns, Parks, Golf Courses, Cemeteries, etc.				
good condition: grass cover on 75% or more	39	61	74	80
fair condition: grass cover on 50% to 75%	49	69	79	84

Usually, the best source of information for determining the SCS soil group for a particular drainage area is the *Soil Survey of Galveston County, Texas* [SCS, 1988].

Example of Hydrograph Computation for Hydrologic Methods I and II

As an example of the Malcom method of developing a hydrograph, we will develop a runoff hydrograph for a 535-acre watershed which is 5.1% impervious.

According to Table 3.3, the 100-year, 24-hour rainfall total for Galveston County is 13.00 inches.

Assume that the Soil Survey for Galveston County indicates that the soils in the watershed are predominately SCS Soil Group C. The undeveloped portion of the watershed consists of woodlands in good condition. Therefore, according to Table 3.9, the SCS Curve Number should be about 70.

Referring to Figure 3.7, the total runoff volume from the undeveloped portion of the watershed is 8.2 inches. However, there are no infiltration losses for the 5.1% of the watershed which is impervious, so the runoff from that area is 12.17 inches. Therefore, the runoff volume in cubic feet is computed as follows:

$$V_1 = \frac{94.9\% \times 535 \text{ ac} \times 43{,}560 \text{ sq ft / ac} \times 8.20 \text{ in}}{12 \text{ in / ft}}$$

$$= 15.11 \text{ million cubic feet}$$

$$V_2 = \frac{5.1\% \times 535 \text{ ac} \times 43{,}560 \text{ sq ft / ac} \times 12.17 \text{ in}}{12 \text{ in / ft}}$$

$$= 1.21 \text{ million cubic feet}$$

$$V = V_1 + V_2 = 16.32 \text{ million cubic feet}$$

The time to peak is computed using Equation 3.7:

$$T_P = \frac{V}{1.39 Q_P}$$

$$T_P = \frac{16{,}320{,}000 \text{ cu ft}}{1.39 \times 901 \text{ cu ft / sec} \times 60 \text{ sec / min}} = 222 \text{ min}$$

The computed time interval for hydrograph computations is $T_P/10$.

$$t = 137/10 = 22 \text{ min}$$

For convenience, the computed hydrograph will be based on 20-minute intervals. For $t_i \leq 1.25 T_P$ (277.5 minutes), use equation 3.8. For $t_i > 1.25 T_P$ (277.5 minutes), use equation 3.9. Table 3.11 lists the computed runoff hydrograph.

TABLE 3.11 Computed Runoff Hydrograph for Example Watershed

Time t (min)	Time t (hr)	Flow Rate q (cfs)	Equation Used
0	0.00	0	3.8
20	0.33	19	3.8
40	0.67	73	3.8
60	1.00	160	3.8
80	1.33	270	3.8
100	1.67	395	3.8
120	2.00	525	3.8
140	2.33	649	3.8
160	2.67	756	3.8
180	3.00	838	3.8
200	3.33	887	3.8
220	3.67	901	3.8
240	4.00	876	3.8
260	4.33	816	3.8
280	4.67	731	3.9
300	5.00	648	3.9
320	5.33	575	3.9
340	5.67	510	3.9
360	6.00	452	3.9
380	6.33	401	3.9
400	6.67	356	3.9
420	7.00	316	3.9
440	7.33	280	3.9
460	7.67	249	3.9

METHOD III HYDROLOGIC ANALYSIS

Hydrologic analyses of watersheds larger than 500 acres shall be completed using the HEC-1 computer program. A stream network model which simulates the runoff response of a drainage basin to rainfall over that basin can be developed with the HEC-1 computer program through the appropriate combination of runoff and routing computations. The following sections describe the elements required to develop a HEC-1 computer model.

Background of Hydrologic Method III

The HEC-1 computer program was developed by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers. The HEC-1 program is a widely-accepted tool for watershed analysis. The hydrologic processes which may be simulated using the HEC-1 program include rainfall, infiltration, and runoff.

The use of the HEC-1 program allows the division of large watersheds into a number of sub-watersheds. A runoff hydrograph from each individual sub-watershed may then be computed, routed downstream, and combined with runoff hydrographs from other sub-watersheds. This feature of the HEC-1 program, along with capabilities which facilitate the modeling of flow diversions and other special hydrologic conditions, allows large, complex watersheds to be divided into smaller, more homogeneous units which may be more easily handled. This in turn increases the level of detail associated with the analysis of the watershed.

The HEC-1 program computes ***hydrographs***, or relationships between flow rate and time, at user-specified locations in a watershed. The program performs computations at even intervals of time throughout the duration of the storm event being analyzed. At each interval, HEC-1 determines how much rainfall occurs over the watershed or sub-watershed and computes the infiltration loss and runoff rate. The interval of time used by the program is specified by the user and is termed the "computation interval." This same computation interval is used in routing or combining runoff hydrographs.

Rainfall

The first major component in any HEC-1 model is rainfall data. The data consists of a total rainfall depth and a relationship which defines the distribution of rainfall over the storm duration. Total rainfall depths are obtained using NOAA Technical Memorandum NWS HYDRO-35, "*Five- to 60-Minute Precipitation Frequency for the Eastern and Central United States*," and U.S. Weather Bureau Technical Paper No. 40, "*Rainfall Frequency Atlas of the United States*."

Rainfall events are often described in terms of their ***recurrence interval***. The recurrence interval of a storm event is the long-term average number of years between occurrences of the event. For example, a 5-year storm event has a recurrence interval of 5 years. However, it is possible for several 5-year events to occur within the same year; the recurrence interval is merely a *long-term* average.

A ***10-Year storm event*** is a rainfall event which has a 10% chance of occurring in any given year, or a rainfall event which, over an extended period of time, is expected to occur once every 10 years.

A ***50-Year storm event*** is a rainfall event which has a 2% chance of occurring in any given year, or a rainfall event which, over an extended period of time, is expected to occur once every 50 years.

A ***100-Year storm event*** is a rainfall event which has a 1% chance of occurring in any given year, or a rainfall event which, over an extended period of time, is expected to occur once every 100 years.

A ***500-Year storm event*** is a rainfall event which has a 0.2% chance of occurring in any given year, or a rainfall event which, over an extended period of time, is expected to occur once every 500 years.

Table 3.3 summarizes data relating rainfall depth to storm frequency and duration. These data values are used to develop rainfall distributions for use in the HEC-1 model.

The rainfall distributions developed for each storm event are assembled using a method developed by the Galveston District of the U.S. Army Corps of Engineers and used by the Harris County Flood Control District. This method results in the distribution of rainfall over a 24-hour period, with the most intense rainfall centered around the 16th hour. Six rainfall distributions are developed for each storm event. Each distribution corresponds to a different drainage area. The total rainfall associated with each distribution decreases as the drainage area increases. This reflects the fact that severe rainfall events over large areas are less likely to occur than are events which involve only small areas.

Figure 3.9 presents the rainfall distribution for the 100-year storm event. There are actually six distributions for the 100-year storm event. Each distribution corresponds to a different total drainage area ranging from 0.1 square mile to 300 square miles. As illustrated, the smaller drainage areas are assumed to experience more intense rainfall during the peak periods of the rainfall event. Figure 3.9 also illustrates that the total depth of rainfall experienced during the 24-hour storm also declines slightly with drainage area.

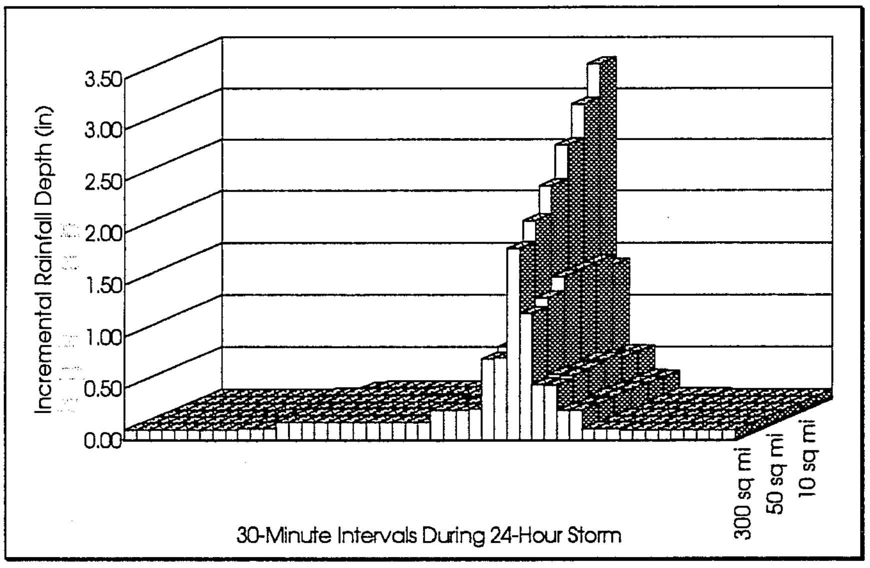


FIGURE 3.8 Rainfall Hyetographs for Various Drainage Areas

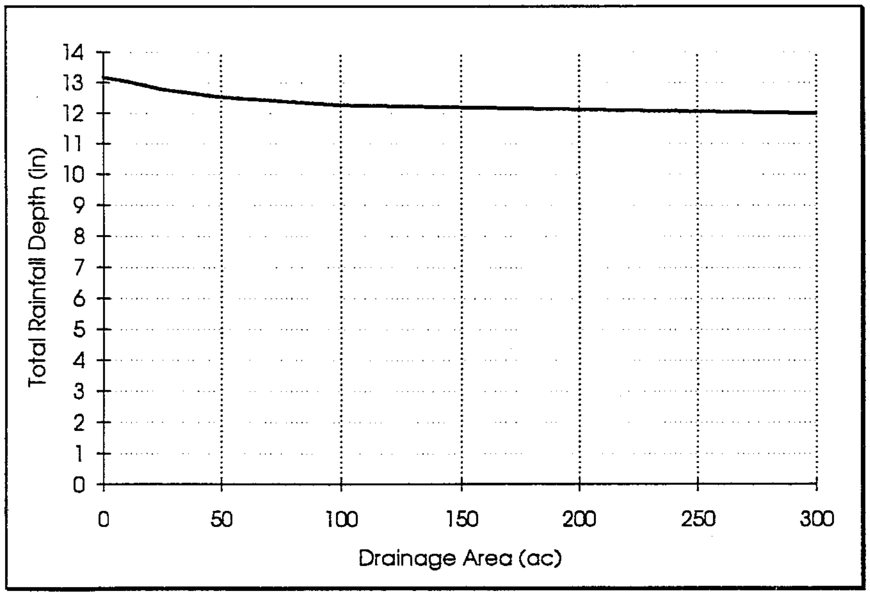


FIGURE 3.9 Total Rainfall Depth for Various Drainage Areas

100-yr

JD: 13.18 0.01
JD, 13.05 10.0
JD 12.78 25
JD 12.52 50
JD 12.26 100
JD 11.99 300
(in) sq mi

Figure 3.10 illustrates how these storm distributions are entered into the HEC-1 computer program. In each distribution, the IN record is used to input the time interval on which the incremental rainfall data on PI records are based, as well as the starting date and time of the rainfall event. The JD record is used to input the total rainfall depth and drainage area for each distribution. For each hydrograph operation (compute, combine, or route), the HEC-1 program computes an index hydrograph corresponding to each of the six drainage areas specified on JD records. The program then interpolates a hydrograph using the actual total drainage area.

FIGURE 3.10 100-Year Rainfall Depth vs. Area Relationship

12.92 , 10

```
IN    30 01JAN92     0000
JD 13.18       .01
PI   .09       .09    .09    .09    .09    .09    .09    .09    .09    .10
PI   .10       .10    .16    .17    .17    .17    .17    .17    .17    .17
PI   .17       .17    .17    .17    .27    .27    .28    .28    .78    .79
PI  3.31      1.35    .52    .52    .28    .27    .10    .10    .10    .10
PI   .09       .09    .09    .09    .09    .09    .09    .09
IN    30 01JAN92     0000
JD 13.05        10
PI   .09       .09    .09    .09    .09    .09    .09    .09    .09    .09
PI   .10       .10    .17    .17    .17    .17    .17    .17    .18    .18
PI   .18       .18    .18    .18    .28    .28    .28    .29    .77    .78
PI  2.98      1.45    .54    .53    .28    .28    .10    .10    .10    .09
PI   .09       .09    .09    .09    .09    .09    .09    .09
IN    30 01JAN92     0000
JD 12.78        25
PI   .09       .09    .09    .09    .09    .09    .09    .09    .09    .09
PI   .09       .09    .17    .17    .17    .17    .17    .17    .17    .17
PI   .17       .17    .18    .18    .28    .29    .29    .29    .78    .79
PI  2.65      1.45    .58    .58    .29    .29    .09    .09    .09    .09
PI   .09       .09    .09    .09    .09    .09    .09    .09
IN    30 01JAN92     0000
JD 12.52        50
PI   .09       .09    .09    .09    .09    .09    .09    .09    .09    .10
PI   .10       .10    .16    .16    .16    .16    .16    .17    .17    .17
PI   .17       .17    .17    .17    .29    .29    .29    .30    .82    .83
PI  2.32      1.45    .56    .56    .30    .29    .10    .10    .10    .10
PI   .09       .09    .09    .09    .09    .09    .09    .09
IN    30 01JAN92     0000
JD 12.26       100
PI   .09       .09    .09    .09    .09    .10    .10    .10    .10    .10
PI   .10       .10    .17    .17    .17    .17    .17    .18    .18    .18
PI   .18       .18    .18    .18    .27    .28    .28    .28    .84    .85
PI  2.05      1.31    .57    .56    .28    .28    .10    .10    .10    .10
PI   .10       .10    .10    .09    .09    .09    .09    .09
IN    30 01JAN92     0000
JD 11.99       300
PI   .10       .10    .10    .10    .10    .10    .10    .10    .10    .11
PI   .11       .11    .17    .17    .17    .17    .17    .17    .17    .17
PI   .17       .17    .17    .17    .29    .29    .29    .30    .79    .80
PI  1.85      1.23    .54    .53    .29    .29    .11    .11    .11    .10
PI   .10       .10    .10    .10    .10    .10    .10    .10
```

Loss Rates

Only a portion of the rainfall which occurs over a watershed during a storm event actually becomes runoff. The remainder is intercepted by infiltration, depression storage, evaporation and other mechanisms. The portion of rainfall which becomes runoff is termed the ***excess rainfall.*** The difference between the observed total rainfall hyetograph and the excess rainfall hyetograph is termed ***abstraction***, or ***loss***.

A portion of the rainfall over a given watershed infiltrates into the soil. The remainder of the rainfall becomes runoff and contributes to storm flows in downstream areas. The HEC-1 program internally determines what portion of the rainfall is lost to infiltration. A number of loss methods, each of which utilizes different mathematical functions to represent the process of infiltration, are available to the HEC-1 user. For this analysis of the Dickinson Bayou watershed, losses in storm water runoff volume due to infiltration are modeled using the Green-Ampt loss function. The Green-Ampt function is an empirical method which relates infiltration loss function parameters to soil structure.

When the Green-Ampt loss function is specified, the HEC-1 program begins infiltration loss computations with a user-specified initial abstraction (variable name STRTL). All rainfall is assumed to infiltrate into the ground until the cumulative rainfall depth is equal to the initial abstraction. Once the initial abstraction is satisfied, the Green-Ampt loss function determines cumulative infiltration losses by applying the following empirical equations:

$$F(t) = \frac{PSIF - DTHETA}{\left(\frac{f(t)}{XKSAT}\right) - 1} \quad \text{for } f(t) > XKSAT$$

Equation 3.10

$$f(t) = r(t) \quad \text{for } f(t) \le XKSAT$$

Equation 3.11

in which:

$F(t)$ = cumulative infiltration

$f(t)$ = infiltration rate

$r(t)$ = rainfall intensity

PSIF = wetting front suction, the capillary suction which draws water into the soil, measured at the wetting front, or the limit of penetration of water into the soil. The wetting front separates wet soil from dry soil.

DTHETA = soil moisture deficit, the difference between the maximum amount of moisture which a soil can hold and the amount of moisture actually present in the soil at any given time.

XKSAT = hydraulic conductivity of saturated soil, the ratio of the rate of flow of water through a soil to the energy gradient causing that flow. Hydraulic conductivity is a measure of soil permeability.

The recommended Green-Ampt loss function parameters reflect a clay or clay loam soil. The values actually selected to represent each of the parameters are as follows:

- *STRTL* = 0.50 inches
- *PSIF* = 11.20
- *DTHETA* = 0.360
- *XKSAT* = 0.02.

Impervious areas, or areas covered with materials such as concrete and asphalt which allow no infiltration, are assumed by the HEC-1 program to have 100% runoff. Typical values for the percentage of impervious cover corresponding to various types of development are given in Table 3.12. These values should be used when only the general type of planned development is known; once the actual level of development has been determined for a specific area, a refined value should be used to reflect the actual percent of impervious cover.

A value of 35% has been used for a number of years by the HCFCD to represent the average percentage of impervious cover which may be typically expected in large areas of mixed residential and commercial development. This value is acceptable for planning studies.

TABLE 3.12 Typical Average Values for Impervious Cover

Source: SCS TR-55 (SCS, 1986)

Type of Development	Impervious Cover
Commercial and Business Areas	85%
Industrial Areas	72%
Residential Areas, Average lot size 1/8 Acre or less	65%
Residential Areas, Average lot size 1/4 Acre	38%
Residential Areas, Average lot size 1/3 Acre	30%
Residential Areas, Average lot size 1/2 Acre	25%
Residential Areas, Average lot size 1 Acre	20%

Sub-Basin Hydrograph Computations

The Harris County Flood Hazard Study was completed during the early 1980's. As a part of that study, the Harris County Flood Control District developed a standard methodology for the computation of sub-basin hydrograph parameters. The following watershed parameters or characteristics are used in the HCFCD standard hydrologic methodology.

- **Drainage Area (*A*):** Area within the watershed being analyzed, in square miles.
- **Watershed Length (*L*):** Total length of longest watercourse, from the outflow point to the upstream watershed boundary, in river miles.
- **Length to Centroid (L_{ca}):** Length along longest watercourse from the outflow point to a point opposite the computed centroid of the drainage area, in river miles.
- **Channel Slope (*S*):** Weighted channel slope, measured along the longest watercourse and computed between stations equal to 10 percent and 85 percent of L, in feet per mile.
- **Watershed Slope (S_o):** Watershed slope, measured along an average overland watercourse, from the overbank of the main watercourse to the watershed divide, and computed between stations equal to 10 percent and 85 percent of the total overland watercourse length, in feet per mile.
- **Percent Urban Development (*UD*):** Portion(s) of drainage area for residential, industrial, commercial, and institutional use, measured from aerial photos, in percent of total drainage area.

- **Percent Channel Improvement (*CI*):** Portion(s) of longest watercourse with improved channel sections, measured from aerial photos or construction drawings, in percent of total definable channel length.
- **Percent Average Channel Conveyance (*CC*):** Ratio of discharge carried in channel to total discharge, measured at several representative cross-sections along the main watercourse from the outflow point to the upstream end of the main channel at the watershed boundary and averaged, in percent.
- **Percent Watershed Development (*WD*):** A composite factor based on 30 percent × Percent Urban Development plus 70 percent × Percent Channel Improvement, in percent.
- **Percent Ponding (*P*):** Portion(s) of drainage area where runoff is retarded from reaching a watercourse because of physical obstructions (i.e., leveed field, swamps, etc.), measured in percent of total drainage area.
- **Percent Impervious (*RTIMP*):** Portion(s) of drainage area covered by impervious materials such as concrete and asphalt, in percent of total drainage area.

Clark Unit Hydrograph

Given the design storm excess rainfall, it is necessary to determine the storm runoff hydrograph at the point of interest utilizing the HEC-1 program. A ***unit hydrograph*** represents the hydrologic response of a watershed to excess rainfall. A Unit Hydrograph is a hydrograph, or relationship between flow rate and time, which results from 1 inch of rainfall excess (1 inch of runoff). The Clark method of computing unit hydrographs is one of the methods available within the HEC-1 computer program, and is widely accepted.

The Clark unit hydrograph for a drainage area is described by three parameters: *TC*, *R* and a time-area curve. Figure 3.11 illustrates the Clark Unit Hydrograph and its parameters.

1. *TC* represents the ***time of concentration***. The time of concentration is the time required for storm runoff to flow from the most remote point in a watershed to the outlet point.
2. *R* is a ***watershed storage coefficient***. The storage coefficient is an indicator of the available stormwater storage volume within a watershed. This storage volume may be provided in depressions or low areas, ponds, or in the channel and flood plain of the stream under consideration. The relative amount of storage volume within a watershed varies directly with the storage coefficient, i.e., the higher the storage coefficient, the greater the storage volume.
3. The ***time-area curve*** defines the cumulative area of the watershed contributing runoff to the design point as a function of time. The Time-Area Curve relates the fraction of the total watershed area contributing runoff at the watershed

outlet to the fraction of the Time of Concentration which has elapsed since the beginning of runoff. The entire watershed is considered to be contributing runoff at the outlet when the elapsed time is equal to the Time of Concentration. HEC-1 offers a standard time-area curve based on an assumed watershed shape. This standard time-area curve is used in most situations. The HEC-1 time-area curve is defined by the following equations:

Equation 3.12

$$AI = 1.414T^{3/2} \qquad 0 \le T < 0.5$$

Equation 3.13

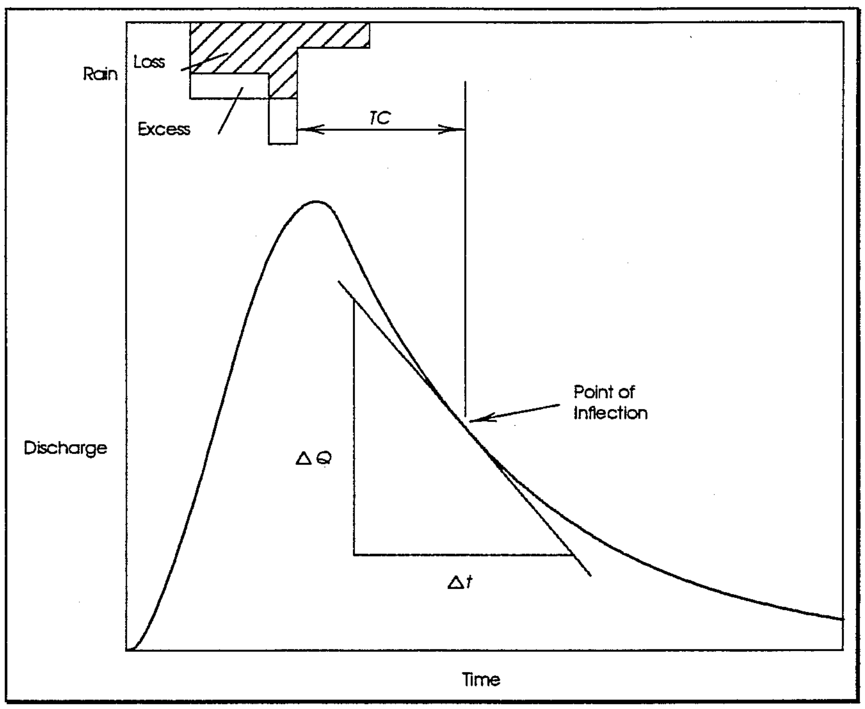
$$1 - AI = 1.414(1-T)^{3/2} \qquad 0.5 < T < 1$$

in which:

AI = cumulative area as a fraction of total watershed area

T = fraction of the time of concentration.

FIGURE 3.11 HEC-1 Clark Unit Hydrograph



The following steps are completed by the HEC-1 program in developing a unit hydrograph using the Clark method:

1. X and Y ordinates from the Time-Area Curve are multiplied by the watershed area and time of concentration, respectively, to obtain a relationship between the amount of drainage area contributing flow at the watershed outlet and elapsed time since the beginning of runoff;
2. Values of total watershed area are multiplied by 1 inch of runoff to yield a relationship between runoff volume and elapsed time since the beginning of runoff;
3. Incremental runoff volumes are determined for equal time intervals and divided by that time interval to yield a

"translation hydrograph" relating flow rate (volume per unit time) and elapsed time from beginning of runoff;

4. Using routing coefficients computed from Watershed Storage Coefficient, the translation hydrograph is routed through a linear reservoir which simulates the effects of storage within the watershed.

The relationships used in the routing process are:

$$Q_2 = (CA \times I) + (CB \times I)$$ Equation 3.14

$$QUNGR = \frac{Q_1 + Q_2}{2}$$ Equation 3.15

in which:

Q_2 = instantaneous flow at the end of a time interval

Q_1 = instantaneous flow at the beginning of a time interval

I = the flow rate from the translation hydrograph for the given time interval

$$CA = \frac{dt}{R + dt/2}$$

$CB = 1 - CA$

dt = the computation time interval

R = the watershed storage coefficient

$QUNGR$ = the unit hydrograph flow rate at the end of the given time interval.

The resulting unit hydrograph may be used to compute a runoff hydrograph for the watershed by multiplying each unit hydrograph ordinate by the computed total rainfall excess, or runoff, in inches. In the HEC-1 program, a separate runoff hydrograph is computed for each computation time interval using the unit hydrograph and the rainfall excess for that interval. The ordinates of all of the individual runoff hydrographs are totaled for each time interval to obtain a complete runoff hydrograph for the watershed.

The following equations are used in the HCFCD's standard hydrologic methodology to relate the watershed characteristics to the Clark Unit Hydrograph parameters, TC and R:

$$TC = 2.46(1 - 0.0062WD)\left(\frac{L_{ca}}{\sqrt{S}}\right)^{1.06} \qquad So \le 20 \text{ ft./mi.}$$ Equation 3.16

$$TC = 3.79(1 - 0.0062WD)\left(\frac{L_{ca}}{\sqrt{S}}\right)^{1.06} \qquad 20 \text{ ft./mi.} < So \le 40 \text{ ft./mi.}$$ Equation 3.17

$$TC = 5.12(1 - 0.0062WD)\left(\frac{L_{ca}}{\sqrt{S}}\right)^{1.06} \qquad So > 40 \text{ ft./mi.}$$ Equation 3.18

Equation 3.19

$$WD = 0.30(UD) + 0.70(CI)$$

Equation 3.20

$$TC + R = 7.25\left(\frac{L}{\sqrt{S}}\right)^{0.706} \qquad \text{(if } \%UD \le 18\text{)}$$

Equation 3.21

$$TC + R = 4295UD - 0.678CC - 0.967\left(\frac{L}{\sqrt{S}}\right)^{0.706} \qquad \text{(if } \%UD > 18\text{)}$$

in which:

L = Watershed Length (miles)

L_{ca} = Length to Centroid (miles)

S = Channel Slope (feet/mile)

UD = Percent Urban Development

CI = Percent Channel Improvement

WD = Percent Watershed Development

CC = Percent Channel Conveyance

TC = Time of Concentration (hours)

R = Watershed Storage (hours)

Ponding Adjustment

The adjustment for ponding is made by increasing the Storage Coefficient, R, for Clark's unit hydrograph method. The equation used to determine the adjusted storage coefficient is:

$$R_p = R \times RM$$

where:

R_p = storage coefficient adjusted for ponding

R = unadjusted storage coefficient

RM = storage coefficient adjustment factor.

The value of RM is computed using the percent ponding and the following equation:

$$RM = aP^b$$

in which:

P = Percent Ponding

a, b = values determined from Figures 3. and 3.. In both figures, the values of a and b depend upon the percentage of the watershed area affected by the ponding. This depends upon the location of the ponded area. For example, a ponded area located at the mouth of the watershed would affect 100% of the watershed area.

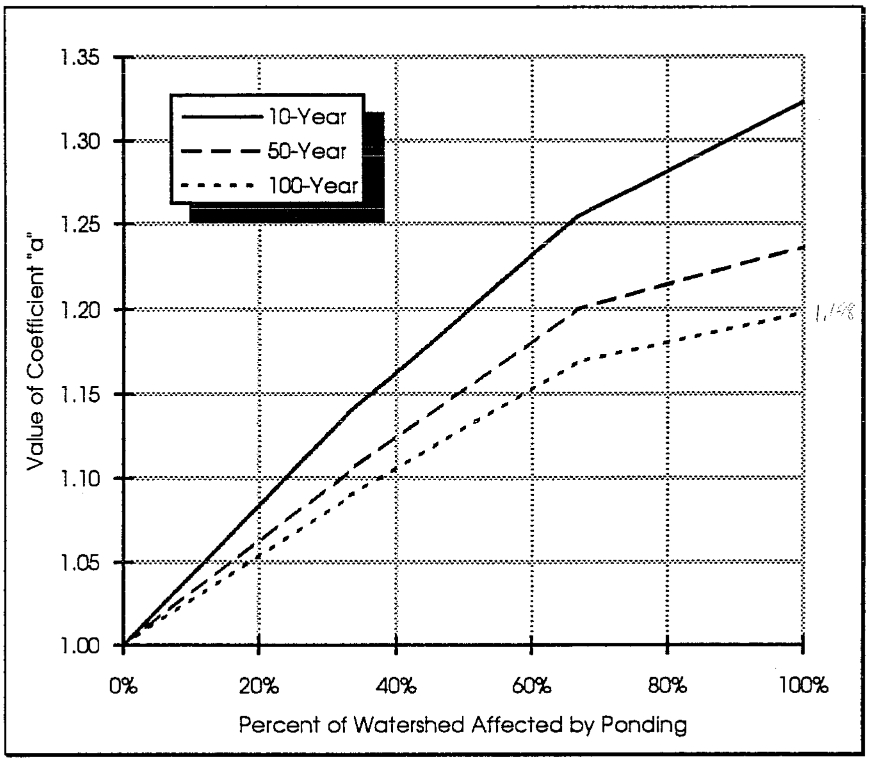


FIGURE 3.12 Value of Coefficient "a" in Ponding Adjustment Equation

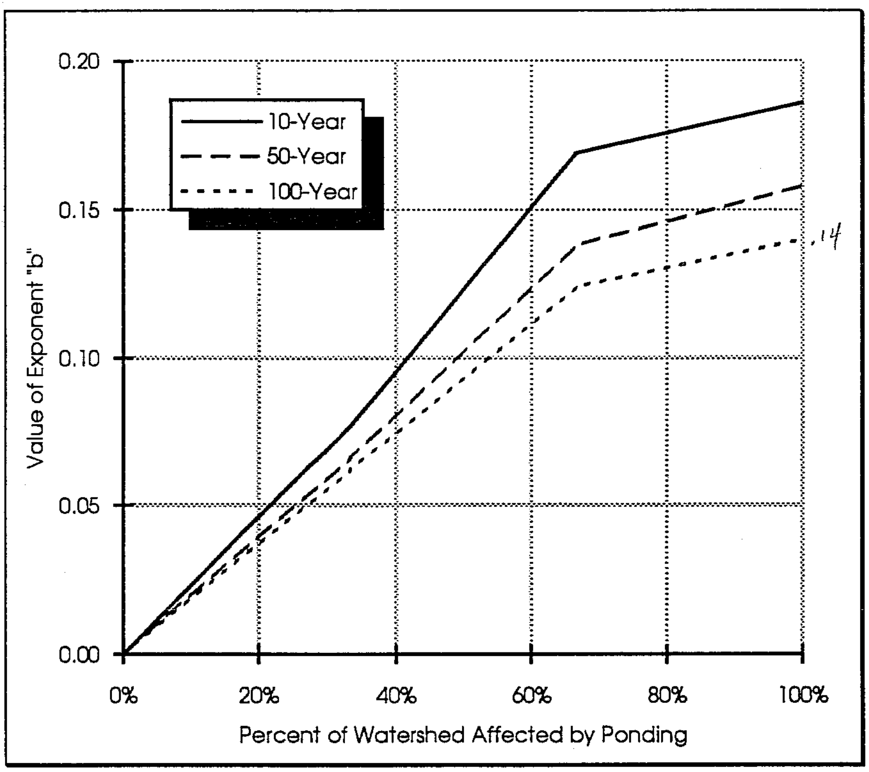


FIGURE 3.13 Value of Exponent "b" in Ponding Adjustment Equation

The procedure for determining the appropriate storage coefficient adjustment factor is as follows:

1. Compute the percent ponding area and the percent of the total watershed area affected by ponding in the subject watershed;
2. Use the computed value of the percent area affected by ponding along with the graphs to determine the appropriate coefficients for use in the *RM* equation;
3. Use the coefficients determined in step 2 and the computed value of percent ponding area to compute the storage coefficient adjustment factor.

Consider as an example a watershed which has 15% ponding area and 40% area affected by ponding area. Using the graphs, the value of the constant to be used in the *RM* calculation is 1.109, while the value of the exponent is 0.080. The storage coefficient adjustment factor may be computed as follows:

$$RM = 1.109(15)^{0.080}$$

$$RM = 1.387$$

For a ponding area equal to 15%, the current Harris County methodology yields a value of *RM* equal to 1.73. The current Fort Bend County methodology yields a value of *RM* equal to 1.29 for a ponding area of 15% and a total area affected by ponding equal to 40%.

This ponding adjustment method is based on data published by the U.S. Soil Conservation Service in Technical Release No. 55, "Urban Hydrology for Small Watersheds" (January 1975). The SCS data consists of tables of peak flow rate reduction coefficients which correspond to values of percent ponding area. Three separate tables are presented, each table corresponding to a different location of ponding areas:

1. Ponding areas located at the watershed outlet,
2. Ponding areas located in the central portion of the watershed, and
3. Ponding areas located in the upper reaches of the watershed.

Channel Flood Routing

As a flood wave passes downstream through a channel or detention facility, it is altered due to the effects of storage and travel time. The procedure for determining how the shape of the flood hydrograph changes is termed ***flood routing***.

Flood routing can be classified into two broad but related categories: open channel routing and reservoir routing. ***Reservoir routing*** is generally used to determine the effectiveness of stormwater detention, which reduces downstream peak flow rates. ***Open channel routing*** is a refinement of the description of an area's rainfall-runoff process. It modifies the time rate of runoff due to storage and travel time within the channel and its overbanks. Analysis of areas with very flat overbanks and wide flood plains should include channel routing to determine possible peak discharge attenuation and lagging.

Modified Puls Method

The movement of flood waters from a given point in a watershed to a downstream location is simulated in the HEC-1 program through the use of flood routing reaches. The recommended technique for both channel and reservoir routing is the ***Modified Puls*** method. The Modified Puls method is based on the assumption of an invariable discharge-storage relationship and a constant level pool in the storage reach of interest. The relationships used to route flows through each routing reach are based on the continuity equation:

$$dS = I - O \qquad \text{Equation 3.22}$$

in which:

dS = change in storage volume in routing reach

I = inflow to routing reach

O = outflow from routing reach.

HEC-2 Data

The required storage-discharge relationships for this routing technique can be obtained through use of the HEC-2 backwater program for a variety of flow conditions. HEC-2 is generally used to determine storage volumes and travel times for a wide range of flow rates.

The storage volumes computed using the HEC-2 program are input to the HEC-1 program along with corresponding flow rates to establish a relationship between storage volume and the rate of outflow from the routing reach. The number of steps used for each routing reach is determined by dividing the average travel time by the HEC-1 computation interval.

Care must be taken in developing these storage-discharge relationships with HEC-2. Cross-sections should be provided to adequately define all of the flood plain storage available at various water levels. However, only the effective area of the cross-section should be used to establish the proper discharge-water level relationship. For a detailed discussion of the Modified Puls routing technique and other methodologies, see the *Handbook of Applied Hydrology* [Chow, 1964].

Stream Flow Diversions

Diversions of stream flows occur when a portion of the total flow in a given stream leaves that stream. The diversion of flows may be temporary in that the diverted flow may re-enter the stream at some downstream location, or the diversion may be permanent. The HEC-1 program allows diversions of flow to be represented using a relationship between total flow and diverted flow. The flow diversion relationship is used to determine the amount of flow diverted and the amount of flow remaining in the given stream for each time interval during the storm event. The resulting relationship between diverted flow and time yields a diversion hydrograph. These hydrographs are given an identifying label and may be recalled by the user at any point for routing or combining with other hydrographs.

Chapter 4. Hydraulic Analysis and Design

The purpose of this chapter is to introduce the hydraulic design and analysis concepts needed for plan or study preparation in the Dickinson Bayou watershed.

Design Storm Frequencies

All open channels in the Dickinson Bayou watershed shall be designed to contain the runoff from the 100-year frequency 24-hour duration storm within the right-of-way. In addition, the channel must be designed to have sufficient freeboard to provide for adequate drainage of lateral storm sewers during the 25-year storm.

In those cases where channel modifications are necessary to control increased flow from proposed development, proposed water surface profiles are restricted such that the 100-year flood profile under existing conditions shall not be increased. If the capacity of the existing channel downstream of the project is less than the 100-year design discharge, consideration shall be given for more frequent events to ensure that the severity and frequency of downstream flooding are not increased.

Closed Conduit Design

This section describes the use of closed conduit systems in place of channel sections.

