8

ORDINANCE NO. 79-11

AN ORDINANCE FOR THE PURPOSE OF ADOPTING STORM DRAINAGE STANDARDS FOR THE CITY OF FORT SMITH

BE IT ORDAINED AND ENACTED BY THE BOARD OF DIRECTORS OF THE CITY OF FORT SMITH, ARKANSAS, THAT:

SECTION 1: There is hereby adopted the 2011 Storm Drainage Standards, three (3) hard copies of which are now filed in the Office of the City Clerk and may also be viewed electronically at http://www.fortsmithar.gov/engineering/default.aspx ("Fort Smith Storm Drainage"), and which are hereby adopted and incorporated as fully as if set out verbatim herein, and the provisions thereof shall be controlling within the corporate limits of the City of Fort Smith, Arkansas.

SECTION 2: Emergency Clause. It is hereby found and determined by the Board of Directors of the City of Fort Smith that an emergency exists in the City of Fort Smith, Arkansas, requiring adoption of the 2011 Storm Drainage Standards. This Ordinance being necessary for the immediate preservation of the public health, safety and welfare shall be immediately effective as of the date of its adoption.

PASSED AND APPROVED THIS 4th DAY OF October, 2011.

APPROVED:

Sandy Sanders
Mayor

ATTEST:

Sherri Gard
City Clerk

Approved as to form:

City Attorney
Publish 1 Time

2011 Storm Drainage Standards

The City of Fort Smith ARKANSAS

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STANDARDS AND REFERENCES

These drainage standards were developed from the following standards and references:

AASHTO Model Drainage Manual, 1991 (Chapter 6)
AASHTO Model Drainage Manual, 2005 (Chapters 2, 3, 5, & 6)
AHTD Drainage Manual, 1982 (Chapter 4)
City of Fayetteville, Arkansas, Drainage Criteria Manual (Chapters 1 & 5)
City of Fort Smith, Arkansas, Minimum Storm Drainage Standards, 1975 (Chapters 1 – 4, & 6)
City of Fort Smith, Arkansas, Drainage Policy, 1987 (Chapters 1, 3, & 6)
FHWA, HDS No. 5, Hydraulic Design of Highway Culverts, 1985 (Chapter 4)
NRCS, TR-55, Urban Hydrology for Small Watersheds, 1986 (Chapter 2)
The Stormwater Manager's Resource Center, www.stormwatercenter.net (Chapter 5)

All other references are as noted within this document.

CHAPTER 1 – GENERAL REQUIREMENTS

1.1 GENERAL

No storm drainage facility – whether an enclosed structure, pipe, or an open channel, ditch or stream – shall be constructed, altered, or reconstructed within a subdivision, planned development, or a developed area, within a public right-of-way or easement, or discharge into, upon, or under a public right-of-way or easement, or a subdivision or planned development or developed area within the planning jurisdiction of the City of Fort Smith, without first obtaining written approval from the Department of Engineering.

1.2 DESIGN DATA, MAPS, COMPUTATIONS, PLANS, AND SPECIFICATIONS

All designs, plans and specifications submitted to the City for approval for the construction of public drainage works as required herein shall be prepared by a registered professional engineer, licensed in the state of Arkansas, and shall meet the minimum standards specified herein. All public improvements shall be constructed in accordance with the City of Fort Smith Standard Specifications for Public Works Construction and all applicable revisions and the City of Fort Smith Standard Drawings.

For private drainage works, all designs, plans and specifications submitted to the City for approval shall be prepared by a registered professional engineer, licensed in the state of Arkansas, and shall meet the minimum standards specified in Chapters 1 and 2. Drainage ways traversing private developments (i.e. ditches, culverts, storm drains) shall be public, and therefore, shall be subject to the minimum standards specified for public drainage works.

1.2.1 *Drainage Report*

A drainage report must be submitted for review and approval by the Engineering Department. The drainage report shall include the following:

1.2.1.1 Title and Date

The drainage report must include a title page which shows the title of the project and date of submission.

1.2.1.2 Location of Project

The street address and a vicinity map shall be included. The vicinity map must show the location of the project with respect to well-known roads, streets, subdivisions, survey lines, and/or City monuments.

1.2.1.3 Description of Project

A brief description of the project and the planned improvements must be included.

1.2.1.4 Contact Information

The name, address, and telephone number of the Project Owner and Developer must be included.

1.2.1.5 Drainage Area Map

A topographic area map with the pre- and post-development drainage area(s) outlined shall be provided. The map shall have a minimum of two foot contour intervals and a minimum scale of 1" = 200'.

1.2.1.6 Area Drainage Issues

A description of any known on- or off-site drainage/flooding problems shall be provided.

1.2.1.7 Written Summary of the Proposed Improvements

The summary must include on-site improvements, off-site improvements, condition of the downstream receiving areas, existing problems, any increases in flows, detention or lack of detention, and final conclusions.

1.2.1.8 Pre- and Post-Development Flowrates

All pre- and post-development flowrate calculations must be included for the 10-, 25-, 50-, and 100-year storm events.

1.2.1.9 Storm Water Detention Design

All calculations must be included.

1.2.1.10 Open Channel Flow Design

All calculations must be included.

1.2.1.11 Pavement Drainage Design

Calculations for gutter spread must be included.

1.2.1.12 Culvert Design

All calculations (both inlet and outlet control) must be included.

1.2.1.13 Inlet Design

Capture efficiency calculations must be included.

1.2.1.14 Storm Sewer Design

All storm sewer design and hydraulic grade line calculations must be included.

1.2.1.15 100-Year Water Surface Elevation and Minimum Floor Elevation

Calculations must be included for the water surface elevation resulting from the 100-year storm for all overland flow, including flow in the streets, parking lots, swales, and between lots. Minimum floor elevations must also be included. Minimum floor elevations shall be one foot above the calculated 100-year water surface elevation of open channels, swales, or overland flow.

1.2.1.16 Federal and State Requirements

Copies of documents which show compliance with all applicable Federal and State requirements (Corps 404 Permit, ADEQ Notice of Intent, FEMA CLOMR, etc.) must be included. Proof of permit approvals must be submitted before construction may begin.

1.2.1.17 Certification by Registered Professional Engineer

The title sheet of the drainage report shall be sealed, signed, and dated by a Professional Engineer registered in the state of Arkansas.

1.2.2 *Plan Requirements*

Plans shall be submitted on 22" x 34" sheets, unless another size is approved by the Engineering Department. All plans submitted for review shall include the following:

1.2.2.1 Location of Project

A vicinity map must be included. The vicinity map must show the location of the project with respect to well-known roads, streets, subdivisions, survey lines, and/or City monuments.

1.2.2.2 Plan and Profile Views of All Proposed Improvements

Plan and profile views must show the location, size, flowline elevations, gradients, materials, boring information and rock elevations (where applicable), and depths and sizes of adjacent or crossing utilities and structures.

1.2.2.3 North Arrow and Scale

A north arrow and scale must be shown on every applicable sheet. The top of each page shall either be north or east, unless otherwise approved by the Engineering Department. Plan drawings shall be prepared with a horizontal scale of 1" = 20' or larger. Profile drawings shall be prepared with the same horizontal scale as the plan drawings and a vertical scale of 1" = 5' or larger. Cross sections shall have a horizontal scale of 1" = 10' or larger and a vertical scale of 1" = 5' or larger. Special cases may warrant the use of larger or smaller scale drawings and may be used with prior permission of the Engineering Department.

1.2.2.4 Bench Mark

At least two permanent bench marks must be established. Bench marks must be tied to the City of Fort Smith Coordinate System.

1.2.2.5 Right-of-Way

Plans must show the existing and proposed right-of-way or easements.

1.2.2.6 Existing Structures and Utilities

Plans must show the location of all existing structures, streets, driveways, storm drains, fences, trees, landscaping, utilities, and other features within 25 feet of proposed improvements. The flowline elevations of all existing drainage facilities must be shown. Where conflicts may occur between existing underground utilities and new construction, the elevations of the existing utilities must be determined by excavation methods.

1.2.2.7 Cross Sections

Cross sections must be provided at a maximum of 50 foot intervals along the centerline of proposed improvements for a minimum width of 50 feet, or as necessary to ensure drainage and define existing conditions of adjacent lands. Cross sections must show surface elevations, flowline elevations and sizes of all proposed improvements, and flowline elevations and sizes of all crossing or adjacent utilities or structures.

1.2.2.8 Stationing

Stationing shall be provided along the centerline of the proposed improvements. The stationing of street plans and profiles shall be from left to right and downstream to upstream in the case of channel improvement/construction projects, unless otherwise approved by the Engineering Department.

1.2.2.9 Details

Details must be provided on the plans for any structure requiring special design and not included in the City of Fort Smith Standard Drawings.

1.2.2.10 Floodplain and Floodway

FEMA regulated floodplain and floodway boundaries must be shown on the plans.

1.2.2.11 100-Year Water Surface Elevation and Minimum Floor Elevation

The water surface elevation resulting from the 100-year storm for all overland flow, including flow in the streets, parking lots, swales, and between lots, shall be shown on the plans. Minimum floor elevations for all lots must also be shown on the plans. Minimum floor elevations shall be one foot above the calculated 100-year water surface elevation of open channels, swales, or overland flow.

1.2.2.12 Erosion and Sediment Control Plan (Stormwater Pollution Prevention Plan); Grading Plan (Site Plan)

For construction sites that will disturb one or more acres, an Erosion and Sediment Control Plan (Stormwater Pollution Prevention Plan) identifying the type, location, and schedule for implementing erosion and sediment control measures, including appropriate provisions for maintenance and disposition of temporary measures, must be included in the plans. A Grading Plan (Site Plan) which shows the area to be disturbed and expected project sequencing, as well as the location and type of erosion controls to be installed, must be included with the Erosion and Sediment Control Plan. The Erosion and Sediment Control Plan and the Grading Plan must comply with the City of Fort Smith Fill and Grading Ordinance and with all applicable regulations set forth by the Arkansas Department of Environmental Quality.

1.3 RIGHT-OF-WAY AND EASEMENTS

All public drainage improvements shall be located in street right-of-way or in an easement dedicated to public use of the minimum widths as shown below:

1.3.1 *Enclosed Structures*

Enclosed structures require a minimum width of 15 feet or the width of the structure plus 10 feet, whichever is larger. Where required, an access easement shall also be provided.

1.3.2 *Open Channels*

Open channel easements shall be required to contain the entire channel design width including freeboard or the 100-year design storm, whichever is larger. The minimum width of an open channel easement shall be15 feet. Where required, an access easement shall also be provided.

1.3.3 *Stormwater Ponds and Wetlands*

A minimum width of 25 feet shall be provided around the 100-year flood pool connecting the tributary pipes and discharge system, as well as a 20-foot wide access easement.

1.3.4 *Access Easements*

Access easements shall be required to provide street access to drainage easements and right-of-way. Since every development differs in size and scope, the number and location of access easements must be determined by the Engineering Department on a case-by-case basis. The minimum width of access easements shall be 20 feet.

1.4 DRAINAGE WAYS

Storm water runoff shall not be discharged from a public drainage way onto private property within the boundaries of a development or subdivision.

Any drainage way which traverses a new development or subdivision shall be located within a public drainage easement or right-of-way.