Specific Design Flow Frequency for Closed Conduits

The recommended design flow frequency criteria to be used for continuous closed-conduit systems are given below:

1. **General Requirement**: For all drainage areas, the design flows shall be determined using a 3-year rainfall intensity. A 25-year design water surface should be assumed in the outfall channel.
2. **Areas Over 100 Acres**: For portions of the system serving areas between 100 acres and 200 acres, it is additionally required that the 25-year hydraulic grade line be at or below the gutter line for the portion of the system which drains 100 or more acres. A 25-year design water surface should be assumed in the outfall channel.
3. **Areas Over 200 Acres**: For portions of the system serving an area larger than 200 acres, the 100-year flow for fully developed conditions shall be used to insure that the 100-year hydraulic grade line will be below the natural ground elevation at all points along this portion of the closed system. A 25-year design water surface should be assumed in the outfall channel.
4. **Inlet Capacity**: For systems designed in accordance with (2) or (3), sufficient additional inlet capacity shall be provided to allow for entry into the closed-conduit system of runoff in excess of the runoff conveyed through the storm sewer system up to the design capacity of the closed-conduit system.

5. **Overland Flow**: For all areas, overland flow shall be considered. Closed systems adjoined to an upstream open channel shall be designed for the 100-year ultimate discharge.

Storm sewers generally drain into open channels. In the design of storm sewer systems, therefore, it is required that the existing and ultimate 25-year water surface elevations be computed for the outfall channel, with the higher being used as the starting point for hydraulic grade line computations for the design of storm sewers.

General Design Criteria for Closed Conduits

Storm sewers shall be designed to carry the design storm peak flow. Acceptable methods for computing peak flow rates are described in Chapter 3 of this manual.

For all storm sewer systems or enclosed reaches of open channels, hydraulic calculations and hydraulic profiles along with the construction plans of the closed-conduit system must be submitted for review.

The following specific criteria and requirements shall apply to the design and construction of storm sewers:

1. **Starting Elevation**: Calculation of the hydraulic grade line for design conditions in a specific branch of storm sewer shall proceed upstream from the level of the 25-year water surface elevation in the outfall channel.
2. **Minimum Diameter**: The minimum diameter of a pipe in a storm sewer line shall be 24". A diameter of 18" may be acceptable in limited situations if approved by the applicable drainage regulatory authority.
3. **Manning Coefficient**: The Manning's "n" value to be used in a reinforced concrete pipe storm sewer shall be 0.013. For corrugated metal pipe, the "n" value shall be as shown in Table 4.1.
4. **Flow Velocities**: The minimum velocity of flow to be allowed in a section of storm sewer flowing full shall be 3 fps. The maximum velocity shall be 10 fps.
5. **Overland Flow**: Provisions must be made for all adjacent undeveloped areas with natural drainage patterns directing overland flow into and across planned areas of development.
6. **Required Items**: Before a particular storm sewer design will be reviewed, the following items must be presented:
   a) **Drainage Area Map**: A contour and drainage area map showing all pertinent sub-areas, including contributing off-site areas.
   b) **Flow Calculations**: A listing of all relevant hydrologic design flow calculations, which shall include all contributing off-site flows.

c) **Hydraulic Calculations**: Calculations for determining the hydraulic gradient, along with a profile which illustrates the results.

d) **Plan View Drawing**: A plan showing the location of all manholes and inlets, and the alignment of all storm sewers in the right-of-way.

e) **Profile View Drawing**: A profile showing the placement of storm sewers and the locations of all pipe size changes, grade changes, and pipe intersections.

7. **Construction Specifications**: All storm sewers and appurtenant construction shall conform to the requirements of City of Houston Form E-14-62 [City of Houston, 1980], City of Houston Drawing Nos. 529-S-1, 530-S-1, 530-S-2, and all subsequent revisions, or equivalent approved by the applicable drainage regulatory entity.

8. **Concrete Pipe**: All storm sewers shall be constructed with reinforced concrete pipe or approved equal. Corrugated galvanized metal pipe, or other approved equal, may be used only at the storm sewer outfall into grass-lined channels.

9. **Alignment**: All cast-in-place concrete storm sewers shall follow the alignment of the right-of-way or easement. All precast concrete pipe storm sewers shall be typically designed in a straight line or shall conform to the City of Houston Form E-14-62 [City of Houston, 1980], Drawing Numbers 529-S-1, 530-S-1, 530-S-2 and all subsequent revisions, or equivalent approved by the applicable drainage regulatory entity.

10. **Inlet Lead Alignment**: All storm sewer inlet leads shall be designed in a straight line alignment.

11. **Right-of-Way**: Storm sewers shall be located in public street rights-of-way or in easements that will not prohibit future maintenance access. In most cases where easements are restricted to storm sewers, the pipe should be centered within the limits of the easement.

12. **Soil Borings**: For all storm sewers having a cross-sectional area equivalent to a forty-two inch (42") inside diameter pipe or larger, soil borings with logs shall be made along the alignment of the storm sewer at intervals not to exceed five hundred feet (500') and to a depth not less than three feet (3') below the proposed invert of the sewer. The required bedding of the storm sewer as determined from these soil borings shall be shown in the profile of each respective storm sewer. The design engineer shall inspect the open trench and may authorize changes in the bedding indicated on the plans. Such changes shall be shown on the record drawings and, along with soil boring logs, submitted to the appropriate drainage regulatory authority. All bedding shall be constructed as specified in the City of Houston Form E-14-62 [City of Houston, 1980] and all subsequent revisions, or

equivalent approved by the applicable drainage regulatory entity.

13. **Outfall Erosion Control**: All storm sewer outfalls shall conform with the requirements and specifications defined in Chapter 6, and Figure 4.1.

Design of a storm sewer system should proceed as follows:

1. **Starting Elevation**: Determine the 25-year water surface elevation in the receiving channel at the storm sewer outfall using appropriate backwater calculations.
2. **Peak Flow Rates**: Determine the design flow rates for all sections of storm sewer based on drainage area size, time of concentration, density of development.
3. **Size Pipes**: Assuming storm sewer pipes are full at design flows, determine the appropriate sizes for all sections of storm sewer using Manning's equation and assuming uniform flow conditions.
4. **Compute Backwater**: Begin backwater calculations at the 25-year water surface elevation in the outfall channel and plot the hydraulic gradient through the system for the design storm. Include all relevant energy losses. The hydraulic gradient must not exceed the roadway gutter flow-line elevation.

Friction Losses in Storm Sewers

Friction losses in storm sewer systems shall be computed using Manning's equation.

TABLE 4.1 Values of Manning Roughness Coefficient for Corrugated Metal Pipe

Source: (HCFCD, 1984)

Corrugation (Span x Depth)	Manning "n"
2-2/3" x 1/2"	0.024
3" x 1"	0.027
5" x 1"	0.027
6" x 2"	0.030

Minor Losses in Storm Sewers

Head losses at structures such as inlets and manholes, usually termed "minor losses," shall be determined in the design of closed conduits. The design engineer should determine the relative significance of the minor losses and their applicability to the design. If they are insignificant, they may be omitted.

The equation for head loss at the entrance to a pipe is given as follows:

Equation 4.1

$$\text{Head Loss} = K\frac{V^2}{2g}$$

in which:

K = entrance loss coefficient. (See Table 4.2)

V = flow velocity in pipe (fps).

Type of Entrance	Coefficient K
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.20
Projecting from fill, sq. cut end	0.50
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.20
Square-edge	0.50
Rounded (radius = 1/12D	0.20
Mitered to conform to fill slope	0.70
Inlet or Manhole at beginning of line	1.25

TABLE 4.2 Coefficients for Entrance Losses

Sources: (FHWA, 1985), (City of Waco, undated)

The equation for the head loss (feet) at an inlet or manhole is as follows:

Equation 4.2

$$\text{Head Loss} = \frac{V_2^2 - KV_1^2}{2g}$$

in which:

V_1 = velocity in the upstream pipe (fps).

V_2 = velocity in the downstream pipe (fps).

K = junction or structure coefficient of loss. (See Table 4.3).

Type of Structure	Coefficient K
Inlet on main line	0.50
Inlet on main line with branch lateral	0.25
Manhole on main line with 22-1/2 degree lateral	0.75
Manhole on main line with 45 degree lateral	0.50
Manhole on main line with 60 degree lateral	0.35
Manhole on main line with 90 degree lateral	0.25

TABLE 4.3 Coefficients for Losses at Structures

Source: (City of Waco, undated)

Appurtenant Storm Sewer Structures

Appurtenant storm sewer structures include storm sewer manholes, inlets, and outfall.

Storm Sewer Manholes

Manholes shall be placed at the location of all pipe size or cross section changes, pipe sewer intersections, pipe sewer grade changes, street intersections, at maximum intervals of 500 feet measured along the center-line of the pipe sewer, and at all inlet lead intersections with the pipe sewer where precast concrete pipe sewers are designed.

Storm Sewer Inlets

Two types of inlets are recommended; the Type "BB" Inlet and the Type "C-1" Inlet. All inlets shall be constructed as specified in the City of Houston Form E-14-62 [City of Houston, 1980] and all subsequent revisions, or equivalent acceptable to applicable drainage regulatory entity.

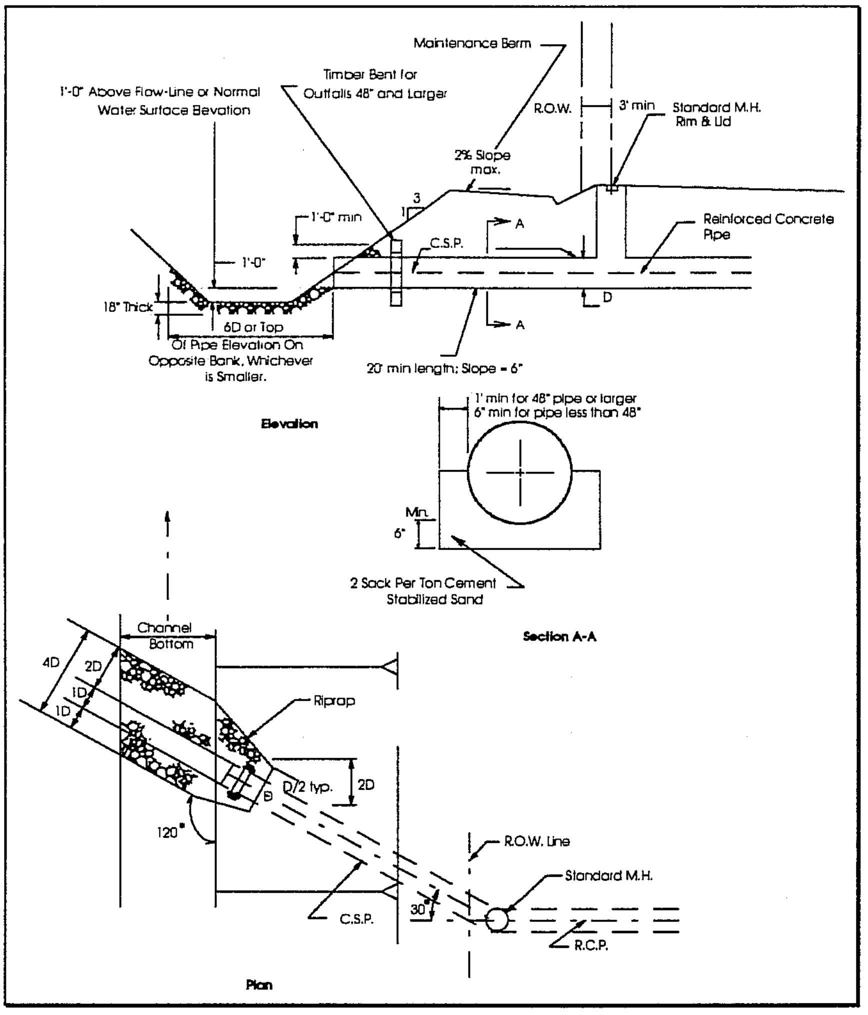
The capacity of inlets shall be determined as shown in Step 7 on page 4-14 of this manual. All inlets shall be designed to carry at least the design storm frequency runoff.

Curb inlets must be spaced to handle the design storm discharge so that the hydraulic gradient does not exceed the roadway gutter elevation. Inlets shall be spaced so that the maximum travel distance of water in the gutter will not exceed six hundred feet (600') in one direction for residential streets and three-hundred feet (300') in one direction on major thoroughfares and streets within commercial developments. Curb inlets shall be located on side streets which intersect major thoroughfares in all original designs or developments. Special conditions warranting other locations of inlets shall be determined on a case-by-case basis.

Storm Sewer Outfalls

Storm sewer outfalls shall be designed in accordance with Figure 4.1.

FIGURE 4.1 Typical Storm Sewer Outfall Structure



Secondary Drainage and Overland Flow Design

It is often infeasible in certain areas to convey the runoff from extreme rainfall events entirely via an underground storm sewer system. Local flooding will occur in areas away from the primary drainage channels because it is simply uneconomical to provide a storm sewer pipe large enough to totally carry the infrequent, severe storm events. For this reason, a sheet flow analysis is required so that street design and alignment assure that excess runoff from extreme storm events will be safely conveyed to primary drainage channels. Sheet flow corridors shall be designated and all required right-of-way dedicated to the appropriate agency. Special consideration must also be given for off-site sheet flows and their impacts on planned developments.

The discussion presented in this section will be directed primarily at curb-and-gutter streets with underground storm sewers.

Storm Frequency and Drainage System Design

Flooding in this area is generally associated with one of two types of severe rainfall events. The first type is a localized high-intensity rainfall of short duration which floods a small localized area and causes ponding of water and interruption of traffic flow. The second type is a more generalized rainfall of longer duration which can cause more widespread flooding and can result in severe damage and loss of life. This second type of storm event is generally used to design drainage channels which serve large watersheds.

In designing storm sewers for draining small developments, it is the localized high intensity, short duration rainfall event which is used. However, since these storm sewers usually drain into open channels, which are used to convey the runoff from larger areas, the design must take into consideration the interaction of these two systems.

Figures 4.2 through 4.4 illustrate the effect of three outlet conditions on the hydraulic grade line of a storm sewer. Assuming the outlet channel is at its 25-year water level, it can be seen from Figure 4.2 that the hydraulic grade line for the standard design condition remains at or below the gutter level at the furthest inlet. For this condition, there is no street ponding and the storm sewers are functioning at or below their design capacity.

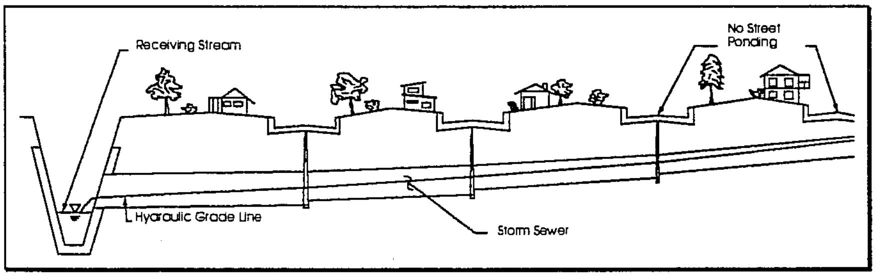


FIGURE 4.2 Effect of Low Starting Elevation on Storm Sewers

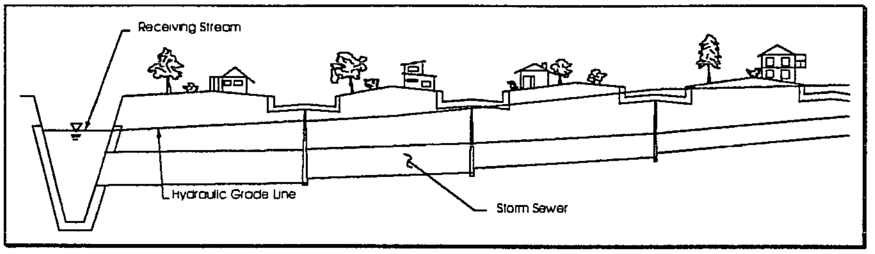


FIGURE 4.3 Effect of Medium Starting Elevation on Storm Sewers

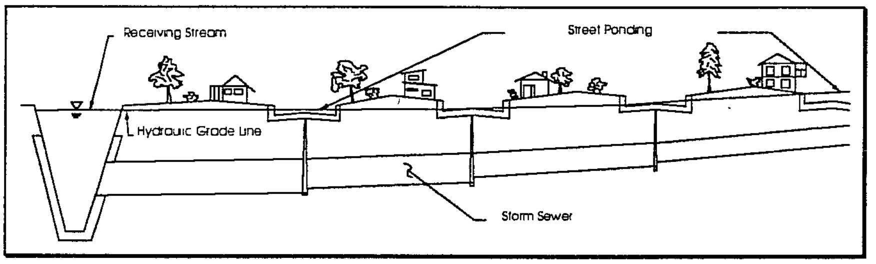


FIGURE 4.4 Effect of High Starting Elevation on Storm Sewers

Figures 4.3 and 4.4 illustrate cases where the tailwater condition is above the design level. Street ponding begins to occur throughout the storm sewer drainage system, as the storm sewers are unable to operate at their design capacity. This local flooding situation could also occur when the tailwater is below design conditions if local rainfall is in excess of that used in the design of the storm sewer system. As this widespread street ponding starts to occur, provisions must be made to limit the depth of ponding to a level below that which will cause significant property damage. In general, 100-year flood elevations shall be considered unacceptable when they exceed the lowest of the following:

1. one foot above natural ground;
2. one foot over top of curb; or
3. one foot below the lowest slab elevation.

Overland Flow

When the capacity of the underground system is exceeded and street ponding begins to occur, careful planning can reduce or eliminate the flood hazard for adjacent properties. Street layout and pavement grades are the key components in developing a successful system which can convey excess storm runoff to an outfall channel designed to carry the 100-year storm runoff.

Land Plan and Street Layout

Designing an effective overland drainage system must begin with the land plan and street layout. Awareness of overland flow problems in this early phase of the development process can reduce costly revisions and delays later on in the project. When designing drainage systems, attention should be given to special problems created by the topography. Excessive street cuts which can create ponding levels that hamper vehicle access and/or present a flood hazard must be avoided. Proper engineering foresight in the design of items such as emergency relief swales or underground systems can solve these potential problems.

The maximum allowable ponding level in a street is the lowest of the following elevations:

1. one foot above natural ground
2. one foot above top of curb
3. one foot below the lowest slab elevation.

The design engineer must determine whether the storm sewer system can convey flows from a 100-year storm event without ponding water in the street at levels that exceed the maximum allowable level. The 100-year discharge can be obtained by following the procedures outlined in Chapter 3. A 25-year tailwater condition should be assumed in the outlet channel. If storm sewer backwater calculations indicate that the allowable level is exceeded, the engineer must analyze the street system and verify that the excess flows will be able to reach the outfall channel without exceeding the maximum allowable ponding level. In making this analysis, the engineer can account for the portion of flows that would be carried by the sewer system in addition to the street system, assuming a 25-year tailwater condition.

Figure 4.5 illustrates a cul-de-sac street which slopes downhill and is designed so that sheet flow can only escape through building lots. Figure 4.6 illustrates a more acceptable alternative.

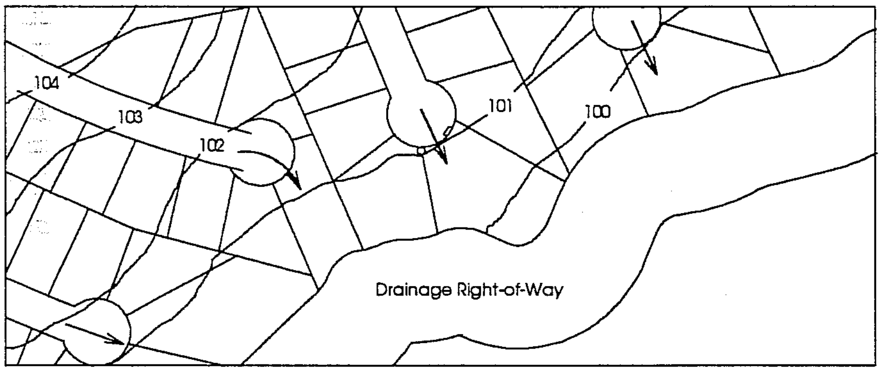


FIGURE 4.5 Undesirable Overland Flow Pattern for Cul-de-sac Street

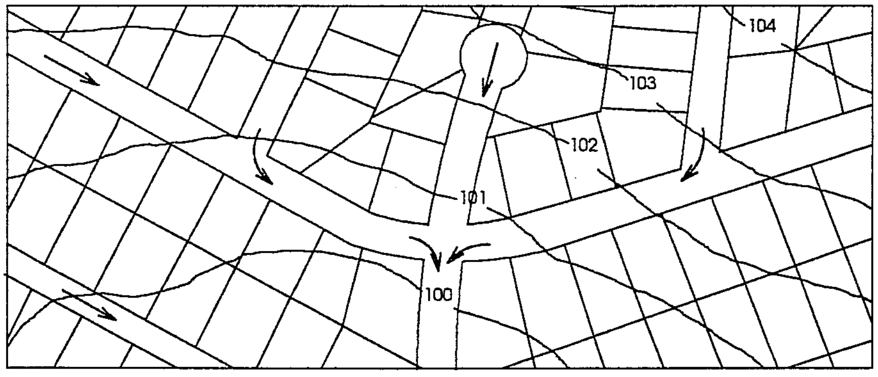


FIGURE 4.6 Acceptable Overland Flow Pattern for Cul-de-sac Street

Figure 4.7 illustrates a curve or turn in a roadway which is placed in a low area such that sheet flow entering that curve or turn can escape only through existing building lots. Figure 4.8 illustrates an acceptable alternative.

FIGURE 4.7 Undesirable Overland Flow Pattern at Roadway Curve

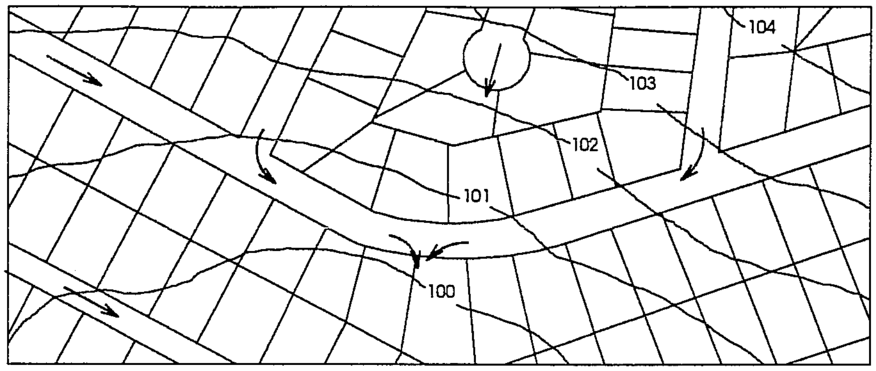


FIGURE 4.8 Acceptable Overland Flow Pattern at Roadway Curve

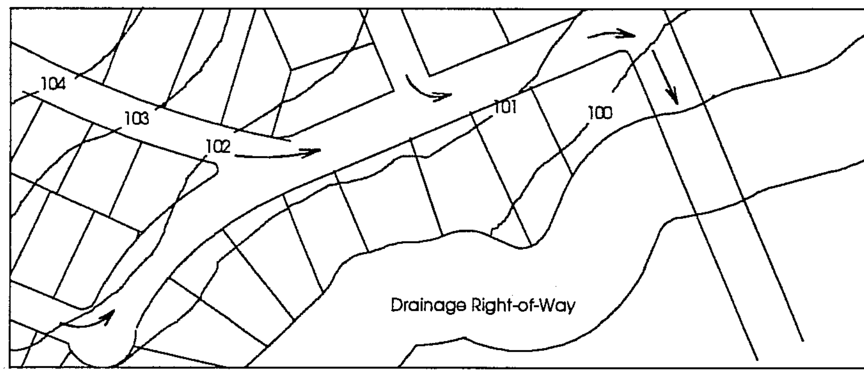


Figure 4.9 illustrates a situation in which many streets intersect a single street which is lower than the intersecting streets so that sheet flow down the streets can escape only through existing building lots. Figure 4.10 illustrates an acceptable alternative.

FIGURE 4.9 Undesirable Overland Flow at Street Intersections

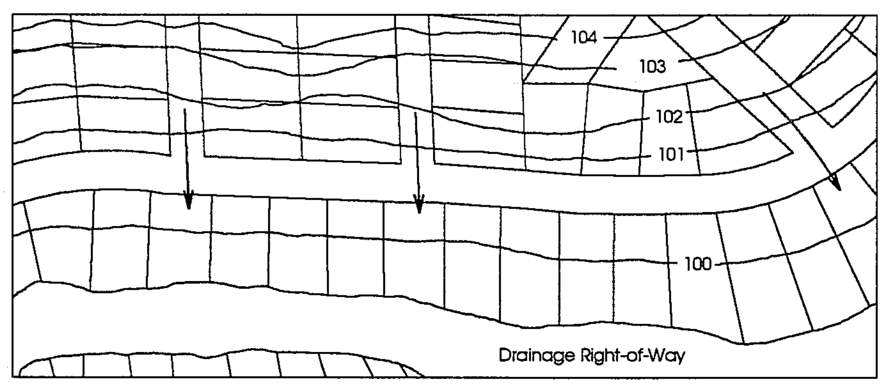
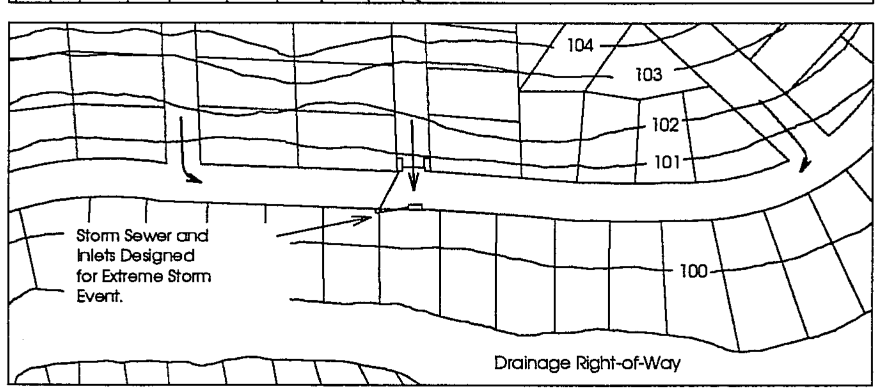


FIGURE 4.10 Acceptable Overland Flow at Street Intersections



Conveyance of Surface Flow to Primary Channels

Once it has been determined that ponding levels are excessive and where the collective sheet flow is going to go, provisions must be made to get the overflows into the appropriate drainage channel. This may be done through the use of additional pipe capacity and inlets or by using a surface swale. An underground conveyance system can be included in the storm sewer construction and maintenance program with minimal cost increase. Also, landowners are less likely to disturb an underground pipe than a surface swale. However, the surface swale will function for a wider range of flow conditions than the pipe system.

If surface swales are used, they should be concrete-lined to reduce the possibility of adjacent landowners disturbing them. The surface flow conveyance system shall be contained within an easement dedicated to the appropriate agency. The easement shall be of sufficient width to operate and maintain the system.

Since a surface swale system would act only under emergency conditions and would not function under normal circumstances, all precautions must be taken to insure that the relief system will function when needed. The recommended design procedure for sizing pipe outfall structures for sheet flow conveyance is presented beginning on page 4-12. The design procedure recommended for sizing of the surface swale is similar to the procedure for the pipe outfall. First, the appropriate values from steps one and two are computed, then the required swale cross-section is determined by normal depth calculations, sizing the swale such that an acceptable water surface is achieved.

Roadside Ditch Drainage

Under certain conditions, roadside ditch drainage is acceptable as an alternative to curb-and-gutter systems. However, a similar potential for flooding exists when flow in roadside ditches exceeds capacity. Provisions must be made to assure that the amount of water ponded behind an elevated roadway does not reach damaging levels. (See Chapter 7 for typical roadside ditch interceptor drain detail).

Preliminary approval for the use of roadside ditch systems must be obtained prior to the submittal of contour and drainage area maps, and hydrologic and hydraulic calculations.

The following requirements must also be met in the design of roadside ditch systems:

1. **Design Flow**: The design flow shall be determined based on the projected land use.
2. **Side Slope**: Minimum acceptable ditch section shall have a side slope no steeper than 3 horizontal to 1 vertical.
3. **Bottom Width**: The minimum bottom width for roadside ditches shall be two feet.
4. **Manning Coefficient**: The "n" coefficient for the ditch calculations shall be a minimum of 0.040. All values must be justified.

5. **Minimum Grade**: The minimum grade or slope of the ditches shall be 0.10%.
6. **Hydraulic Computations**: Hydraulic design computations must be submitted for each drainage ditch system.
7. **Freeboard:** The computed water surface of the ditches shall be a minimum of 0.5 foot below natural ground elevations along the street right-of-way lines.
8. **Erosion Control**: The entire ditch must be revegetated immediately after construction to minimize erosion. Erosion control methods shall be utilized where velocities of flow are calculated to be greater than five feet per second or where soil conditions dictate their need.
9. **Depth**: The minimum depth of the ditches shall be 18 inches and the maximum depth shall be 4 feet.

Extreme Event Storm Sewer Analysis

This section outlines the procedure recommended for designing an underground pipe system to convey overflows to a primary drainage channel. In subdivisions designed with curb-and-gutter streets, modification of the last storm sewer reach is generally all that is necessary to handle the overflow.

The recommended procedure is given below along with an example based on the drainage system presented in Figure 4.4.

1. **Peak Flow Rate**: Determine the 100-year peak flow at the point of concentration from all existing and future contributing drainage areas for 100% development conditions. In the example, the contributing drainage area is 40 acres and the 100-year discharge is 147 cfs.
2. **Starting Elevation**: Determine the 25-year frequency water-surface elevation in the drainage channel at the pipe outfall point. Based on a 25-year backwater profile, the water surface elevation in the channel for the example is 97.0 feet.
3. **Compute Available Head**: Determine the maximum energy head, *H*, available between the outfall point and ponding area by subtracting the maximum allowable ponding elevation in the ponding area from the channel's 25-year water surface elevation. With a slab elevation of 101.5 feet and a top of curb and natural ground elevation of 100.0 feet in this example, the maximum allowable ponding elevation is the lowest of the following:

1. one foot over natural ground;
2. one foot over the top of curb; or
3. one foot below the lowest floor elevation.

In this case, the maximum elevation is controlled by the lowest floor elevation and is 100.5 feet. There are 3.5 feet of head available (*H*).

4. **Compute Pipe Loss**: Establish a size of the storm sewer pipe and compute the head loss using the following equation:

Equation 4.3

$$H_P = 4.66\frac{Q^2 n^2 L}{D^{16/3}}$$

in which:

H_p = head loss in feet

Q = 100-year discharge in cubic feet per second

n = Manning's "n" value

D = diameter of pipe in feet

L = length of pipe in feet

For this example, 65 linear feet of 60-inch corrugated metal pipe (CMP) with a Manning's n value of 0.024 and 120 linear feet of 60-inch reinforced concrete pipe (RCP) with a Manning's "n" value of 0.013 is selected. The head loss is as follows:

$$HL_P = 4.66\frac{Q^2\left(n_{CMP}^2 L_{CMP} + n_{RCP}^2 L_{RCP}\right)}{D^{16/3}}$$

$$= 4.66\frac{(147)^2\left[(0.024)^2(65)+(0.013)^2(120)\right]}{5^{16/3}} = 1.09 \text{ ft}$$

5. **Compute Lead Head Loss**: Compute the head loss through the leads, h_1, using Equation 4.3. Experience has shown that 24-inch diameter leads generally cause excessive head loss. 30-inch diameter leads are satisfactory in most cases, while 36-inch leads are too large for the most common street inlets type "B-B" and "C-1." Therefore, the 30-inch diameter is selected.

   Estimate the percentage of 100-year runoff flowing through each lead. Assume the 147 cfs to be divided between three leads as follows:

   Lead 1: 20-foot lead with a flow of 56 cfs.

   Lead 2: 20-foot lead with a flow of 56 cfs.

   Lead 3: 45-foot lead with a flow of 37 cfs.

$$HL_1 = 4.66\frac{Q^2 n^2 L}{D^{16/3}} = 4.66\frac{(56)^2(0.013)^2(20)}{(2.5)^{16/3}} = 0.37 \text{ ft}$$

$$HL_2 = HL_1$$

$$HL_3 = 4.66\frac{(37)^2(0.013)^2(45)}{(2.5)^{16/3}} = 0.37 \text{ ft}$$

6. **Compute Inlet Head**: Determine the energy head available at each inlet using the equation:

Equation 4.4

$$H_i = H - H_P - HL$$

If H_i is negative, the hydraulic grade line is above the maximum ponding elevation. Increase the capacity of the system and repeat steps 4, 5, and 6. If H_i is positive, check the elevation of the hydraulic grade line relative to the maximum ponding elevation. For grade lines above the gutter line, use H_i as the energy head on the inlet; otherwise, make the value of H_i equal to the maximum ponding elevation minus the gutter elevation. For this example, assume the hydraulic grade line is above the gutter elevation. Since the head loss through the three leads in the example are similar, the available head at each inlet is:

$$H_i = 3.5 - 1.1 - 0.37 = 2.03$$

7. **Determine Inlet Type**: Determine the type of inlets required to handle the portion of the 100-year flow reaching the ponding area. The flow through the inlet(s) must be equal to or greater than the flows estimated in Step 5 for each lead. Use the following orifice equation to compute the flow into each inlet.

Equation 4.5

$$Q = CA\sqrt{2gH_i}$$

in which:

Q = discharge in cubic feet per second.

C = orifice coefficient (0.8 for inlets).

A = area of inlet opening. (Type "B-B" 2.14 square feet and Type "C-1" 6.50 square feet.)

g = acceleration of gravity (32.2 ft/sec^2)

H_i = as defined in Step 6.

Type "C-1" inlets are selected for Inlet 1 and Inlet 2 and Type "B-B" inlets are selected for Inlet 3 across the street.

$$Q_{C-1} = 0.8(6.50)\sqrt{(64.4)(2.0)} = 59 \text{ cfs}$$

$$2Q_{B-B} = 2(0.8)(2.14)\sqrt{(64.4)(2.0)} = 38 \text{ cfs}$$

Thus, a Type "C-1" inlet at Inlet 1, a Type "C-l" inlet at Inlet 2, and two Type "B-B" inlets at Inlet 3 will convey the 100-year sheet flow to the channel with the energy head available. If this inlet choice is adequate, the design is complete.

8. **Repeat Analysis if Necessary**: Repeat Steps 4 through 7 until the combination of storm sewer pipe, leads, and inlets adequately conveys the 100-year sheet flow to the channel with the energy head available, and is the most economical.

Off-Site Overland Flow

Sheet flow from undeveloped areas into an existing or a proposed subdivision can create a localized flood hazard by overloading street inlets and/or flooding individual lots. Any drainage plan for a proposed subdivision must address the drainage of all adjacent lands, both under undeveloped and fully developed conditions. A plan which may be adequate under conditions of ultimate development can be severely deficient during intermediate conditions of development due to sheet flow from adjacent undeveloped land. Provisions must be made to divert 100-year sheet flows to a channel system or to the secondary street and storm sewer system.

Redirection of the sheet flow can usually be achieved through the use of drainage swales located in temporary drainage easements along the periphery of the subdivision. As the adjacent area develops to the point at which the street system can effectively handle the sheet flow condition, the temporary drainage swales and easements may be abandoned. The drainage swales should be relatively shallow, with the excavation spoiled continuously along the subdivision side of the swale to prevent flow from overrunning the swale. The swale should have sufficient grade to avoid standing water, but not enough to create erosion problems. Generally, a minimum grade of 0.10% should be maintained with the maximum grade strongly dependent on local soil conditions.

Such temporary drainage swales may be directed to inlets in the storm sewer system or, preferably, to the appropriate primary outfall channel. If an undeveloped area is to be drained to a storm sewer, additional inlet and storm sewer capacity must be provided to prevent prolonged street ponding in the subdivision resulting from flow from the undeveloped area. Provisions for this flow must also be included in the design of the street drainage overflow system. The design of temporary drainage swales directed to drainage channels must include adequate provisions to drop the flow into the channel through an approved structure in order to avoid excessive erosion of the channel banks.

Outfalling the temporary swale into the backslope drainage system for the channel is unacceptable because the backslope drainage interceptor structures are not adequate to convey flow from an off-site swale. A typical approved structure is shown in Chapter 7, with the exception of the pipe dimension. The pipe must be sized to handle the 100-year flow from the off-site area.

Open Channel Design Criteria

The proper hydraulic design of a channel is of primary importance in insuring that flooding, sedimentation and erosion problems do not occur. This section summarizes the practical considerations, technical principles, and criteria necessary for proper design of open channels. The analysis of open channel flow also aids in determining other flow-related concerns, such as culvert tailwater depths, time of concentration calculations (travel times), and flood elevations.

In a major drainage system, open channels offer significant advantages over closed conduits in regard to cost, flow capacity, flood storage, recreation, and aesthetics. However, open channels require considerable right-of-way and maintenance. Careful consideration must be given in the design process to insure that disadvantages are minimized and benefits are maximized. When a design approach not covered in this manual is to be used, it should be reviewed and discussed with the appropriate drainage regulatory agency prior to commencing significant portions of the design effort.

Channel Location and Alignment

Due consideration must be given to the maintainability of man-made channels. Channel designs which incorporate measures that may hinder the efforts of maintenance personnel should be avoided. Sufficient right-of-way should be provided to allow easy access by maintenance equipment.

Typical Channel Cross-Sections

Figure 4.11 illustrates a cross-section of a typical grass-lined channel. The following are minimum requirements to be used in the design of all grass-lined channels:

Grass-Lined Trapezoidal Channels.

1. **Channel Side Slopes**: In grass-lined channels, the normal maximum side slope will be 3 horizontal to 1 vertical (3:1), which is the practical limit for mowing equipment. In some areas, local soil conditions may dictate the use of side slopes flatter than 3:1 to ensure slope stability.
2. **Channel Bottom Width**: Minimum bottom width is six (6) feet.
3. **Channel Right-of-Way**: A minimum maintenance berm is required on both sides of the channel. The width of the berms shall be between 15 and 30 feet depending upon channel size. For channel top widths of 30 feet or less, 15-foot berms are acceptable; for top widths between 30 and 60 feet, 20-foot berms are required; and for top widths of 60 feet or greater, 30-foot berms are required along both sides of the channel. The elevation of the top of the berm should be at natural ground along the channel reach. See Chapter 6.
4. **Channel Backslope Drains**: Backslope interceptor structures are necessary at a maximum spacing of 800 feet to prevent sheet flow over the ditch side slopes. See Chapter 7.
5. **Channel Erosion Control**: Channel slopes must be revegetated immediately after construction to minimize erosion. See Chapter 7 of this manual.
6. **Ditch Interceptor Structures**: Flow from roadside ditches must be conveyed to the channel through a roadside ditch interceptor structure and pipe. See the ditch interceptor structure and pipe detail in Chapter 7 of this manual.
7. **Geotechnical Report**: Unless waived by the applicable drainage regulatory authority, a geotechnical investigation and report must be provided.

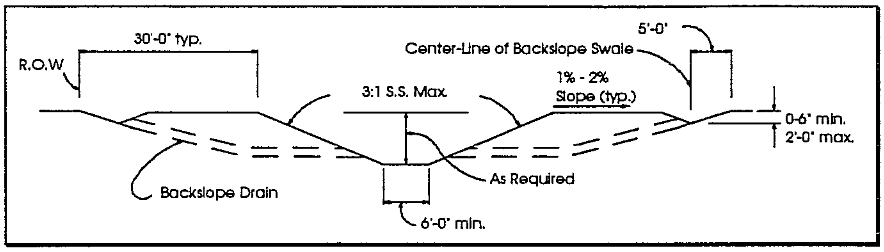


FIGURE 4.11 Typical Section - Grass-Lined Trapezoidal Channel

Concrete-Lined Trapezoidal Channels

Figure 4.12 illustrates a cross-section of a typical concrete-lined channel. All partially or fully concrete-lined trapezoidal channels must meet or exceed the following minimum design requirements:

1. **Class A Concrete**: All concrete shall be Class A concrete unless noted otherwise.
2. **Channel Bottom Width**: Fully lined cross-sections shall have a minimum bottom width of eight (8) feet.
3. **Channel Right-of-Way**: A minimum maintenance berm is required on both sides of the channel. The width of the berm will be 20 feet on one side of the channel and 10 feet on the other side. The elevation of the top of the berm should be at natural ground along the channel reach. See Chapter 6.
4. **Reinforcement**: Concrete slope protection shall have the minimum thickness and reinforcement indicated in Table 4.4. Cast-in-place concrete side slopes should not be steeper than 1.5:1.
5. **Toe Walls**: All slope paving shall include a minimum 18-inch toe wall at the top and sides and a 24-inch toe wall across or along the channel bottom for clay soils. In sandy soils, a 36-inch toe wall is required across the channel bottom.
6. **Backslope Drains**: In instances where the channel is fully lined, backslope drainage structures may not be required. Partially lined channels will require backslope drainage structures.
7. **Weep Holes**: Weep holes shall be used to relieve hydrostatic head behind lined channel sections. The specific type, spacing and construction method for the weep holes will be based on the recommendations of the geotechnical report.
8. **Seal Slab**: Where construction is to take place under muddy conditions or where standing water is present, a seal slab of Class C concrete shall be placed in the channel bottom prior to the placement of concrete slope paving.
9. **Control Joints**: Control joints shall be provided at a spacing of approximately twenty-five feet on center. The use of a sealing agent shall be utilized to prevent moisture infiltration.

In most cases, a minimum 20-foot rip-rap protection blanket located on the downstream side of the paving will be necessary to protect concrete toe walls.

FIGURE 4.12 Typical Section - Concrete-Lined Trapezoidal Channel

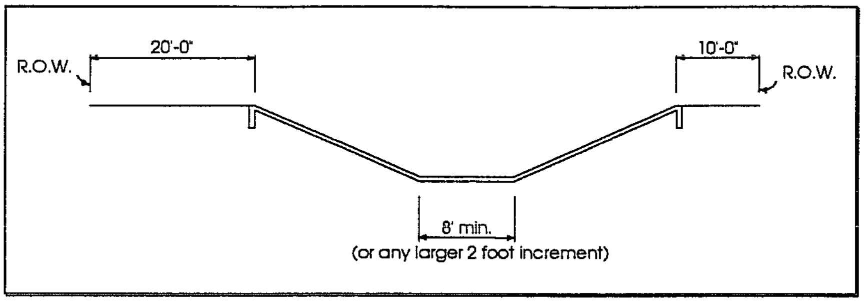


TABLE 4.4 Minimum Thickness and Reinforcement for Concrete Slope Paving

Note: Reinforcement equivalent to the stated minimum will be acceptable.

Channel Side Slopes (H:V	Minimum Concrete Thickness	Minimum Reinforcement
3:1	4 inches	6 x 6 x W2.9 x W2.9 welded wire fabric
2:1	5 inches	6 x 6 x W4.0 x W4.0 welded wire fabric
1.5:1	6 inches	4 x 4 x W4.0 x W4.0 reinforcement

Rectangular Concrete Pilot Channels (Low Flow Sections)

Figure 4.13 illustrates a cross-section of a typical concrete-lined channel with a rectangular low flow section. For purposes of illustration only, the channel in Figure 4.13 has one concrete-lined slope and one grass-lined slope. Normally, both slopes would be either concrete-lined or grass-lined. In areas where it is necessary to use a vertical-walled rectangular section, the following minimum requirements are to be addressed:

1. **Class A Concrete**: All concrete shall be Class A concrete unless noted otherwise.
2. **Reinforcement**: The structural steel design should be based on ASTM A-615, Grade 60 steel.
3. **Channel Bottom Width**: Minimum bottom width shall be eight (8) feet.
4. **Channel Bottom Slope**: For bottom widths twelve (12) feet or greater, the channel bottom shall be graded toward the channel center line at a slope of 1/2 inch per foot.
5. **Vertical Wall Height:** Minimum height of vertical walls shall be four (4) feet. Heights above this shall be in two (2) foot increments. Exceptions shall be on a case by case basis.
6. **Escape Stairways:** Escape stairways shall be located at the upstream side of all street crossings, but not to exceed 1,400 feet intervals.
7. **Future Slope Paving**: For rectangular concrete pilot channels with grass side slopes, the top of the vertical wall should be constructed to allow for future placement of concrete slope paving.
8. **Weep Holes**: Weep holes should be used to relieve hydrostatic pressures. The specific type, spacing and construction method for the weep holes will be based on the recommendations of the geotechnical report.
9. **Seal Slab**: Where construction is to take place under muddy conditions or where standing water is present, a seal slab of

Class C concrete should be placed in the channel bottom prior to the placement of concrete slope paving.

10. **Maintenance Shelves**: Concrete pilot channels may be used in combination with slope paving or a maintenance shelf, as illustrated in Figure 4.14 Horizontal paving sections should be analyzed as one-way paving capable of supporting maintenance equipment having a concentrated wheel load of up to 1,350 lbs.
11. **Control Joints**: Control joints shall be provided at a spacing of approximately twenty-five feet on center. The use of a sealing agent shall be utilized to prevent moisture infiltration.
12. **Structural Calculations**: Structural calculations shall be provided for all concrete pilot channels.

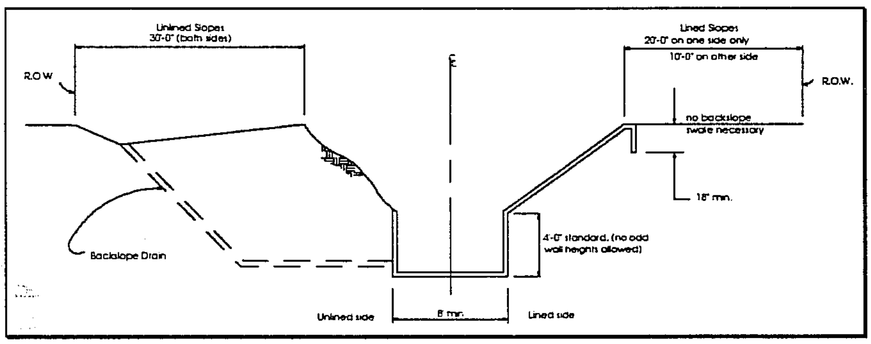


FIGURE 4.13 Typical Section - Concrete-Lined Low Flow Channel

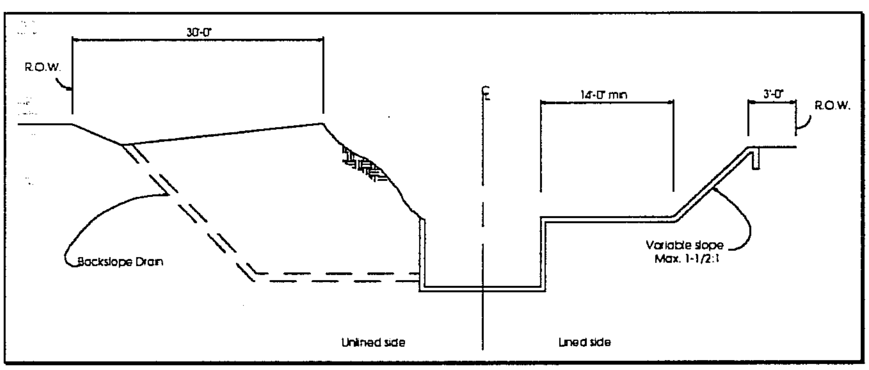


FIGURE 4.14 Concrete-Lined Low Flow Channel with Maintenance Shelf

Riprap-Lined Trapezoidal Channels

Rip-rap is broken concrete rubble or well-rounded stone. The use of rip-rap is encouraged because of its proven past performance, its flexibility, and its high Manning's "n" value (approximately 0.04), which reduces channel velocities. A discussion of rip-rap design can be found in Corps EM 1110-2-1601 [USACE, 1970].

Minimum requirements and criteria for rip-rap installation are as follows:

1. **Mat Thickness**: Minimum mat thickness is 18 inches. At the edges of the rip-rap, particularly in the channel bottom and partially up the sides, a thickened section is sometimes needed to reduce the chance of failure due to channel bottom degradation downstream of the rip-rap.

2. **Block Description**: Use evenly graded, 40-pound to 265-pound blocks. A wide gradation range is necessary to heop heold the rip-rapo in place. Minimum 6-inch thickness per block. No exposed steel in broken concrete rubble. For installations in which severe hydraulic conditions are expected, 40-640 pound blocks are required. Sacks of ready-mix concrete alone are not acceptable for use as rip-rap because of the lack of gradation allows water penetration and soil mining under the rip-rap installation.
3. **Side Slope**: Maximum steepness of the side slope is 2 (horizontal) to 1 (vertical).
4. **Filter Fabric Bedding**: Filter fabric is required where rip-rap is placed on a soil with sand or silt. On clay soils with very little sand content, filter fabric is not required.

Manning n Values for Improved Channels

The values of the Manning roughness coefficient listed in Table 4.5 should be used in the design of channel improvements.

TABLE 4.5 Manning Roughness Coefficient for Improved Channels

Channel Cover	"n" value
Grass-lined	0.040
Concrete-lined	0.015
Rip-rap-lined	0.040

Allowable Velocities for Channels

Excessive flow velocities in open channels can cause erosion and destabilize side slopes, and may pose a threat to safety. Velocities which are too low may allow the deposition of sediment and channel clogging. Table 4.6 provides desirable average and maximum allowable velocities based on 25-year flow rates.

TABLE 4.6 Allowable 25-Year Flow Velocities for Channel Design

Source: (HCFCD, 1984)

Channel Description	Average Velocity (fps)	Maximum Velocity (fps)
Grass Lined: Predominantly Clay Soil	3.0	5.0
Grass Lined: Predominantly Sand Soil	2.0	4.0
Rip-rap Lined	5.0	8.0
Concrete Lined	6.0	10.0

In areas of the channel where the maximum velocities given in Table 4.6 are exceeded, or where determined by minimum erosion protection requirements, a structural erosion protection such as cellular concrete articulated mats, concrete slope paving, rip-rap, revetment mats, gabions, etc. must be installed. The slope protection must at least extend up the banks to the 25-year flood level.

Channel Flowline Slope

Slope of the channel flow-line (invert) is generally governed by topography and the energy head required for flow. Since flow-line slope directly affects channel velocities, channels should have sufficient grade to prevent significant siltation. However, slopes should not be so large as to create erosion problems. The minimum recommended channel flow-line slope is 0.05 percent. The maximum channel invert slope will be limited by the maximum flow velocities given in Table 4.6. Appropriate channel drop structures may be used to limit channel invert slopes in steep areas.

Design of Channel Confluences and Splits

The angle of intersection between tributary and main channels should be between 15 degrees and 45 degrees. Angles in excess of 45 degrees are permissible but are discouraged. Angles in excess of 90 degrees are not permitted.

See Chapter 7 for erosion control requirements at channel confluences.

Design of Channel Transitions

Expansions and contractions should be designed to create minimal flow disturbance and thus minimal energy loss. Transition angles should be less than 12 degrees, or about five units parallel to the channel center-line to 1 unit perpendicular to the invert (5:1). When connecting rectangular to trapezoidal channels, a warped or wedge-type transition is recommended.

Design of Channel Bends

In general, center-line curves should be as gradual as possible and not have a radius of less than three times the design top width. A smaller radius may be used where erosion protection is provided, but the radius may not be less than 100 feet. The maximum deflection angle for any curve in a man-made channel should be 90 degrees.

Design of Channel Crossings

For small drainage areas, the most economical means of moving open channel flow beneath a road or railroad is generally with culverts. Discussion in this section will address procedures for determining the most cost effective culvert size and shape given a design discharge and allowable headwater elevation.

Hydraulic Design Criteria for Bridges

This section includes a discussion of the hydraulic and hydrologic considerations pertinent to bridge design. This section considers all designs to be completed for ultimate development. Where appropriate, the actual construction of a crossing may be phased as development occurs. In this case, both the ultimate and the interim crossings must be shown on the construction plans. Calculations for each must be submitted for approval. The ultimate right-of-way is required even for an interim phase of construction.

The following basic procedure should be followed in the hydraulic design of bridge structures:

1. **Right-of-Way**: Determine the ultimate right-of-way width and the dimensions of the required ultimate channel cross-section at the crossing location.
2. **Water Surface Elevations**: Determine existing and ultimate 100-year water surface elevations at the proposed crossing location.
3. **Bridge Elevation**: Establish the minimum low chord elevation of the bridge as the higher of: a) at least one foot above the existing 100-year flood elevation, b) at least one foot above the ultimate 100-year flood elevation, or c) at or above the level of natural ground.
4. **Bridge Length**: Establish the total length of the bridge to allow the accommodation of the ultimate channel section with a minimum of structural modification.

5. **Bridge Piers**: Locate the bridge pier bents in such a way as to keep piers as far away from the channel center-line as possible or, if possible, to eliminate them entirely from the channel bottom. Due consideration should be given to the existing as well as the ultimate channel sections when locating the pier bents.
6. **Effects of Bridge**: Use the HEC-2 computer program, to determine the effect of the bridge structure on existing and ultimate 100-year flood elevations upstream of the crossing.
7. **Erosion Protection**: Use the results of the HEC-2 or alternative hydraulic analysis to determine existing and ultimate flow velocities through the bridge opening. Determine the extent of slope protection required to prevent erosion damage in the vicinity of the bridge.

Design Storm Frequency

At a minimum, bridges must be designed to pass the fully developed 100-year design flow without causing backwater problems, structural damage, or erosion. No increase in 100-year water surface elevations will be allowed either upstream or downstream of the bridge unless authorization is given by the appropriate drainage regulatory authority.

Bridge Alignment

Wherever possible, bridges shall intersect the channel at an angle of 90 degrees.

Bridge Length

Newly constructed bridges must be designed to completely span the existing or proposed channel so that the channel will pass under the bridge without modification. Bridges and bents constructed on existing or interim channels shall be designed to accommodate the ultimate channel section with a minimum of structural modification. Energy losses due to flow transitions shall be minimized. In addition, provision must be made for future channel enlargements should they become necessary.

Piers and Abutments

Bents and abutments must be aligned parallel to the longitudinal axis of the channel so as to minimize obstruction of the flow. Bents shall be placed as far away from the channel center-line as possible and if possible should be eliminated entirely from the channel bottom.

Minimum Low Chord Elevation

The low chord of all bridges must be located at least one foot above the 100-year flood elevation, or at or above the level of natural ground, whichever is higher.

Erosion Control

Increased turbulence and velocities associated with flow in the vicinity of bridges requires the use of erosion protection in affected areas. Chapter 6 of this manual contains information concerning erosion protection requirements for open channels.

Hydraulic Design Criteria for Culverts

Design Storm Frequency

At a minimum, culverts must be designed to pass the fully developed 100-year design flow without causing backwater problems, structural damage, or erosion. No increase in 100-year water surface elevations will be allowed either upstream or downstream of the culvert unless authorization is given by the appropriate drainage regulatory authority.

Culvert Alignment

Culverts shall be aligned parallel to the longitudinal axis of the channel to insure maximum hydraulic efficiency and minimum erosion. In areas where a change in alignment is necessary, the change shall be accomplished upstream of the culvert crossing in the open channel. Appropriate erosion protection shall be provided.

Culvert Length

Culverts shall be designed to completely span the road or railroad right-of-way.

Culvert Headwalls and Endwalls

Headwalls and endwalls shall be utilized to control erosion and scour, to anchor the culvert against lateral pressures, and to insure bank stability. All headwalls shall be constructed of reinforced concrete and may be either straight and parallel to the channel, flared, or warped, with or without aprons, as required by site and hydraulic conditions. Protective guardrails should be included along culvert headwalls.

In general, the following conditions are favorable for the use of ***parallel*** headwalls and endwalls [City of Austin, undated]:

1. Approach velocities are less than 6 fps.
2. Backwater pools may be permitted.
3. Approach channel is undefined.
4. Ample right-of-way or easement is available.
5. Downstream channel protection is not required.

The wings of ***flared*** headwalls and endwalls should be located with respect to the direction of the approaching flow instead of the culvert axis. The following conditions are favorable for the use of a flared headwall and endwall:

1. Channel is well defined.
2. Approach velocities are greater than 6 fps.
3. Medium amounts of debris exist.

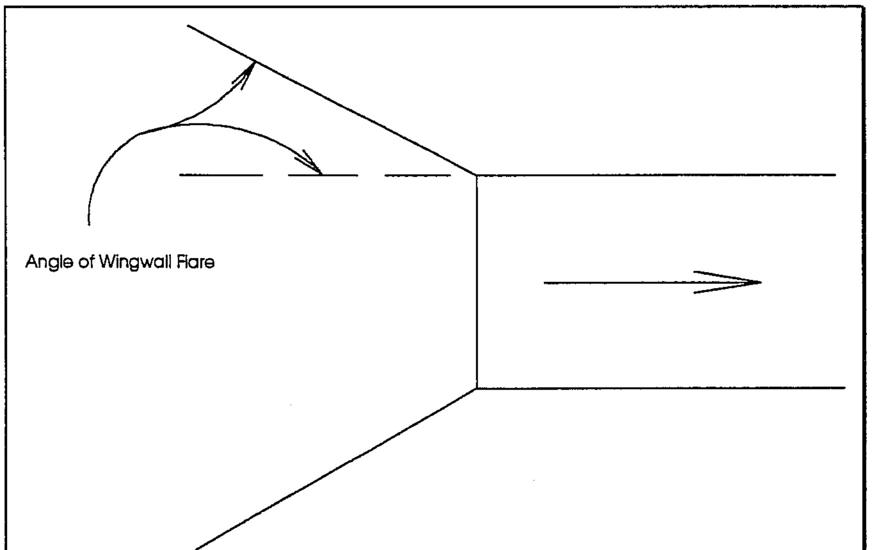


FIGURE 4.15 Flared Wingwalls

Warped headwalls are effective with drop-down aprons to accelerate flow through culvert, and are effective headwalls for transitioning flow from closed conduit flow to open channel flow. This type of headwall should be used only where the drainage structure is large and right-of-way or easement is limited. The following conditions are favorable for the use of a warped headwall and endwall:

1. Channel is well defined and concrete lined.
2. Approach velocities are greater than 8 fps.
3. Medium amounts of debris exist.

Minimum Culvert Sizes

The minimum pipe culvert diameter shall be 24 inches and the minimum box culvert dimensions shall be 2 feet by 2 feet. These restrictions are made to guard against flow obstruction. Sizes less than these shall be considered on a case by case basis.

Manning's "n" Values for Culverts

The minimum Manning's "n" value to be used in concrete culverts shall be 0.013. For corrugated metal, the "n" value shall be as follows:

TABLE 4.7 Manning Roughness Coefficient for Corrugated Metal Pipe "n"

Corrugation (Span x Depth)	Manning "n"
2-2/3" x 1/2"	0.024
3" x 1"	0.027
5" x 1"	0.027
6" x 2"	0.030

Erosion at Culverts

Culverts, because of their hydraulic characteristics, generally increase the velocity of flow over that found in the natural channel. For this reason, the tendency for erosion, especially at the outlet, must be addressed. In general, culvert discharge velocities in unprotected channels should not exceed allowable channel velocities as defined in Table 4.6. Chapter 7 contains information concerning erosion protection requirements for open channels.

Structural Requirements for Culverts

The following structural requirements shall be met for culvert design:

1. **Concrete Pipe Culverts**: All precast reinforced concrete pipe shall be ASTM C-76 (minimum).
2. **Box Culverts**: All precast reinforced concrete box culverts with more than two feet of earth cover shall be ASTM C789-79. All precast reinforced concrete box culverts with less than two feet of cover shall be ASTM 850-79.
3. **Corrugated Metal Culverts**: All corrugated metal pipes shall be ASTM A-760.
4. **Loading**: ASSHTO HS20-44 loading should be used for all culverts.
5. **Guardrails**: Guardrails are suggested at all roadway culvert crossings. The approach ends of the guardrail shall be flared away from the roadway and properly anchored.

6. **Joint Sealant**: Joint sealing material for precast concrete culverts shall comply with the "AASHTO Designation M-198 74 I, Type B, Flexible Plastic Gasket (Bitumen)" specification.
7. **Backfill**: Two-sack-per-ton cement-stabilized sand shall be used for backfill around culverts.
8. **Bedding**: A 6-inch bedding of two-sack-per-ton cement-stabilized sand is required for all precast concrete box culverts.

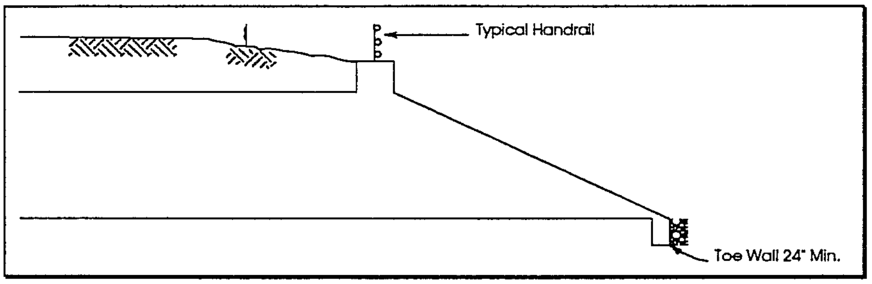


FIGURE 4.16 Profile of Typical Culvert

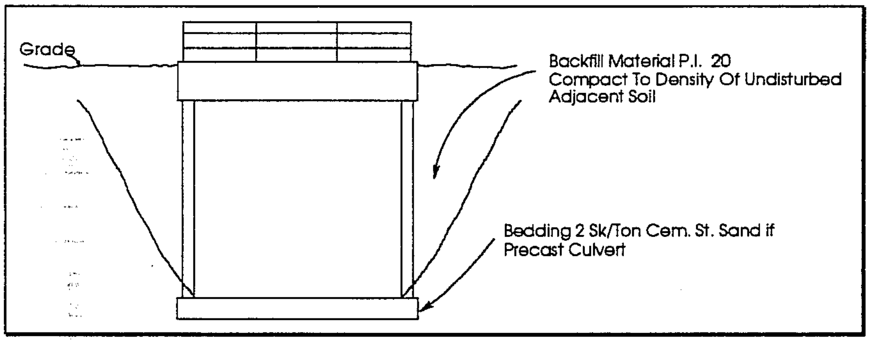


FIGURE 4.17 Section of Typical Culvert

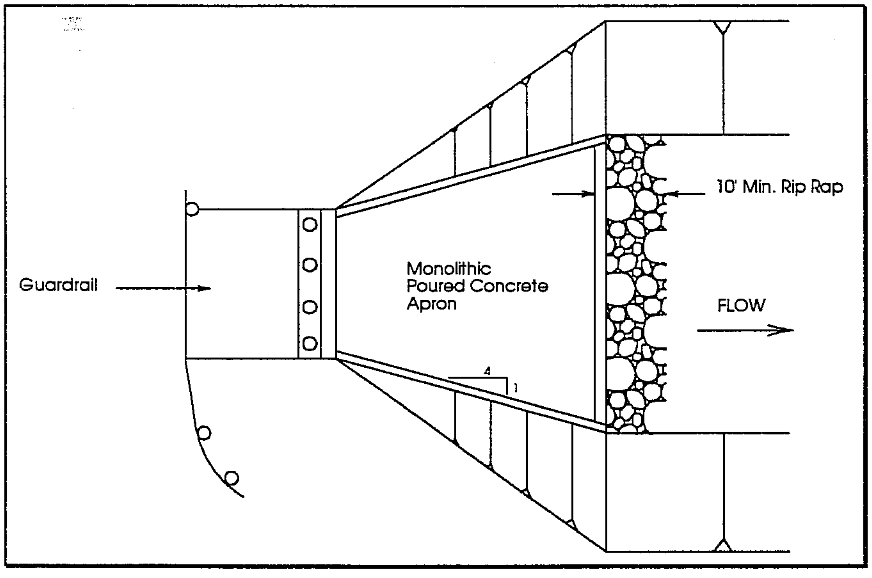


FIGURE 4.18 Typical Culvert Outlet

Pipeline and Utility Crossings

Design requirements and considerations for pipeline and utility crossings are included here.

These requirements include minimum depth of cover under streams, minimum low chord elevation, span considerations, compatibility with ultimate channel design, and others.

Guidelines are presented for the relocation of utility lines upon widening or deepening of drainage facilities.

Open Channel Analysis

Several methods exist which can be used to compute water-surface profiles in open channels. The methodology selected depends on the complexity of the hydraulic design and the level of accuracy desired.

For an existing or proposed channel with flow confined to uniform cross-sections, a hand-calculated normal depth or direct step computation is sufficient. Manning's equation should be used for computing normal depths. For evaluating non-uniform channels for existing conditions or designing a proposed channel with flow in the overbanks, the standard step method is recommended.

At least three computer programs which make use of the standard step method are available: the HEC-2 program developed by the U.S. Army Corps of Engineers, the WSP-2 program developed by the Soil Conservation Service, and the WSPRO (HY-7) computer program developed by the Federal Highway Administration. The use of HEC-2 is encouraged because it is widely accepted and it offers flexibility in designing channels.

Channel Cross Section Data

Channel cross-sections should be oriented from left to right, with these directions determined while looking downstream. Each segment of the cross-section should generally be aligned so that it is perpendicular to the direction of flow across that segment. The end points of each cross-section should be higher than the computed energy grade line elevation.

Channel stationing should begin with 0+00 at the downstream end of the channel (usually at the flow-line of the stream into which the channel being surveyed empties) and increase in the upstream direction. Stationing should be measured along the flow-line of the channel. Channel cross-sections should be identified by the channel station at which the cross-section intersects the channel center-line.

The spacing of channel cross-sections is very important, as it can have a significant impact on computations of water surface elevations. In general, the maximum distance between cross-sections should be 500 feet for unimproved channels and 2,000 feet for improved, regular channels. These distances should be measured along the center-line of the stream. Additional cross-sections should be inserted wherever discontinuities or irregularities are encountered. These include transitions, curves and bends, drop structures, bridges, and culvert crossings.

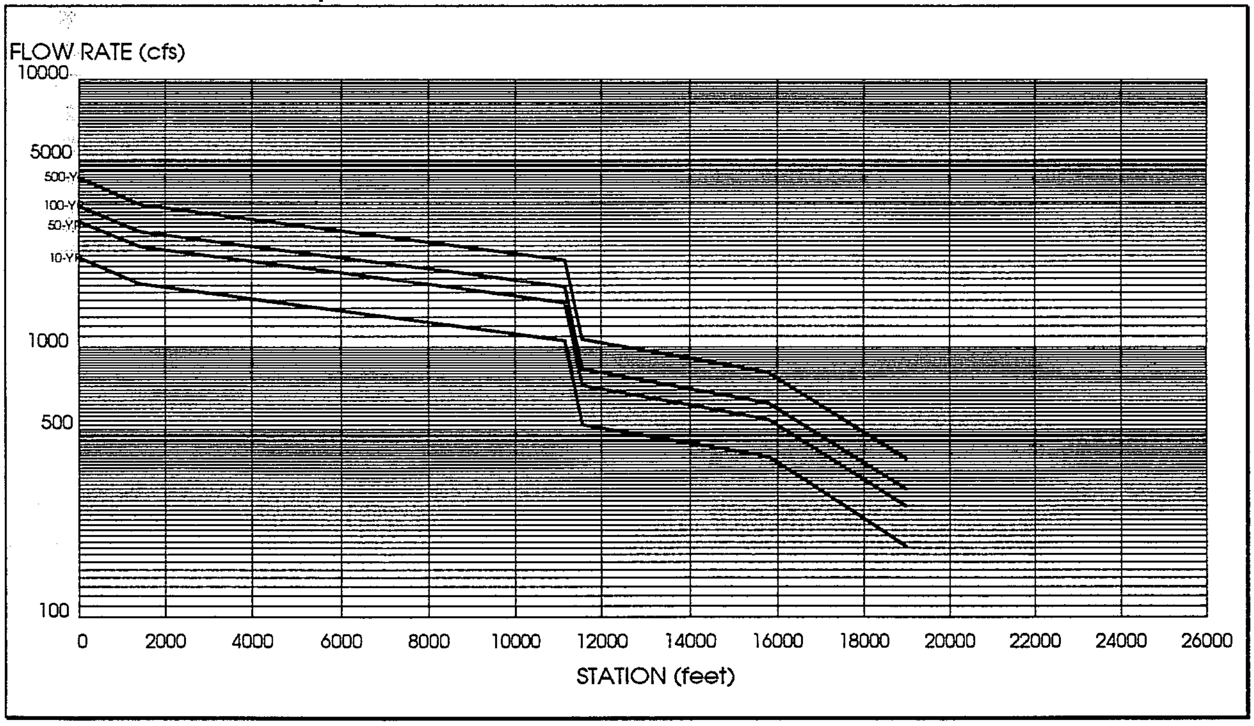
A minimum of five (5) points is usually required for the channel portion of a surveyed cross-section. This includes one point at the top of each channel bank, one point at the toe of each side slope, and one point at the channel flow-line. Additional points may be required when discontinuities in channel cross-sections are encountered. Conversely, there are some situations in which fewer points are required. The number of cross-section points required for overbank areas is dependent on the width of the cross-section and on the character of the terrain in the overbank. As a general rule, enough points should be obtained to give a true representation of the channel shape and overbank terrain, and to define any breaks or discontinuities in topography which may exist.

Channel Flow Rates

Peak flow rates are generally computed at a number of points along major stream channels using the HEC-1 model of the watershed. Peak flow rates for the 500-year storm event are determined at each HEC-1 analysis point by plotting the computed 10-year, 50-year, and 100-year peak flow rates on log-probability graph paper. A straight line is fit to the three points plotted for each HEC-1 analysis point and extended to the 500-year frequency to determine the corresponding peak flow rate.

The computed 10-year, 50-year, 100-year, and 500-year peak flow rates for each stream are then plotted versus stream station on semi-logarithmic graph paper, as illustrated in Figure 4.19. Consecutive flow rates for each frequency are connected with straight lines. This graph is then used to determine the peak flow rates at desired locations along the stream.

FIGURE 4.19 Plot of Computed Flow Rates Versus Stream Station



Peak flow rates for the 10-year, 50-year, and 100-year storm event are generally computed by the HEC-1 watershed model at the mouth of each of the major tributary streams. Log-probability paper may be used to determine 500-year flow rates at the mouth of each stream, as described previously. However, peak flow rates may be required at several points along each of these tributaries for use in the HEC-2 models assembled for each stream. The method for determining peak flow rates for tributaries at locations other than the mouth will be as follows:

1. Record the existing drainage area, percent urban development, and peak discharge at the mouth of each sub-area within which flow rates must be determined at locations upstream of the HEC-1 computation point.
2. Determine the total drainage area and percent urban development at two or three points along the tributary upstream of the mouth.
3. Use HEC-1 to compute a "fully developed" peak flow rate for each tributary. Unit hydrograph parameters for this condition are estimated by setting the Percent Urban Development and Percent Channel Improvement equal to 100.
4. Use the existing drainage area, existing percent urban development, and existing and fully developed conditions flow rates at the mouth of each tributary to develop an equation for computing upstream peak discharges as follows:

Equation 4.6

$$Q_{ud} = \frac{Q_{ex} - Q_{fd}(UD)}{1 - UD}$$

Equation 4.7

$$k_{ud} = \frac{Q_{ud}}{A^{0.779}}$$

Equation 4.8

$$k_{fd} = \frac{Q_{fd}}{A^{0.779}}$$

Equation 4.9

$$Q = \left[(\%UD) \times \left(k_{fd} - k_{ud}\right) + k_{ud}\right] A^{0.779}$$

in which:

Q_{ud} = peak flow rate corresponding to completely undeveloped conditions

Q_{ex} = existing conditions peak flow rate

Q_{fd} = peak flow rate corresponding to fully developed conditions

k_{ud} = y-intercept of area vs. discharge equation for undeveloped conditions

k_{fd} = y-intercept of area vs. discharge equation for fully developed conditions

A = drainage area at selected location along the stream

$\%UD$ = existing Percent Urban Development at selected location.

The slope of the equation, which plots as a straight line on log-log graph paper, is 0.779. This value is determined by plotting the computed 100-year peak flow rate for each sub-area in the Dickinson Bayou HEC-1 model with less than 5% percent urban development versus drainage area and fitting a straight line throught the plotted points. This line represents a relationship between the rate of increase of drainage area and peak discharge.

5. Use the equation developed for each tributary to compute peak discharge rates at the computation points upstream of the HEC-1 computation point.

Starting Water Surface Elevations

The starting elevation for water surface profile computations may be specified in one of three ways:

1. as critical depth
2. as a known elevation
3. by the slope-area method

Starting at critical depth is appropriate only at locations where critical or near-critical flow conditions are known to exist for the range of discharges being computed, e.g., a drop structure or weir.

When an accurate rating curve is available, the appropriate starting elevation can be specified as a known value. Alternatively, the starting elevation may be specified as the water surface elevation in the receiving stream. Care must be exercised when using the latter approach, however. It is important to make sure that the use of the water surface elevation in a receiving stream does not result in a coincident storm frequency greater than the design storm frequency for the stream being analyzed. For instance, the use of 100-year flow rates in the analysis of a tributary stream along with a starting water surface elevation equal to the 100-year water surface elevation in the receiving stream may result in a coincident storm return period of greater than 100 years.

If critical flow conditions do not exist, and the starting water surface elevation for the stream cannot be determined from a rating curve or other source of information, the slope-area method must be used. For beginning backwater computations by this method, the slope of the energy grade line is specified. As a first trial, the starting slope may be set equal to the physical slope of the channel. A trial and error approach should be used to refine the estimate of the slope of the energy grade line until the specified slope at the first channel cross-section is consistent with the computed energy slope at several subsequent cross-sections.

Head Losses

Manning's equation should be used to determine energy losses due to channel friction and resistance. The Manning equation is an empirical relationship which relates friction slope, flow depth, channel roughness, and channel cross-sectional shape to flow rate. The ***friction slope*** is a measure of the rate at which energy is being lost due to channel resistance. When the channel slope and the friction slope are equal ($S_f = S_o$) the flow is uniform and the Manning equation may be used to determine the depth for uniform flow, commonly known as the ***normal depth***.