All drainage outfalls, whether public or private, must be constructed at an elevation that allows for positive drainage onto the adjacent property.

1.5 BRIDGE DESIGN

Bridge design has not been addressed and is beyond the scope of this document. Any required bridge design will be reviewed by the Engineering Department on a case-by-case basis.

1.6 DRAINAGE REPORT/PLAN REQUIREMENT CHECKLIST

The Engineer must complete the Drainage Report/Plan Requirement Checklist found in Appendix 1A, including the certification, and submit it with the drainage report and construction plans for review. The drainage report and construction plans will not be reviewed until the Drainage Report/Plan Requirement Checklist is submitted.

APPENDIX 1A

DRAINAGE REPORT/PLAN REQUIREMENT CHECKLIST

THE CITY OF FORT SMITH

DRAINAGE REPORT/PLAN REQUIREMENT CHECKLIST

Project Name: ____

Project Address: ____

Owner/Developer: ____

Engineering Firm: ____

Revision No.: ____________________ **Date:** ________________________

Engineer: **Initial beside each number below in the space provided to acknowledge that the task is complete. If not applicable, mark "N/A" in the space. Sign the certification at the end of the document prior to submittal.**

DRAINAGE REPORT:

_____ 1. **Title and Date** – Include a title sheet which shows the project title and date of submission.

_____ 2. **Location of Project** – Include the street address and a vicinity map.

_____ 3. **Description of Project** – Include a brief description of the project.

_____ 4. **Contact Information** – Include the name, address, and telephone number of the Project Owner and Developer.

_____ 5. **Drainage Area Map** – Provide a topographic map with pre- and post-development drainage area(s) outlined.

_____ 6. **Area Drainage Issues** – Provide a description of any known on- or off-site drainage/flooding problems.

_____ 7. **Written Summary of the Proposed Improvements** – Include a summary of on- and off-site improvements, any increases in flows, detention or lack of detention, and final conclusions.

_____ 8. **Pre- and Post-Development Flowrates** – Include pre- and post-development flowrate calculations for the 10-, 25-, 50-, and 100-year storm events.

_____ 9. **Storm Water Detention Design** – Include all calculations.

_____ 10. **Open Channel Flow Design** – Include all calculations.

_____ 11. **Pavement Drainage Design** – Calculations for gutter spread must be included.

_____ 12. **Culvert Design** – All calculations (both inlet and outlet control) must be included.

_____ 13. **Inlet Design** – Capture efficiency calculations must be included.

_____ 14. **Storm Sewer Design** – All storm sewer design and hydraulic grade line calculations must be included.

_____ 15. **100-Year Water Surface Elevation and Minimum Floor Elevation** – Calculations must be included for the water surface elevation resulting from the 100-year storm for all overland flow, including flow in the streets, parking lots, swales, and between lots. Minimum floor elevations must also be included. Minimum floor elevations shall be one foot above the calculated 100-year water surface elevation of open channels, swales, or overland flow.

_____ 16. **Federal and State Requirements** – Include copies of documents which show compliance with all applicable federal and state requirements (Corps 404 Permit, ADEQ Notice of Intent, FEMA CLOMR, etc.).

_____ 17. **Certification by Registered Professional Engineer** – The title sheet must be sealed, signed, and dated by a Professional Engineer registered in the state of Arkansas.

Drainage Report/Plan Requirements Checklist
Project Name: ______________________
Owner/Developer: ____________________

PLAN REQUIREMENTS:

_____ 1. **Location of Project** – Include a vicinity map.

_____ 2. **Plan and Profile Views of All Proposed Improvements** – Plan and profile views must show the location, size, flowline elevations, gradients, materials, boring information and rock elevations (where applicable), and depths and sizes of adjacent or crossing utilities and structures.

_____ 3. **North Arrow and Scale** – Include a north arrow and scale on every applicable sheet.

_____ 4. **Bench Mark** – At least two permanent bench marks must be established and shown on the plans.

_____ 5. **Right-of-Way** – Plans must show the existing and proposed right-of-way and easements.

_____ 6. **Existing Structures and Utilities** – Plans must show the location of all existing structures, streets, driveways, storm drains, fences, trees, landscaping, utilities, and other features within 25 feet of the proposed improvements.

_____ 7. **Cross Sections** – Provide cross sections at a maximum of 50 foot intervals along the centerline of proposed improvements for a minimum width of 50 feet. Cross sections must show surface elevations, flowline elevations and sizes of all proposed improvements, and flowline elevations and sizes of all crossing or adjacent utilities or structures.

_____ 8. **Stationing** – Stationing shall be provided along the centerline of the proposed improvements.

_____ 9. **Details** – Provide details on the plans for any structure requiring special design and not included in the City of Fort Smith Standard Drawings.

_____ 10. **Floodplain and Floodway** – FEMA regulated floodplain and floodway boundaries must be shown on the plans.

_____ 11. **100-Year Water Surface Elevation and Minimum Floor Elevation** – The water surface elevation resulting from the 100-year storm for all overland flow, including flow in the streets, parking lots, swales, and between lots, shall be shown on the plans. Minimum floor elevations shall be one foot above the calculated 100-year water surface elevation of open channels, swales, or overland flow.

_____ 12. **Erosion and Sediment Control Plan (SWPPP); Grading Plan (Site Plan); Fill and Grading Permit** – An Erosion and Sediment Control Plan and Grading Plan must be included in the plans. The Fill and Grading Permit application must also be submitted for review and approval.

CERTIFICATION:

I hereby certify that the drainage report and accompanying plans for the project referenced above have been prepared in accordance with the requirements of the City of Fort Smith.

____

Engineer's Signature

____

Engineer's Printed Name

CHAPTER 2 – HYDROLOGY

2.1 GENERAL

Hydrology is generally defined as a science that addresses the interrelationship between water on and under the earth and in the atmosphere. For this manual, hydrology will address estimating flood magnitudes as the result of precipitation. In the design of drainage structures, floods are usually considered in terms of peak runoff or discharge in cubic feet per second (ft^3/s) and hydrographs as discharge per time. For structures that are designed to control the volume of runoff (e.g., detention storage facilities) or where flood routing through culverts is used, then the entire discharge hydrograph will be of interest.

2.2 DESIGN FLOW RATES

When designing drainage facilities, a design storm of a given return period is used to develop a design flow rate. For the given roadway classification, public drainage facilities within the City of Fort Smith shall be designed for the return period specified in the Table 2-1 below:

TABLE 2-1. Design Storm Selection Guidelines

Drainage Facility/Classification	Exceedence Probability	Return Period
Channels:		
All Classifications	4%	25-yr
Culverts:		
Major and Minor Arterial	2%	50-yr
All Other Classifications	4%	25-yr
Storm Drains:		
Residential	10%	10-yr
Residential Collector	10%	10-yr
Residential Collector (Restricted Parking)	10%	10-yr
Major Collector	4%	25-yr
Minor Arterial	2%	50-yr
Major Arterial	2%	50-yr
Boulevard	2%	50-yr
Industrial	4%	25-yr
All Other Systems	10%	10-yr

Provisions shall be made in drainage facilities to safely transmit the overflow from the 100-year storm event without flooding houses, buildings, structures, etc., and to provide for the health, safety, and welfare of persons and their property.

2.3 TIME OF CONCENTRATION

The time of concentration, which is denoted as t_c, is defined as the time required for a particle of water to flow from the hydraulically most distant point in the watershed to the outlet or design point. Factors that affect the time of concentration are the length of flow, the slope of the flow path and the roughness of the flow path. For flow at the upper reaches of a watershed, rainfall characteristics, most notably the intensity, may also influence the velocity of the runoff.

Various methods can be used to estimate the time of concentration of a watershed. When selecting a method to use in design, it is important to select a method that is appropriate for the flow path. Some estimation methods were designed and calibrated to be used for an entire watershed. These methods have t_c as the dependent variable. Other methods are intended for one segment of the principal flow path and produce a flow velocity that can be used with the length of that segment of the flow path to compute the travel time on that segment. With this method, the time of concentration equals the sum of the travel times on each segment of the principal flow path.

In classifying these methods so that the proper method can be selected, it is useful to describe the segments of flow paths. Sheet flow occurs in the upper reaches of a watershed. Such flow occurs over short distances and at shallow depths prior to the point where topography and surface characteristics cause the flow to concentrate in rills and swales. The depth of such flow is usually 0.8 in to 1.2 in or less. Concentrated flow is runoff that occurs in rills and swales and has depths on the order of 1.5 in to 4.0 in. Part of the principal flow path may include pipes or small streams. The travel time through these segments would be computed separately. Velocities in open channels are usually determined assuming bank-full depths.

2.3.1 *Sheet-Flow Travel Time*

Sheet flow is a shallow mass of runoff on a plane surface with the depth uniform across the sloping surface. Typically, flow depths will not exceed 2 in. Such flow occurs over relatively short distances. Sheet flow rates are commonly estimated using a version of the kinematic wave equation. The original form of the kinematic wave time of concentration is:

$$t_c = \frac{0.93}{I^{0.4}}\left(\frac{nL}{\sqrt{S}}\right)^{0.6} \tag{2.1}$$

in which t_c is the time of concentration (min), n is the roughness coefficient, L is the flow length (ft), I is the rainfall intensity (in./h) for a storm that has a return period T and duration of t_c (min), and S is the slope of the surface in ft/ft. Values of n can be obtained from Table 2-2.

The maximum allowable length for sheet flow shall be 100 feet, unless there is documented engineering justification to use a longer length. In no instance shall the length used exceed 300 feet.

TABLE 2-2. Roughness Coefficients (Manning's n) for Sheet Flow (AASHTO)

Surface Description	n[1]
Smooth surfaces (concrete, asphalt, gravel, bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤ 20 percent	0.06
Residue cover > 20 percent	0.17
Grasses:	
Short grass prairie	0.15
Dense grasses	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods:[2]	
Light underbrush	0.40
Dense underbrush	0.80

[1]The n values are a composite of information compiled in Reference (3).

[2]When selecting n, consider cover to a height of about 1 in. This is the only part of the plant cover that will obstruct sheet flow.

Some hydrologic design methods, such as the Rational method, assume that the storm duration equals the time of concentration. Thus, the time of concentration is entered into the IDF curve to find the design intensity. However, for Equation 2.1, i depends on t_c and t_c is not initially known. Therefore, the computation of t_c is an iterative process. An initial estimate of t_c is assumed and used to obtain i from the intensity-duration-frequency curve for the locality. The t_c is computed from Equation 2.1 and used to check the initial value of i. If they are not the same, then the process is repeated until two successive t_c estimates are the same.

2.3.2 *Velocity Method*

The velocity method can be used to estimate travel times for sheet flow, shallow concentrated flow, pipe flow, or channel flow. It is based on the concept that the travel time (T_t) for a flow segment is a function of the length of flow (L) and the velocity (V):

$$T_t = \frac{L}{60V} \tag{2.2}$$

in which T_t, L, and V have units of minutes, feet and feet/second, respectively. The travel time is computed for the principal flow path. Where the principal flow path consists of segments that have different slopes or land covers, the principal flow path should be divided into segments and Equation 2.2 used for each flow segment. The time of concentration is then the sum of travel times:

$$t_c = \sum_{i=1}^{k} T_{ti} = \sum_{i=1}^{k} \left(\frac{L_i}{60V_i} \right) \tag{2.3}$$

in which k is the number of segments and the subscript i refers to the flow segment.