The Manning equation is as follows:

Equation 4.10

$$V = \frac{1.49}{n} R^{2/3} \sqrt{S_f}$$

or

Equation 4.11

$$Q = \frac{1.49}{n} A R^{2/3} \sqrt{S_f}$$

in which:

Q = total discharge (cfs)

V = velocity of flow (ft/sec)

n = Manning coefficient of roughness

A = cross-sectional area of the flow (ft^2)

R = hydraulic radius of the channel (ft) (flow area/wetted perimeter)

S_f = friction slope, the rate at which energy is lost due to channel resistance

Normal depth may be determined by using Equation 4.11. The area (A) and the hydraulic radius (R) are written in terms of the depth (y). Knowing the discharge (Q), Manning "n" value, and the channel slope (S_o), Equation 4.11 can be solved by trial to find normal depth (y_o).

Head losses should be incorporated into the backwater computations for bends with a radius of curvature less than three times the channel top width. Energy loss due to curve resistance is computed using the following equation:

Equation 4.12

$$h_L = C_f \frac{V^2}{2g}$$

in which:

h_L = head loss (feet)

C_f = coefficient of resistance

V = average channel velocity (feet per second)

g = gravitational acceleration (32.2 ft/sec^2).

Guidelines for selecting appropriate values of C_f are available [Chow, 1959].

The HEC-2 computer program does not incorporate a bend loss computation. If HEC-2 is used and bend losses are significant, the loss must be added at the appropriate point in the computation. Bends with a radius of curvature greater than three times the top width of the channel generally have insignificant losses and no computation is required.

Manning n Values for Channels and Overbanks

Manning "n" value is an experimentally derived constant which represents the effect of channel roughness in the Manning equation. Considerable care must be given to the selection of an appropriate "n' value for a given channel due to its significant effect on the results of the Manning equation. Tables 4.8 through 4.11 provide a listing of "n" values for various channel conditions.

TABLE 4.8 Manning Roughness Coefficient for Lined or Built-Up Channels

Source: (Chow, 1959)

Type of Channel and Description	Minimum	Normal	Maximum
Metal			
Unpainted Smooth steel surface	0.011	0.012	0.014
Painted smooth steel surface	0.012	0.013	0.017
Corrugated metal	0.021	0.025	0.030
Cement			
Neat, surface	0.010	0.011	0.013
Mortar	0.011	0.013	0.015
Wood			
Planed, untreated	0.010	0.012	0.014
Planed, creosoted	0.011	0.012	0.015
Unplaned	0.011	0.013	0.015
Wood plank with battens	0.012	0.015	0.018
lined with roofing paper	0.010	0.014	0.017
Concrete			
Trowel finish	0.011	0.013	0.015
Float finish	0.013	0.015	0.016
Finished, with gravel on bottom	0.015	0.017	0.020
Unfinished	0.014	0.017	0.020
Gunite concrete, good section	0.016	0.019	0.023
Gunite concrete, wavy section	0.018	0.022	0.025
Concrete on good excavated rock	0.017	0.020	—
Concrete on irregular excavated rock	0.022	0.027	—
Concrete Bottom, Float Finished			
sides of dressed stone in mortar	0.015	0.017	0.020
sides of random stone in mortar	0.017	0.020	0.024
sides of cement rubble masonry	0.020	0.025	0.030
sides of cement rubble masonry, plastered	0.016	0.020	0.024
sides of dry rubble or riprap	0.020	0.030	0.035
Gravel Bottom			
sides of Formed concrete	0.017	0.020	0.025
sides of Random stone in mortar	0.020	0.023	0.026
sides of Dry rubble or rip-rap	0.023	0.033	0.036
Brick			
Glazed	0.011	0.013	0.015
in cement mortar	0.012	0.015	0.018
Rubble Masonry			
Cemented	0.017	0.025	0.030
Dry	0.023	0.032	0.035
Dressed ashlar	0.013	0.015	0.017
Asphalt			
Smooth	0.013	0.013	—
Rough	0.016	0.016	—
Vegetated lining	0.030	—	0.500

TABLE 4.9 Manning Roughness Coefficient for Excavated or Dredged Channels

Source: (Chow, 1959)

Type of Channel and Description	Minimum	Normal	Maximum
Earth, straight and uniform			
Clean, recently completed	0.016	0.018	0.020
Clean, after weathering	0.019	0.022	0.025
Gravel, uniform section, clean	0.022	0.025	0.030
With short grass, few weeds	0.022	0.027	0.033
Earth, winding and sluggish			
No vegetation	0.023	0.025	0.030
Grass, some weeds	0.025	0.030	0.033
Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
Earth bottom and rubble sides	0.028	0.030	0.035
Stony bottom and weedy banks	0.025	0.035	0.040
Cobble bottom and clean sides	0.030	0.040	0.050
Dragline-excavated or dredged			
No vegetation	0.025	0.028	0.033
Light brush or banks	0.035	0.050	0.060
Rock cuts			
Smooth and uniform	0.025	0.035	0.040
Jagged and irregular	0.035	0.040	0.050
Channels not maintained, weeds and brush uncut			
Dense weeds, high as flow depth	0.050	0.080	0.012
Clean bottom, brush on sides	0.040	0.050	0.080
Same, highest stage of flow	0.045	0.070	0.110
Dense brush, high stage	0.080	0.100	0.140

TABLE 4.10 Manning Roughness Coefficient for Minor Natural Streams

Source: (Chow, 1959) Note: A "minor stream" is one which has a top width of less than 100 feet at flood stage. A major stream is one with a top width of more than 100 feet at flood stage. The n value is less than that for minor streams of similar description because banks offer less effective resistance.

Type of Channel and Description	Minimum	Normal	Maximum
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but some stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070

TABLE 4.11 Manning Roughness Coefficient for Major Natural Streams

Source: (Chow, 1959)

Type of Channel and Description	Minimum	Normal	Maximum
Regular section with no boulders or brush	0.025	—	0.060
Irregular and rough section	0.035	—	0.100

TABLE 4.12 Manning Roughness Coefficient for Flood Plains

Source: (Chow, 1959)

Type of Channel and Description	Minimum	Normal	Maximum
Pasture, no brush			
Short grass	0.025	0.030	0.035
High grass	0.030	0.035	0.050
Cultivated areas			
No crop	0.020	0.030	0.040
Mature row crops	0.025	0.035	0.045
Mature field crops	0.030	0.040	0.050
Brush			
Scattered brush, heavy weeds	0.035	0.050	0.070
Light brush and trees, in winter	0.035	0.050	0.060
Light brush and trees, in summer	0.040	0.060	0.080
Medium to dense brush, in winter	0.045	0.070	0.110
Medium to dense brush, in summer	0.070	0.100	0.160
Trees			
Dense willows, summer, straight	0.110	0.150	0.200
Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
Same as above, but with flood stage reaching branches	0.100	0.120	0.160

The following equation presents a method to compute a composite roughness coefficient based on various channel characteristics [Chow, 1959]:

$$n = (n_o + n_1 + n_2 + n_3 + n_4)m$$

Equation 4.13

in which:

n = Computed Value of Manning Roughness Coefficient

Table 4.13 defines and lists representative values for the other terms in the equation.

TABLE 4.13 Parameters Used in Computing Manning Roughness Coefficient

Source: (Chow, 1959)

Parameter	Accounts for	Representative Values
n_0	Channel Material	0.020 for Earth 0.025 for Rock Cut 0.024 for Fine Gravel 0.028 for Coarse Gravel
n_1	Degree of Irregularity	0.000 for Smooth 0.005 for Minor Irregularities 0.010 for Moderate Irregularities 0.010 for Severe Irregularities
n_2	Variation of Channel Cross-Section	0.000 for Gradual Variations 0.005 for Alternating Occasionally 0.010 to 0.015 for Alternating Frequently
n_3	Relative Effect of Obstructions	0.000 for Negligible Obstructions 0.010 to 0.015 for Minor Obstructions 0.020 to 0.030 for Appreciable Obstructions 0.040 to 0.060 for Severe Obstructions.
n_4	Vegetation	0.005 to 0.010 for Low Vegetation 0.010 to 0.025 for Medium Vegetation 0.025 to 0.050 for High Vegetation 0.050 to 0.100 for Very High Vegetation
m	Degree of Meandering	1.000 for Minor Meandering 1.150 for Appreciable Meandering 1.300 for Severe Meandering

Analysis of Channel Transitions

A discussion of the computation of expansion and contraction losses at channel transitions is given here. The equation for these losses and loss coefficients are also included in this section. Also, the use of these coefficients in the hydraulic analysis of streams is discussed.

Analysis of Channel Bends

A discussion of the calculation of bend losses is given here.

Guidelines for selecting loss coefficient values are included. The input of bend losses in HEC-2 is discussed. Guidelines for erosion protection locations are presented.

Hydraulic Analysis of Bridges

There are numerous methods available to compute the energy losses associated with flow through bridges or culverts. Sources of energy loss in these structures include flow resistance, channel transitions, and direct obstructions to the flow such as piers. Each structure should be examined individually to determine the best approach. The bridge modeling routines found in HEC-2 are recommended for their versatility and flexibility. Brief descriptions of what they do and when they should be used are discussed in the following sections.

HEC-2 Normal Bridge Method

The Normal Bridge Method computes energy losses through a bridge in the same way that losses in a normal stream reach are computed. That is, the computed losses are due primarily to friction and are calculated using the standard step method. The friction loss calculations are based on Manning's equation. The HEC-2 program accounts for the presence of the bridge by subtracting the area of that portion of the bridge below the water surface from the total flow area and by increasing the wetted perimeter where water is in contact with the bridge structure.

The normal bridge method should be used when friction losses are the predominant consideration. This includes long culverts under low flow conditions and cases where the bridge and abutments are small obstructions to the flow. Because the special bridge method requires a trapezoidal approximation of the bridge opening for low flow solutions, the normal bridge method can be used when the flow area cannot be reasonably approximated by a trapezoid. Also, when highly submerged weir flow occurs over a bridge, the normal bridge method is preferred.

HEC-2 Special Bridge Method

The Special Bridge Method has the capability to compute energy losses through a bridge for a number of different flow conditions, including low flow, pressure (orifice) flow, weir flow, or any possible combination of these conditions. The Special Bridge uses hydraulic formulas to determine what flow conditions exist, what portion of the total flow rate falls into each condition, and what change in energy head and water surface elevation will occur through the bridge for a given total flow rate.

The special bridge method is capable of solving flow problems where losses are due primarily to factors other than friction. In general, the Special Bridge method should be used for most bridges, unless conditions clearly indicate that it is not a valid approach for a particular bridge structure.

Whenever flow crosses critical depth in a structure, the special bridge method should be used. In the HEC-2 Special Bridge Method, three different regimes are possible for flows through a bridge structure. These regimes are denoted Class A, Class B, and Class C low flow. ***Class A Low Flow*** occurs when the water surface through the bridge is above critical. In other words, the flow is sub-critical. ***Class B Low Flow*** can exist for either sub-critical or super-critical flow conditions. For either condition, Class B Low Flow occurs when the water surface profile passes through critical depth within the bridge constriction. ***Class C Low Flow*** occurs where the water surface profile stays super-critical through the bridge constriction.

Special Culvert Method

The Special Culvert Method utilizes methods developed by the Federal Highway Adminstration (FHWA) to determine energy losses through culvert crossing structures. The Special Culvert Method may be applied to a variety of flow conditions through a single culvert or through two or more identical culverts. Flows overtopping the roadway are treated as weir flow, as is the case with the Special Bridge Method.

Alternatives to HEC-2 for Bridge and Culvert Analysis

The use of alternative means for computing bridge- and culvert-related losses is encouraged when the engineer is properly aware of how and why such a strategy is appropriate and its results are reasonable. One example of such an alternative method involves the use of the procedures described in Federal Highway Administration's (FHWA) *Hydraulics of Bridge Waterways* [Bradley, 1970]. Another is presented in the FHWA's *Hydraulic Design of Highway Culverts* [FHWA, 1985].

Caution must be exercised to insure that the losses calculated by alternative methods are properly used in the HEC-2 program. For example, the FHWA technique provides the increase in water surface elevation above the normal water surface elevation without the bridge. Therefore, it includes the effects of contraction and expansion losses and the loss caused by the structure, but it does not reflect the normal friction loss that would occur without the bridge.

Alternative Hydraulic Analysis of Culverts

As an alternative to the use of the HEC-2 Special Culvert Option, the culvert can be analyzed using the methods described in the following sections.

The hydraulic capacity of a culvert is said to be either inlet-controlled or outlet-controlled. ***Inlet control*** means that the discharge in the culvert is limited by the hydraulic and physical characteristics of the inlet alone. These include headwater depth, culvert barrel shape, barrel cross-sectional area, and the type of inlet edge. For inlet control, the barrel roughness, length, and slope are not factors in determining culvert capacity.

Under ***outlet control***, the discharge capacity of the culvert is dependent on all of the hydraulic variables of the structure. These include headwater depth, tailwater depth as well as barrel shape, cross-sectional area, barrel roughness, slope, and length.

In all culvert design, ***headwater***, or depth of ponding at the entrance to the culvert, is an important factor in culvert capacity. The headwater depth (*HW*) is the vertical distance from the invert at the culvert entrance to the energy grade line of the approaching flow. Due to low velocities in most entrance pools and the difficulty in determining velocity head in any flow, the energy line can often be assumed to be the same as the water surface.

For culverts under outlet control, ***tailwater depth*** is an important factor in computing both headwater depth and the hydraulic capacity of the culvert. If flow in the channel downstream of the culvert is sub-critical, a computer-aided backwater analysis or calculation of normal depth is warranted to determine the tailwater elevation. If the downstream flow is super-critical, tailwater depth is not a factor in determining the culvert's hydraulic capacity.

Inlet-Controlled Flow

Under inlet control, the culvert entrance may or may not be submerged. However, in all cases inlet-controlled flow through the culvert barrel is free surface flow. When the culvert inlet is submerged, the most reliable means for determining discharge is with standard empirical relationships. Nomographs which plot headwater vs. discharge for various culvert sizes and shapes under inlet control have been developed on the basis of laboratory research with models and full scale prototypes. Appendix A provides a copy of these nomographs.

Outlet-Controlled Flow

Culverts with outlet control flow with the culvert barrel full or partially full for part or all of the barrel length. Both the headwater and tailwater may or may not submerge the culvert.

If the culvert is flowing full, the energy required to pass a given quantity of water is stored in the head (*H*). From energy considerations it can be shown that *H* is the difference between the hydraulic grade line at the outlet and the energy grade line at the inlet (expressed in feet).

When a given discharge passes through a culvert, stored energy, represented by the total head (*H*) is dissipated in three ways. A portion is lost to turbulence at the entrance (H_e); a portion is lost to frictional resistance in the culvert barrel (H_f); and a portion is lost as the kinetic energy of flow through the culvert is dissipated in the tailwater (H_o) From this, the following relationship is evident:

Equation 4.14

$$H = H_e + H_f + H_o$$

The entrance loss (H_e) is determined by multiplying the culvert velocity head by an entrance loss coefficient k_e. Table 4.14 through 4.16 list values for the entrance loss coefficient. In these tables, the entrances described as "End section conforming to fill slope" are the sections commonly available from manufacturers.

From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design, have a superior hydraulic performance.

TABLE 4.14 Entrance Loss Coefficients for Concrete Pipe Culverts

Source: (FHWA, 1985)

Type of Structure and Design of Entrance	Coefficient k_e
Projecting from fill, socket end (groove-end)	0.20
Projecting from fill, square cut end	0.50
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.20
Square-edge	0.50
Rounded (radius = 0.5D)	0.20
Mitered to conform to fill slope	0.70
End section conforming to fill slope	0.50
Beveled edges (33.7 degree or 45 degree bevels)	0.20
Side- or slope-tapered Inlet	0.20

TABLE 4.15 Entrance Loss Coefficients for Corrugated Metal Culverts

Source: (FHWA, 1985)

Type of Structure and Design of Entrance	Coefficient k_e
Projecting from fill (no headwall)	0.90
Headwall or headwall and wingwalls (square-edge)	0.50
Mitered to conform to fill slope (paved or unpaved slope)	0.20
End section conforming to fill slope	0.50
Beveled edges (33.7 degree or 45 degree bevels)	0.20
Side- or slope-tapered inlet	0.20

TABLE 4.16 Entrance Loss Coefficients for Concrete Box Culverts

Source: (FHWA, 1985)

Type of Structure and Design of Entrance	Coefficient k_e
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.50
Rounded on 3 edges to radius of 1/12 barrel dimension or beveled edges on 3 sides	0.20
Wingwalls at 30 degree to 75 degree to barrel	
Square-edged at crown	0.40
Crown edge rounded to radius of 1/12 barrel dimension or beveled top edge	0.20
Wingwalls at 10 degrees to 25 degrees to barrel	
Square-edged at crown	0.50
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.70
Side- or sloped-tapered Inlet	0.20

The exit loss (H_o) is generally set equal to the culvert velocity head, the downstream flow velocity being assumed to be zero. An expression for the friction loss (H_f) is derived from Manning's equation:

$$H_f = \left(\frac{29n^2L}{R^{4/3}}\right)\frac{V^2}{2g}$$

Equation 4.15

in which:

n = Manning's roughness coefficient

L = culvert barrel length (ft)

R = the hydraulic radius (ft)

g = the gravitational constant (32.2 ft/sec^2)

V = mean velocity of flow in the culvert (ft/sec).

Rearranging Equation 4.14 it is seen that for full flow:

Equation 4.16

$$H = 1 + K_e + \left(\frac{29n^2L}{R^{4/3}}\right)\frac{V^2}{2g}$$

Equation 4.16 may be solved using the full flow nomographs located in the Texas Department of Transportation Bridge Division Hydraulics Manual. Each nomograph is drawn for a particular barrel shape and material and a single value of Manning's "n" as noted on the respective charts. These nomographs may be used for other values of "n" by modifying the culvert length as directed in the instructions for use of the full-flow nomographs.

Figure 4.20 represents the various hydraulic elements of pressure, or full, flow through a culvert and reveals graphically that the head (H) is equivalent to the vertical distance between the energy grade line at the inlet and the energy grade line at the outlet. It also reveals the following relationship for full flow conditions:

Equation 4.17

$$H = H_e + H_f + H_o = HW + S_oL - \frac{V_d^2}{2g} - TW$$

in which:

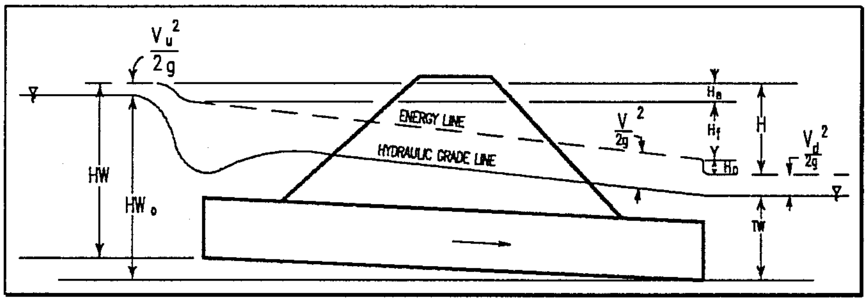
HW = headwater depth (feet)

TW = tailwater depth (feet)

S_o = culvert barrel slope (ft/ft)

FIGURE 4.20 Hydraulic Elements of Pressure Flow Through Culverts

(Source: (FHWA, 1985))



If the downstream flow velocity is neglected, equation 4.17 becomes:

Equation 4.18

$$H = HW + S_oL - TW$$

In culvert design it is generally required that the depth of the headwater (HW) be determined. Rearranging Equation 4.18, the following expression for HW is derived:

Equation 4.19

$$HW = H + d_2 - LS_o$$

d_2 is the depth of flow at the culvert outlet. When the culvert outlet is submerged by the tailwater, d_2 is simply the tailwater depth. However, when the tailwater is below the crown of the culvert, d_2 is taken as the greater of the following two values:

1. TW
2. $(d_c + D) / 2$

in which:

d_c = critical depth in the culvert as read from the appropriate chart (ft)

TW = tailwater depth above the invert of the culvert outlet (ft)

D = height of the culvert (ft).

Step by Step Culvert Design Procedure

It is possible by involved hydraulic computations to determine the probable type of flow under which a culvert will operate for a given set of conditions. However, such computations can be avoided by determining the headwater necessary for a given discharge under both inlet and outlet flow conditions. The larger of the two will define the type of control and the corresponding headwater depth.

The culvert design procedures presented here are based on information provided in the Federal Highway Administration publication *Hydraulic Design of Highway Culverts* [FHWA, 1985]. The nomographs included in Appendix A of this manual cover the range of pipe and box culverts commonly used in drainage design.

The following is the recommended procedure for selection of culvert size:

Step 1: List design data.

a) Design discharge (Q), in cfs, with return period.

b) Approximate length (L) of culvert, in feet.

c) Slope of culvert. If grade is given in percent, convert to slope in feet per feet.

d) Allowable headwater depth, in feet, which is the vertical distance from the culvert invert (flow-line) at the entrance to the water surface elevation permissible in the headwater pool or approach channel upstream from the culvert.

e) Flow velocities in the channel upstream and downstream of the proposed culvert location.

f) Type of culvert for first trial selection, including barrel material, barrel cross-sectional shape and entrance type.

Step 2: Determine the first trial culvert size.

Since the procedure given is one of trial and error, the initial trial size can be determined in several ways:

a) Past experience and engineering judgement.

b) By using an approximating equation such as $Q/6 = A$ from which the trial culvert dimensions are determined. A is the culvert barrel cross-sectional area and 6 is an estimate of barrel velocity in feet per second.

c) Initially, utilize the inlet control nomographs for the culvert type selected. An HW/D must be assumed, say $HW/D = 1.5$, along with the given Q to determine a trial size.

Note: If any trial size is too large in dimension because of limited height of embankment or availability of size, multiple culverts may be used by dividing the discharge appropriately among the number of barrels used. Raising the embankment height or the use of pipe arch and box culverts with width greater than height should also be considered. Final selection should be based on applicability and costs.

Step 3: Find headwater depth for trial size culvert.

a) Assuming Inlet Control

1. Using the trial size from Step 2, find the headwater depth (*HW*) by use of the appropriate inlet control nomograph. Tailwater (*TW*) conditions are to be neglected in this determination. *HW* in this case is found by multiplying HW/D obtained from the nomographs by the height of culvert (D).

2. If *HW* is greater or less than allowable, try another trial size until *HW* is acceptable for inlet control before computing *HW* for outlet control.

b) Assuming Outlet Control

1. Approximate the depth of tailwater (*TW*), in feet, above the invert at the outlet for the design flood condition in the outlet channel.

2. For tailwater (*TW*) elevation equal to or greater than the top of the culvert at the outlet, set d_2 equal to *TW* and find *HW* by the following equation:

Equation 4.20

$$HW = H + d_2 - LS_o$$

in which:

HW = vertical distance in feet from culvert invert at entrance to the pool surface

H = head loss in feet as determined from the appropriate nomograph (Charts 8-14)

d_2 = vertical distance in feet from culvert invert at outlet to the hydraulic grade line

S_o = slope of barrel (feet/feet)

L = culvert length (feet).

3. For tailwater (*TW*) elevations less than the top of the culvert at the outlet, find headwater *HW* by Equation 4.20 as in Step b(2) above except that:

   $$d_2 = (d_c + D)/2 \text{ or } TW \text{ (whichever is greater)}$$

   in which:

   d_c = critical depth in feet. Note: d_c cannot exceed D

   D = height of culvert opening (feet).

   Note: Headwater depth determined in Step b(3) becomes increasingly less accurate as the headwater computed by this method falls below the value:

   $$D + (1 + k_e)\frac{V^2}{2g}$$

c) Compare the headwater depths obtained in Step 3a and Step 3b (Inlet Control and Outlet Control). The higher headwater governs and indicates the flow control existing under the given conditions for the trial size selected.

FIGURE 4.21 Culvert Design Form (Source: (FHWA, 1985))

Project: ______________________

Station: 1+00
Sheet 1 of 1

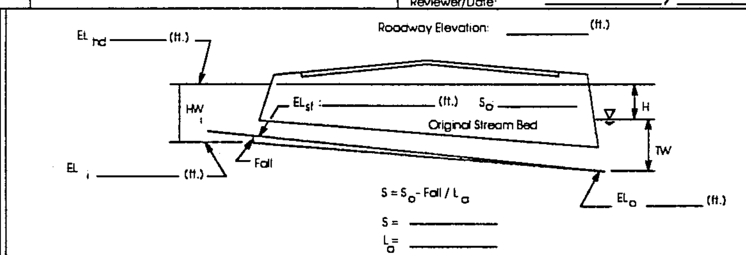
CULVERT DESIGN FORM
Designer/Date: ________ / ________
Reviewer/Date: ________ / ________

See Additional Sheets   HYDROLOGICAL DATA

- ☐ Method: ______________
- ☐ Drainage Area ________ ☐ Stream Slope ________
- ☐ Channel Shape: ______________
- ☐ Routing: ________ ☐ Other: ________

DESIGN FLOWS/TAILWATER

Recurrence Interval (yr)	Flow Rate (cfs)	Tailwater (ft)



CULVERT DESCRIPTION: Material - Shape - Size - Entrance	Total Flow Q (cfs)	Flow per Barrel Q/N (1)	HEADWATER CALCULATIONS — Inlet Control: HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	Outlet Control: TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	Control HW Elev.	Outlet Flow Velocity	Comments

TECHNICAL FOOTNOTES:

(1) Use Q/NB for Box Culverts

(2) $HW_i/D = HW/D$ or HW_f/D from Design Charts

(3) Fall $= HW_i - (EL_{hd} - EL_{sf})$: Fall is Zero

(4) $EL_{hi} = HW_i + EL_i$ (Invert of Inlet Control Section)

(5) TW Based on Downstream Control or Flow Depth in Channel.

(6) $h_o = TW$ or $(d_c + D/2)$ (whichever is greater)

(7) $H = \left[1 + k_e + (29n^2 L)/R^{1.33}\right] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

a - Approximate
hd - Design Headwater
hi - Headwater in Inlet Control
ho - Headwater in Outlet Control
i - Inlet Control Section
sf - Streambed at Culvert Face
f - Culvert Face
o - Outlet
tw - Tailwater

COMMENTS/DISCUSSION

CULVERT BARREL SELECTED:

Size: ______________
Shape: ______________
Material: ______________ n: ______
Entrance: ______________

d) If outlet control governs and the *HW* is higher than is acceptable, select a larger trial size and find *HW* as instructed under Step 3b. (Inlet control need not be checked, since the smaller size was satisfactory for this control as determined under Step 3a.)

Step 4: Try additional culvert types or shapes.

Determine their size and *HW* by the above procedure.

Step 5: Compute outlet velocities for size and types to be considered in selection and determine need for channel protection.

a) If outlet control governs in Step 3c above, outlet velocity equals Q/A_o, where A_o is the cross-sectional area of flow in the culvert barrel at the outlet. If d_c or TW is less than the height of the culvert barrel, use A_o corresponding to d_c or TW depth, whichever gives the greater area of flow. A_o should not exceed the total cross-sectional area A of the culvert barrel.

b) If inlet control governs in Step 3c, outlet velocity can be assumed to equal mean velocity in open-channel type flow in the barrel as computed by Manning's equation for the rate of flow, barrel size, roughness and slope of culvert selected.

Step 6: Record final selection of culvert with size, type, required and computed headwater, outlet velocity and economic justification.

Figure 4.21 provides a culvert design form which may be used to record the culvert computations and related data.

Example of Culvert Design Procedure

This section contains a complete example of the step-by-step culvert design procedure presented in the previous section.

Step 1: List design data.

a) Design discharge (Q) = 200 cfs for the 25-year storm event.

b) Approximate length (L) of culvert = 200 feet.

c) Natural Stream Bed Slope = 1% = 0.01 ft/ft. Set the inlet invert at the natural streambed elevation (no fall).

d) Base the design headwater on the shoulder elevation of 110.0 with a two foot freeboard. Therefore, the design headwater is 108.0 - 100.0 = 8.0 feet.

e) Flow velocities in the channel upstream and downstream of the proposed culvert location.

f) Design a circular pipe culvert for this site. Consider the use of a corrugated metal pipe with standard 2-2/3 by 1/2 inch corrugations and beveled edges and concrete pipe with a groove end.

Step 2: Determine the first trial culvert size.

The initial trial size for the corrugated metal pipe culvert may be computed using the approximation of $A = 200/6 = 33.3$ square feet. Therefore, the pipe diameter, $D = \sqrt{4 \times 33.3/\pi} = 6.5$ ft = 78 inches. A more standard pipe size of 72 inches will be used for the first trial.

FIGURE 4.22 Example Culvert Design Form (Source: (FHWA, 1985))

Project: Culvert Design Example Problem
Montgomery County Drainage Criteria Manual

Station: 1+00
Sheet 1 of 1

CULVERT DESIGN FORM
Designer/Date: KNW / 7/18
Reviewer/Date: RDD / 7/19

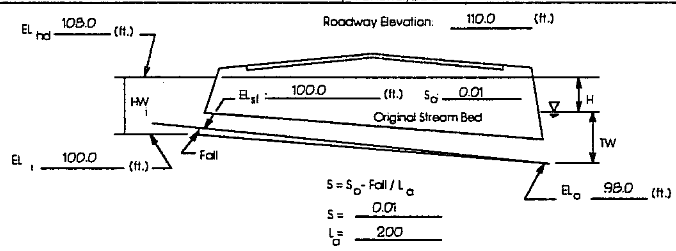
See Additional Sheets

HYDROLOGICAL DATA

- ☐ Method: Dickinson Bayou Runoff Rate Curves
- ☐ Drainage Area 125 ac ☐ Stream Slope: 1.0%
- ☐ Channel Shape: Trapezoidal
- ☐ Routing: N/A ☐ Other:

DESIGN FLOWS/TAILWATER

Recurrence Interval (yr)	Flow Rate (cfs)	Tailwater (ft)
25	200	3.5



CULVERT DESCRIPTION: Material - Shape - Size - Entrance	Total Flow Q (cfs)	Flow per Barrel Q/N (1)	HEADWATER CALCULATIONS: Inlet Control: HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	Outlet Control: TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	Control HW Elev.	Outlet Flow Velocity	Comments
C.M.P. / Circular / 72 in. / Bevel 45 deg. in headwall	200	200	0.95	5.7		105.7	3.5	3.9	4.9	4.9	0.20	2.5	105.4	105.7	9.2	Try 60" CMP
C.M.P. / Circular / 60 in. / Bevel 45 deg. in headwall	200	200	1.36	6.8		106.8	3.5	4.0	4.5	4.5	0.20	6.0	108.5	108.5	11.8	Try 60" RCP
R.C.P. / Circular / 60 in. / Groove End	200	200	1.35	6.7		106.7	3.5	4.0	4.5	4.5	0.20	3.0	105.5	106.7	15.6	Try 54" RCP
R.C.P. / Circular / 54 in. / Groove End	200	200	1.77	8.0		108.0	3.5	4.0	4.3	4.3	0.20	4.7	107.0	108.0	15.2	

TECHNICAL FOOTNOTES:

(1) Use Q/NB for Box Culverts
(2) $HW_i/D = HW/D$ or HW_i/D from Design Charts
(3) Fall $= HW_i - (EL_{hd} - EL_{sf})$; Fall is Zero
(4) $EL_{hi} = HW_i + EL_i$ (Invert of Inlet Control Section)
(5) TW Based on Downstream Control or Flow Depth in Channel.
(6) $h_o = TW$ or $(d_c + D/2)$ (whichever is greater)
(7) $H = \left[1 + k_e + (29n^2 L/R^{1.33})\right] V^2/2g$
(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:
a - Approximate
hd - Design Headwater
hi - Headwater in Inlet Control
ho - Headwater in Outlet Control
i - Inlet Control Section
sf - Streambed at Culvert Face
f - Culvert Face
o - Outlet
tw - Tailwater

COMMENTS/DISCUSSION
High Outlet Velocity - Outlet protection or larger conduit may be necessary.

CULVERT BARREL SELECTED:
Size: 54 in.
Shape: Circular
Material: Concrete n: 0.012
Entrance: Groove End

Step 3: Find headwater depth for trial size culvert.

a) Assuming Inlet Control

Using a pipe diameter of 72 inches, the headwater depth (*HW*) is determined using FHWA Chart 2 in Appendix A. The computed headwater depth $HW = 0.96 \times 72 = 69.12" = 5.80'$.

b) Assuming Outlet Control

1. Assume that the tailwater depth for 25-Year Flood is 3.5 feet.
2. The tailwater elevation is less than the top of the culvert.
3. Critical depth d_c is determined using FHWA Chart 4 in Appendix A. For this culvert, $d_c = 3.9$ feet. Therefore, $d_2 = (d_c + D)/2 = (3.9 + 6)/2 = 4.9$ feet. The headwater depth is computed using equation 4.7:

$$HW = H + d_2 - LS_o = 2.5 + 3.9 - 200 \times 0.01 = 5.7$$

c) Since the computed Inlet Control Headwater (5.8 feet) is higher than the computed Outlet Control Headwater (5.7 feet) for this culvert, the Inlet Control Headwater governs.

d) The computed headwater for the 72-inch corrugated metal pipe culvert is lower than the allowable headwater. Therefore, reduce the pipe diameter for the second trial.

Step 4: Try additional culvert types or shapes.

The culvert design form shown in Figure 4.22 shows the sequence of trial sizes and configurations for the example culvert. As indicated, the second trial indicates that the computed headwater depth for a 60-inch corrugated metal pipe culvert is greater than the allowable value. Therefore, a third trial is performed using a 60-inch groove end concrete pipe. For the fourth and final trial, the diameter of the concrete pipe is reduced to 54 inches, which provides a reasonable headwater value.

Step 5: Compute outlet velocities for size and types to be considered in selection and determine need for channel protection.

a) Inlet control governs the pipe size for the final trial.

b) Outlet velocity is computed assuming normal depth in the culvert barrel. The computed normal depth for a flow rate of 200 cfs in a 54-inch concrete pipe at a 1% slope is 3.5 feet, and the resulting flow velocity is 15.2 feet per second. This outlet velocity high enough to require erosion protection at the culvert outlet.

Step 6: Record final selection of culvert with size, type, required and computed headwater, outlet velocity and economic justification.

Figure 4.22 presents an example of a culvert design form which has been completed for the example described in this section.

Chapter 5. Detention System Analysis and Design

The introduction of impervious cover and improved runoff conveyance serves in many cases to increase flood peaks quite dramatically over those for existing conditions. When physical, topographic, and economic conditions allow it, channel improvements downstream of the development are often used to prevent increased flooding. When this is not feasible, a widely used practice is runoff detention or retention storage, wherein the storm volume is held back in the watershed and released at an acceptable rate. This chapter of the manual presents information on storage techniques, including guidance for the design of appropriate storm runoff storage facilities.

Development in a watershed can have complex and far-reaching consequences on the overall hydrologic regime. For this reason, careful plans for anticipating and meeting the long term flood control and drainage needs of the Dickinson Bayou watershed are being drawn up. The watershed "master plan" has been formulated to provide the most practical and efficient basin-wide approach to the hydrologic consequences of ongoing or future development, including proper coordination of storm detention facilities and channel improvements. Accordingly, the appropriate drainage regulatory agency must be consulted concerning preferred watershed flood control strategies and alternatives.

FLOOD STORAGE CONCEPTS

In a ***retention*** storage facility, runoff is captured and released only after the storm event is over and the downstream water surface has subsided. A retention storage system is seldom used. Special outlet devices or pumps are usually required for such systems. Figure 5.1 illustrates the effect of retention storage on developed conditions runoff hydrographs.

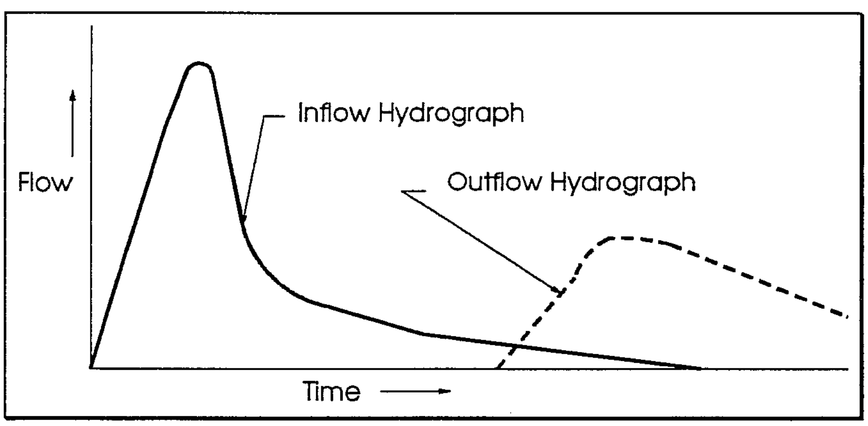
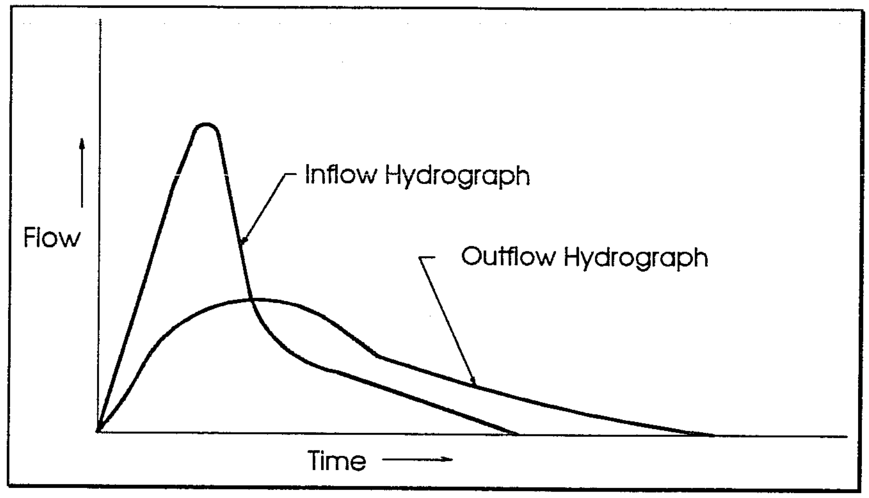


FIGURE 5.1 Effect of Retention Storage on Hydrographs

The vast majority of flood control storage is handled by ***detention*** facilities. The purpose of detention storage is to hold storm runoff back but release it continuously at an acceptable rate through a flow-limiting outlet structure, thus controlling downstream peak flows. Figure 5.2 illustrates the typical effect of detention storage facilities on developed conditions runoff hydrographs.