The velocity of Equation 2.2 is a function of the type of flow (overland, sheet, rill and gully flow, channel flow, pipe flow), the roughness of the flow path, and the slope of the flow path. Some methods also include a rainfall index such as the 2-yr, 24-h rainfall depth. A number of methods have been developed for estimating the velocity.

After short distances, sheet flow tends to concentrate in rills and then gullies of increasing proportions. Such flow is usually referred to as shallow concentrated flow. The velocity of such flow can be estimated using an empirical relationship between the velocity and the slope:

$$V = kS^{0.5} \tag{2.4}$$

in which V is the velocity (ft/s) and S is the slope (%). The value of k is a function of the land cover, with values for selected land covers given in Table 2-3.

TABLE 2-3. Intercept Coefficients for Velocity vs. Slope Relationship of Equation 2.4

k	Land Cover/Flow Regime
0.249	Forest with heavy ground litter; hay meadow (overland flow)
0.499	Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland (overland flow)
0.699	Short grass pasture (overland flow)
0.899	Cultivated straight row (overland flow)
1.001	Nearly bare and untilled (overland flow); alluvial fans in western mountain regions
1.499	Grassed waterway (shallow concentrated flow)
1.611	Unpaved (shallow concentrated flow)
2.031	Paved area (shallow concentrated flow); small upland gullies

2.3.3 *Open Channels*

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs or where blue lines (indicating streams) appear on USGS quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bankfull condition.

Manning's equation is:

$$V = \frac{1.486 \; R^{2/3} \; S^{1/2}}{n} \qquad (2.5)$$

where:

V = average velocity, ft/s

R = hydraulic radius, ft (equal to A/WP)

A = cross sectional flow area, ft^2

WP = wetted perimeter, ft

S = slope of the hydraulic grade line, ft/ft

n = Manning's roughness coefficient

After average velocity is computed using Equation 2.5, T_t for the channel segment can be estimated using Equation 2.2.

2.4 HYDROLOGIC METHODS

There are numerous methods of rainfall computations on which the design of storm drainage and flood control systems are based. The methods to be used in the City of Fort Smith and the circumstances for their use are listed below:

- The Rational Method shall only be used for drainage areas less than 200 acres. If a watershed or basin involves a design time of concentration in excess of 30 minutes, then the applicability of the Rational Method must be checked.
- The NRCS TR-55 Tabular Hydrograph Method shall only be used for drainage areas between 100 and 2,000 acres.
- Suitable computer programs (e.g., HYDRO, HEC-1, HEC-HMS and TR-20) may be used to facilitate tedious hydrologic calculations. Programs must be approved by the Engineering Department.
- The 100-year discharges specified in the FEMA flood insurance study shall be used to analyze the impacts of a proposed crossing on a regulatory floodway. However, if these discharges are deemed to be outdated, the discharges based on current methods may be used subject to receipt of necessary regulatory approvals.
- Other methods may be used for drainage areas in excess of 2,000 acres, subject to approval by the Engineering Department.

2.4.1 *Rational Method*

The Rational method may be used for estimating the design storm peak runoff for areas as large as 200 acres. The Rational formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient and mean rainfall intensity for a duration equal to the time of concentration (the time required for water to flow from the most remote point of the basin to the location being analyzed). The Rational formula is expressed as follows:

$$Q = CIA \tag{2.7}$$

where:

Q is defined as the peak rate of runoff in cubic feet per second. Actually, Q is in units of acre-inches per hour, but calculator results differ from cubic feet by less than one percent. Since the difference is so small, the Q value calculated by the equation is universally taken as cubic feet per second, or *CFS*.

C is the dimensionless coefficient of runoff represented in the ratio of the amount of runoff to the amount of rainfall.

I is the average intensity of rainfall in inches per hour for a period of time equal to the critical time of full contribution of the drainage area under consideration. This critical time for full contribution is commonly referred to as time of concentration.

A is the area in acres that contributes to runoff at the point of design or the point under consideration.

Basic assumptions associated with use of the Rational Method are as follows:

- The computed peak rate of runoff to the design point is the function of the average rainfall rate during the time of concentration to that point.
- The time of concentration is the critical value in determining the design rainfall intensity and is equal to the time required for water to flow from the hydraulically most distant point in the watershed to the point of design.
- The ratio of runoff to rainfall, C, is uniform during the entire duration of the storm event.
- The rate of rainfall or rainfall intensity, I, is uniform for the entire duration of the storm event and is uniformly distributed over the entire watershed area.

2.4.1.1 Infrequent Storms

The coefficients given in Tables 2-5 and 2-6 are applicable for storms of 5-year to 10-year frequencies. Less frequent, higher intensity storms will require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff. See Reference *(4)*. The adjustment of the Rational method for use with major storms can be made by multiplying the right side of the Rational formula by a frequency factor C_f. The Rational formula now becomes:

$$Q = CC_fIA \tag{2.8}$$

C_f values are listed in Table 2-4.

TABLE 2-4. Frequency Factors for Rational Formula

Recurrence Interval (years)	C_f
25	1.1
50	1.2
100	1.25

The product of C_f times C shall not exceed 1.0.

TABLE 2-5. Recommended Coefficient of Runoff Values (For Various Selected Land Uses)

Description of Area	Runoff Coefficients
Business: Downtown areas	0.70–0.95
Neighborhood areas	0.50–0.70
Residential: Single-family areas	0.30–0.50
Multi units, detached	0.40–0.60
Multi units, attached	0.60–0.75
Suburban	0.25–0.40
Residential (2.5 ac lots or more)	0.30–0.45
Apartment dwelling areas	0.50–0.70
Industrial: Light areas	0.50–0.80
Heavy areas	0.60–0.90
Parks, cemeteries	0.10–0.25
Playgrounds	0.20–0.40
Railroad yard areas	0.20–0.40
Unimproved areas	0.10–0.30

Source: HDS No. 2 *(1)*.

TABLE 2-6. Coefficients for Composite Runoff Analysis

Surface	Runoff Coefficients
Streets: Asphalt	0.70–0.95
Concrete	0.80–0.95
Drives and walks	0.75–0.85
Roofs	0.75–0.95

Source: HDS No. 2 *(1)*.

2.4.1.2 Runoff Coefficient

The runoff coefficient, C, is the variable of the Rational method least amenable to precise determination and requires the judgment and understanding of the designer. Although engineering judgment will always be required in the selection of runoff coefficients, a typical coefficient represents the integrated effects of many drainage basin parameters; the following discussion considers only the effects of land use.

Two methods for determining the runoff coefficient are presented based on land use (Table 2-5) and a composite coefficient for complex watersheds (Table 2-6).

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surface in the drainage area. Composites can be made with Tables 2-5 and 2-6. The composite procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide to the selection of reasonable values of the coefficient for an entire area.

2.4.1.3 Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate (in./h) for a duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from Rainfall-Intensity-Duration curves. Curves for use within the City of Fort Smith are given in Appendix 2A.

2.4.1.4 Drainage Area

The drainage area (A) is measured in acres when using the Rational Method. Drainage areas should be delineated and calculated using 2-foot contour information. Such information may be obtained through the Engineering Department.

2.4.1.5 Example Problem – Rational Method

Following is an example problem that illustrates the application of the Rational method to estimate peak discharges.

Preliminary estimates of the maximum rate of runoff are needed at the inlet to a culvert for a 10-yr and 50-yr return period.

Site Data

From a topographic map and field survey, the area of the drainage basin upstream from the point in question is found to be 5 ac. In addition, the following data were measured:

Length of overland flow = 265 ft
Length of grassed waterway = 65 ft
Average overland slope = 0.3 percent
Average grassed waterway slope = 0.5 percent

Land Use

From existing land-use maps, land use for the drainage basin was estimated to be:

Residential (1.2-ac lots) = 80 percent
Playground = 20 percent

Step 1 Calculate Time of Concentration

For overland flow (short grass pasture) from Table 2-3, k = 0.699:

$$V = (0.699)(0.3)^{0.5} = 0.4 \text{ ft/s}$$

$$T_t = \frac{265\ ft}{(0.4\ ft/s)(60\ s/\min)} = 11.1 \text{ min}$$

For grassed waterway (shallow concentrated flow) from Table 2-3, k = 1.499:

$$V = (1.499)(0.5)^{0.5} = 1.1 \text{ ft/s}$$

$$T_t = \frac{65 ft}{(1.1 ft/s)(60 s/\min)} = 1.0\ min$$

$$T_c = T_t \text{ (overland flow)} + T_t \text{ (grassed waterway)} = 12.1 \text{ min}$$

Step 2 Determine Rainfall Intensity

From Appendix 2B with duration equal to 12.1min:

I_{10} = 5.8 in./hour
I_{50} = 7.3 in./hour

Step 3 Determine C, Runoff Coefficient

A weighted runoff coefficient C for the total drainage area is determined in the following table by utilizing the values from Table 2-6

:

Land Use	(1) Percent of Total Land Area	(2) Runoff Coefficient	(3) Weighted Runoff Coefficient[1]
Residential (1.2 ac lots)	80%	0.3	0.24
Playground	20%	0.2	0.04
Total Weighted Runoff			0.28
[1]Column 3 equals Column 1 multiplied by Column 2.			

Step 4 Determine Peak Runoff

From the Rational equation:

$$Q_{10} = CIA = (0.28)(5.8 \text{ in./h})(5.0 \text{ ac}) = 8 \text{ ft}^3/\text{s}$$

$$Q_{50} = C_fCIA = (1.2)(0.28)(7.3 \text{ in./h})(5.0 \text{ ac}) = 12 \text{ ft}^3/\text{s}$$

Note: $C_f = 1.2$ from Table 2-4.

These are the estimates of peak runoff for a 10- and 50-yr design storm for the given basin.

2.4.2 *NRCS TR-55, Tabular Hydrograph Method*

The NRCS tabular method is a synthetic hydrograph method developed specifically for use in urbanized and urbanizing areas. This method is similar to the Rational Method in that runoff is directly related to rainfall amounts through use of Runoff Curve Numbers. The basic equation used with the tabular method is also very similar to the one used for the Rational Method:

$$q = q_t A_m Q \tag{2.9}$$

where:

q = hydrograph coordinate (ft^3/s) at hydrograph time t
q_t = tabular hydrograph unit discharge (csm/in.)
A_m = drainage area of individual subarea (mi^2)
Q = accumulated direct runoff (in.)

Note: csm/in. = cubic feet per second per square mile per inch of runoff

Hydrograph coordinates are computed from the hydrograph distribution data in the TR-55 Manual. A coordinated value is computed for each time shown in the distribution data. The calculated q results, when plotted against the corresponding times, constitute the runoff hydrograph.

The tabular method is useful in analyzing watersheds involving several subareas with complex runoff patterns. The method is most useful in analyzing changes in runoff volume due to development and in evaluating runoff control measures. The NRCS tabular method as described here shall be used in all cases where watershed problems involve two or more interacting subareas.