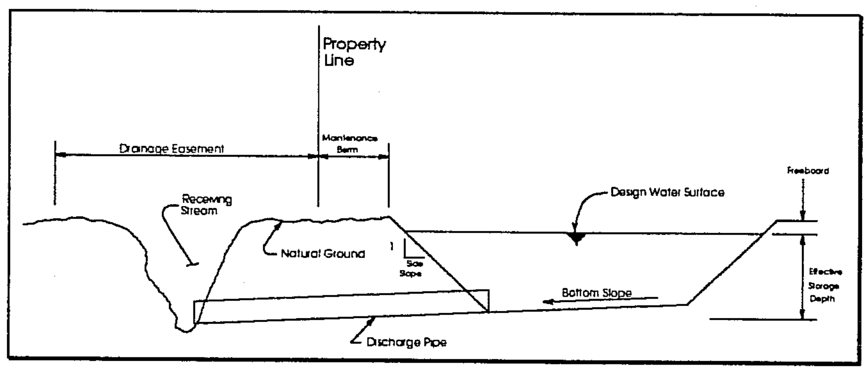
FIGURE 5.2 Effect of Detention Storage on Hydrographs



In-Line versus Off-Line Detention

Storage systems may be classified as either on-line or off-line facilities. They may be designed for either detention or retention of stormwater. Figures 5.3 and 5.4 illustrate a typical detention facility.

FIGURE 5.3 Cross-Section of Typical Detention Basin



An ***off-line detention facility*** is one in which storm runoff does not begin to flow into the storage facility until the discharge in the channel reaches some critical value above which unacceptable downstream flooding will occur. An off-line facility serves to store only the runoff volume associated with the high flow rate portions of the flood event.

An ***on-line detention facility*** is one in which the total storm runoff volume passes through the retention or detention facility's outflow structure.

An ***in-stream detention facility*** is a special type of on-line facility created by restricting the discharge of a segment of a drainage channel. In-stream detention facilities are acceptable if the drainage channel receives runoff only from the property for which the detention capacity is being provided. However, in-stream detention facilities which receive runoff from other upstream properties will be approved only if a written agreement is reached among all upstream property owners specifying the operation and maintenance of the in-stream detention facility under existing and future conditions of development.

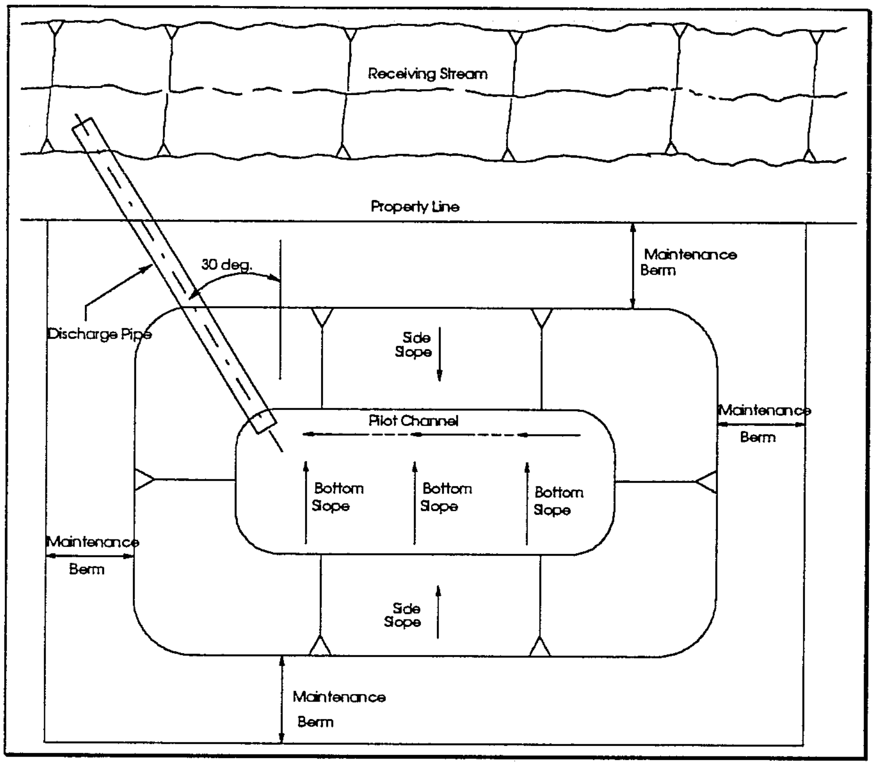


FIGURE 5.4 Typical Detention Basin Configuration

DETENTION DESIGN CONSIDERATIONS

Location of Facility

Detention basins should be located on or near primary drainage channels. This allows for a direct connection between the basin and the stream which receives discharges from it. In addition, detention basin locations should be chosen to facilitate the drainage of storm runoff into the basin.

Maintenance of Facility

Care should be exercised to insure that detention facilities are constructed in locations which allow easy access for maintenance purposes. Before any storm water detention or retention facility is constructed, a perpetual maintenance commitment must be obtained from an entity acceptable to the applicable drainage regulatory authority. Entities which may assume perpetual maintenance include, but are not necessarily limited to the following:

- Municipal Utility Districts, or other special-purpose districts created according the provisions of Texas law with the authority to maintain drainage facilities.
- Property Owners' Associations
- The Property Owner (for detention basins serving a single property owner)

Storm Sewer Hydraulic Gradients

The hydraulic gradients in storm sewers shall be determined using procedures outlined in Chapter 4. The starting water surface elevation for these calculations shall be the 25-year maximum pond elevation.

Allowances for Extreme Storm Events

Design consideration must be given to storm events in excess of the 100-year flood. An emergency spillway, overflow structure, or swale must be provided as necessary to effectively handle the extreme storm event. In places where a dam has been utilized to provide detention directly in the channel, due consideration must be given the consequences of a failure, and if a significant hazard exists, the dam must be adequately designed to prevent such hazards.

In addition, detention facilities which measure greater than six feet in height are subject to regulation by the Texas Water Commission [TWC, 1986]. The height of a detention facility or dam is the distance from the lowest point on the crest of the dam (or embankment), excluding spillways, to the lowest elevation on the center-line or downstream toe of the dam (or embankment), including the natural stream channel. Texas Water Commission regulations classify dam sizes and hazard potential and specify required failure analyses and spillway design flood criteria [TWC, 1986].

Multi-Purpose Use of Detention Facilities

The amount of land required for a stormwater detention facility is generally quite substantial. For this reason, storage facilities may serve a secondary role as parks or recreational areas whenever possible. Conversely, parking areas may serve a secondary role as storage facilities as long as the 100-year ponding depth within the parking area is six inches or less where cars are parked. Such dual use areas will be allowed only after proper review of the design scenario and approval of the specific project by the appropriate review agency or agencies.

When a dual use facility is proposed, a joint use agreement is required between the appropriate drainage regulatory agency and the entity sponsoring the secondary use. This agreement must specify the maintenance responsibilities of each party.

For privately maintained or dual use systems, each stormwater detention facility will be reviewed and approved only if: 1) The facility has been designed to meet or exceed the requirements contained within this manual; and 2) Provisions are made for the facility to be adequately maintained.

Aesthetic Aspects of Detention Facility Design

Due consideration should be given to aesthetic aspects of detention facility design. The use of reduced (flatter) side slopes, landscaping, and other measures to improve the appearance of detention basins should be considered.

Safety Considerations in Detention Facility Design

Safety should also be given careful consideration in detention basin designs. Embankment slopes, railings, fences, grates, and other features should be incorporated into the design of the facility wherever appropriate. Appropriate warning signs should be placed around the perimeter of the facility.

Designs of detention outlet structures should, wherever possible, incorporate grates and other appropriate safety features. Flow velocities should be limited to avoid the formation of dangerous undertows.

Wherever possible, the depth of water near the edge of the basin should be limited to reduce hazards to persons venturing near the water's edge. Limiting the embankment slope to a value which would allow a person to easily escape from the basin should also be considered.

SELECTION OF METHODS FOR DETENTION ANALYSIS

The three methods available for detention analysis are as follows:

- Method I: Corresponds to Hydrology Method I
- Method II: Corresponds to Hydrology Method II
- Method III: Corresponds to Hydrology Method III

See Chapter 3 for a description of Hydrology Methods I, II, and III.

Selection of Method on the Basis of Drainage Area

Drainage area is the primary basis for selecting detention method:

- Method I: <50 acres
- Method II: 50 - 500 acres
- Method III: >500 acres (can be applied to smaller areas if desired)

Other Criteria for Selecting Detention Method

Method III is always required when a detention facility affects an existing Dickinson Bayou watershed model.

- **Basins in Series**: Use *Inter-Connected Pond Routing* (ICPR) program from Streamline Technologics, Inc. of Orlando, FL as acceptable alternative to HEC-1.
- **Variable Tailwater Conditions and Interior Storage Areas**: Use *Interior Flood Hydrology* (HEC-IFH) program from the US Army Corps of Engineers as acceptable alternative to HEC-1.

METHOD I DETENTION ANALYSIS

The maximum allowable release rate from the detention facility during the 100-year storm event is the 100-year peak flow rate from the watershed of the detention facility under pre-development condition. The undeveloped peak flow rate shall be determined using the Rational Method.

The volume of flood control storage to be provided by the facility for the 100-year storm event is to be determined using the triangular hydrograph method illustrated in Figure 5.5. The required volume may be computed using the following formulas:

$$B = \frac{43560 V_g}{0.5 I} \qquad \text{Equation 5.1}$$

$$S = \frac{0.5 B (I - O)}{43560} \qquad \text{Equation 5.2}$$

in which:

B = duration of inflow to the basin (seconds)

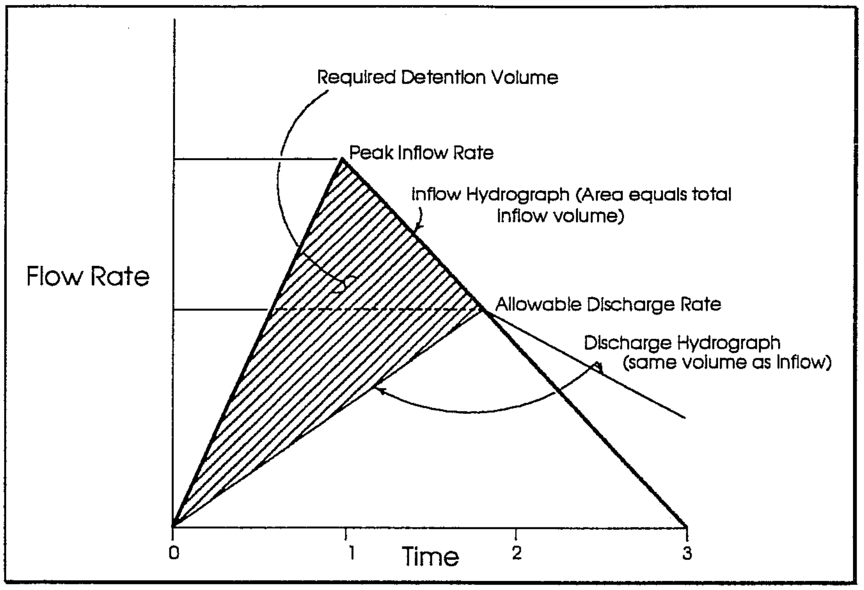
V_R = total basin inflow volume (acre-feet)

S = required flood storage volume (acre-feet)

I = peak inflow rate (cubic feet per second)

O = peak discharge rate (cubic feet per second).

FIGURE 5.5 Required Detention Volume for Less Than 50 Acres



This storage volume must be provided below the proposed maximum 100-year water surface elevation in the basin. The required storage volume for the 25-year storm event should be computed in the same way. The 25-year ponding elevation should be determined as the elevation below which the computed storage volume may be provided within the detention basin.

The size of the outlet pipe that is require to pass the maximum allowable release rate during the 100-year storm is to be computed assuming outlet control (See Chapter 4), by establishing a maximum ponding level in the detention facility during the 100-year storm and determining the appropriate a tailwater elevation in the outfall channel.

METHOD II DETENTION ANALYSIS

For drainage areas greater than or equal to 50 acres but less than 500 acres, an inflow hydrograph must be developed and routed through the detention facility. The inflow hydrograph may be assembled using the drainage area versus peak discharge curves and the Small Watershed Method of hydrograph development, both of which are described in Chapter 3. Alternatively, the inflow hydrograph may be developed using the HEC-1 computer program and the guidelines for HEC-1 applications presented in Chapter 3.

Routing of flows through the detention facility may be accomplished using the ***Modified Puls*** method. This method is described by the equation:

Equation 5.3

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

in which:

I = instantaneous inflow rate at the beginning of a routing period (cfs)

O = instantaneous outflow rate at the beginning of a routing period (cfs)

S = instantaneous storage volume at the beginning of a routing period (cfs)

Δt = duration of routing period (seconds).

The HEC-1 computer program may be used to perform detention routing computations using the Modified Puls method. Other programs which utilize the Modified Puls method are available. The routing equation given above may also be solved graphically and used in manual routing computations.

The existence of flooding problems in downstream areas may make the use of the HEC-1 program essential for the design and analysis of some detention basins which fall within this category. The appropriate drainage regulatory agency should be consulted to determine whether a detailed analysis of downstream impacts using the HEC-1 program is required for a particular watershed or detention basin location.

METHOD III DETENTION ANALYSIS

For drainage areas greater than or equal to 500 acres, the HEC-1 computer program will be used to analyze the operation of the proposed detention facility and to insure that downstream flooding conditions will not be increased. An existing conditions HEC-1 model of the entire watershed should first be established in conjunction with the appropriate drainage regulatory agency. Once existing conditions are established, the proposed development and detention facility will be analyzed for the 10-, 25-and 100-year storm events (and smaller events if the downstream channel has less than 10 year capacity). The detention facility will be sized to allow an appropriate release rate that will not cause any increase in flood levels in downstream areas.

DETENTION BASIN DESIGN DETAILS

All detention facilities shall be designed to attenuate developed conditions peak flow rates from the 25-year and 100-year frequency, 24-hour duration storm to existing conditions levels. No increase in downstream flow rates or flood levels will be allowed.

The maximum 100-year water surface elevation in all detention facilities shall be a minimum of 1 foot below the minimum top of bank elevation of the basin. In addition, all detention facilities must be designed to have sufficient freeboard to provide for adequate drainage of lateral storm sewers during the 25-year storm.

Detention Basin Geometry

The structural design of detention facilities is very similar to the design of open channels. For this reason, all requirements from Chapter 4 pertaining to the design of concrete-lined or grass-lined channels shall also apply to concrete-lined or grass-lined detention facilities.

In addition, the following guidelines are applicable:

1. **Pond Bottom**: A pilot channel shall be provided in detention facilities to insure that proper and complete drainage of the storage facility will occur. Concrete pilot channels shall have a minimum depth of two inches and a minimum flow-line slope of 0.0005 ft/ft. Grass-lined pilot channels shall have a minimum depth of two feet, a minimum flow-line slope of 0.001 ft/ft, and maximum side slopes of 3:1.

   The bottom slopes of the detention basin should be graded toward the pilot channel at a minimum slope of 0.005 ft/ft, and a recommended slope of 0.0075 ft/ft.

   Detention basins which make use of a channel section for detention storage may not be required to have a pilot channel, but should be built in accordance with the requirements for open channels as outlined in Chapter 4.

2. **Outlet Structure**: The outlet structure for a detention pond is subject to higher than normal head water conditions and erosive velocities for prolonged periods of time. For this reason the erosion protective measures are very important.

   Reinforced concrete pipe used in the outlet structure should conform to ASTM C-76 Class III with compression type rubber gasket joints conforming to ASTM C-443. Pipes, culverts and conduits used in the outlet structures should be carefully constructed with sufficient compaction of the backfill material around the pipe structure as recommended in the geotechnical analysis. Generally, compaction density should be the same as the rest of the structure. The use of pressure grouting around the outlet conduit should be considered where soil types or conditions may prevent satisfactory backfill compaction. Pressure grouting should also be used where headwater depths could cause backfill to wash out around the pipe.

Geotechnical Investigation

Before initiating final design of a detention pond with a surface area of 5 acres of more at the 100-year design water surface elevation, a detailed soils investigation by a geotechnical engineer should be undertaken. The following minimum requirements, taken from the *Criteria Manual for Design of Flood Control and Drainage Facilities in Harris County, Texas* [HCFCD, 1984], shall be addressed:

1. The ground water conditions at the proposed site;
2. The type of material to be excavated from the pond site and its suitability for additional use;
3. If a dam is to be constructed, adequate investigation of potential seepage problems though the dam and attendant control requirements, the availability of suitable embankment material and the stability requirements for the dam itself;

4. Potential for structural movement or areas adjacent to the pond due to the induced loads from existing or proposed structures and methods of control that may be required;
5. Stability of the pond side slopes.

Detention Basin Outlet Structure Design

The primary detention outlet structure shall be designed to convey the maximum 100-year detention discharge. The following types of detention outlet structures may be utilized:

1. Outflow Pipes or Culverts
2. Horizontal Weirs
3. V-Notch Weirs
4. Orifices
5. Outlet Pipes with Risers

The use of other types of outlet structures should be approved by the appropriate drainage regulatory agency prior to the completion of any detailed design computations.

Design Tailwater Depth for Detention Facilities

There are two tailwater conditions which may be applied to detention basin design; a constant tailwater elevation or tailwater elevations which vary with time. In reality, the water level in the outfall channel will always vary with time during a runoff event due to flow from the watershed upstream of the detention pond outfall as well as the outflow from the pond. Routing a hydrograph through a detention pond should incorporate the effect of the variable tailwater on the outflow. However, in most cases the development of a storm hydrograph in the outfall channel requires extensive watershed modeling.

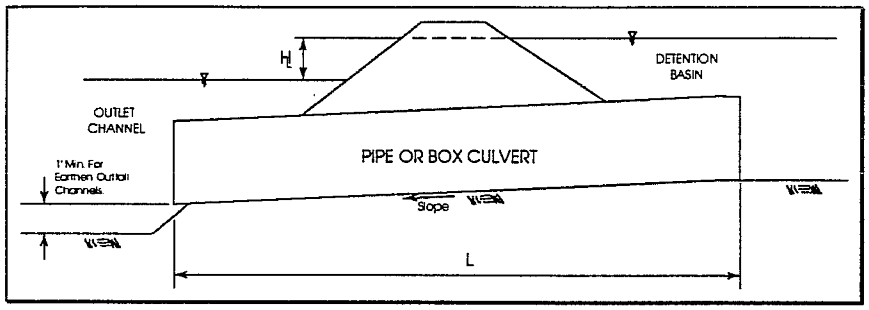
For detention facilities which outfall at a location on a channel where the upstream drainage area is greater than 2,000 acres, the use of variable tailwater elevations is recommended. Check with the applicable drainage regulatory agency to find out what hydrologic information is available for the subject watershed and, if necessary, to discuss procedures for developing hydrographs.

For detention facilities which outfall at a location on a channel where the upstream drainage area is less than 2,000 acres, the use of a constant tailwater elevation is allowed. For the 100-year storm, the tailwater elevation used should be two feet below the maximum 100-year water surface elevation in the detention pond or the maximum 100-year water surface elevation in the outfall channel, whichever is lower. For the 25-year storm, the tailwater elevation should be equal to the maximum 25-year water surface elevation in the outfall channel. In no instance, however, should the design tailwater elevation be less than that of the top (crown) of the outlet pipe.

Outflow Pipes or Culvert Outlet Structures

Figure 5.6 illustrates a typical detention outlet pipe. The minimum cross-sectional dimension is 24 inches. The minimum culvert slope is that required for a flow velocity of 3 fps at full gravity flow conditions. Detention outflow pipes are generally examples of culverts operating under outlet control conditions, and may thus be analyzed using the appropriate methods described in Chapter 4.

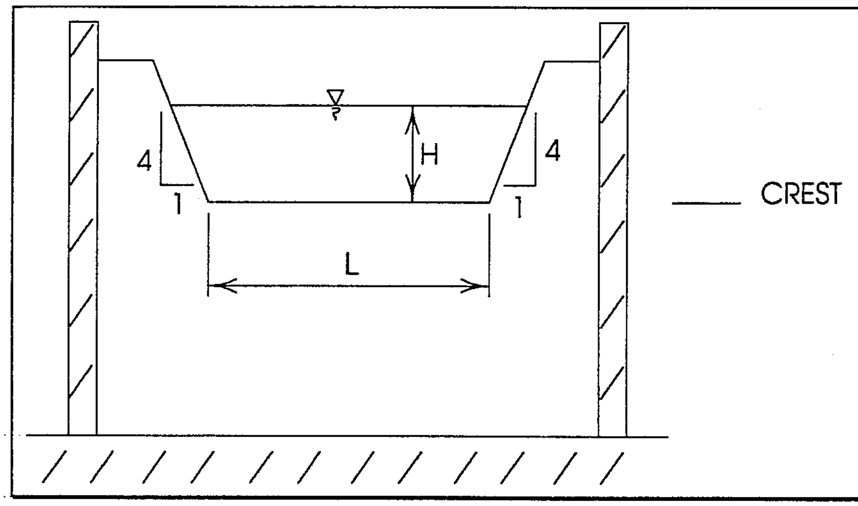
FIGURE 5.6 Typical Pipe or Box Culvert Outlet Structure



Horizontal Weir Outlet Structures

Figure 5.7 shows a typical sharp-crested horizontal weir. Horizontal weirs are useful when a large rate of discharge must be developed with a relatively small head loss.

FIGURE 5.7 Sharp-Crested Rectangular Weir



The 4:1 side slope on the sides of the weir offset the effects of end contractions and allow the full width of the weir (L) to be used in the weir flow equation:

Equation 5.4

$$Q = CLH^{3/2}$$

in which:

Q = the flow capacity of the weir (cfs).

C = the weir flow coefficient. Values are available in most hydraulics textbooks.

H = the head on the weir (ft), measured above the crest of the weir.

L = the length of the weir crest (ft.

Figure 5.8 shows a sharp-crested weir in cross-section view.

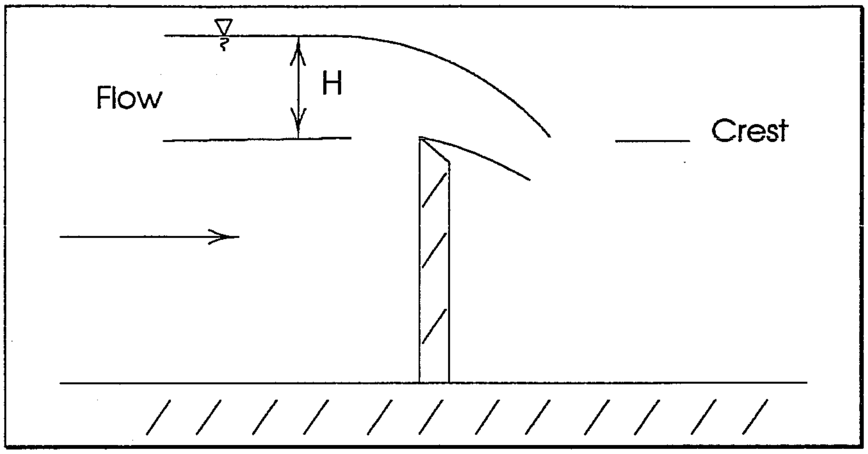


FIGURE 5.8 Section Through Sharp-Crested Weir

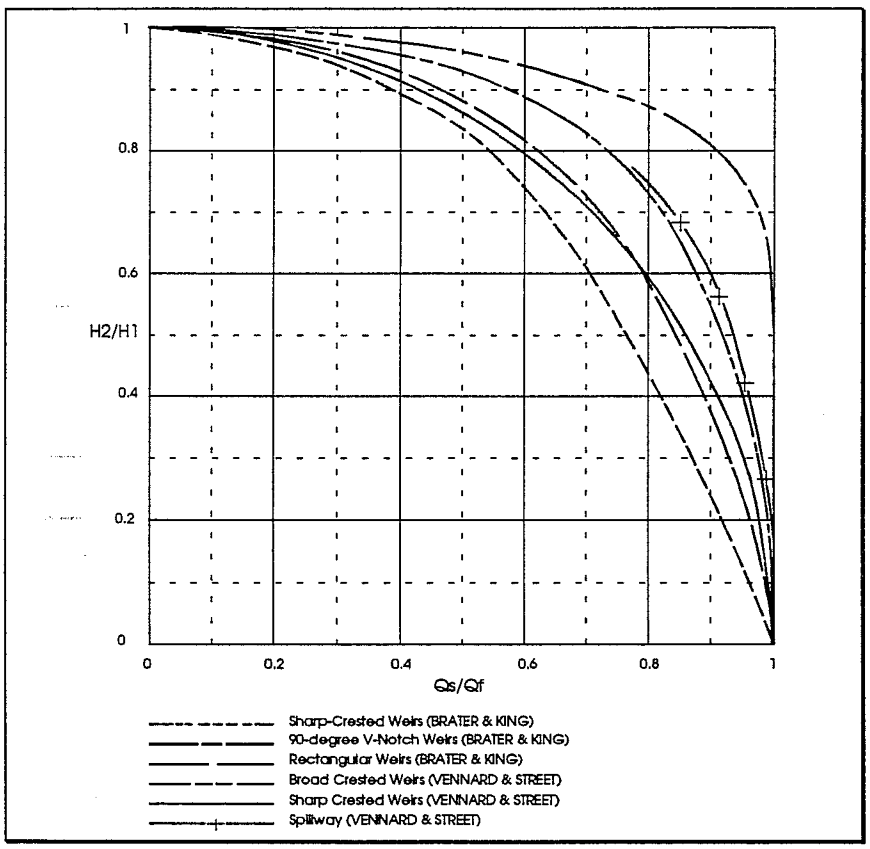


FIGURE 5.9 Capacity Adjustments for Submerged Weirs

The flow capacity of a weir decreases under tailwater conditions high enough to result in ***submergence***. Figure 5.9 illustrates the adjustments necessary to account for submergence for various types of weirs. In this figure, the following variables are used:

H2 = Weir head, measured using the water surface elevation on the upstream side of the weir

H1 = Weir head, measured using the water surface elevation on the downstream side of the weir

Qf = The weir flow capacity assuming no submergence

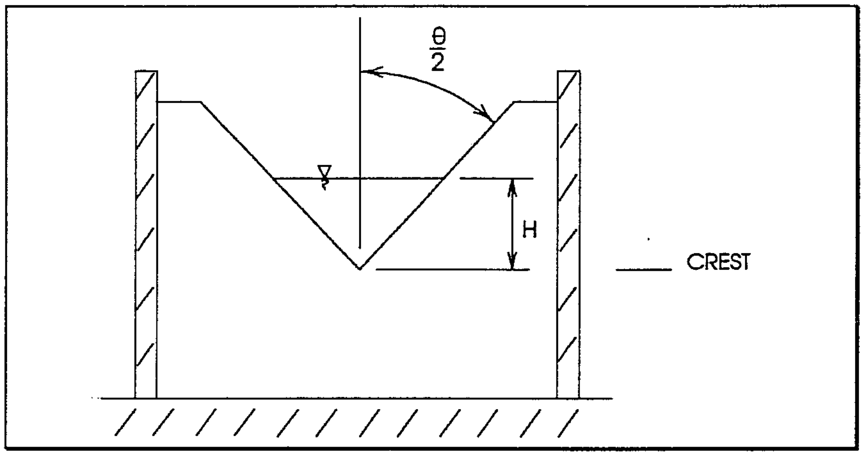
Qs = The weir flow capacity after correction for submergenc

The ratio H2/H1 represents the ratio of upstream to downstream head on the weir. This ratio is zero if the water surface elevation on the downstream side of the weir is at or below the weir crest elevation, and no submergence adjustment is made. When the water surface elevation on the downstream side of the weir is equal to the water surface elevation on the upstream side, no flow occurs.

V-Notch Weir Outlet Structures

Figure 5.10 illustrates a V-notch weir. These types of weirs are useful when the flow rate must increase more slowly with each increment of head.

FIGURE 5.10 Sharp-Crested V-Notch Weir



The following equation is used to compute the flow capacity of a V-notch weir:

Equation 5.5

$$Q = 2.5 \tan\frac{\theta}{2} H^{5/2}$$

in which:

Q = the flow capacity of the weir (cfs).

θ = the angle illustrated in Figure 5.10 (radians).

H = the head on the weir (ft), measured from the lowest point in the notch.

Orifice Outlet Structures

Orifices may be of any shape and may be used in lieu of pipes under certain conditions. Circular and rectangular orifices are illustrated in Figures 5.11 and 5.12, respectively. Orifices may be submerged or unsubmerged, as illustrated in Figures 5.13 and 5.14.

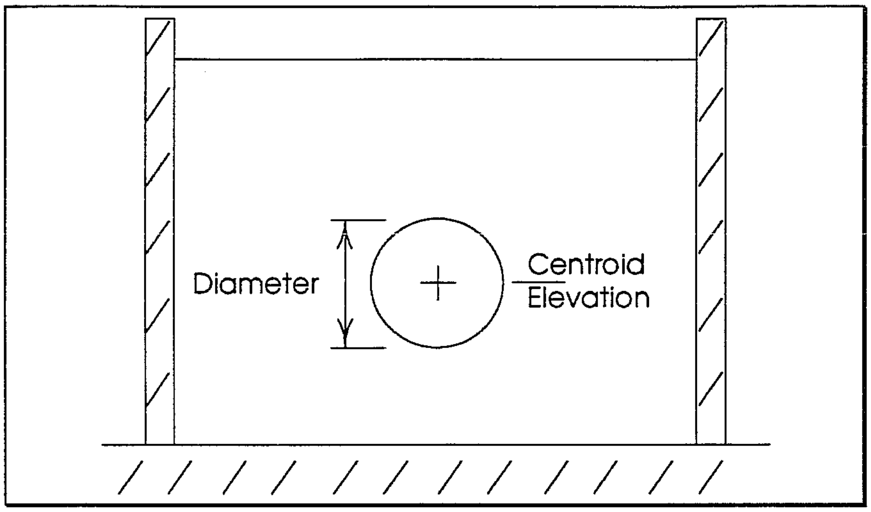


FIGURE 5.11 Circular Orifice

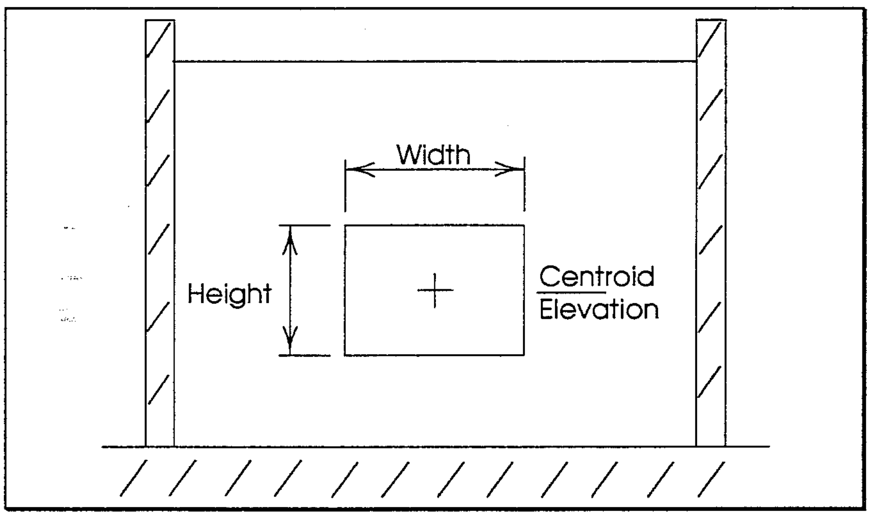


FIGURE 5.12 Rectangular Orifice

The capacity of an unsubmerged orifice may be computed using the following equation:

$$Q = CA\sqrt{2gh_1}$$ Equation 5.6

in which:

Q = the flow capacity of the orifice (cfs).

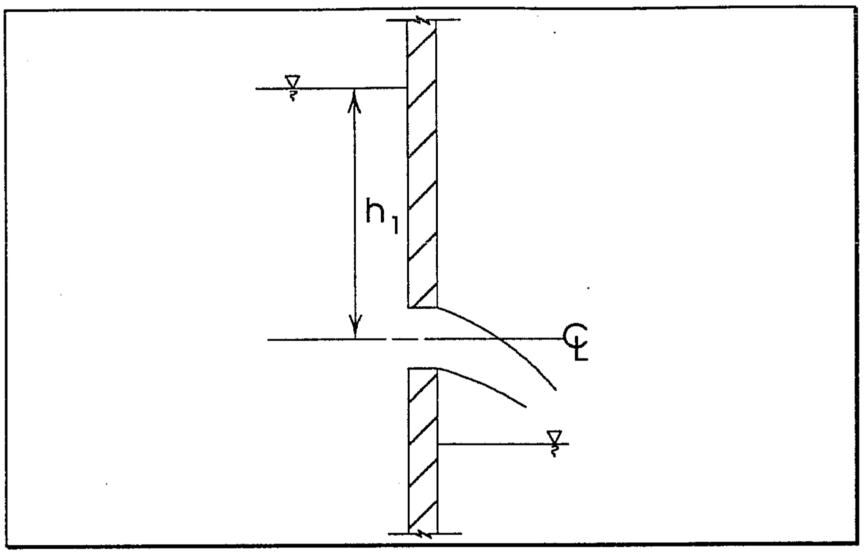
C = the orifice flow coefficient, which may be determined from most hydraulics textbooks.

A = the cross-sectional area of the orifice (sq ft).

g = acceleration of gravity = 32.2 ft/sec^2.

h_1 = head on the orifice (ft), measured from the centroid of the cross-sectional area (see Figure 5.13).

FIGURE 5.13 Unsubmerged Orifice



If the orifice is submerged, the following equation may be used to compute the capacity:

Equation 5.7

$$Q = CA\sqrt{2g(h_1 - h_2)} = CA\sqrt{2g\Delta h}$$

in which:

Q = the flow capacity of the orifice (cfs).

C = the orifice flow coefficient, which may be determined from most hydraulics textbooks.

A = the cross-sectional area of the orifice (sq ft).

g = acceleration of gravity = 32.2 ft/sec^2.

h_1 = the upstream head on the orifice (ft), measured from the centroid of the cross-sectional area.

h_2 = the downstream head on the orifice (ft), measured from the centroid of the cross-sectional area.

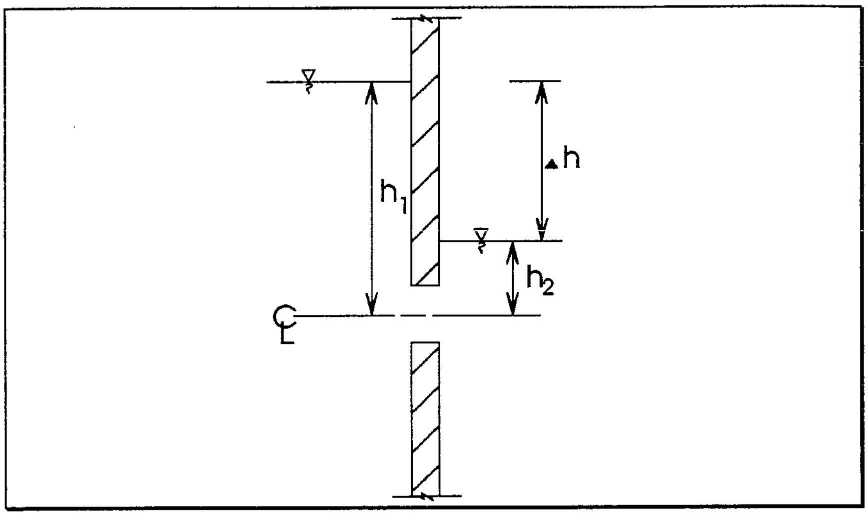


FIGURE 5.14 Submerged Orifice

Pipe Spillway with Riser Outlet Structures

Pipe spillways with risers are useful in certain situations. Figure 5.15 illustrates a typical riser pipe configuration. Hydraulically, the riser pipe acts as a weir as long as the outlet pipe has sufficient capacity to prevent submergence. The riser crest may be analyzed using the weir flow equation (Equation 5.4), with the head on the weir set equal to the distance *h* in the figure. When the riser pipe itself begins to fill, however, the entire structure begins to act as a pressure conduit and should be analyzed using the methods described in Chapter 4 for such conditions.