2.4.2.1 Accumulated Direct Runoff

A relationship between accumulated rainfall and accumulated runoff was derived by NRCS from experimental plots for numerous soils and vegetative cover conditions. Data for land-treatment measures (e.g., contouring, terracing) from experimental watersheds were included. The equation was developed mainly for small watersheds for which only daily rainfall and watershed data are ordinarily available. It was developed from recorded storm data that included the total amount of rainfall in a calendar day but not its distribution with respect to time. The NRCS runoff equation is therefore a method of estimating direct runoff from a 24-hour or 1-day storm rainfall. The equation is:

$$Q = (\mathrm{P} - \mathrm{I_a})^2 / (\mathrm{P} - \mathrm{I_a}) + \mathrm{S} \tag{2.10}$$

where:

Q = accumulated direct runoff (in.)
P = accumulated rainfall (potential maximum runoff) (in.)
I_a = initial abstraction including surface storage, interception and infiltration prior to runoff (in.)
S = potential maximum retention (in.)

The relationship between I_a and S was developed from experimental watershed data. It removes the necessity for estimating I_a for common usage. The empirical relationship used in the NRCS runoff equation is:

$$\mathrm{I_a} = 0.2\mathrm{S} \tag{2.11}$$

Substituting 0.2S for I_a in Equation 2.10, the NRCS rainfall-runoff equation becomes:

$$Q = (\mathrm{P} - 0.2\mathrm{S})^2 / (\mathrm{P} + 0.8\mathrm{S}) \tag{2.12}$$

The accumulated rainfall, P, is based on the 24-hour rainfall amount for the design recurrence interval of interest. The 24-hour rainfall amounts for the City of Fort Smith are taken from the U.S. Weather Bureau's *Technical Paper No. 40* and are listed below:

10-year frequency	P = 6.3 in.
25-year frequency	P = 7.3 in.
50-year frequency	P = 8.2 in.
100-year frequency	P = 9.2 in.

The potential maximum retention, S, is related to the soil and cover conditions of the watershed through the NRCS runoff curve number, CN. CN has a range of 0 to 100, and S is related to CN by:

$$S = (1000 / CN) - 10 \tag{2.13}$$

The runoff curve number is discussed in greater detail in Section 2.4.2.2 below. Typical values for runoff curve numbers are shown in Table 2-8.

Figure 2-1 shows a graphical solution of equation 2.12, which enables the precipitation excess from a storm to be obtained if the accumulated rainfall and watershed curve number are known.

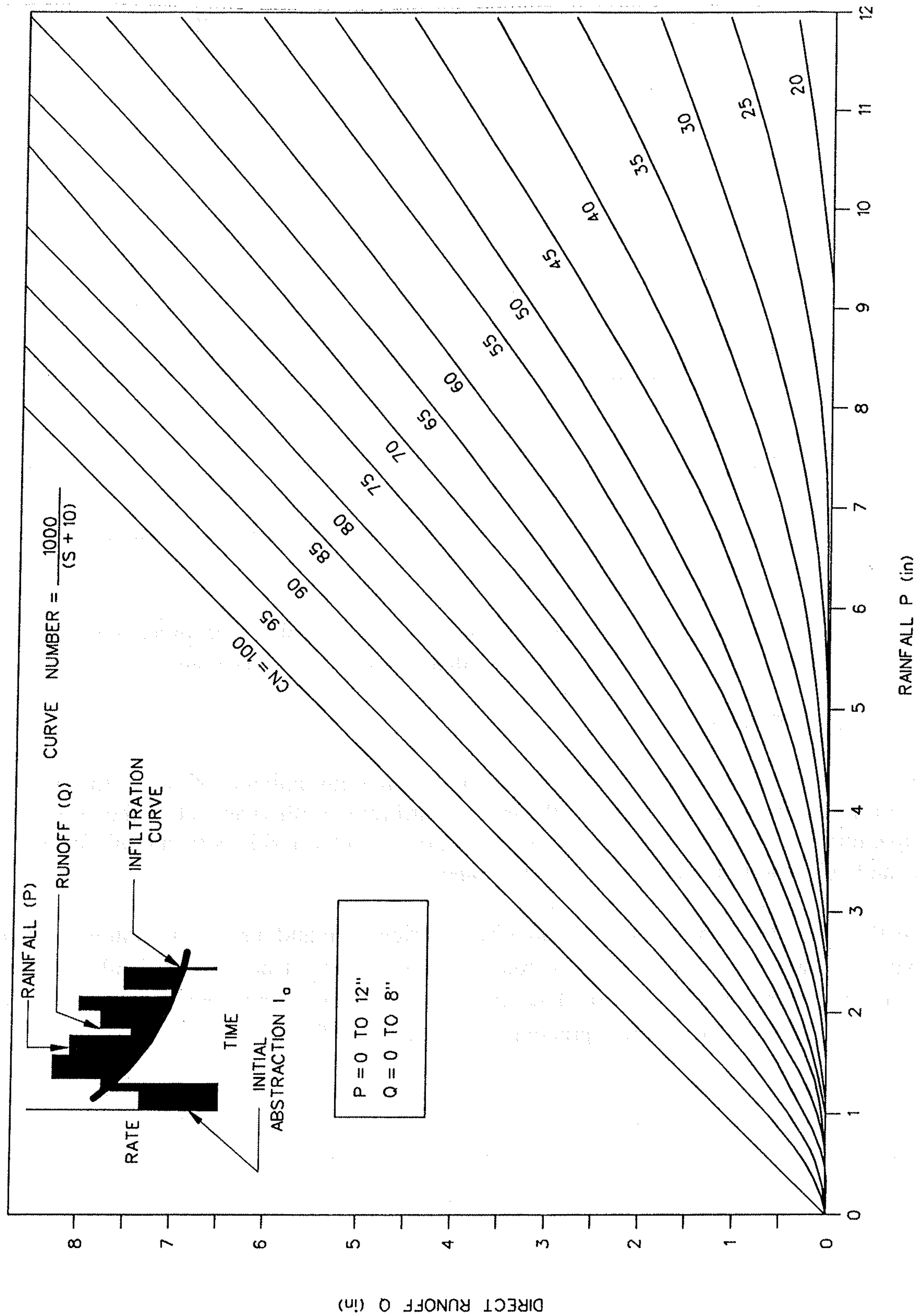
2.4.2.2 Runoff Factor

Often times, runoff is referred to as rainfall excess or effective rainfall, all defined as the amount by which rainfall exceeds the capability of the land to infiltrate or otherwise retain the rainwater. The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types and land slope.

Land use is the watershed cover, and it includes both agricultural and nonagricultural uses. Items such as type of vegetation, water surfaces, roads and roofs are all part of the land use. Land treatment applies mainly to agricultural land use, and it includes mechanical practices (e.g., contouring, terracing) and management practices (e.g., rotation of crops).

NRCS uses a combination of soil conditions and land use (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area when the soil is not frozen. The higher the CN, the higher the runoff potential.

FIGURE 2-1. NRCS Relation Between Direct Runoff, Curve Number and Precipitation



Soil properties influence the relationship between runoff and rainfall because soils have differing rates of infiltration. Infiltration is the movement of water through the soil surface into the soil. Based on infiltration rates, NRCS has divided soils into four hydrologic soil groups as follows:

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

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

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

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

Soil classifications important to the City of Fort Smith are given in Appendix 2B.

Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. Also, runoff curve numbers vary with the antecedent soil moisture conditions, defined as the amount of rainfall occurring in a selected period preceding a given storm. In general, the greater the antecedent rainfall, the more direct runoff there is from a given storm. A five-day period is used as the minimum for estimating antecedent moisture conditions. Antecedent soil moisture conditions also vary during a storm; heavy rain falling on a dry soil can change the soil moisture condition from dry to average to wet during the storm period.

Table 2-7 presents the NRCS curve number values for the different land uses, treatments and hydrologic conditions; separate values are given for each soil group. For example, the CN for a wooded area with good cover and soil group B is 55; for soil group C, the CN would increase to 70. If the cover (on soil group B) is poor, the CN will be 66.

Table 2-7 is based on an average antecedent moisture condition; i.e., soils that are neither very wet nor very dry when the design storm begins. Curve numbers should be selected only after a field inspection of the watershed and a review of zoning and soil maps. Table 2-8 gives conversion factors to convert average curve numbers to wet curve numbers. **Only curve numbers for wet conditions shall be used in the design of storm drainage systems.**

TABLE 2-7. Runoff Curve Numbers (Average Watershed Condition, $I_a = 0.2S$) (After Reference *(4)*)

Cover Type	Hydrologic Condition[d]	Curve Numbers for Hydrologic Soil Group A	B	C	D
Fully developed urban areas[a] (vegetation established)					
Lawns, open spaces, parks, golf courses, cemeteries, etc.					
Good condition; grass cover on 75% or more of the area		39	61	74	80
Fair condition; grass cover on 50% to 75% of the area		49	69	79	84
Poor condition; grass cover on 50% or less of the area		68	79	86	89
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads					
Paved with curbs and storm sewers (excluding right-of-way)		98	98	98	98
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Paved with open ditches (including right-of-way)		83	89	92	93
	Average % impervious[b]				
Commercial and business areas	85	89	92	94	95
Industrial districts	72	81	88	91	93
Row houses, town houses, and residential with lots sizes $^1/_8$ ac or less	65	77	85	90	92
Residential: average lot size					
$^1/_4$ ac	38	61	75	83	87
$^1/_3$ ac	30	57	72	81	86
$^1/_2$ ac	25	54	70	80	85
1 ac	20	51	68	79	84
2 ac	12	46	65	77	82
Western desert urban areas:					
Natural desert landscaping (pervious areas only)		63	77	85	88

Artificial desert landscaping (impervious weed barrier, desert shrub with 1-in. to 2-in. sand or gravel mulch and basin borders)			96	96	96	96
Developing urban areas[c] (no vegetation established) Newly graded area			77	86	91	94
Cultivated Agricultural Land: Fallow						
Straight row or bare soil			77	86	91	94
Conservation tillage		Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row	Poor	72	81	88	91
		Good	67	78	85	89
	Conservation tillage	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured	Poor	70	79	84	88
		Good	65	75	82	86
	Contoured and tillage	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured and terraces	Poor	66	74	80	82
		Good	62	71	78	81
	Contoured and terraces and conservation tillage	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Conservation tillage	Poor	64	75	83	86
		Good	60	72	80	84
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	Contoured and tillage	Poor	62	73	81	84
		Good	60	72	80	83
	Contoured and terraces	Poor	61	72	79	82
		Good	59	70	78	81
	Contoured and terraces and conservation tillage	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadows[e]	Straight row	Poor	66	77	85	89
		Good	58	72	81	85
	Contoured	Poor	64	75	83	85
		Good	55	69	78	83
	Contoured and terraces	Poor	63	73	80	83
		Good	57	67	76	80
Noncultivated agricultural land						
Pasture or range	No Mechanical treatment[i]	Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
		Fair	25	59	75	83
		Good	6	35	70	79
Meadow—continuous grass, protected from grazing and generally mowed for hay			30	58	71	78
Forestland—grass or orchards—evergreen or Deciduous		Poor	55	73	82	86
		Fair	44	65	76	82
		Good	32	58	72	79
		Poor	48	67	77	83

Brush—brush-weed-grass mixture with brush the major element[g]	Fair	35	56	70	77
	Good	30[f]	48	65	73
Woods	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30[f]	55	70	77
Woods—grass combination (orchard or tree farm)[h]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Farmsteads		59	74	82	86
Forest-range					
Herbaceous—mixture of grass, weeds, and low-growing brush, with brush the minor element	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen—mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon—juniper - pinyon, juniper, or both grass understory	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sage-grass	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub—major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite and cactus	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

[a] For land uses with impervious areas, curve numbers are computed assuming that 100 percent of runoff from impervious areas is directly connected to the drainage system. Pervious areas (lawn) are considered to be equivalent to lawns in good condition and the impervious areas have a *CN* of 98.

[b] Includes paved streets.

[c] Use for the design of temporary measures during grading and construction. Impervious area percent for urban areas under development vary considerably. The user will determine the percent impervious. Then using the newly graded area *CN*, the composite *CN* can be computed for any degree of development.

[d] For conservation tillage poor hydrologic condition, 5 to 20 percent of the surface is covered with residue (less than 750 lb/ac row crops or 300 lb/ac small grain). For conservation tillage good hydrologic condition, more than 20 percent of the surface is covered with residue (greater than 750 lb/ac row crops or 300 lb/ac small grain).

[e] Close-drilled or broadcast.

- For noncultivated agricultural land:
  - Poor hydrologic condition has less than 25 percent ground cover density.
  - Fair hydrologic condition has between 25 percent and 50 percent ground cover density.
  - Good hydrologic condition has more than 50 percent ground cover density.
- For forest-range:
  - Poor hydrologic condition has less than 30 percent ground cover density.
  - Fair hydrologic condition has between 30 percent and 70 percent ground cover density.
  - Good hydrologic condition has more than 70 percent ground cover density.

[f] Actual curve number is less than 30: Use $CN = 30$ for runoff computations.

[g] *CN*s shown were computed for areas with 50 percent woods and 50 percent grass (pasture) cover. Other combinations of conditions may be computed from the *CN*'s for woods and pasture.

[h] Poor: < 50 percent ground cover.
Fair: 50 to 75 percent ground cover.
Good: > 75 percent ground cover.

[i] Poor: < 50 percent ground cover or heavily grazed with no mulch.
Fair: 50 to 75 percent ground cover and not heavily grazed.
Good: > 75 percent ground cover and lightly or only occasionally grazed.

TABLE 2-8. Conversion from Average Antecedent Moisture Conditions to Dry and Wet Conditions

CN For Average Conditions	CN For Wet Conditions
100	100
95	98
90	96
85	94
80	91
75	88
70	85
65	82
60	78
55	74
50	70
45	65
40	60
35	55
30	50
25	43
15	30
5	13

Source: Reference *(2)*.

2.4.2.3 Estimation of CN Values for Urban Land Uses

The CN table (Table 2-7) includes CN values for a number of urban land uses. For each of these, the CN is based on a specific percentage of imperviousness. For example, the CN values for commercial land use are based on an imperviousness of 85 percent. Curve numbers for other percentages of imperviousness can be computed using a weighted CN approach, with a CN of 98 used for the impervious areas and the CN for open space (good condition) used for the pervious portion of the area. Thus, CN values of 39, 61, 74, and 80 are used for hydrologic soil groups A, B, C, and D, respectively. These are the same CN values for pasture in good condition. Thus, the following equation can be used to compute a weighted CN:

$$CN_w = CN_p(1-f) + f(98) \tag{2.14}$$

in which f is the fraction (not percentage) of imperviousness. To show the use of Equation 2.14, the CN values for commercial land use with 85 percent imperviousness are:

A soil: $39(0.15) + 98(0.85) = 89$
B soil: $61(0.15) + 98(0.85) = 92$
C soil: $74(0.15) + 98(0.85) = 94$
D soil: $80(0.15) + 98(0.85) = 95$

These are the same values shown in Table 2-7.

2.4.2.4 Parameter I_a/P

I_a/P is a parameter that is necessary to determine the tabular hydrograph unit discharges, q_t, from the tables shown in Appendix 2C. I_a denotes the initial abstraction, and P is the 24-hour rainfall depth for a selected return period. The I_a/P value can be obtained from Table 2-10 for a given CN and P. For a given 24-h rainfall distribution, I_a/P represents the fraction of rainfall that must occur before runoff begins.

2.4.2.5 Limitations of the NRCS TR-55, Tabular Hydrograph Method

The Tabular Hydrograph method does have several limitations. This method shall not be used if: T_t is greater than 3 hours, T_c is greater than 2 hours, drainage areas of individual subareas differ by a factor of 5 or more, the entire composite flood hydrograph or entire runoff volume is required for detailed flood routings, or the time of peak discharge must be more accurate than that obtained through the Tabular Hydrograph method. If any of these conditions apply, a hydrograph method such as TR-20 shall be used *(3)*.

TABLE 2-10. I_a/P for Selected Rainfall Depths and Curve Numbers

Rainfall (in.)	Curve Number											
	40	45	50	55	60	65	70	75	80	85	90	95
0.4	*	*	*	*	*	*	*	*	*	*	*	0.27
0.8	*	*	*	*	*	*	*	*	*	0.45	0.28	0.13
1.2	*	*	*	*	*	*	*	*	0.42	0.3	0.19	+
1.6	*	*	*	*	*	*	*	0.42	0.32	0.22	0.14	+
2	*	*	*	*	*	*	0.44	0.34	0.25	0.18	0.11	+
2.4	*	*	*	*	*	0.46	0.36	0.28	0.21	0.15	+	+
2.8	*	*	*	*	0.48	0.39	0.31	0.24	0.18	0.13	+	+
3.1	*	*	*	*	0.42	0.34	0.27	0.21	0.16	0.11	+	+
3.5	*	*	*	0.46	0.38	0.3	0.24	0.19	0.14	0.1	+	+
3.9	*	*	*	0.42	0.34	0.27	0.22	0.17	0.13	+	+	+
4.3	*	*	0.46	0.38	0.31	0.25	0.2	0.15	0.12	+	+	+
4.7	*	*	0.42	0.35	0.28	0.23	0.18	0.14	0.11	+	+	+
5.1	*	0.48	0.39	0.32	0.26	0.21	0.17	0.13	0.1	+	+	+
5.5	*	0.44	0.36	0.3	0.24	0.2	0.16	0.12	+	+	+	+
5.9	*	0.41	0.34	0.28	0.23	0.18	0.15	0.11	+	+	+	+
6.3	0.48	0.39	0.32	0.26	0.21	0.17	0.14	0.11	+	+	+	+
6.7	0.45	0.37	0.3	0.24	0.2	0.16	0.13	0.1	+	+	+	+
7.1	0.42	0.34	0.28	0.23	0.19	0.15	0.12	+	+	+	+	+
7.5	0.4	0.33	0.27	0.22	0.18	0.14	0.11	+	+	+	+	+
7.9	0.38	0.31	0.25	0.21	0.17	0.14	0.11	+	+	+	+	+
8.3	0.36	0.3	0.24	0.2	0.16	0.13	0.1	+	+	+	+	+
8.7	0.35	0.28	0.23	0.19	0.15	0.12	0.1	+	+	+	+	+
9.1	0.33	0.27	0.22	0.18	0.15	0.12	+	+	+	+	+	+
9.4	0.32	0.26	0.21	0.17	0.14	0.11	+	+	+	+	+	+
9.8	0.3	0.25	0.2	0.17	0.14	0.11	+	+	+	+	+	+
10.2	0.29	0.24	0.2	0.16	0.13	0.11	+	+	+	+	+	+
10.6	0.28	0.23	0.19	0.15	0.13	0.1	+	+	+	+	+	+
11	0.27	0.22	0.18	0.15	0.12	0.1	+	+	+	+	+	+
11.4	0.26	0.21	0.18	0.14	0.12	+	+	+	+	+	+	+
11.8	0.25	0.21	0.17	0.14	0.11	+	+	+	+	+	+	+
12.2	0.25	0.2	0.16	0.13	0.11	+	+	+	+	+	+	+
12.6	0.24	0.19	0.16	0.13	0.11	+	+	+	+	+	+	+
13	0.23	0.19	0.15	0.13	0.1	+	+	+	+	+	+	+
13.4	0.22	0.18	0.15	0.12	0.1	+	+	+	+	+	+	+
13.8	0.22	0.18	0.15	0.12	0.1	+	+	+	+	+	+	+
14.2	0.21	0.17	0.14	0.12	+	+	+	+	+	+	+	+
14.6	0.21	0.17	0.14	0.11	+	+	+	+	+	+	+	+
15	0.2	0.16	0.13	0.11	+	+	+	+	+	+	+	+
15.4	0.2	0.16	0.13	0.11	+	+	+	+	+	+	+	+
15.7	0.19	0.16	0.13	0.1	+	+	+	+	+	+	+	+

* $I_a/P = 0.50$ should be used.
+ $I_a/P = 0.10$ should be used.

2.4.3 *Computer Methods*

Suitable computer programs (such as TR-20, HEC-1, HEC-HMS, etc.) may be used to determine and route runoff hydrographs. All programs used must have approval of the Engineering Department.

2.5 REFERENCES

(1) FHWA. *Highway Hydrology.* Hydraulic Design Series No. 2, FHWA-SA-96-067. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 1996.

(2) Soil Conservation Service. *A Method for Estimating Volume and Rate of Runoff in Small Watersheds.* Technical Publication 149. Natural Resources Conservation Service, Washington, DC, Revised April 1973.

(3) Soil Conservation Service. *Urban Hydrology for Small Watersheds.* Technical Release No. 55, Ratural Resources Conservation Service, Washington, DC, 1986.

(4) Wright-McLaughlin Engineers. *Urban Drainage and Flood Control Criteria Manual and Handbook.* Denver Regional Council of Government in Denver, Colorado, 1969.