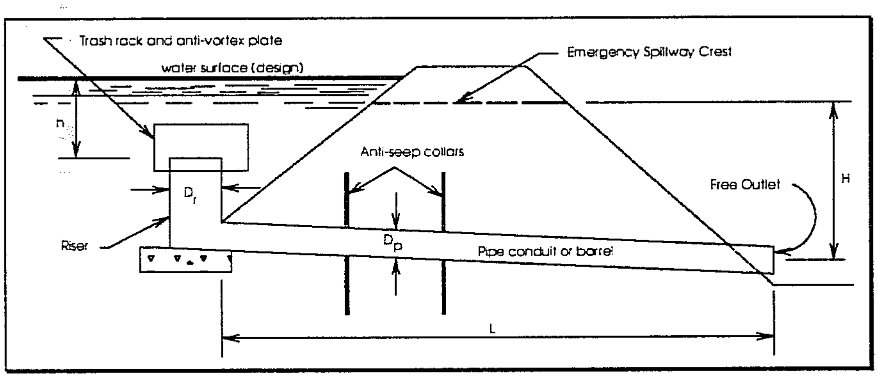


FIGURE 5.15 Pipe Outlet Structure with Drop Structure Inlet

Note: SCS Technical Note 210-15-TX1 provides further details on the design and analysis of riser pipe outlet structures.

Extreme Event Design

In order to accommodate extreme storm events which are less frequent than the 100-year design storm, an emergency overflow system shall be provided for all detention basins. This system, which may consist of an overflow swale, a weir, or other structure approved by the appropriate drainage regulatory authority, shall be designed to carry the 100-year allowable detention basin discharge at full-bank conditions (water surface elevation equal to minimum basin top of bank elevation). The emergency overflow system shall direct flows into an outfall channel and prevent flow in the direction of developed areas.

Pump Detention Facilities

Pumped detention systems will be approved for use only under the following conditions (taken from the *Criteria Manual for Design of Flood Control and Drainage Facilities in Harris County, Texas* [HCFCD, 1984]):

1. **Feasibility**: A gravity system is not feasible from an engineering and economic standpoint;
2. **Backup Pump**: At least two pumps are provided, each of which is sized to pump the design flow rate; if a triplex system is used, any two of the three pumps must be capable of pumping the design flow rate;
3. **Downstream Effects**: The selected design outflow rate must not aggravate downstream flooding. (Example: A pump system designed to discharge at the existing 100-year flow rate each time the system comes on-line could aggravate flooding for more frequent storm events).
4. **Security**: Fencing of the control panel is provided to prevent unauthorized operation and vandalism;
5. **Maintenance and Operation**: Adequate assurance is provided that the system will be operated and maintained on a continuous basis;
6. **Backup Power**: Emergency source of power is provided.

If a pump system is desired, review of the preliminary conceptual design by the appropriate drainage regulatory authority is recommended before any detailed engineering is performed.

Chapter 6. Property Rights

Introduction

This section describes the acquisition and transfer of property rights for flood control facilities.

The purposes of this chapter are to:

1. Define when property rights are required.
2. Establish the criteria for determining the areal extent of required property rights.
3. Define the type of property rights required.
4. Define the appropriate manner of transferring property rights.

The policy of the drainage regulatory agencies is to acquire the minimum property rights necessary for the operation and maintenance of flood control facilities. This generally consists of a dominant easement for flood control and drainage.

For facilities constructed by others who wish to transmit the constructed facility to a drainage regulatory agency for future operation and maintenance, the following conditions must be met:

1. **Consistent Criteria**: The facility must be constructed according to the criteria and specifications stated in this manual, or as amended by the applicable drainage regulatory authority.
2. **Consistent Purpose**: The facility must serve a flood control purpose which is consistent with the master plan for the watershed.
3. **Adequate Property Rights**: The facility must be transferred with sufficient property rights to allow the public entity to legally perform operations and maintenance activities.

In determining the property rights required, joint use and shared use of property should be considered. Such considerations should include but not necessarily be limited to transportation facilities, other utilities, recreational and open-space needs, and environmental mitigation or preservation.

Areal Extent of Property Rights

This section states the minimum right-of-way sizes for various drainage facilities, including:

- Open Channels
- Detention Basins
- Closed Conduits
- Overland Flow Corridors
- Access Easements or Corridors

Open Channels

The amount of right-of-way required for open channels shall be based on full development of the watershed and is dependent on channel top width and channel type (grass-lined or concrete-lined) as required to accommodate the discharge resulting from the 100-year, 24-hour rainfall event. Adequate area must be set aside for both the channel itself and the adjacent berm required for channel maintenance. Minimum right-of-way requirements include the channel top width from bank to bank plus the maintenance berm areas on both sides and shall be dedicated at the time of platting of the adjacent property. However, if additional right-of-way is required to serve upstream development prior to downstream platting, sufficient right-of-way must be dedicated to accommodate the improved channel and provide adequate maintenance berms. See Table 6.10.

TABLE 6.1 Channel Right-of-Way Requirements

Channel Type	Top Width	Maintenance Berm Width
Grass-Lined	Less than 30 feet	15 feet on both sides
Grass-Lined	30 feet to 60 feet	20 feet on both sides
Grass-Lined	60 feet or greater	30 feet on both sides
Concrete-Lined	All	10 feet on one side, 20 feet on one side

Note: If concrete lining does not extend all the way up to the top of bank, the maintenance berm requirements for grass-lined channels apply.

Detention Basins

A 30-foot wide access and maintenance easement shall be provided around the top of the entire detention pond. This is in addition to the dedication required for the pond itself.

Closed Conduit Systems

This section describes the total property width required for enclosed systems.

Storm sewers shall be located in public street rights-of-way or in easements adjoining and parallel to a street right-0of-way. The location of storm sewers shall not be within side lot or back lot easements than prohibit future maintenance access unless approved by the applicable drainage regulatory authority.

For storm sewers located adjacent to public or semi-public permanent rights-of-way such as: drainage district ditch, county drainage ditch, municipal drainage ditch, lighting and power company, public street or road, and pipeline companies easements and/or fee strips, the following are the basic easement width requirements:

1. The basic minimum width shall be ten feet (10').
2. For storm sewers greater than ten feet (10') and less than fifteen feet (15') in diameter or width, the minimum width of the easement shall be twenty-five feet (25').
3. For storm sewers greater than fifteen feet (15') in diameter or width, the minimum width of the easement shall be determined by the applicable drainage regulatory authority.

4. For storm sewers whose depth to the flow-line is greater than fifteen feet (15'), add five feet (5') to the minimum easement width specified above.

5. For all easements specified in this list, a minimum distance of five feet (5') must be maintained from the easement line to the outside edge of the storm sewer, and a minimum distance of two feet (2') shall be maintained from the right-of-way line to the outside edge of the storm sewer.

Where approvals are granted for easements restricted to storm sewers located as side lot or back lot easements, the following are the basin easement width requirements:

1. The basic minimum width shall be twenty feet (20') with the storm sewer centered in the easement.

2. For storm sewer greater than ten feet (10') and less than fifteen feet (15') in diameter or width, the minimum width of the easement shall be twenty-five feet (25').

3. For storm sewers greater than fifteen feet (15') in diameter or width, the minimum width of the easement shall be determined by the applicable drainage regulatory authority.

4. For storm sewers whose depth to the flow-line is greater than fifteen feet (15'), add five feet (5') to the minimum easement width specified above.

5. For all easements specified above, a minimum distance of five feet (5') must be maintained from the easement line to the outside edge of the storm sewer.

Where approvals are granted for special use or combination easements located as side lot or back lot easements, the following are the basic easement width requirements:

1. The basic minimum width shall be twenty-five feet (25').

2. For storm sewer greater than ten feet (10') and less than fifteen feet (15') in diameter or width, the minimum width of the easement shall be twenty-five feet (25').

3. For storm sewers greater than fifteen feet (15') in diameter or width, the minimum width of the easement shall be determined by the applicable drainage regulatory authority.

4. For storm sewers whose depth to the flow-line is greater than fifteen feet (15'), add five feet (5') to the minimum easement width specified above.

5. The additional easement requirements specified above for larger and deeper storm sewers shall also be applied to the basic easement width immediately above.

In all easements restricted to storm sewers, the storm sewer shall be centered within the limits of the easement.

All cast-in-place concrete storm sewers shall follow the alignment of the right-of-way or easement.

Overland Flow Corridors

The total property width required for sheet flow swales and other overland flow swales shall be a minimum of 30 feet.

Access Easements or Rights-of-Way

Some right-of-way width may occasionally be required for access to drainage facilities which are not otherwise conveniently accessible. In such cases, the right-of-way width shall be a minimum of 20 feet.

Chapter 7. Erosion and Sediment Control

This section provides recommended criteria for control of channel erosion and siltation. The primary objective is to maintain the capacity and stability of open channels. The erosion potential must be addressed in all designs of open channels or hydraulic structures.

Erosion Control Requirements

A good grass cover must be established on all areas within the right-of-way (except the channel bottom) disturbed by channel improvements or by any type of construction. An adequate grass cover on the banks helps stabilize the channel and minimizes erosion caused by overbank flow and high velocities in the channel.

Establishing a good grass cover requires preparing the seedbed, seeding properly, keeping the seed in place, fertilizing, and watering regularly. As a minimum requirement, the Harris County Flood Control District specification entitled *Hydro Mulch Seeding* must be followed on all reseeding operations. Other methods of retaining the soil and seeds such as asphalt mulch, jute mesh, or paper mesh may be used with prior approval from the appropriate drainage regulatory agency. Solid sodding or sprigging are two recommended methods in areas where hydro mulch may not be successful.

Geotechnical Considerations

Two basic considerations which must be analyzed are the flow characteristics in terms of velocity and turbulence, and the properties of the affected soils. For all new channels and for major channel improvements, a soils report which addresses erosion and slope stability must be submitted. Erosion protection should be installed in areas where recommended by the geotechnical engineer. For minor channel alterations, check with the appropriate drainage regulatory agency to determine whether erosion control measures are necessary.

Channel Parameter Considerations

Channel erosion is generally caused by excessive velocities in the channel; by flow over the banks of the channel; or by secondary flows at junctions, bends, and transitions. Each of these sources of erosion can be minimized if erosion protection measures are included in the design and construction of the channel and its appurtenances. Adequate grass cover or a structural lining in the channel often can minimize problems due to excessive velocities and secondary flow. Backslope drainage systems can intercept flow within the right-of-way to prevent overbank flow and erosion.

Erosion protection is necessary to insure that channels maintain their capacity and stability and to avoid excessive transport and deposition of eroded material. The three main parameters which affect erosion are vegetation, soil type, and the magnitude of flow velocities and turbulence. In general, silty and sandy soils are the most vulnerable to erosion.

The necessity for erosion protection should be anticipated in the following settings:

1. **Channel Bends**: Areas of channel curvature, especially where the radius of the curve is less than three times the design flow top width.
2. **Bridges**: Around bridges where channel transitions create increased flow velocities.
3. **Steep Sections**: When the channel invert is steep enough to cause excessive flow velocities.
4. **Sheet Flow**: Along grassed channel side slopes where significant sheet flow enters the channel laterally.
5. **Channel Confluences**: Where tributaries enter a channel.
6. **Erosion-Prone Soils**: In areas where the soil is particularly prone to erosion.

Sound engineering judgement and experience should be used in locating areas which require erosion protection. It is often prudent to analyze potential erosion sites following a significant storm event to pinpoint areas of concern.

Requirements for Channel Confluences

Figure 7.1 presents the minimum requirements for determining when erosion protection or channel lining are necessary given the angle of the confluence of two channels. A healthy cover of grass must also be established from the top edge of the lining to the top of the channel bank. The top edge of the lining shall extend to the 25-year water surface elevation.

TABLE 7.1 Minimum Erosion Protection for Channel Confluences

Source: (HCFCD, 1984) Note: See Figure 7.1 for illustration of θ.

25-Year Velocity in Side Channel (fps)	Angle of Intersection θ	
	15 to 45 degrees	45 to 90 degrees
4 or more	Protection Required	Protection Required
2 to 4	No Protection Required	Protection Required
2 or less	No Protection Required	No Protection Required

TABLE 7.2 Minimum Extent of Erosion Protection for Channel Confluences

Source: (HCFCD, 1984) Note: See Figure 7.1 for illustration of *a*, *b*, *R*, and *T*.

Location	Minimum Distance
a	20 ft
b	larger of 50 ft or 0.75 T_m / tan θ
c	20 ft

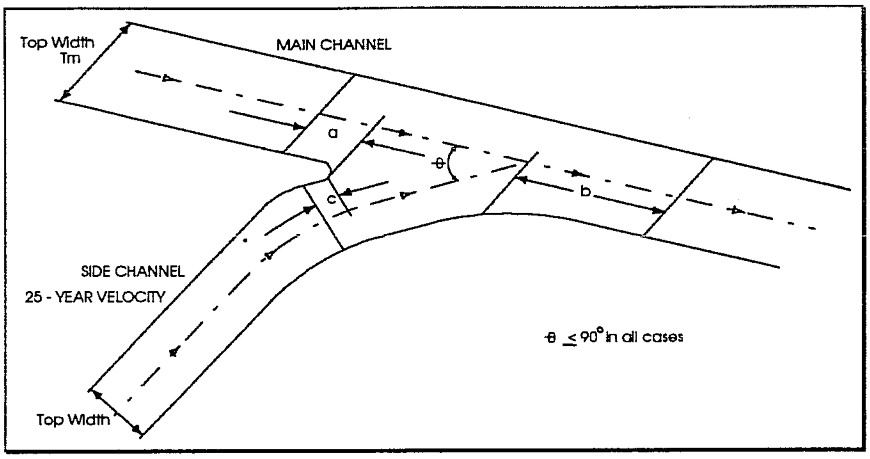


FIGURE 7.1 Typical Channel Confluence

Requirements for Channel Bends

Slope protection is required for channel bends with a radius of curvature measured from the center-line of less than three times the top width of the ultimate channel. When required, erosion protection must extend along the outside bank of the bend and at least 20 feet downstream of it. Additional protection on the channel bottom and inside bank, or beyond 20 feet downstream, will be required if maximum allowable velocities are exceeded. See Table 7.4 for allowable 25-year flow velocities. Figure 7.2 illustrates the minimum erosion protection requirements for channel bends.

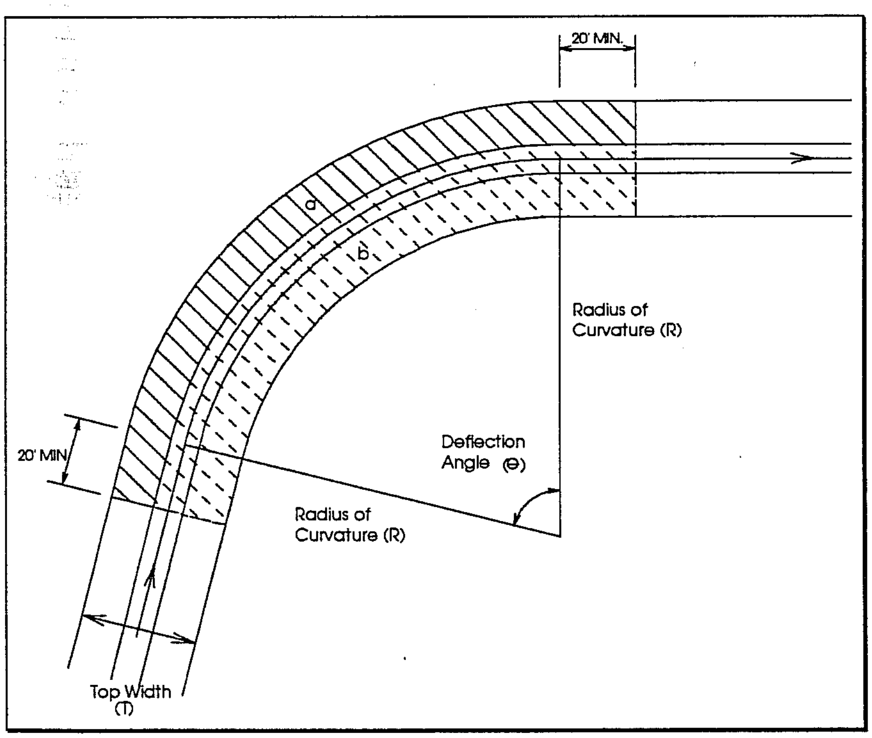


FIGURE 7.2 Typical Channel Bend

TABLE 7.3 Minimum Erosion Protection for Channel Bends

Source: (HCFCD, 1984) Note: See Figure 7.2 for illustration of *a, b, R,* and *T.*

Location	Erosion Protection Requirements
a	Slope protection required if $R/T \leq 3.0$, or if 25-year flow velocities exceed allowable values given in Table 7.4
b	Slope protection required if 25-year flow velocities exceed allowable values given in Table 7.4

TABLE 7.4 Allowable 25-Year Flow Velocities for Channel Design

Source: (HCFCD, 1984)

Channel Description	Average Velocity (fps)	Maximum Velocity (fps)
Grass Lined: Predominantly Clay Soil	3.0	5.0
Grass Lined: Predominantly Sand Soil	2.0	4.0
Riprap Lined	5.0	8.0
Concrete Lined	6.0	10.0

Requirements for In-Channel Energy Dissipators

This section describes the design procedures for the following types of in-channel energy dissipators:

1. Straight-Drop Spillways
2. Sloping Drops
3. Baffled Chutes

Requirements for Straight-Drop Spillway

The straight drop spillway is commonly installed in drainage channels to adjust channel gradients which are too steep for design conditions. This type of spillway design is based on the hydraulics of the aerated free-falling nappe. Figure 7.3 illustrates the configuration of a typical straight drop spillway constructed of steel sheet piling.

FIGURE 7.3 Cross-Section of Typical Straight Drop Spillway

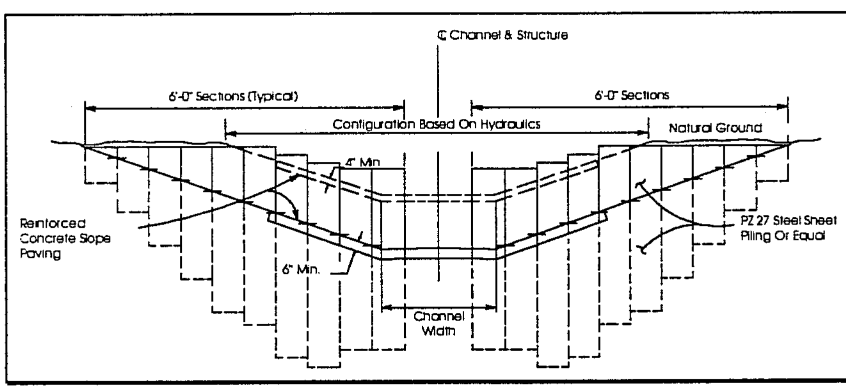
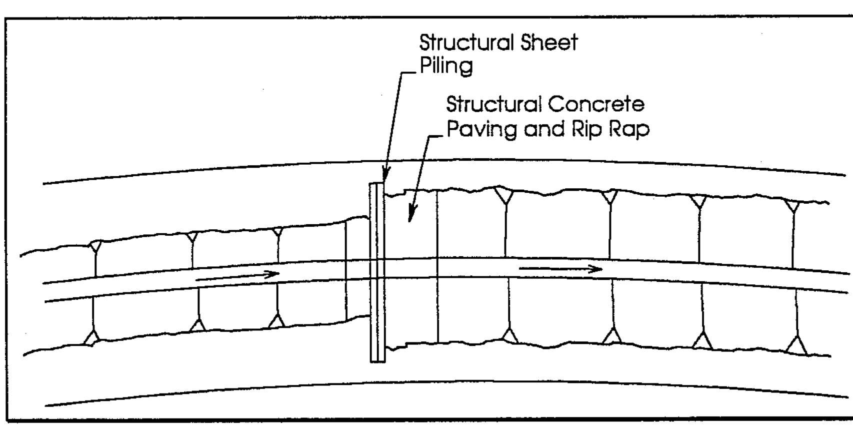


FIGURE 7.4 Plan View of Typical Straight Drop Spillway



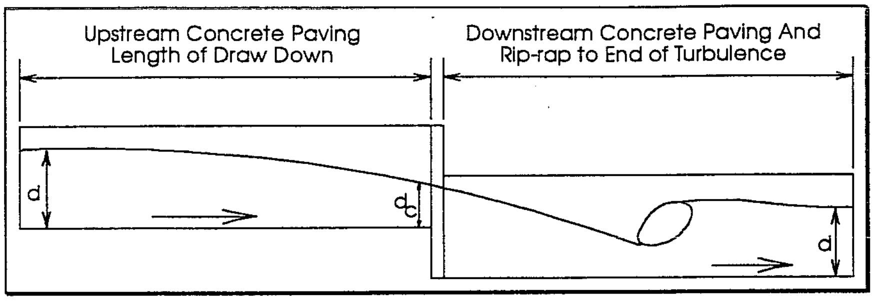


FIGURE 7.5 Profile View of Typical Straight Drop Spillway

Hydraulics of Straight-Drop Spillways

Figure 7.6 illustrates the flow geometry of a straight drop spillway. The aerated free falling nappe in a straight drop spillway will reverse its curvature and turn smoothly into super-critical flow on the apron. The flow geometry at straight drop spillways can be described by functions of the ***drop number***, *D*, which is defined as:

$$D = \frac{q^2}{gh^3} \qquad \text{Equation 7.1}$$

in which:

q is the discharge per unit width of the crest of overfall.

g is the acceleration of gravity (32.2 ft/sec^2).

h is the height of the drop.

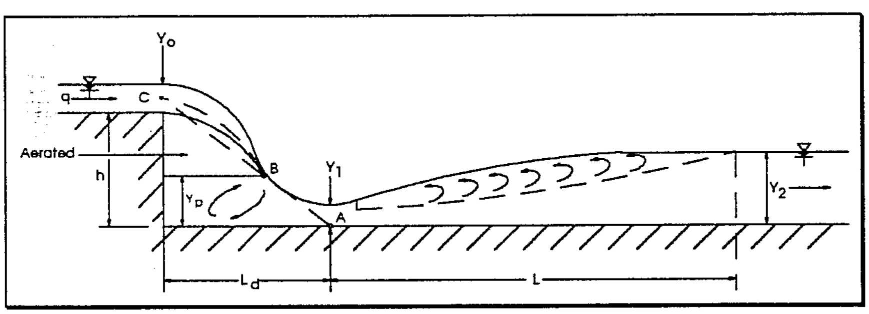


FIGURE 7.6 Flow Geometry of Straight Drop Spillway

The functions of the drop number used to describe the flow geometry are:

$$\frac{L_D}{h} = 4.30D^{0.27} \qquad \text{Equation 7.2}$$

$$\frac{Y_P}{h} = 1.00D^{0.22} \qquad \text{Equation 7.3}$$

$$\frac{Y_1}{h} = 0.54D^{0.425} \qquad \text{Equation 7.4}$$

$$\frac{Y_2}{h} = 1.66D^{0.27} \qquad \text{Equation 7.5}$$

in which:

L_D = drop length, or distance from the drop to the position of the depth Y_1

Y_p = pool depth under the nappe

Y_1 = depth at the toe of the nappe or the beginning of the hydraulic jump

Y_2 = the tailwater depth sequent to Y_1.

If the tailwater depth is less than Y_2, the hydraulic jump will recede downstream. If the tailwater depth is greater than Y_2, the jump will be submerged. As the tailwater rises, the spillway crest may finally be submerged. The spillway will still be effective if the submergence does not reach the control depth on the spillway crest.

These relationships consider the flow at the straight drop spillway to be a two-dimensional flow that practically corresponds to the flow near the center of a wide channel. That is, the spillway crest is assumed to be the same width as the channel. The design of the approach channel should analyze carefully the effects of any end contractions which may cause the ends of the nappe to land beyond the basin apron and the side walls. The flow geometry at the drop depends on the discharge per unit width, q, the height of drop, h, and the depth of uniform flow in the channel upstream and downstream from the drop structure. Aeration of the nappe is important to the proper functioning of the drop and the structural stability of the drop structure.

Experimental studies have demonstrated that the depth, Y_O, at the brink of the drop is approximately 70 percent of critical depth, and that critical depth actually occurs a distance of about four times the critical depth upstream from the brink. For example, a typical channel section carrying 3,000 cfs has a critical depth of 6 to 8 feet. If the drop structure opening was designed with the same dimensions as the channel, critical depth could occur 30 to 40 feet upstream of the brink. Due to this draw down, velocities at critical depth and upstream of critical depth would be in excess of acceptable velocities for grass-lined channels.

In order to avoid excessive drawdown of the upstream water surface and resulting high velocity, the opening in the structure must have less cross-sectional area than the channel. Critical depth is a function of the discharge rate and geometry. By reducing the area of the opening, critical depth will be forced to occur at the structure rather than in the upstream channel, thereby creating a backwater condition. The structure opening can be designed from the bottom up using a range of flows in an iterative process. Special attention must be given to the energy grade line during design. An analysis of the water-surface drawdown should be performed to determine the limits of erosion protection required upstream of the structure.

The length of the downstream hydraulic jump cannot be easily determined by theory, but it has been investigated experimentally. Various technical references give experimental results for jump lengths. Figure 7.7 illustrates a curve based on data and recommendations of the U.S. Bureau of Reclamation [USBUREC, 1984]. This curve allows the determination of jump lengths in rectangular channels. In the absence of more extensive data, this curve may be used to approximate jump lengths in trapezoidal channels.

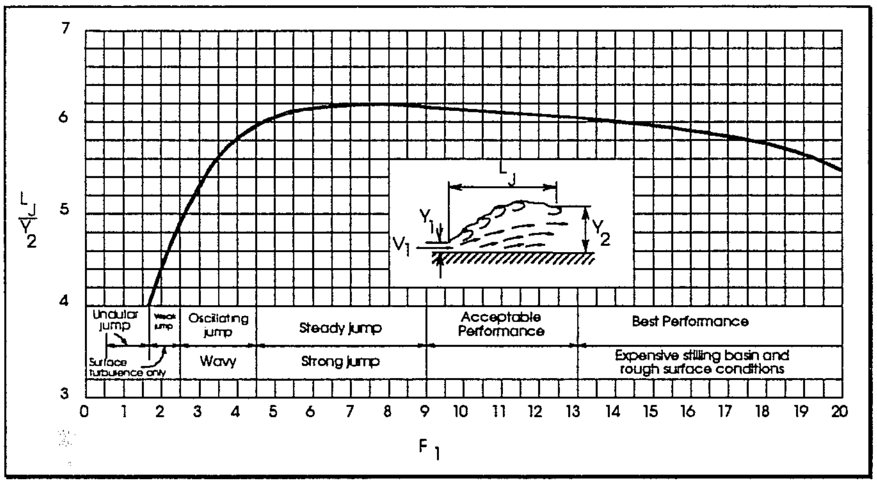


FIGURE 7.7 Length of Hydraulic Jumps

Design Procedure for Straight Drop Spillways

The following procedure may be used to design straight drop spillways:

1. **Design Flow Rate**: Determine the peak 100-year flow rate at the drop structure. This may be accomplished through a HEC-1 analysis or by using drainage area versus peak flow rate curves for the particular watershed in which the stream under consideration is located.

2. **Channel Hydraulics**: Compute the normal depth (D_N), flow velocity (V_N), and energy head (H_N) in the channel upstream of the structure for 20% of the peak 100-year flow rate using Manning's Equation.

3. **Determine Lowest Opening Width**: Design the lowest opening of the drop for 20% of the peak 100-year flow using the following equations:

$$d_c = \frac{2}{3} H_N$$ Equation 7.6

$$V_C = \sqrt{d_c g}$$ Equation 7.7

$$A = \frac{Q}{V_c}$$ Equation 7.8

Equation 7.9

$$W_1 = \frac{A}{D_c}$$

in which:

d_c = approximate depth of critical flow

V_c = approximate velocity of critical flow

g = acceleration due to gravity (32.2 ft/sec/sec)

A = area of flow

W_1 = width of single opening to produce necessary area for critical flow.

4. **Determine Other Opening Widths**: Determine opening widths for several percentages of the 100-year flow by completing steps 2 and 3 for each flow rate. Use the following equation to determine the widths of multiple openings:

Equation 7.10

$$W_i = \frac{A_i - A_{i-1}}{d_{ci} - d_{ci-1}}$$

in which:

W_i = width of successive openings in drop structure, when more than one are used

d_{ci} = depth of critical flow for successive openings in drop structure

A_i = area of successive openings in drop structure

5. **Design Openings**: Use the results of steps 2 through 4 to determine the dimensions of a number of openings (usually from 2 to 5) in the sheet piling which will yield a configuration similar to that indicated by those results.

6. **Compute Profiles**: Use HEC-2 to compute water surface profiles in the channel for several different flows. Adjust the drop structure configuration as necessary to yield the desired water surface elevations and velocities upstream of the drop.

7. **Upstream Slope Protection**: Examine the HEC-2 results to determine the required length of upstream slope protection. The slope protection should extend far enough upstream to reach a point where the flow velocity is below the accepted maximum for all flows (see Chapter 4). The minimum length of upstream protection is 40 feet. At least 20 feet of the total length should be riprap, with the balance being concrete slope paving. The riprap should be placed upstream of the concrete paving.

8. **Downstream Slope Protection**: Compute the drop number and functions of the drop number using equations 7.2

through 7.5 (all parameters are defined on Figure 7.6). Using the depth at the toe of the nappe (Y_1), compute the area at the toe of nappe (A_1) and the corresponding velocity (V_1). Then, compute the Froude number (F_1) using the formula:

$$F_1 = \frac{V_1}{\sqrt{gY_1}}$$ Equation 7.11

Using the Froude number F_1 to determine a value of L_J/Y_2 from the curve on Figure 7.7 illustrating the relationship between these parameters, compute the jump length L_J using the values of L_J/Y_2 and Y_2 which have already been determined. Compute the total length of slope protection required by combining the drop length L_D with the jump length L_J.

$$L_{SP} = L_D + L_J$$ Equation 7.12

Repeat this procedure for several percentages of the peak 100-year flow rate. Choose the maximum value of L_{SP} as the required length of slope protection.

9. **Downstream Riprap**: Break the total length of downstream slope protection into a length of concrete slope paving and a length of riprap. The minimum total length of slope protection downstream of a straight drop structure is 50 feet, with a minimum of 20 feet of riprap included in the total. The riprap should be placed downstream of the slope paving on the downstream side of the drop structure.

Example of Straight Drop Spillway Design

This section provides an example of the procedure recommended for designing a straight drop spillway. The procedure below is based on the channel and drop structure shown in Exhibits 7.3 and 7.4. For this example, a 5-foot vertical drop is to be accommodated in a channel with upstream bottom width = 15 feet, side slopes = 3:1 (H:V), channel invert slope S = 0.08%, and Manning's n-Value = 0.04.

1. **Design Flow Rate**: Determine the 100-year frequency flow to the design point from all existing and future contributing drainage areas for 100% development. For this example the drainage area is 3,500 acres, and the 100-year flow is 2,000 cfs. (See Chapter 3 for determination of flow.)

2. **Channel Hydraulics**: Determine the normal depth, D_N, normal velocity, V_N, and energy head, H_N, in the channel upstream of the structure for various percentages of the 100-year flow using Manning's Equation:

$$Q = \frac{1.486}{n} AR^{2/3}\sqrt{S}$$ Equation 7.13

Rearranging and solving for $AR^{2/3}$ yields:

Equation 7.14

$$AR^{2/3} = \frac{nQ}{1.486\sqrt{S}}$$

For 20% of the 100-year flow rate,

Equation 7.15

$$AR^{2/3} = \frac{400 \times 0.04}{1.486 \times \sqrt{0.0008}} = 380.7$$

Determine the normal depth, D_N, using hydraulic tables, graphs, or by trial and error using equations for A and R. For this example, $D_N = 5.38$ ft, and the corresponding area, $A = 168$ sq ft. After determining normal depth, the normal velocity, V_N is determined using the following equation:

$$V_N = \frac{Q}{A} = \frac{400}{168} = 2.38$$

The energy head at normal depth, H_N, is computed as follows:

$$H_N = D_N + \alpha \frac{V_N^2}{2g} = 5.38 + 1.1 \times \frac{(2.38)^2}{2 \times 32.2} = 5.47$$

In this equation, α = the energy coefficient. For trapezoidal channels, $\alpha = 1.1$.

Table 7.5 lists the computed values for Normal Depth, Normal Velocity, and Velocity Head for various flow rates up through the 100-year flow rate.

TABLE 7.5 Example of Computed Hydraulic Values

Q (%)	Q (cfs)	$AR^{2/3}$	D_N (ft)	V_N (fps)	H_N (ft)
20%	400	380.7	5.38	2.38	5.48
40%	800	761.4	7.46	2.86	7.60
60%	1,200	1,142.0	8.99	3.18	9.16
80%	1,600	1,522.7	10.23	3.42	10.43
100%	2,000	1,903.4	11.29	3.62	11.51

3. **Determine Lowest Opening Width**: Design an opening of the straight drop spillway for 20% of the 100-year flow, using Equations 7.6 through 7.9. Note: The equations for critical depth and critical velocity are approximations which simplify initial calculations. These results will be checked later.

   For $Q_{20\%}$ = 400 cfs, H_n = 5.48 ft. Using Equation 7.6, $d_c = 5.48 \times 2/3 =$ 3.65 ft. Equation 7.7 is used to determine

   $V_c = \sqrt{3.65 \times 32.2} = 10.84$ fps. Equation 7.8 is applied to compute the Area, A = 400/10.84 = 36.9 sq ft. Therefore, the width of the lowest opening, W_1 = 36.9/3.65 = 10.1 ft.

4. **Determine Other Opening Widths**: Similar computations are performed for 40%, 60%, 80%, and 100% of the 100-year peak flow rate. Equation 7.9 is used to compute successive widths of multiple openings. Table 7.6 lists the computed widths for each flow rate.

Q (cfs)	H_N (ft)	d_c (ft)	V_c (fps)	A (sq ft)	W_1 (ft)	W_t (ft)
400	5.5	3.7	10.8	36.9	10.1	10.1
800	7.6	5.1	12.8	62.5	12.3	17.9
1,200	9.2	6.1	14.0	85.4	13.9	22.9
1,600	10.4	7.0	15.0	106.7	15.3	23.6
2,000	11.5	7.7	15.8	126.9	16.5	28.8

TABLE 7.6 Example of Computed Opening Widths

5. **Design Openings**: For more practical construction, the five opening widths listed in Table 7.6 are simplified to the three widths listed in Table 7.7.

Depth (ft)	Width (ft)
4	10
7	22
9	28

TABLE 7.7 Example of Simplified Opening Widths

6. **Compute Profiles**: As noted, the critical depth computed in Steps 3) and 4) above are based on an approximation. The critical depth for use in backwater calculations may be calculated by solving the following equation:

$$\alpha \frac{Q^2}{g} = \frac{A^3}{B}$$

Equation 7.16

in which:

Q = the flow rate in cfs

A = the actual area of drop structure opening at the trial depth

B = the top width of the opening at the trial depth

g = the acceleration due to gravity

α = energy coefficient

For a flow rate of 2,000 cfs, $\alpha Q^2/g = 1.1 \times (2000)^2/32.2 = 136{,}645$ ft^5. By solving for A, the critical depth of 8.80 produces $(10 \times 4 + 22 \times 3 + 28 \times 1.80)^3/28 = 136{,}632$ ft^5. Therefore, 8.80 ft is the beginning 100-year water surface elevation at the drop structure.

7. **Upstream Slope Protection**: Analyze the upstream water surface profiles and determine the point upstream of the drop structure where velocities fall below the maximum allowed as given in Table 7.4. A range of flow conditions should be checked. However, the 100-year flow generally results in the highest upstream velocities. Assume that Table 7.0 lists the results of a HEC-2 analysis of the channel upstream of the drop structure. These results are computed assuming a contraction loss coefficient of 0.6.

TABLE 7.8 Example HEC-2 Results for Upstream Channel

Upstream Channel Station (ft)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Depth (ft)	Flow Velocity (fps)
0+00	108.8	111.59	8.8	12.78
0+20	112.9	113.14	12.88	3.8
0+40	113.38	113.51	13.36	2.71
10+00	113.75	113.74	12.78	2.93

It is apparent that the velocities are low enough such that there should be no erosion problem beyond 40 feet upstream. Therefore, the concrete slope protection should extend 20 feet upstream with 20 feet of riprap beyond that.

8. **Downstream Slope Protection**: The final consideration in the hydraulic design of the straight drop structure is the design of downstream slope protection. The length of slope protection is the sum of the drop length, L_D, and the length of the hydraulic jump, L_J. The greatest drop length, L_D, is determined using Equations 7.2 through 7.5. Table 7.7 lists the results.