APPENDIX 2A

I-D-F CURVES
(Source: AHTD)

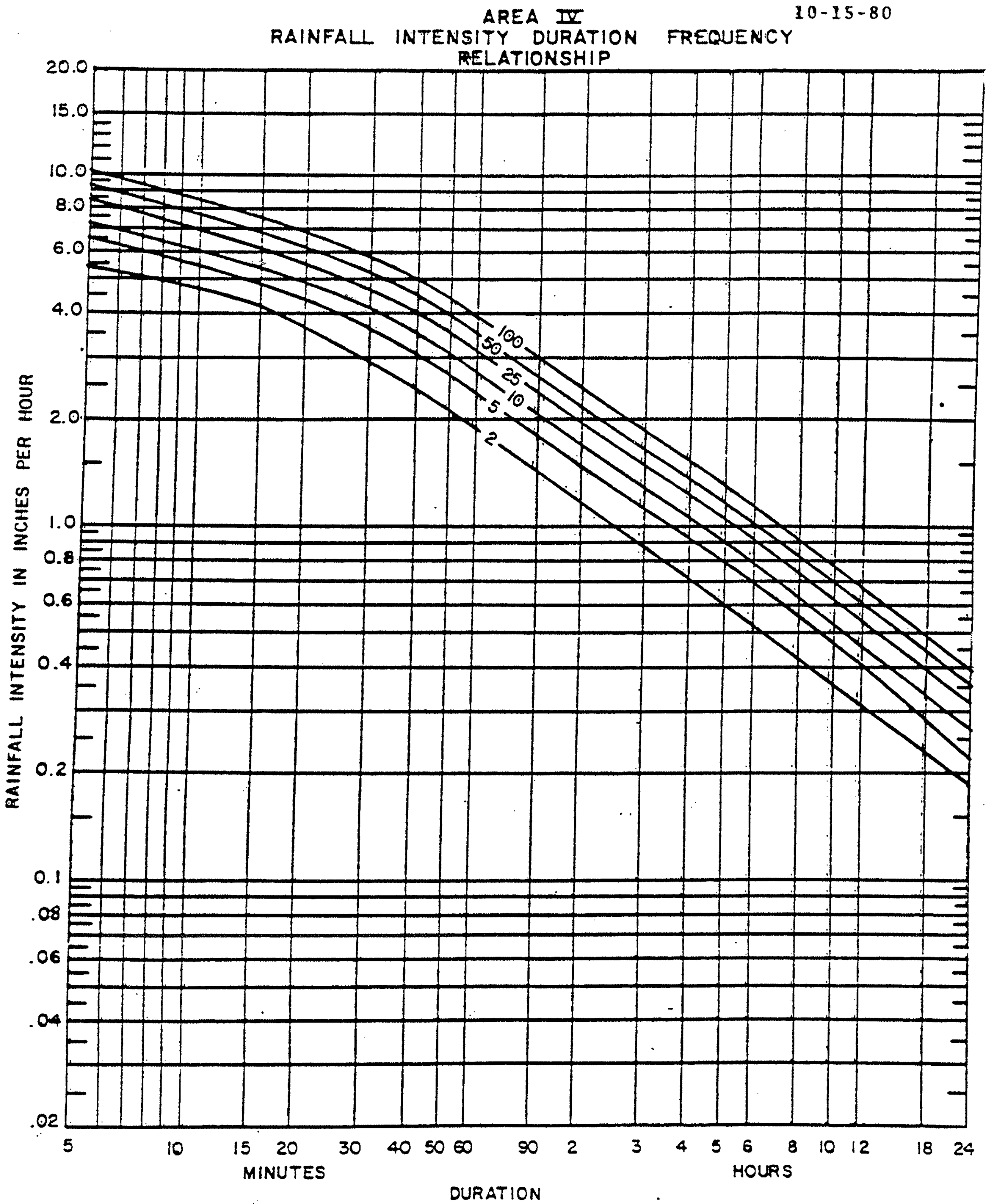


FIGURE 2A-1. Area IV Rainfall Intensity Duration Frequency Relationship

APPENDIX 2B

SOIL CLASSIFICATIONS
(Source: AHTD)

Table 3-7

Soil Names and Hydrological Classification in Arkansas

Acadia D
Adaton D
Agnos B
Alaga A
Allen B
Alligator D
Amagon D
Amy D
Angie C
Apison B
Arkabutla C
Ashton B
Ashwood C
Askew C
Avilla B

Barling C
Baxter B
Beasley C
Beulah B
Bibb D
Billyhaw D
Blevins B
Boden C
Bodine B
Bonn D
Bosket B
Boswell D
Bowdre C
Bowie B
Brandon B
Briley B
Britwater B
Brocket C
Broseley B
Bruno A
BudeC
Buxin D
Brockwell B

Caddo D
Cahaba B
Calhoun D
Calloway C
Cane C
Captina C
Carnasaw C
Carytown D
Cascilla B
Caspiana B
Catalpa C
Ceda B
Chastain D
Chennely C
Cherokee D
Christian C
Clarksville B
Clebit D
Cleora B
Collins C
Commerce C
Conasauga C
Convent C
Corydon D
Coushatta B
Crevassee A
Crowley D
Cuthbert C

Darco A
Dardanelle B
Demopolis C
Desha D
Dexter B
Doniphan B
Dubbs B
Dundee C

Dunning D

Earle D
Egam C
Elsah B
Emory B
Enders C
Ennis B
Estate C
Etowah B
Eutaw D
Eylau C

Falaya C
Falkner C
Fatima B
Fayetteville B
Felker D
Foley D
Forrestdale D
Fountain D

Gallion B
Gasconade D
Gassville C
Geep B
Gladwater D
Goldsboro C
Goldston C
Gore D
Grenda C
Grubbs D
Guin A
Guthrie D
Guyton D

Harleston C
Hartsells B
Hatchie C
Hayti D
Healing B
Hebert C
Hector D
Henry D
Hillemann C
Hollywood D
Holston B
Houlka D
Houston D
Huntington B

Iberia D
Iuka C
Izagora C

Jackport D
Jay C
Jeanerette D
Johnsburg D

Kalmia B
Kamie B
Karma B
Kaufman D
Keo B
Kiematia A
Kipling D
Kirvin C
Kobel D

Lafe D
Lagrange D
Latanier D
Latonia B
Leadvale C
Leaf D

Leeper D
Leesburg B
Lexington B
Lindside C
Lily B
Linker B
Lobelville C
Locust C
Lonoke B
Loring C
Lucy A
Luverne C

Mantachie C
Marietta C
Marvell B
Mashulaville D
Mayes D
Mayhew D
McCrory D
McGehee C
McKamic D
McLaurin B
Melvin D
Memphis B
Mhoon D
Miller D
Millwood D
Moko C
Monongahela C
Montevallo D
Moreland D
Morganfield B
Morse D
Mountainburg D
Muldrow D
Muskogee C
Myatt D

Nacogdoches B
Natchez B
Neila B
Newardk C
Newellton D
Newtonia B
Nixa C
Noark B
Norfolk B
Norwood B
Nugent A

Oaklimeter C
Oklared B
Ochlockonee B
Oktibbeha B
Ora C
Orangeburg B
Ouachita C
Ozan D

Patterson C
Pembroke B
Peridge B
Perry D
Pheba D
Philo C
Pickens D
Pickwick B
Pikeville B
Pirum B
Pipe D
Protia C
Portland D
Prentiss C
Providence C

Ramsey D
Razort B
Rexor A
Rilla B
Roanoke D
Robinsville B
Roellen D
Routon D
Ruston B

Sacul D
Saffell B
Sallisaw B
Samba D
Sardis C
Savannah C
Sawyer C
Secesh B
Sequatchie B
Sessum D
Sharkey D
Sherwood B
Shubuta C
Sidon C
Sloan D
Smithdale B
Smithton D
Sogn D
Spadra B
Stanser B
State B
Steele B
Steprock B
Sterlington B
Stough C
Sturkie B
Stuttgart D
Summit C
Sumter C
Susquehanna D

Taft C
Talbott C
Taloka D
Terouge D
Tiak C
Tichnor D
Tippah C
Tiptonville B
Toine B
Townley C
Trebloc D
Trinity D
Troup A
Tuckerman D
Tunica D
Tuscumbia D
Tutwiler B

Una D

Vaiden D
Ventris D

Wabbaseka D
Wabeh B
Wardell C
Waverly D
Waynesboro B
Weston D
Wickham B
Wideman A
Wilcox D
Wilson S

TABLE 2B-1. Names of Hydrological Classification in Arkansas

APPENDIX 2C

TABULAR HYDROGRAPH UNIT DISCHARGES
(Source: TR-55)

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution

HYDROGRAPH TIME (HOURS)

IA/P = 0.10 * * * TC = 0.1 HR * * *

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
0.0	29	38	57	172	241	425	662	531	345	265	191	130	101	83	68	62	58	54	50	44	41	37	32	27	23	21	19	16	14	13	11	0
.10	26	32	47	98	147	210	353	559	540	410	313	231	164	101	80	67	61	57	53	47	43	39	34	28	24	22	19	17	14	13	11	0
.20	25	31	44	86	127	182	296	471	517	446	357	273	200	117	86	70	63	58	54	48	44	39	34	29	24	22	20	17	14	13	11	0
.30	22	28	37	57	76	110	158	250	398	477	457	390	312	178	111	83	69	62	57	51	45	41	36	31	25	23	20	18	15	13	11	0
.40	21	27	35	53	68	96	137	213	336	430	448	410	345	210	128	90	72	64	58	52	46	41	36	31	26	23	20	18	15	13	11	0
.50	19	24	30	43	49	62	85	120	182	284	382	426	415	305	188	120	86	71	63	55	49	43	38	33	27	24	21	19	15	14	11	0
.75	17	22	27	37	41	49	62	84	120	181	258	327	375	353	264	177	120	88	72	59	52	45	39	34	29	25	22	20	15	14	11	0
1.0	13	17	22	27	30	33	37	43	52	66	91	131	190	315	358	307	220	149	104	72	60	50	43	37	32	27	23	21	16	14	12	0
1.5	9	11	14	18	19	21	23	25	27	29	33	37	44	70	134	229	304	318	269	172	106	68	52	44	38	33	28	24	19	15	12	2
2.0	6	8	10	13	14	15	16	17	19	20	22	24	26	32	45	73	130	207	271	292	216	121	68	51	43	37	32	27	21	16	13	6
2.5	3	4	6	8	9	10	10	11	12	13	14	16	17	20	23	29	38	57	97	189	271	244	136	75	53	44	38	33	24	19	14	9
3.0	1	2	4	5	6	6	7	8	8	9	10	11	12	14	16	19	23	28	38	74	146	256	226	131	74	53	44	37	27	21	14	10

IA/P = 0.30 * * * TC = 0.1 HR * * *

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
0.0	0	0	0	48	106	296	597	496	368	300	221	155	125	106	89	83	79	74	69	62	59	54	47	40	35	32	28	25	22	20	17	0
.10	0	0	0	35	82	225	473	488	408	336	260	190	147	113	94	85	80	75	70	63	59	54	48	40	35	32	29	25	22	20	17	0
.20	0	0	0	7	26	64	171	372	449	422	365	295	225	142	109	92	84	79	74	66	61	56	50	43	36	33	30	26	22	20	17	0
.30	0	0	0	5	19	49	130	291	397	414	381	323	258	161	118	96	86	80	75	68	62	57	50	43	37	33	30	27	22	20	17	0
.40	0	0	0	0	3	14	37	99	227	340	389	384	343	229	152	113	94	85	79	71	65	59	52	46	38	34	31	28	23	21	17	0
.50	0	0	0	0	2	10	28	75	177	286	355	374	354	256	170	123	99	87	80	73	66	60	53	46	39	35	31	28	23	21	18	0
.75	0	0	0	0	0	1	4	13	35	86	161	238	296	325	266	194	141	110	93	80	71	63	56	50	43	37	33	30	24	21	18	0
1.0	0	0	0	0	0	0	0	2	6	19	48	99	165	282	311	264	197	144	112	88	77	67	59	52	45	39	34	31	24	22	18	0
1.5	0	0	0	0	0	0	0	0	0	0	0	1	4	29	99	197	265	277	236	162	113	84	69	60	53	46	39	35	28	23	19	2
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	35	94	172	233	253	196	124	83	68	59	52	45	39	31	25	20	8
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	37	88	184	235	201	122	83	67	59	52	45	34	27	21	13
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	38	110	222	202	131	88	69	60	52	39	31	22	15