TABLE 7.9 Example of Computed Drop Lengths

Q (cfs)	W (ft)	q (cfs/ft)	h (ft)	D	L_D (ft)
400	10	40.0	5.0	0.3972	16.75
800	22	36.4	9.0	0.0564	17.80
1,200	22	54.5	9.0	0.1265	22.14
1,600	22	72.7	9.0	0.2252	25.87
2,000	28	71.4	12.0	0.0916	27.06

The length of the hydraulic jump is determined using Figure 7.7. The jump is dependent on the Froude number, which is computed using Equation 7.11. The Froude number will be greatest for the 100-year flow:

$$Y_1 = 0.54D^{0.425}h = 0.54 \times (0.0916)^{0.425} \times 12.0 = 2.35 \text{ ft}$$

$$Y_2 = 1.66D^{0.27}h = 1.66 \times (0.0916)^{0.27} \times 12.0 = 10.44 \text{ ft}$$

The cross-sectional area at the jump may be computed using the following equation:

$$A = Y_1 \times \frac{W_T + W_B}{2} = 2.35 \times \frac{(15 + 6 \times 2.35) + 15}{2} = 51.8$$

in which:

W_T = top width of flow (feet)

W_B = bottom width of channel (feet)

The velocity at the jump, $V_1 = Q/A = 2000/51.8 = 38.6$ fps

Therefore, the Froude number may be computed using Equation 7.11:

$$F_1 = \frac{V_1}{\sqrt{gY_1}} = \frac{38.6}{\sqrt{32.2 \times 2.35}} = 4.44$$

According to Figure 7.7, $L_J/Y_2 = 5.9$. Therefore, the length of the hydraulic jump $L_J = L_J/Y_2 \times Y_2 = 5.9 \times 10.44 = 61.6$ ft. Total length of slope protection should then be 61.6 ft + 27.1

ft = 88.7 ft, or about 90 feet downstream of the drop structure.

9. **Downstream Riprap**: The slope protection should consist of 70 feet of 6-inch concrete slope paving and 20 feet of riprap.

Requirements for Sloped Drops

Sloped drop structures are recommended when the required drop elevation is small, generally from 1 to 4 feet. They tend to be the most economical and topographically versatile means to accomplish a drop. Sloped drops should be no steeper than 2:1 and no flatter than 4:1 (measured along the channel invert).

Sloped drops shall be constructed of concrete slope paving or of cellular concrete articulated mats. Riprap or appropriate alternate erosion protection shall be provided upstream and downstream of the drop.

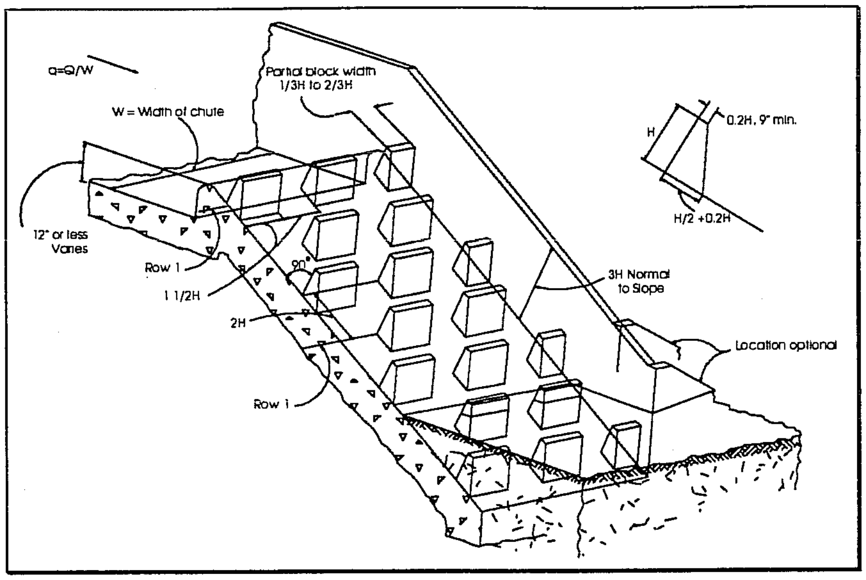
When sub-critical flow approaches a drop, depth decreases and velocity increases as the flow nears critical depth. Accordingly, appropriate erosion protection must be provided sufficiently upstream such that flow velocities are not excessive in any unprotected reach of channel. The minimum recommended distance is 20 feet.

Downstream of the drop, the required length for protection is dependent on the length of the hydraulic jump. As a rough estimate the jump length may be assumed equal to $q/2$, one-half of the design flow per unit width of channel. The use of riprap or a combination of riprap and concrete slope paving is recommended downstream of the drop to force the jump closer to the drop. A minimum of 20 feet of riprap is required downstream of any slope paving used at a drop structure to help reduce velocities and protect the concrete toe. The minimum recommended length of slope paving downstream of a sloped drop is 40 feet.

Requirements for Baffled Chutes

Baffled chutes are used in drainage ways when a relatively large change in elevation is necessary. The baffle blocks prevent undue acceleration of the flow as it passes down the chute. Baffled chutes are generally laid out on a 2:1 slope (no steeper) and can be designed to discharge up to 60 cfs per foot of channel width. The lower end of the chute is constructed to below stream bed level and backfilled as necessary, thereby minimizing degradation or scour of the stream bed. No tailwater or stilling basin is required, as velocities will remain moderate. Figure 7.8 illustrates a baffled chute.

FIGURE 7.8 Typical Baffled Chute



The following simplified step-by-step procedure developed by the Bureau of Reclamation [USBUREC, 1961] is recommended for the design of baffled chutes. Bureau of Reclamation Engineering Monograph No. 25 [USBUREC, 1984] contains an even more detailed discussion. A step-by-step design procedure is presented below:

1. **Design Discharge**: The baffled apron should be designed for the 100-year discharge, Q. The unit discharge $q = Q/W$ may be as high as 60 cubic feet per second per foot of chute width, W. Less severe flow conditions at the base of the chute exist for 35 cubic feet per second and a relatively mild condition occurs for unit discharges of 20 cubic feet per second and less.

2. **Entrance Velocity**: Entrance velocity, V_1 should be as low as practical. Ideal conditions exist when $V_1 = (gq)^{1/3} - 5$ (See Curve D, Figure 7.9). Flow Conditions are not acceptable when $V_1 = (gq)^{1/3}$ (See Curve C, Figure 7.9).

3. **Chute Design**: The vertical offset between the approach channel floor and the chute is used to create a stilling pool or desirable V_1 and will vary in individual installations; Figure 7.8 shows a typical approach pool. Use a short radius curve to provide a crest on the 2:1 sloping chute. Place the first row of baffle piers close to the top of the chute no more than 12 inches in elevation below the crest.

4. **Baffle Height**: The baffle pier height, H, should be about $0.8D_c$ (see Curve B, Figure 7.9). The critical depth on the rectangular chute is $D_c = (q^2/g)^{1/3}$ (see Curve A, Figure 7.9). Baffle pier height is not a critical dimension but should not be less than recommended. The height maybe increased to $0.9D_c$ critical.

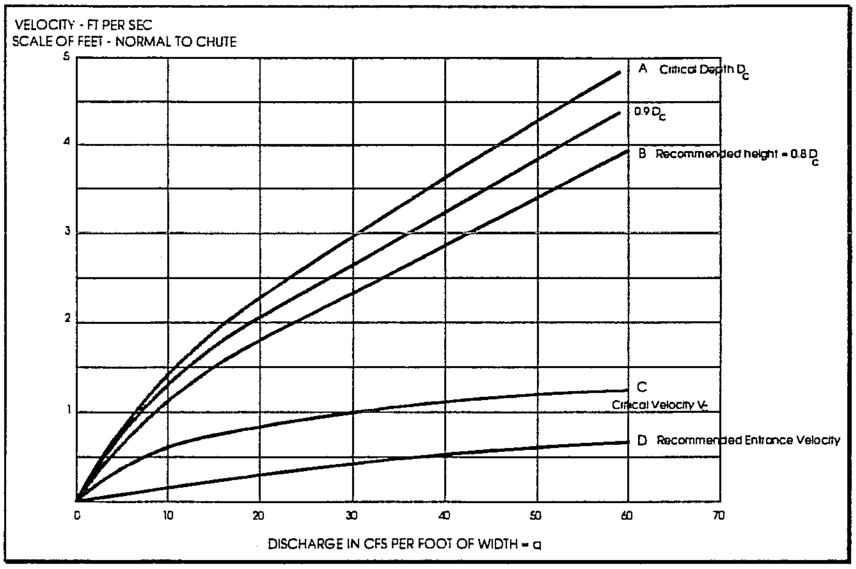


FIGURE 7.9 Recommended Baffle Pier Heights and Allowable Velocities

(Source: (USBUREC, 1961))

5. **Baffle Width**: Baffle pier widths and spaces should be equal, preferably about 1.5*H*, but not less than *H*. Other baffle pier dimensions are not critical; suggested cross section is shown in Figure 7.8. Partial blocks, width 0.33*H* to 0.67*H*, should be placed against the training walls in Rows 1, 3, 5, 7, etc., alternating with spaces of the same width in Rows 2, 4, 6, etc.
6. **Baffle Row Spacing**: The slope distance between rows of baffle piers should be 2*H*, twice the baffle height *H*. When the baffle height is less than 3 feet, the row spacing may be greater than 2*H* but should not exceed 6 feet.
7. **Baffle Alignment**: The baffle piers are usually constructed with their upstream faces normal to the chute surface; however, piers with vertical faces may be used. Vertical face piers tend to produce more splash and less bed scour, but differences are not significant.
8. **Chute Length**: Four rows of baffle piers are generally required to establish full control of the flow, although fewer rows have operated successfully. Additional rows beyond the fourth maintain the control established above, and as many rows may be constructed as is necessary. The chute should be extended to below the normal downstream channel elevation. At least one row of baffles should be buried in the backfill.
9. **Wall Height**: The chute training walls should be three times as high as the baffle piers (measured normal to the chute floor) to contain the main flow of water and splash. It is impractical to increase the wall heights to contain all the splash.
10. **Downstream Riprap**: Riprap consisting of 6-inch to 12-inch stones should be placed at the downstream ends of the training walls to prevent eddies from undermining the walls.

Requirements for Storm Sewer Outfalls in Channels

A major source of erosion in channels is around pipe outfalls, either from storm sewers or from backslope interceptor structures. Erosion can occur in the channel bottom and on the opposite bank due to high flows from such pipes. In addition, improperly installed pipes result in seepage, piping, and erosion around the outside wall of the pipe. The best way to prevent seepage is to construct tight pipe joints, to backfill with cement stabilized sand, and to compact the backfill properly. Chapter 4 presents minimum requirements for pipe outfall construction.

Erosion downstream of the pipe can be prevented by installing riprap or concrete slope paving. The use of riprap is recommended because it is flexible and it reduces the water velocity coming out of the pipe. Problems can occur with concrete slope paving breaking off the outfall pipe due to undermining of the slope paving.

FIGURE 7.10 Erosion Protection for Storm Sewer Outfall Structure

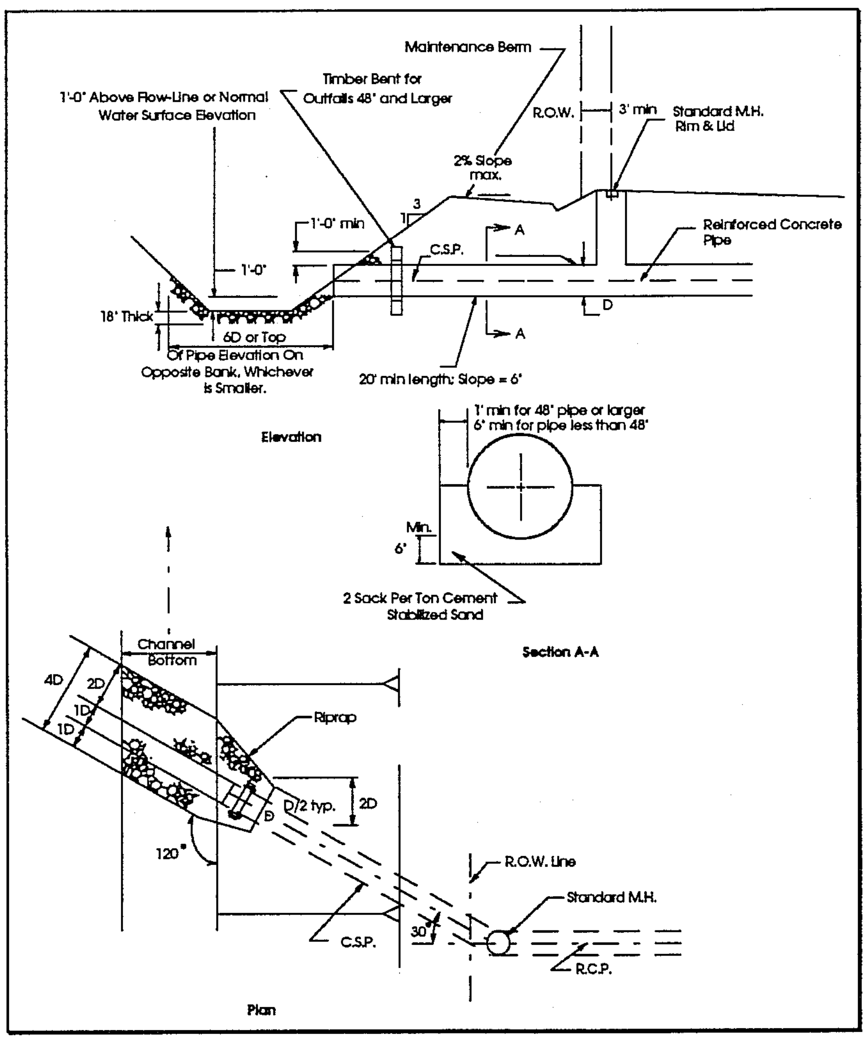


Figure 7.10 illustrate the requirements for erosion protection placement at the end of the pipe. The distance the riprap or slope paving needs to extend across the ditch is given as six times the diameter of the pipe or up to the elevation of the top of pipe on

the opposite bank, whichever results in a shorter distance. The purpose of this dual guideline is to attempt to cover most combinations of pipe and channel sizes. For example, a 6-foot diameter pipe outfalling into a 6-foot wide bottom channel definitely needs opposite bank protection, but the same pipe in a 40-foot wide bottom channel would not need opposite bank protection.

The purpose of installing outfall pipes one foot above the channel flow-line or normal water level is to insure continued operation of the pipe if the channel silts up. A distance larger than one foot would create erosion problems in the channel under the end of the pipe.

Requirements for Backslope Drainage Systems

The use of backslope drains and swales is required for grass-lined channels. These systems collect overland flow from channel overbanks and other areas not draining to the storm sewer collection system. Their purpose is to prevent excessive overland flow from passing over the banks of grass-lined channels and eroding the side slopes. Subject to approval, backslope drains may not be required in undeveloped or sparsely developed areas.

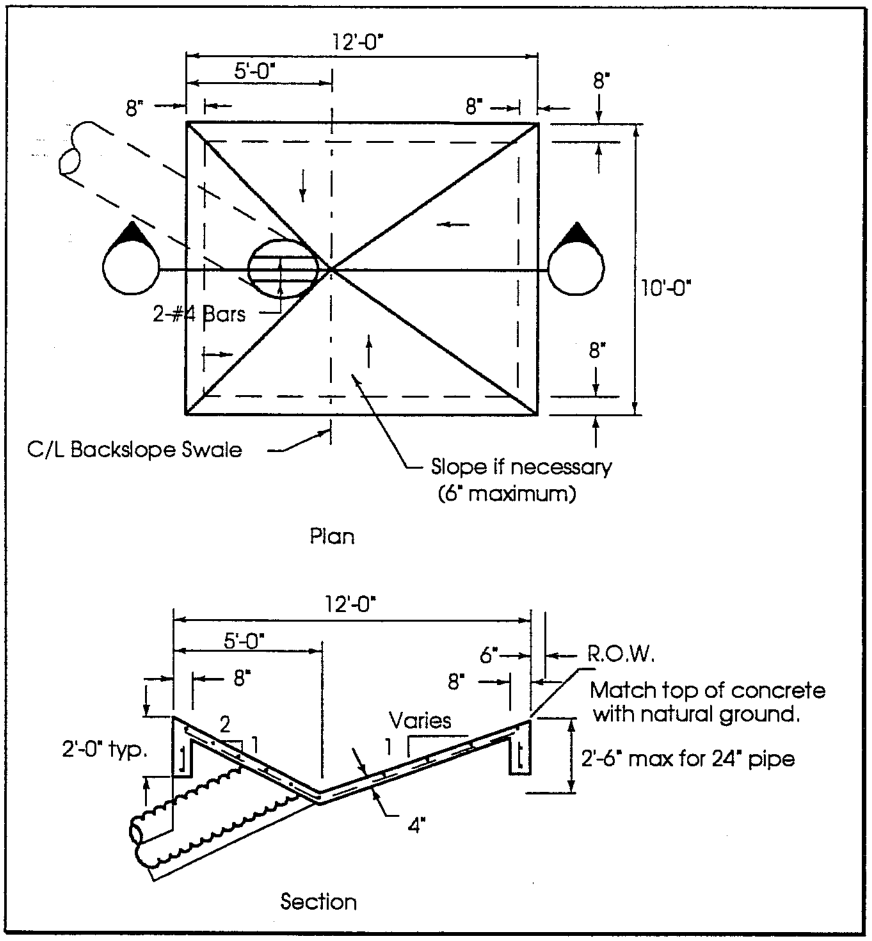
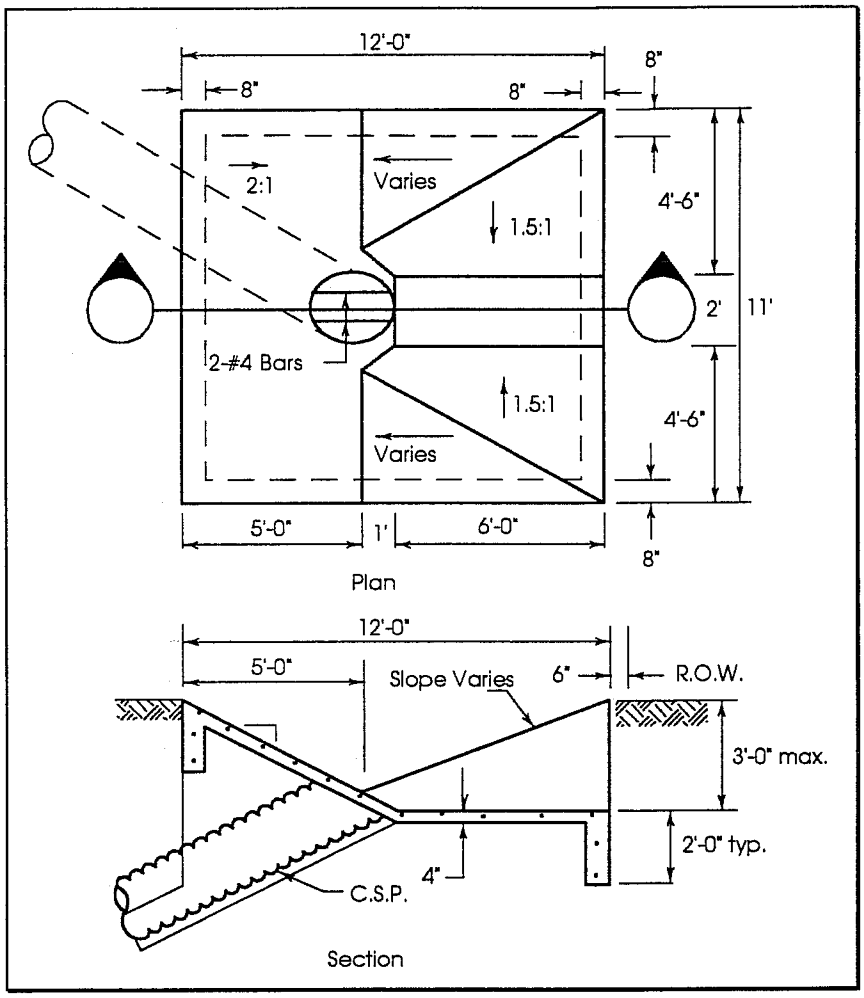


FIGURE 7.11 Typical Backslope Interceptor Structure (24" & 30" CSP Only)

The design engineer should carefully consider the drainage area to be intercepted by such systems, particularly when the channel passes through large areas of undeveloped acreage where large quantities of naturally occurring sheet flow could overload the backslope swale and drainage system. In these areas, the minimum requirements for drain spacing and backslope drainage pipe discussed below may not be adequate. Refer to Figures 7.11 and 7.12 for design of interceptors for backslope drains and offsite ditches.

Documentation of drainage area for each backslope drain system as well as hydraulic pipe and swale sizing calculations must be provided by the engineer.

FIGURE 7.12 Combination Backslope and Offsite Ditch Interceptor Structure (42" max.)



General requirements for backslope drains and swales are as follows:

1. **Minimum Pipe Size**: Minimum backslope drain pipe shall be 24" in diameter.
2. **Maximum Spacing**: Maximum spacing is 800 feet (or 400 feet to the swale high point).
3. **Location**: The drain structure and swale center-line should be five feet inside the channel right-of-way line when a 20-foot maintenance berm is used. When a 30-foot maintenance berm is used, the drain structure and swale center-line should be 7.5 feet inside the channel right-of-way line.

4. **Design Depth**: Minimum design depth in swale is 0.5 feet. Maximum design depth in swale is 2.0 feet.
5. **Grade**: Minimum gradient for swale invert is 0.2%.
6. **Side Slope**: Swale should have a maximum (steepest) side slope of 1.5:1.

Requirements for Roadside Ditch Interceptor Structures

A roadside ditch interceptor structure and adequately sized pipe should be used to convey flow from relatively small ditches into major drainage channels (Figure 7.13). Flow over the banks of grass-lined channels is not acceptable due to potential erosion problems. The interceptor pipe should be sized based on the drainage area served by the small ditch and design frequency for the ditch.

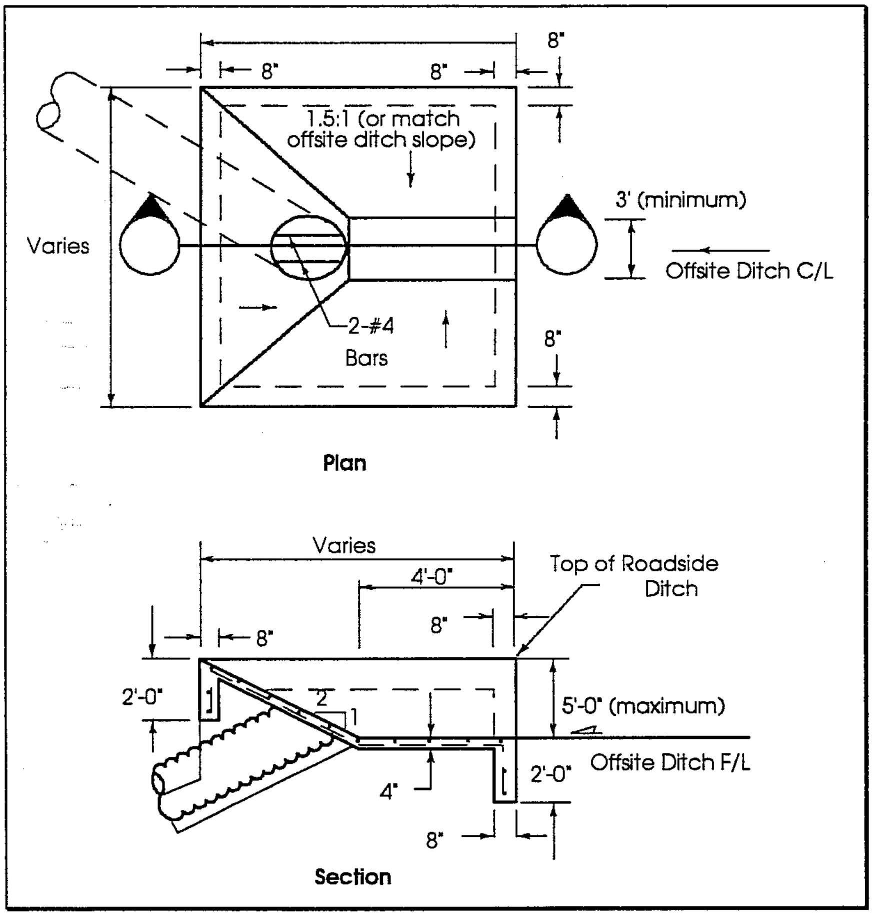


FIGURE 7.13 Typical Offsite Ditch Interceptor Structure (42" max.)

Erosion Control Measures for Detention Facilities

The erosion potential for a detention basin is similar to that of an open channel. For this reason the same types of erosion protection are necessary, including the use of backslope swales and drainage systems, proper revegetation, and pond surface lining where necessary. Proper protection must especially be provided at pipe outfalls into the facility, pond outlet structures and overflow spillways where excessive turbulence and velocities will cause erosion.

Sediment Control Requirements

Sediment deposition can significantly affect the performance of storm water detention basins and flood control channels. Therefore, sediment controls must be considered in the design of these facilities. In general, sediment control requirements follow these guidelines:

1. Wherever possible, sediment deposition within drainage facilities will be minimized.
2. When sediment deposition is unavoidable, an allowance for accumulated sediment volume is required in the design or construction of the facility.

Erosion control measures should be applied during construction of any development to prevent siltation of any affected channels and/or storm sewer systems. Channel siltation caused by construction activities is a major problem, resulting in reduced channel capacity and an additional financial burden on maintenance funds.

Upon completion of a new channel or channel improvements in a subdivision, the appropriate drainage regulatory agency will inspect and accept only the channel portion of the project. Inspectors will not accept the storm sewer outfalls until a request is made for acceptance of the subdivision streets and storm sewers. At that time, the storm sewer outfall must be free of silt and backwater. Any sedimentation which occurs during construction of a subdivision or development must be removed and the channel restored to its design condition.

Appendix A. Culvert Analysis Charts

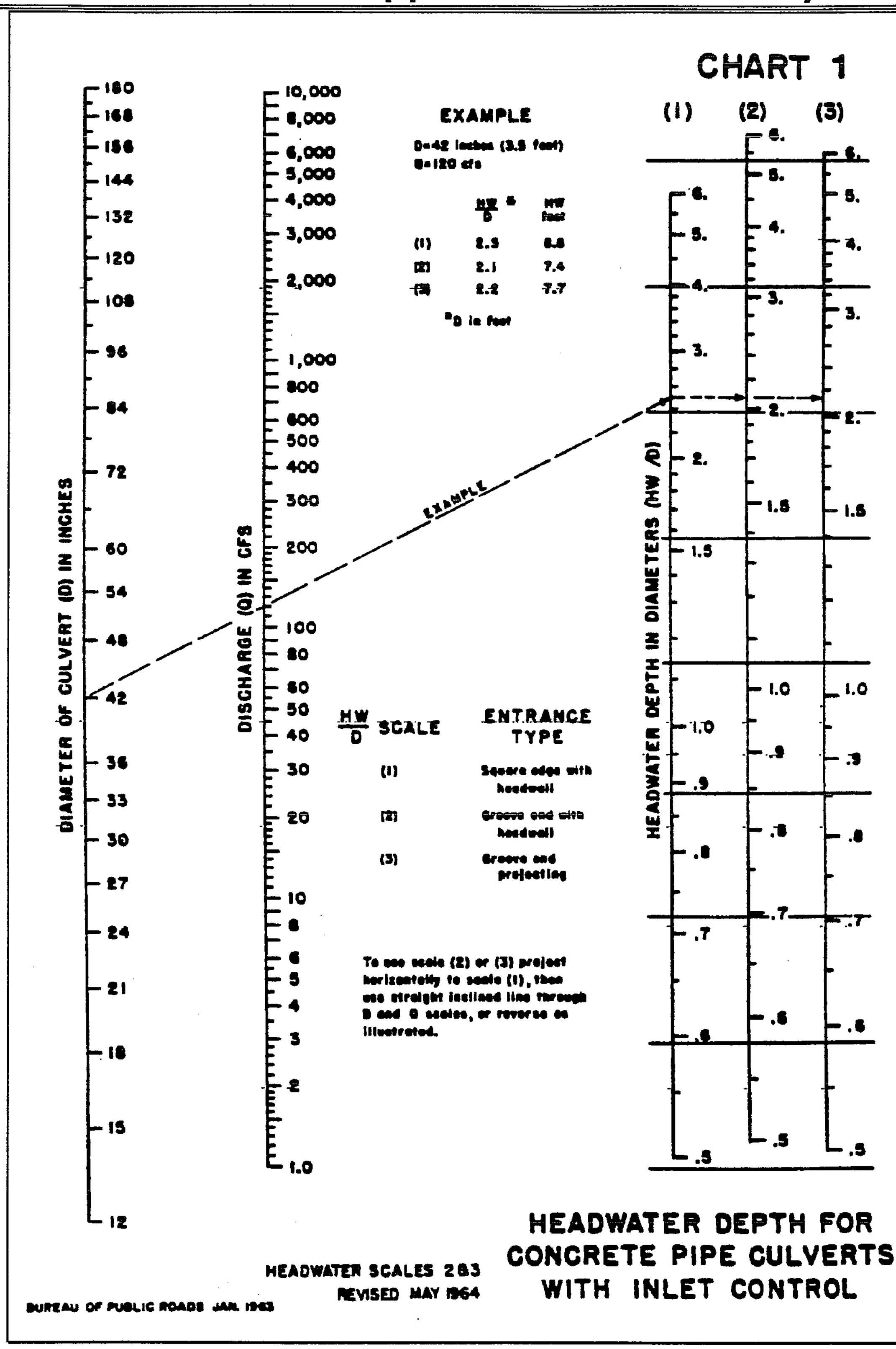
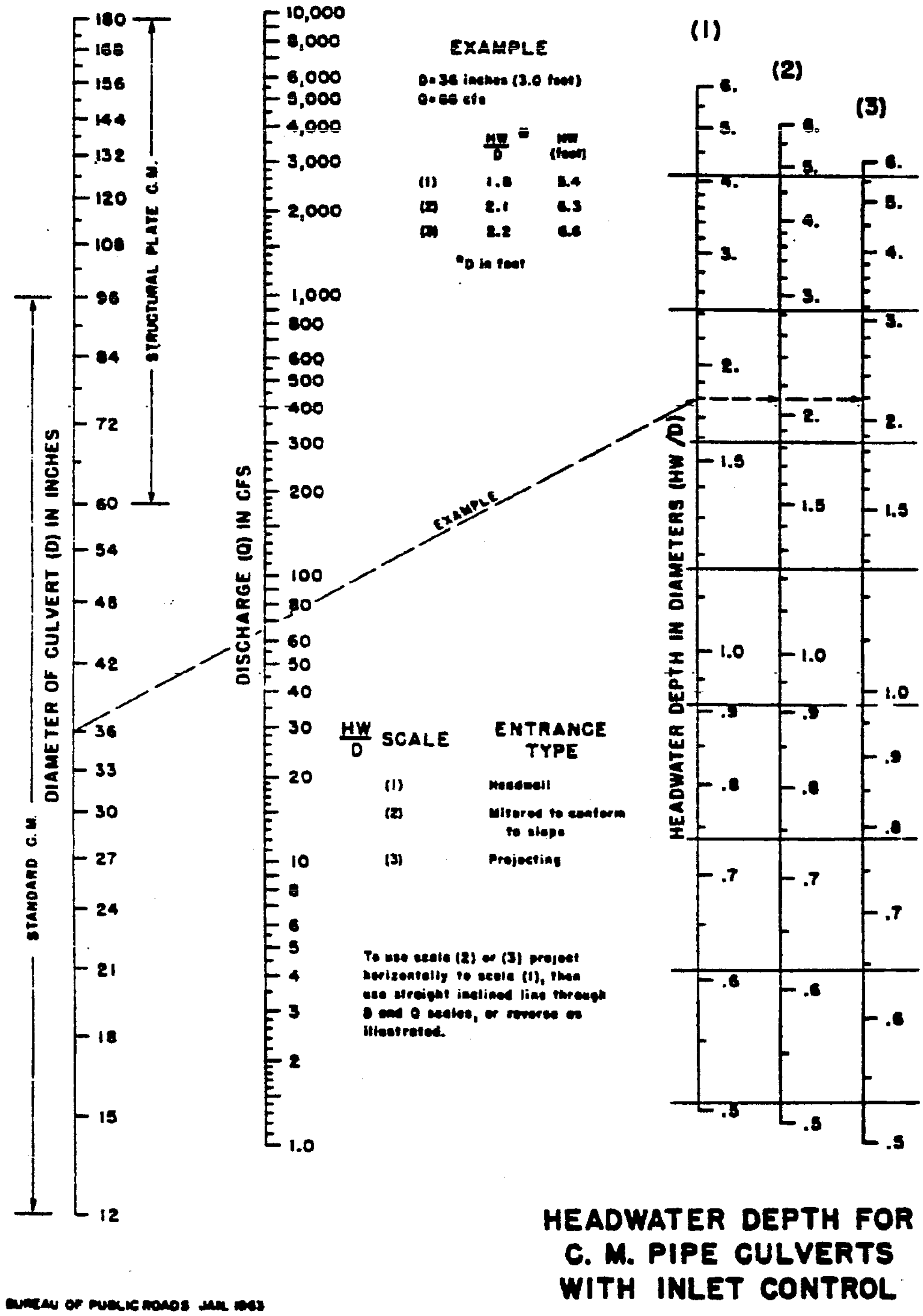
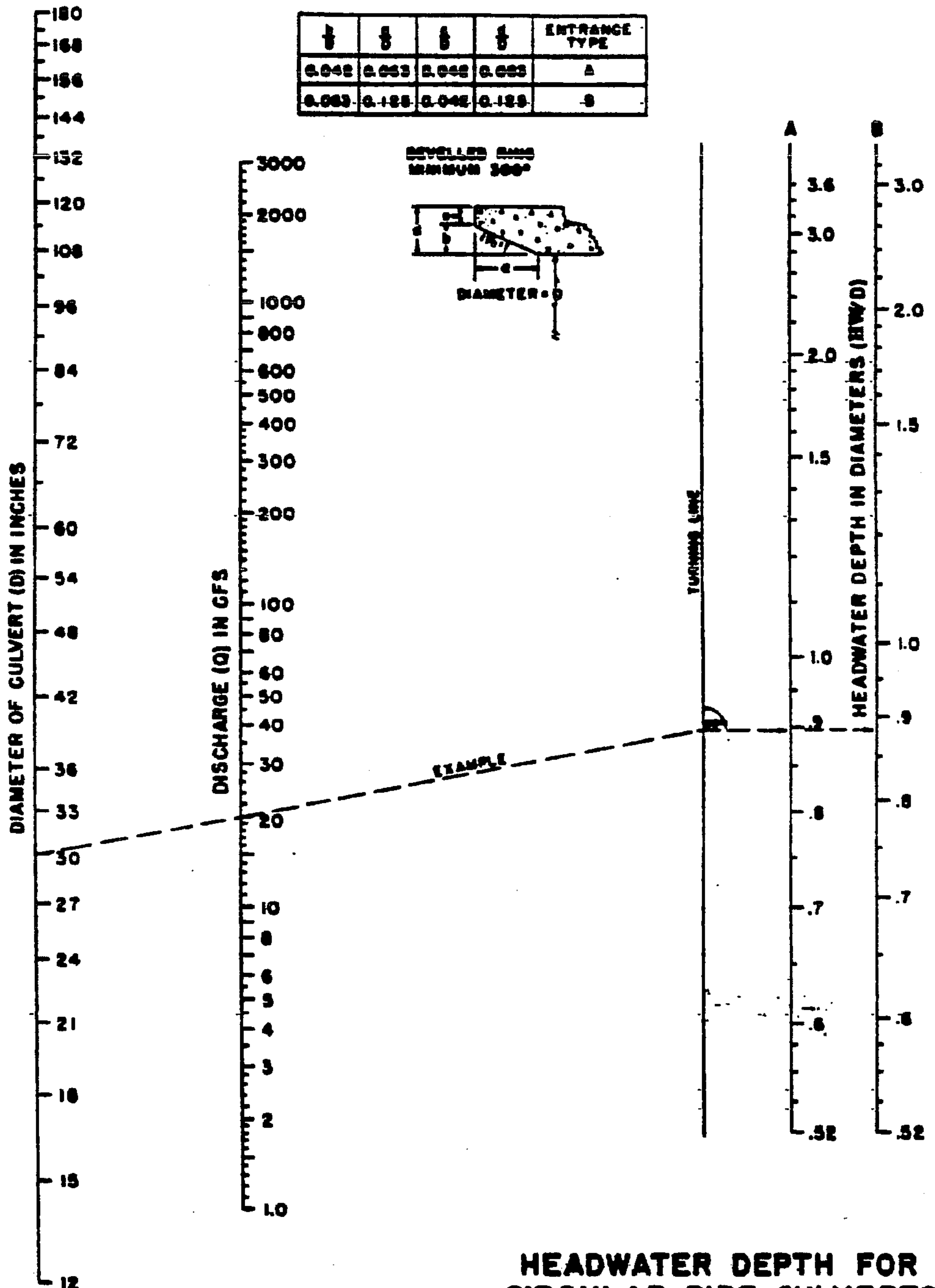