IA/P = 0.50 * * * TC = 0.1 HR * * *

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
0.0	0	0	0	0	0	107	226	282	258	209	155	130	123	107	97	95	91	87	82	78	74	69	61	52	47	43	39	35	32	29	25	0
.10	0	0	0	0	0	71	174	246	254	224	178	146	130	112	100	96	92	88	83	78	75	70	62	53	48	44	40	35	32	29	25	0
.20	0	0	0	0	0	0	48	132	208	239	229	195	162	127	109	99	95	91	87	80	77	72	65	56	49	45	41	36	32	30	25	0
.30	0	0	0	0	0	0	32	99	172	216	225	205	176	136	113	101	96	92	88	81	77	72	65	57	50	45	41	37	32	30	26	0
.40	0	0	0	0	0	0	0	21	73	139	191	213	208	164	131	111	100	95	91	85	79	74	68	60	51	47	43	38	33	30	26	0
.50	0	0	0	0	0	0	0	14	53	110	164	197	204	174	139	116	103	97	92	86	80	75	68	60	52	47	43	39	33	30	26	0
.75	0	0	0	0	0	0	0	5	22	54	96	137	166	180	159	134	115	103	96	89	83	77	70	63	54	48	44	40	33	31	26	0
1.0	0	0	0	0	0	0	0	0	0	2	10	29	60	132	175	169	146	124	109	97	89	81	74	67	59	52	47	43	34	31	27	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	2	17	58	112	150	159	148	122	104	91	81	74	67	59	51	46	38	32	28	1
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	20	54	98	133	149	133	108	90	80	73	66	58	51	42	34	29	7
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	12	35	87	131	141	111	92	81	74	66	59	46	38	30	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	22	63	120	136	110	91	81	73	66	51	42	31	22

RAINFALL TYPE = III * * * TC = 0.1 HR * * * SHEET 1 OF 10

(210-VI-TR-55, Second Ed., June 1986)

TABLE 2C-1. Tabular Hydrograph Unit Discharges

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution—continued

HYDROGRAPH TIME (HOURS)

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
IA/P = 0.10																																
* * * TC = 0.2 HR * * *																																
0.0	27	34	50	119	176	258	448	565	483	358	270	194	137	93	75	65	60	56	52	46	42	38	33	28	24	21	19	17	14	13	11	0
.10	24	30	42	70	103	151	222	372	501	489	402	314	234	128	91	73	64	59	55	49	44	40	35	30	25	22	20	17	15	13	11	0
.20	23	29	39	64	90	131	192	312	438	472	425	351	273	153	101	78	66	61	56	50	45	40	35	30	25	22	20	18	15	13	11	0
.30	21	26	33	48	59	79	114	166	263	380	441	431	378	238	141	96	75	65	60	53	47	42	37	32	26	23	21	18	15	13	11	0
.40	20	25	32	45	54	71	99	144	224	328	404	422	392	271	165	108	81	68	61	54	48	43	37	32	27	23	21	19	15	14	11	0
.50	18	22	28	38	43	51	64	88	125	191	282	363	402	356	241	151	102	78	66	57	51	44	39	34	28	24	22	19	15	14	11	0
.75	15	18	23	30	33	37	42	50	64	87	125	184	253	359	341	260	177	121	89	67	57	48	42	36	31	26	23	21	16	14	12	0
1.0	12	16	20	25	28	30	33	37	43	53	68	94	135	251	345	330	260	181	125	81	63	52	44	38	33	28	24	21	17	14	12	0
1.5	8	10	13	17	18	19	21	23	25	27	30	33	38	55	99	179	266	308	289	201	124	74	54	45	39	34	29	24	19	15	12	2
2.0	5	7	9	12	13	14	15	16	17	19	20	22	24	29	38	57	99	166	238	284	239	141	75	54	44	38	33	28	21	17	13	6
2.5	2	4	5	7	8	9	10	10	11	12	13	14	16	18	22	26	33	46	75	155	240	264	154	83	56	46	39	34	25	19	14	9
3.0	1	2	3	5	5	6	6	7	8	8	9	10	11	13	15	18	21	25	33	59	118	220	250	147	82	56	45	39	28	22	14	10
IA/P = 0.30																																
* * * TC = 0.2 HR * * *																																
0.0	0	0	0	17	49	115	297	489	462	379	307	230	167	119	98	86	81	76	72	64	60	55	48	41	36	32	29	26	22	20	17	0
.10	0	0	0	12	37	89	228	402	442	400	338	266	200	131	104	89	82	78	73	65	61	56	49	42	36	33	29	26	22	20	17	0
.20	0	0	0	1	8	27	69	175	326	403	401	359	297	182	126	100	88	81	76	69	63	58	51	44	37	34	30	27	23	21	17	0
.30	0	0	0	1	6	20	52	134	262	356	386	368	321	208	140	107	91	83	78	70	64	58	52	45	38	34	31	27	23	21	17	0
.40	0	0	0	1	4	15	40	103	209	307	360	365	336	234	157	116	95	85	79	72	65	59	53	46	39	34	31	28	23	21	17	0
.50	0	0	0	0	0	3	11	30	78	165	260	327	352	305	213	147	111	93	84	75	68	61	55	48	41	35	32	29	23	21	18	0
.75	0	0	0	0	0	1	5	14	37	83	150	220	276	311	264	196	144	112	94	80	72	64	56	50	43	37	33	30	24	21	18	0
1.0	0	0	0	0	0	0	0	0	2	7	20	48	94	215	299	286	230	171	128	95	81	69	61	54	47	40	35	32	25	22	18	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	2	13	56	139	224	266	253	186	128	89	71	61	54	48	41	36	29	23	19	3
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	17	58	126	196	245	214	140	89	71	61	54	47	40	32	25	20	9
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	21	57	145	228	213	136	89	70	60	53	46	35	28	21	13
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	23	78	180	215	145	94	72	61	54	41	32	22	15
IA/P = 0.50																																
* * * TC = 0.2 HR * * *																																
0.0	0	0	0	0	0	10	110	206	245	245	208	162	137	117	102	96	93	90	85	79	76	71	63	54	48	44	40	36	32	29	25	0
.10	0	0	0	0	0	7	76	163	217	235	217	181	152	122	106	98	94	90	86	80	76	71	64	55	49	45	41	36	32	29	25	0
.20	0	0	0	0	0	0	5	52	126	187	219	218	193	145	119	104	97	93	89	83	78	73	66	58	50	46	42	37	32	30	26	0
.30	0	0	0	0	0	0	3	36	96	156	198	211	199	155	125	108	99	94	90	84	79	74	67	59	51	46	42	38	32	30	26	0
.40	0	0	0	0	0	0	0	2	24	72	128	175	199	184	147	121	106	98	93	87	81	76	69	61	53	48	44	39	33	30	26	0
.50	0	0	0	0	0	0	0	1	17	54	103	151	183	194	156	127	109	100	95	88	82	76	70	62	54	48	44	40	33	30	26	0
.75	0	0	0	0	0	0	0	0	6	23	52	90	127	174	168	146	124	109	100	92	85	78	72	65	56	49	45	41	34	31	26	0
1.0	0	0	0	0	0	0	0	0	0	0	2	11	29	91	149	169	157	135	117	100	92	83	76	69	61	53	48	43	35	32	27	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	7	33	81	128	155	155	130	109	94	83	75	68	60	53	47	39	33	28	2
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	10	34	72	112	145	139	114	93	82	74	67	60	52	43	35	29	9
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	12	33	83	126	139	112	92	81	74	67	59	47	38	30	13
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	21	60	116	133	110	92	81	73	66	51	42	31	22

RAINFALL TYPE = III

* * * TC = 0.2 HR * * *

sheet 2 of 10

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution—continued

HYDROGRAPH TIME (HOURS)

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
IA/P = 0.10	* * * TC = 0.3 HR * * *																															
0.0	25	31	44	84	124	181	287	441	498	451	358	276	204	118	87	70	63	58	54	48	44	39	34	29	24	22	20	17	14	13	11	0
.10	22	28	37	56	74	108	156	244	375	457	453	389	314	180	113	83	69	62	57	51	46	41	36	31	26	23	20	18	15	13	11	0
.20	21	27	35	53	67	94	136	208	319	411	439	406	345	212	130	91	73	64	59	52	46	42	36	31	26	23	21	18	15	13	11	0
.30	19	24	30	42	49	61	33	118	178	272	365	414	409	305	190	121	87	71	63	55	49	43	38	33	27	24	21	19	15	14	11	0
.40	18	23	29	40	46	56	74	103	153	232	321	383	400	329	217	138	96	75	65	57	50	44	38	33	28	24	21	19	15	14	11	0
.50	16	21	26	34	38	43	52	67	91	132	199	280	349	383	297	196	128	91	73	60	53	46	40	35	30	25	22	20	16	14	11	0
.75	14	18	23	30	33	37	43	52	66	91	131	187	251	347	331	260	182	126	92	68	57	49	42	36	31	26	23	21	16	14	12	0
1.0	11	15	18	23	25	28	30	33	38	44	54	71	98	192	296	334	294	221	154	94	69	55	45	40	34	29	25	22	17	15	12	0
1.5	7	9	12	15	17	18	19	21	23	25	27	30	33	45	75	137	222	287	300	233	149	84	57	47	40	35	30	25	20	16	13	3
2.0	4	6	8	11	12	13	14	15	16	17	19	20	22	26	33	46	76	131	201	278	258	165	85	57	46	40	34	29	22	17	13	7
2.5	2	3	5	7	7	8	9	10	10	11	12	13	14	17	20	24	29	39	60	125	213	260	177	94	60	47	40	35	25	20	14	9
3.0	1	2	3	4	5	5	6	6	7	8	8	9	10	12	14	16	19	23	29	48	S5	195	247	167	93	60	47	40	29	22	15	10
IA/P = 0.30	* * * TC = 0.2 HR * * *																															
0.0	0	0	0	6	22	58	146	308	424	422	367	303	234	145	111	92	84	79	74	67	62	57	50	43	36	33	30	26	22	20	17	0
.10	0	0	0	4	16	44	112	243	364	402	379	328	266	166	120	97	86	80	75	68	62	57	51	44	37	33	30	27	22	21	17	0
.20	0	0	0	3	12	33	86	190	306	370	376	344	292	189	132	103	89	82	77	69	63	58	51	44	37	34	30	27	23	21	17	0
.30	0	0	0	0	2	8	25	65	149	254	331	361	350	261	175	126	100	88	31	73	66	60	53	46	39	35	31	28	23	21	18	0
.40	0	0	0	0	1	6	19	50	116	208	290	338	346	232	195	138	107	91	83	74	67	60	54	47	40	35	32	28	23	21	18	0
.50	0	0	0	0	0	1	4	14	38	90	168	250	308	333	256	180	131	103	89	78	71	63	56	49	42	36	33	29	23	21	18	0
.75	0	0	0	0	0	0	2	6	17	43	89	150	213	299	286	229	171	129	104	85	75	65	58	51	44	38	34	30	24	22	18	0
1.0	0	0	0	0	0	0	0	0	1	3	9	24	53	153	253	288	257	200	150	105	85	72	62	55	48	41	36	32	26	22	19	0
1.5	0	0	0	0	0	0	0	0	0	0	0	1	2	15	59	138	217	258	248	187	130	91	71	62	55	48	41	36	29	23	19	3
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	36	90	159	239	227	158	97	74	63	55	48	42	32	26	20	10
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	12	37	113	194	224	151	96	73	62	55	48	36	29	21	14
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	24	78	176	211	145	95	73	62	54	41	32	22	15
IA/P = 0.50	* * * TC = 0.3 HR * * *																															
0.0	0	0	0	0	0	2	33	116	193	221	221	200	165	129	110	99	95	92	87	81	77	72	65	56	49	45	41	37	32	30	25	0
.10	0	0	0	0	0	1	23	85	157	200	214	205	178	138	115	102	96	92	88	82	77	73	66	57	50	46	42	37	32	30	26	0
.20	0	0	0	0	0	1	15	62	125	175	201	203	187	147	121	105	98	94	89	83	78	73	66	58	50	46	42	38	32	30	26	0
.30	0	0	0	0	0	0	0	10	45	99	149	184	197	174	140	117	104	97	93	86	80	75	69	61	52	47	43	39	33	30	26	0
.40	0	0	0	0	0	0	0	7	32	76	125	164	189	179	148	123	107	99	94	87	81	76	69	62	53	48	44	39	33	30	26	0
.50	0	0	0	0	0	0	0	0	5	23	59	103	144	133	169	141	119	105	98	90	84	78	71	64	55	49	45	41	33	31	26	0
.75	0	0	0	0	0	0	0	0	2	9	27	55	89	148	168	156	135	117	105	94	87	80	73	66	58	51	46	42	34	31	27	0
1.0	0	0	0	0	0	0	0	0	0	0	1	4	14	59	119	157	163	145	126	105	95	85	77	71	63	55	49	44	36	32	27	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	3	19	56	103	139	151	138	116	97	85	77	70	62	54	48	40	33	28	3
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	21	52	91	135	143	120	96	84	76	69	61	54	44	35	29	11
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	22	65	112	136	117	96	83	75	68	60	48	39	30	19
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	13	45	102	131	114	95	83	75	67	53	43	31	22