CHART 2



HEADWATER DEPTH FOR C. M. PIPE CULVERTS WITH INLET CONTROL

CHART 3



HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING INLET CONTROL

Federal Highway Administration
May 1973

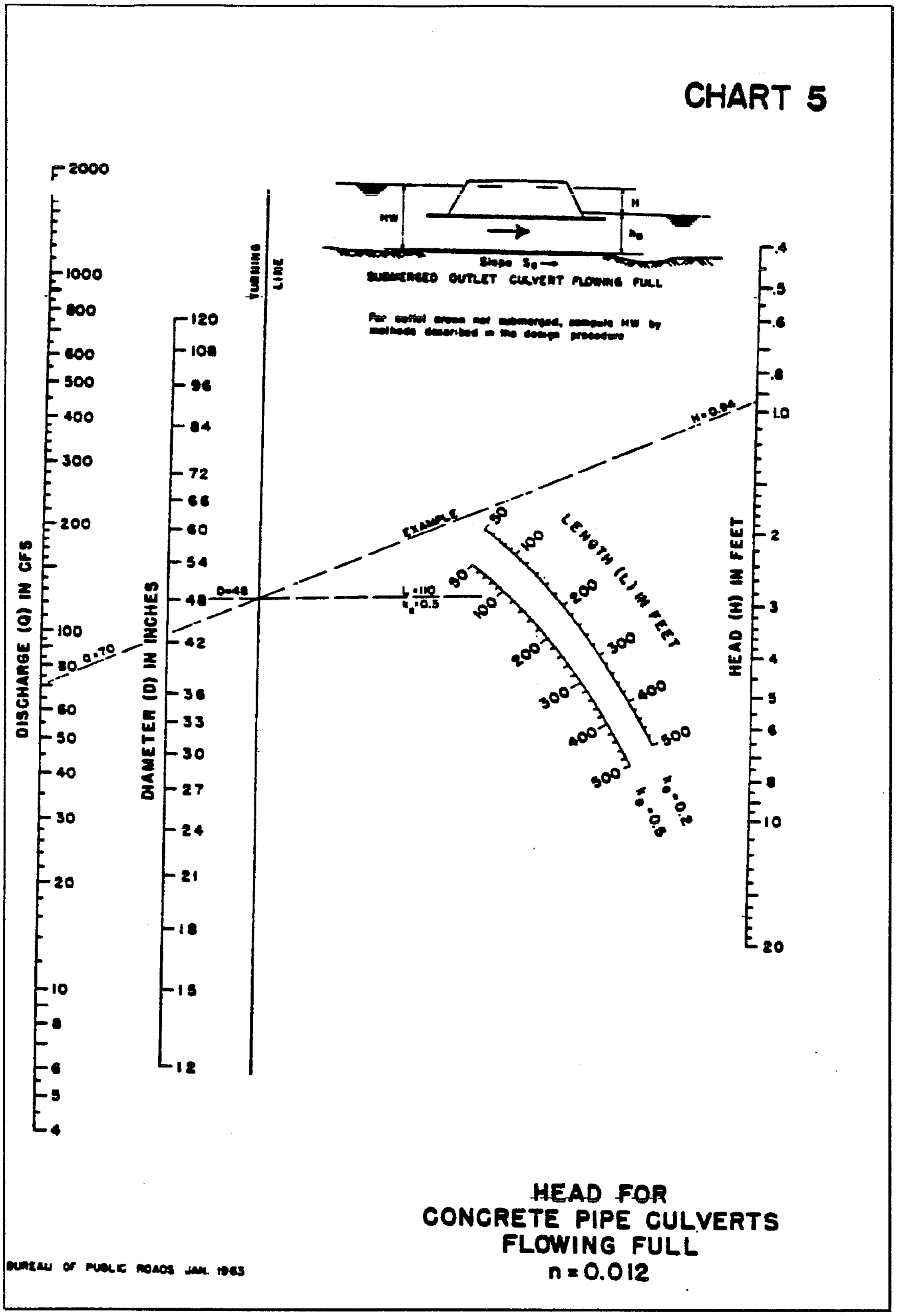
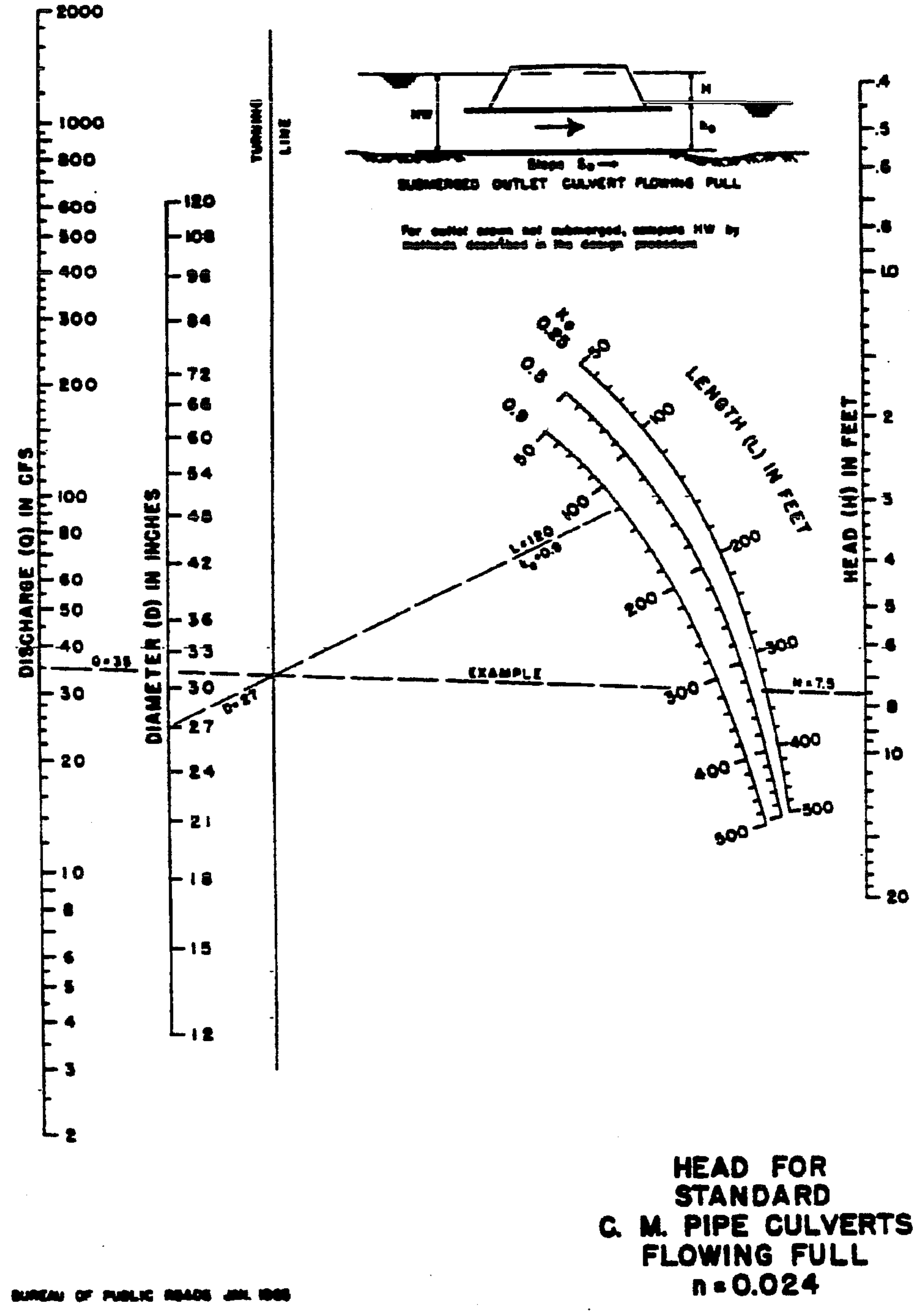


CHART 5
SUBMERGED OUTLET CULVERT FLOWING FULL
For outlet crown not submerged, compute HW by methods described in the design procedure
DISCHARGE (Q) IN CFS
DIAMETER (D) IN INCHES
TURNING LINE
LENGTH (L) IN FEET
HEAD (H) IN FEET
EXAMPLE
Q=70
D=48
H=0.94
BUREAU OF PUBLIC ROADS JAN. 1963
HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

CHART 6



HEAD FOR
STANDARD
C. M. PIPE CULVERTS
FLOWING FULL
n = 0.024

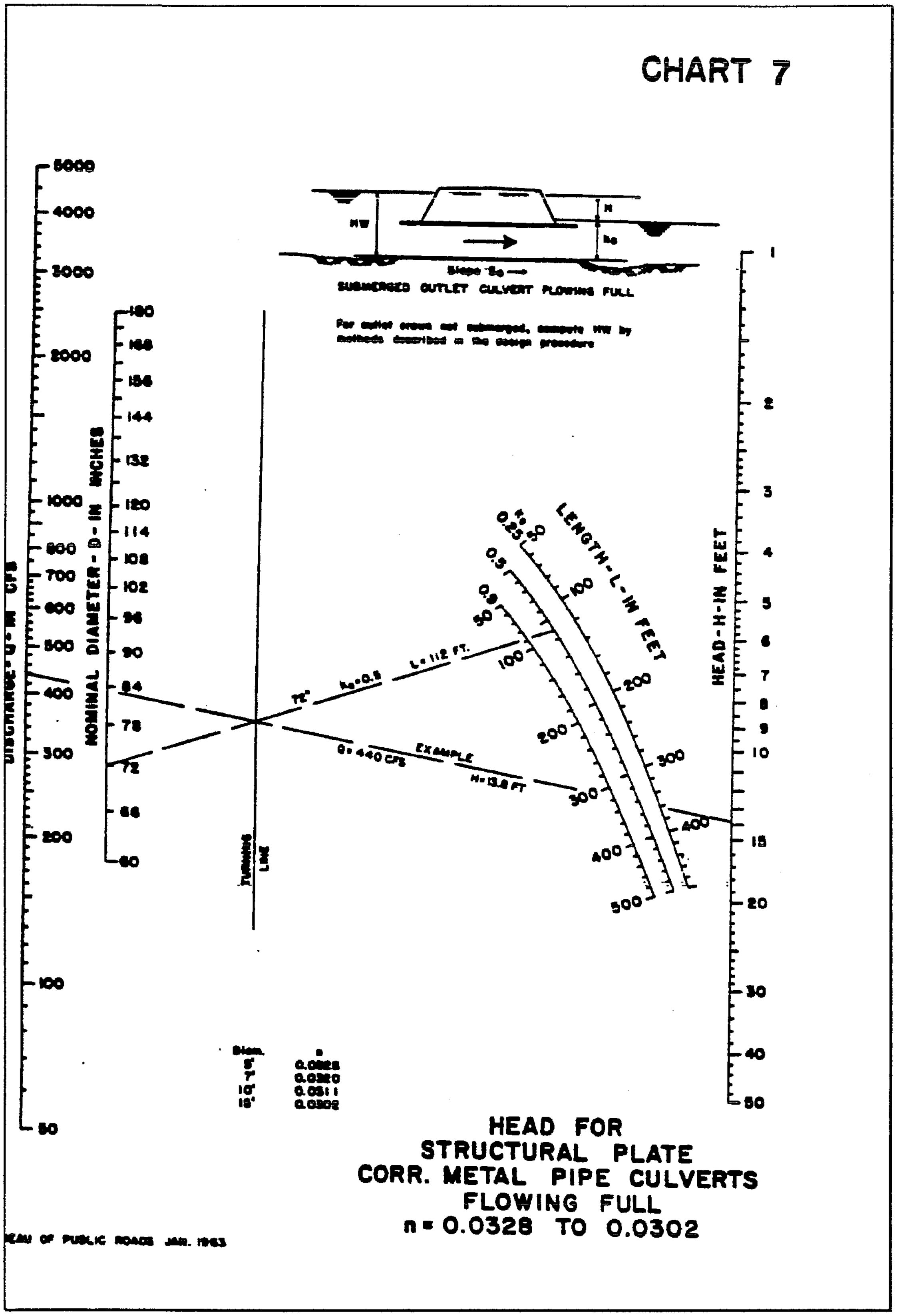


CHART 7
SUBMERGED OUTLET CULVERT FLOWING FULL
HW
H
Slope So
DISCHARGE-Q-IN CFS
NOMINAL DIAMETER-D-IN INCHES
TURNING LINE
LENGTH-L-IN FEET
HEAD-H-IN FEET
EXAMPLE
Q=440 CFS
H=13.8 FT
L=112 FT.
HEAD FOR
STRUCTURAL PLATE
CORR. METAL PIPE CULVERTS
FLOWING FULL
n = 0.0328 TO 0.0302

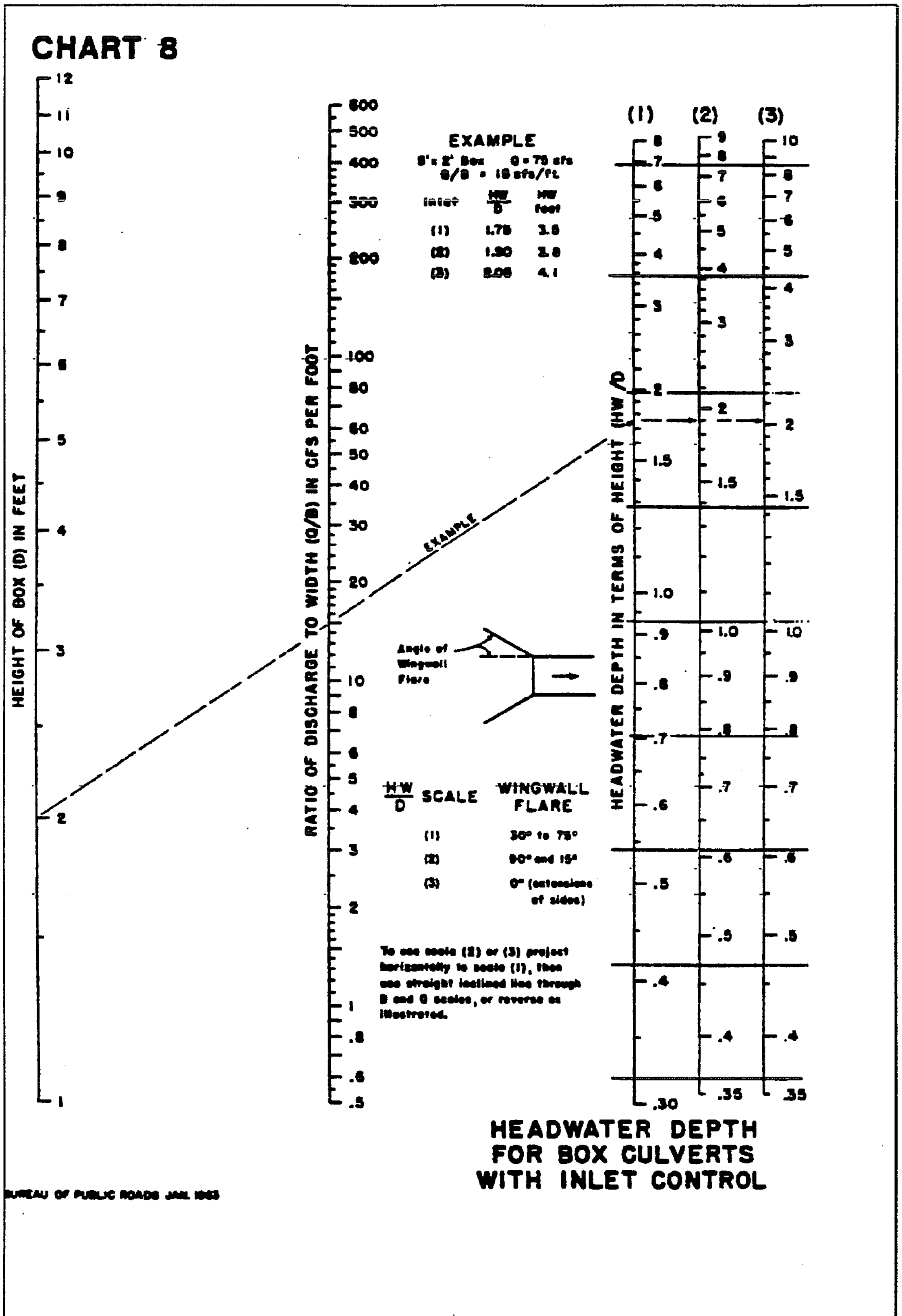
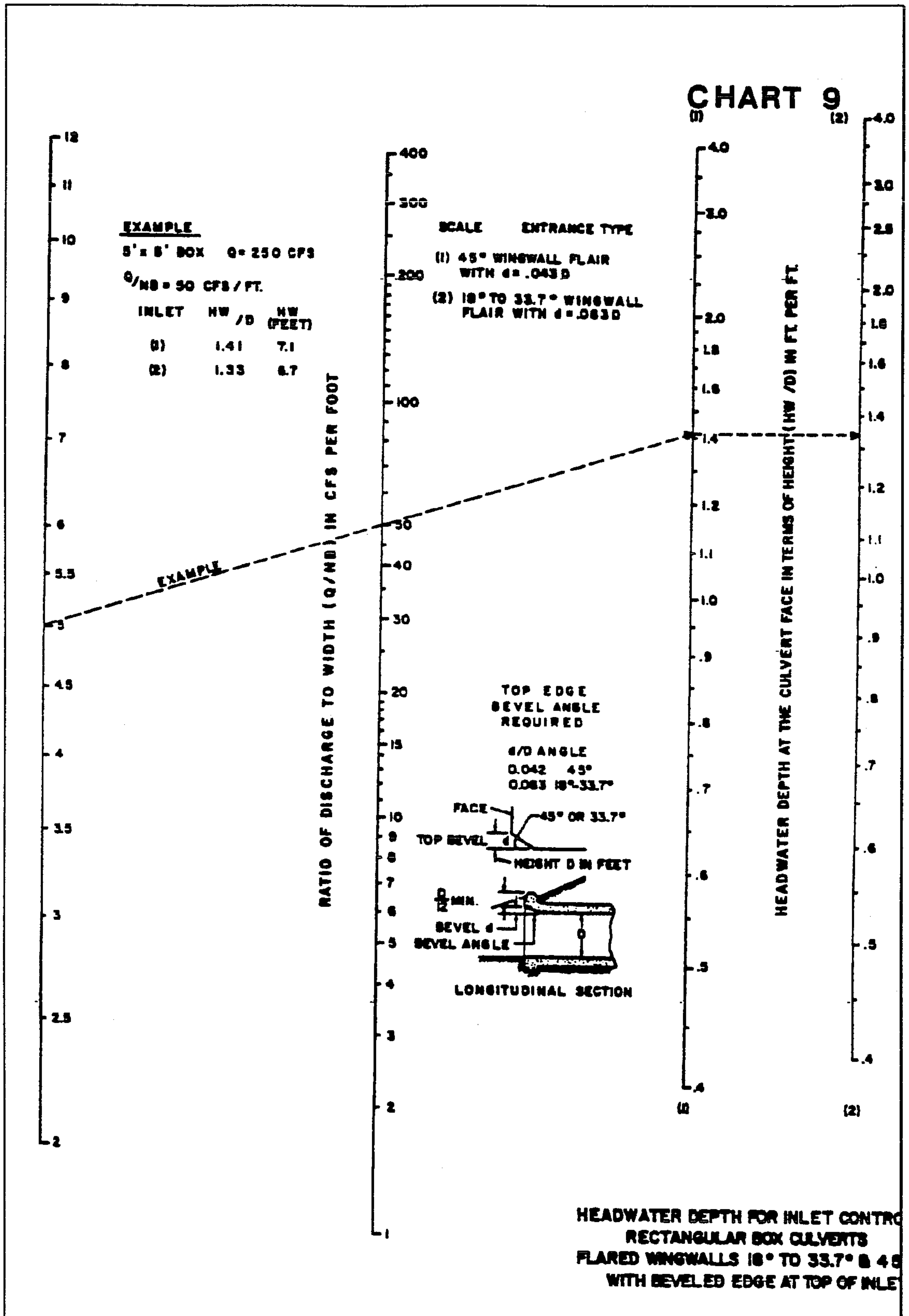
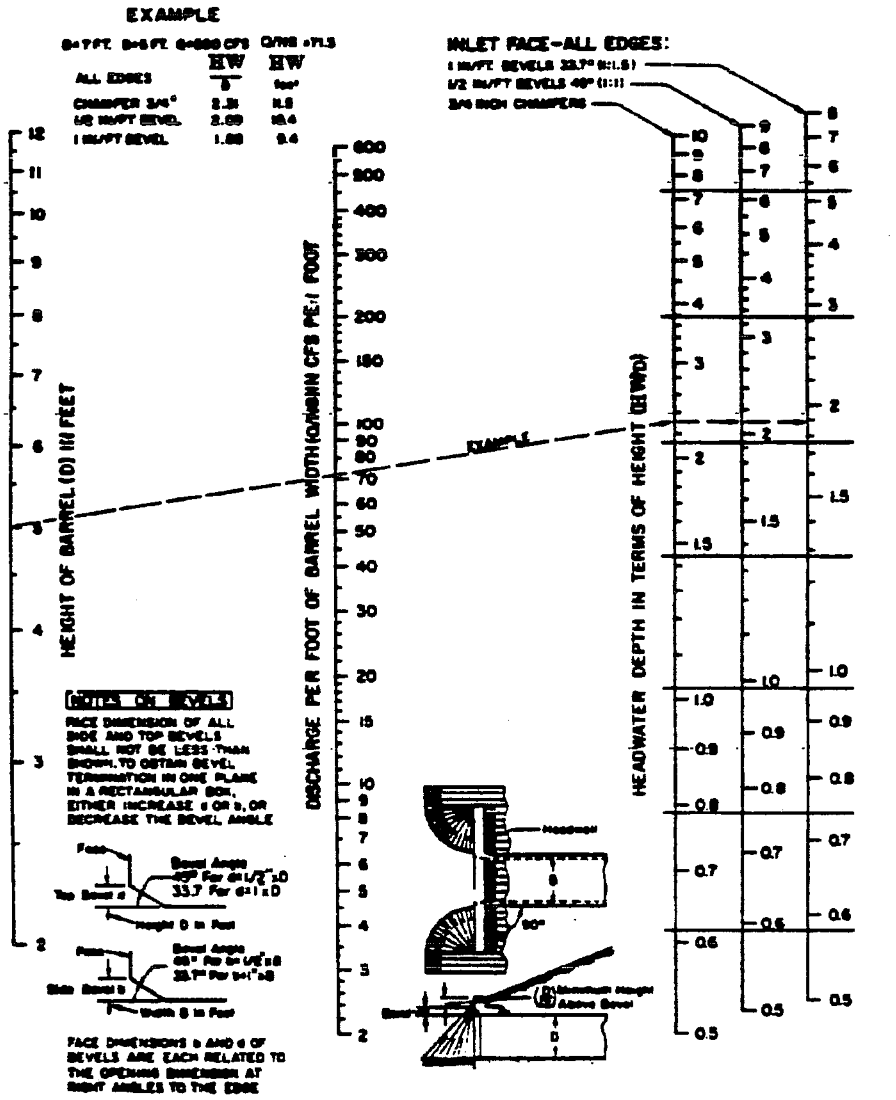


CHART 8
HEIGHT OF BOX (D) IN FEET
RATIO OF DISCHARGE TO WIDTH (Q/B) IN CFS PER FOOT
HEADWATER DEPTH IN TERMS OF HEIGHT (HW/D)
(1) (2) (3)
EXAMPLE
5' x 2' Box Q = 75 cfs
Q/B = 15 cfs/ft
Inlet HW/D HW feet
(1) 1.75 3.5
(2) 1.90 3.8
(3) 2.06 4.1
EXAMPLE
Angle of Wingwall Flare
HW/D SCALE WINGWALL FLARE
(1) 30° to 75°
(2) 90° and 15°
(3) 0° (extensions of sides)
To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated.
HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL
BUREAU OF PUBLIC ROADS JAN. 1963



HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS 18° TO 33.7° & 45°
WITH BEVELED EDGE AT TOP OF INLET

CHART 10



HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
90° HEADWALL
CHAMFERED OR BEVELED INLET EDGES

FEDERAL HIGHWAY ADMINISTRATION
MAY 1973

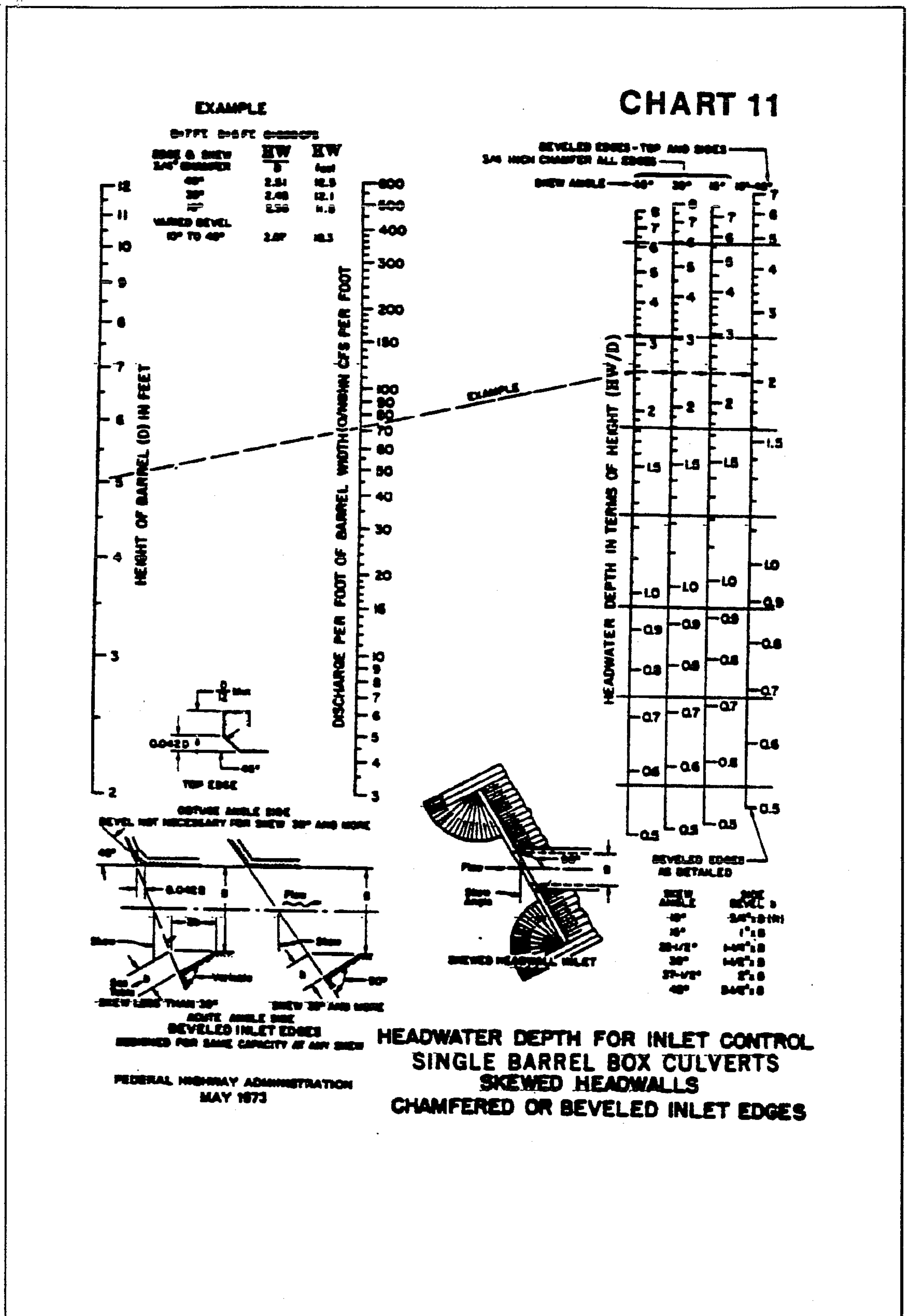
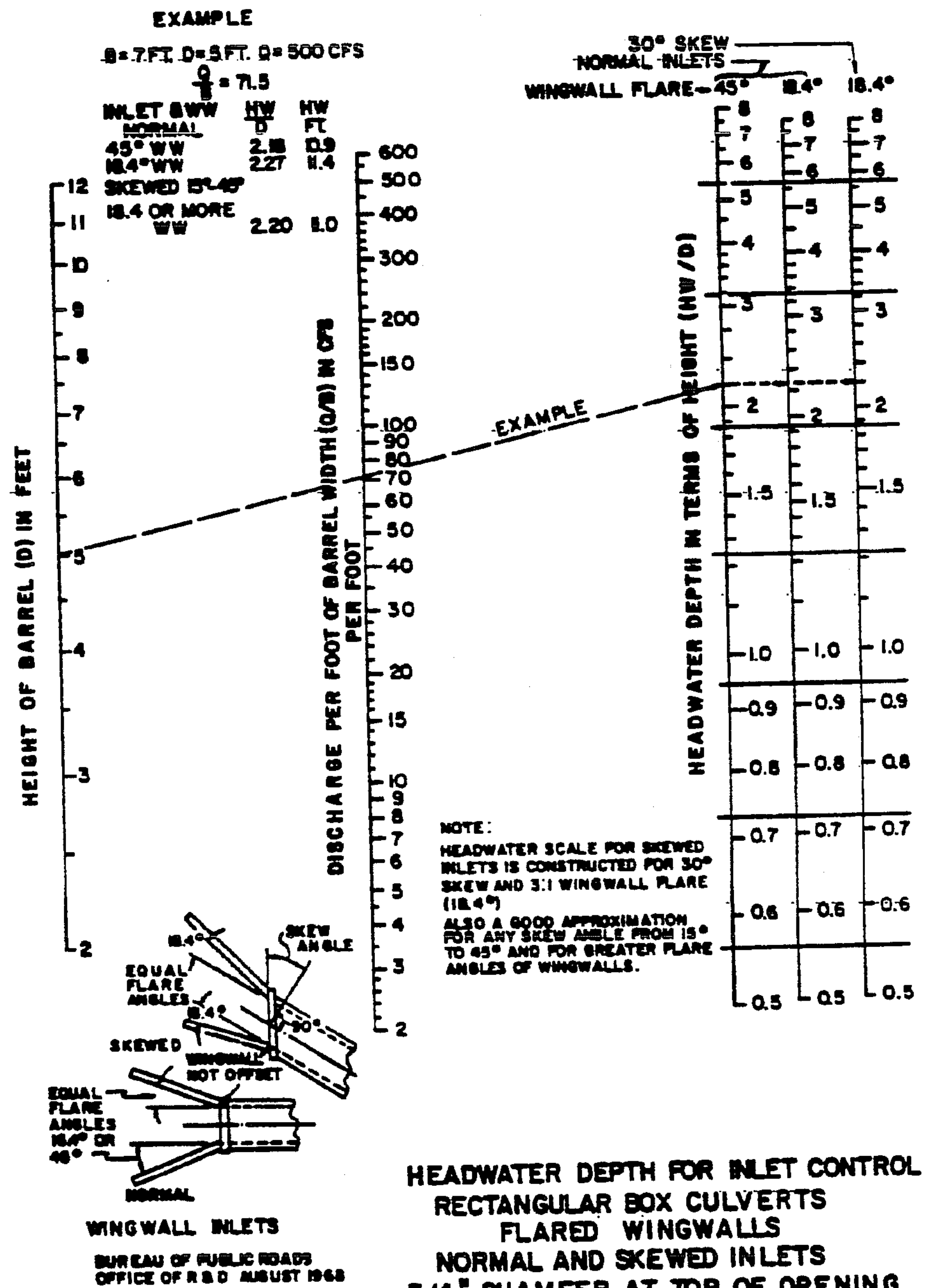


CHART 11
EXAMPLE
HEIGHT OF BARREL (D) IN FEET
DISCHARGE PER FOOT OF BARREL WIDTH (Q/B) IN CFS PER FOOT
HEADWATER DEPTH IN TERMS OF HEIGHT (HW/D)
TOP EDGE
BEVELED INLET EDGES
SKEWED HEADWALL INLET
BEVELED EDGES AS DETAILED
HEADWATER DEPTH FOR INLET CONTROL
SINGLE BARREL BOX CULVERTS
SKEWED HEADWALLS
CHAMFERED OR BEVELED INLET EDGES
MAY 1973

CHART 12



HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS
NORMAL AND SKEWED INLETS
3/4" CHAMFER AT TOP OF OPENING

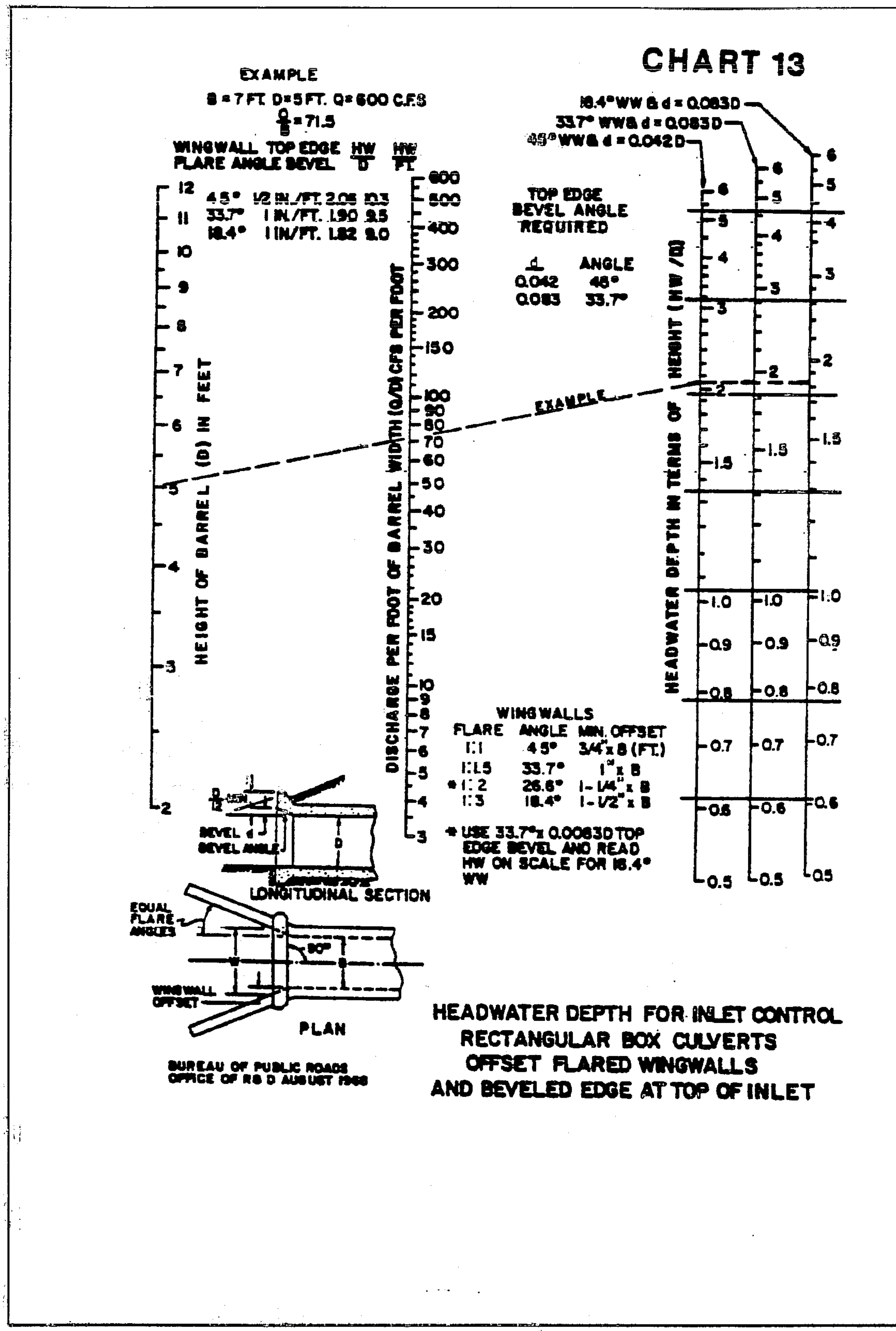


CHART 13
EXAMPLE
B = 7 FT. D = 5 FT. Q = 600 C.F.S.
Q/B = 71.5
WINGWALL FLARE ANGLE
TOP EDGE BEVEL
HW/D
HW FT.
45° 1/2 IN./FT. 2.06 10.3
33.7° 1 IN./FT. 1.90 9.5
18.4° 1 IN/FT. 1.82 9.0
HEIGHT OF BARREL (D) IN FEET
DISCHARGE PER FOOT OF BARREL WIDTH (Q/B) CFS PER FOOT
HEADWATER DEPTH IN TERMS OF HEIGHT (HW/D)
18.4° WW & d = 0.083D
33.7° WW & d = 0.083D
45° WW & d = 0.042D
TOP EDGE BEVEL ANGLE REQUIRED
d ANGLE
0.042 45°
0.083 33.7°
EXAMPLE
WINGWALLS
FLARE ANGLE MIN. OFFSET
1:1 45° 3/4" x B (FT.)
1:1.5 33.7° 1" x B
* 1:2 26.6° 1-1/4" x B
1:3 18.4° 1-1/2" x B
* USE 33.7° x 0.0083D TOP EDGE BEVEL AND READ HW ON SCALE FOR 18.4° WW
BEVEL d
BEVEL ANGLE
LONGITUDINAL SECTION
EQUAL FLARE ANGLES
WINGWALL OFFSET
90°
W
B
PLAN
BUREAU OF PUBLIC ROADS OFFICE OF R & D AUGUST 1968
HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
OFFSET FLARED WINGWALLS
AND BEVELED EDGE AT TOP OF INLET