RAINFALL TYPE = III

* * * TC = 0.3 HR * * *

SHEET 3 OF 10

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution—continued

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
IA/P = 0.10																																
0.0	23	29	39	65	91	132	198	308	422	449	417	345	274	162	108	82	68	61	57	50	45	41	35	30	25	22	20	18	15	13	11	0
.10	20	26	33	48	60	80	114	170	262	368	422	418	370	242	149	102	79	67	60	53	47	42	37	32	27	23	21	18	15	13	11	0
.20	20	25	32	45	55	72	100	147	224	320	388	408	383	272	171	114	85	70	62	54	48	43	37	32	27	23	21	19	15	14	11	0
.30	17	22	28	38	43	51	65	88	127	191	277	351	389	349	244	157	108	81	68	58	51	45	39	34	29	24	22	19	15	14	11	0
.40	17	21	27	36	41	47	59	79	111	165	240	314	378	359	268	178	120	88	72	60	52	45	40	34	29	25	22	20	15	14	11	0
.50	15	19	24	31	34	38	44	54	71	98	142	207	278	364	332	243	163	113	84	65	56	48	41	36	31	26	23	20	16	14	12	0
.75	12	16	20	25	27	30	33	38	44	54	70	97	138	249	333	320	257	185	131	85	65	53	44	39	33	28	24	22	17	14	12	0
1.0	11	13	17	22	23	25	28	30	34	38	46	57	75	145	245	322	311	255	188	114	77	58	47	41	35	30	26	23	18	15	12	1
1.5	6	9	11	14	15	17	18	19	21	23	25	27	30	39	59	105	180	255	292	257	176	98	61	48	41	36	31	26	20	16	13	4
2.0	4	6	8	10	11	12	13	14	15	16	17	19	20	24	30	39	61	103	166	253	272	189	98	61	48	41	35	30	23	18	13	8
2.5	2	3	4	6	7	7	8	9	10	10	11	12	13	16	18	22	26	34	49	100	183	255	198	108	66	50	42	36	26	20	14	9
3.0	1	1	2	4	4	5	5	6	6	7	8	8	9	11	13	15	18	21	26	40	77	169	243	185	106	65	49	41	30	23	15	10
IA/P = 0.30																																
0.0	0	0	0	2	10	30	78	177	306	379	379	347	293	187	133	105	90	82	77	69	63	58	51	44	38	34	30	27	23	21	17	0
.10	0	0	0	2	7	22	59	138	250	336	365	353	313	212	146	112	94	84	78	71	64	59	52	45	38	34	31	27	23	21	17	0
.20	0	0	0	1	5	17	45	107	202	292	341	349	325	235	162	121	98	87	80	72	65	59	53	46	39	34	31	28	23	21	17	0
.30	0	0	0	0	1	4	12	34	83	162	249	310	336	298	215	152	116	96	85	76	68	61	55	48	41	36	32	29	23	21	18	0
.40	0	0	0	0	0	3	9	26	64	130	209	276	324	307	234	168	125	101	88	77	70	62	55	49	41	36	32	29	23	21	18	0
.50	0	0	0	0	0	0	2	7	19	49	103	173	242	313	285	216	157	119	98	82	73	65	57	51	44	37	33	30	24	22	18	0
.75	0	0	0	0	0	0	1	3	9	23	52	97	153	253	285	253	199	151	118	91	78	68	59	53	46	39	35	31	25	22	18	0
1.0	0	0	0	0	0	0	0	0	0	1	4	13	30	104	204	276	276	226	175	120	92	75	64	57	50	43	37	33	26	22	19	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	1	8	36	98	177	236	250	207	148	99	75	63	56	49	42	37	29	24	19	4
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	22	62	124	210	232	176	107	78	65	57	50	43	33	27	20	11
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	24	85	167	219	167	106	77	64	56	49	37	29	21	14
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	15	58	152	207	159	104	76	64	55	42	33	22	15
IA/P = 0.50																																
0.0	0	0	0	0	0	0	10	54	121	182	204	204	191	146	121	106	98	94	90	83	78	73	66	58	50	46	42	38	32	30	26	0
.10	0	0	0	0	0	0	7	38	94	153	187	198	193	155	128	110	100	95	91	84	79	74	67	59	51	46	42	38	32	30	26	0
.20	0	0	0	0	0	0	5	27	71	126	166	187	191	164	134	114	103	96	92	85	80	75	68	60	52	47	43	38	33	30	26	0
.30	0	0	0	0	0	0	0	3	19	54	102	145	173	185	155	129	111	101	95	88	82	76	70	62	54	48	44	40	33	31	26	0
.40	0	0	0	0	0	0	0	2	13	40	81	124	157	180	161	135	116	104	97	90	83	77	71	63	55	49	44	40	33	31	26	0
.50	0	0	0	0	0	0	0	0	1	9	30	64	104	163	174	154	130	113	102	93	86	79	73	66	57	50	46	42	34	31	27	0
.75	0	0	0	0	0	0	0	0	0	3	13	32	59	120	157	162	145	126	111	98	90	82	75	68	60	52	47	43	35	31	27	0
1.0	0	0	0	0	0	0	0	0	0	0	0	2	6	36	90	138	158	152	135	112	98	88	79	72	65	56	50	45	37	32	27	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	1	10	37	80	121	148	143	123	102	87	78	71	64	56	49	41	34	28	4
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	13	36	71	121	140	126	101	86	78	70	63	55	44	36	29	13
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	14	49	97	135	122	100	86	77	70	62	49	40	30	20
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	33	88	130	119	98	85	76	69	54	44	32	22

HYDROGRAPH TIME (HOURS)

* * * TC = 0.4 HR * * *

RAINFALL TYPE = III

SHEET 4 OF 10

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution—continued

HYDROGRAPH TIME(HOURS)

IA/P = 0.10   * * * TC = 0.5 HR * * *

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
0.0	21	27	35	54	70	97	144	217	316	397	411	388	330	214	139	99	78	67	60	52	47	42	36	31	26	23	21	18	15	13	11	0
.10	19	24	30	43	50	64	86	125	186	273	355	392	390	296	194	129	94	75	65	56	49	43	38	33	28	24	21	19	15	14	11	0
.20	18	23	29	40	47	58	77	109	161	235	315	367	382	318	218	145	103	80	68	57	50	44	39	33	28	24	22	19	15	14	11	0
.30	16	21	26	34	38	44	53	69	95	139	203	278	337	367	289	199	135	98	77	62	54	46	40	35	30	25	22	20	16	14	11	0
.40	16	20	25	33	36	41	49	62	84	121	176	244	306	358	306	220	151	107	83	64	55	47	41	35	30	25	23	20	16	14	12	0
.50	14	18	22	28	31	35	39	46	57	75	106	152	213	323	346	282	202	140	102	73	59	50	42	37	32	27	23	21	16	14	12	0
.75	12	16	20	25	28	30	34	38	45	56	75	104	145	246	319	308	252	187	135	89	67	53	44	39	33	28	24	22	17	14	12	0
1.0	10	12	16	20	22	23	25	28	31	34	39	47	60	110	197	280	309	279	220	138	90	63	49	42	37	31	26	23	18	15	12	1
1.5	6	8	10	13	14	15	17	18	19	21	23	25	27	34	49	82	143	218	283	271	203	116	68	51	43	37	32	27	21	16	13	4
2.0	3	5	7	9	10	11	12	13	14	15	16	17	19	22	27	34	50	82	135	226	265	211	114	67	50	42	37	31	23	18	13	8
2.5	2	3	4	6	7	7	8	9	10	10	11	12	13	16	18	22	26	34	50	102	182	249	197	111	67	50	42	36	26	20	14	9
3.0	1	1	2	3	4	4	5	5	6	6	7	8	3	10	12	14	16	19	23	34	63	144	238	201	121	72	52	43	31	23	15	10

IA/P = 0.10   * * * TC = 0.5 HR * * *

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
0.0	0	0	0	1	4	15	40	101	198	295	345	345	325	232	161	122	100	88	80	72	65	59	53	46	39	34	31	28	23	21	18	0
.10	0	0	0	1	3	11	30	77	158	249	313	335	329	253	178	132	106	91	82	73	66	60	53	47	40	35	31	28	23	21	18	0
.20	0	0	0	0	2	8	23	59	125	208	278	316	324	271	196	144	112	95	85	75	67	61	54	47	40	35	32	28	23	21	1e	0
.30	0	0	0	0	0	2	6	17	45	98	171	242	291	313	249	182	136	108	92	80	71	63	56	49	42	36	33	29	24	21	18	0
.40	0	0	0	0	0	1	4	13	34	77	140	208	264	304	263	198	148	115	97	81	72	64	57	50	43	37	33	30	24	21	18	0
.50	0	0	0	0	0	0	1	3	10	26	60	113	177	276	295	244	185	140	111	88	77	67	59	52	45	39	34	31	24	22	18	0
.75	0	0	0	0	0	0	0	1	4	12	29	60	104	204	271	263	222	174	136	101	83	70	61	54	47	40	35	32	25	22	18	0
1.0	0	0	0	0	0	0	0	0	0	1	2	6	16	67	155	235	263	242	198	138	102	80	66	58	51	44	38	34	27	23	19	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	4	22	67	138	205	241	221	167	110	79	66	58	51	44	38	30	24	20	5
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	13	42	93	182	225	191	119	83	67	58	51	44	34	27	21	12
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	15	62	139	213	180	117	82	67	58	51	38	30	22	15
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	10	41	127	203	171	114	81	66	57	43	33	23	16

IA/P = 0.10   * * * TC = 0.5 HR * * *

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
0.0	0	0	0	0	0	0	3	24	68	124	174	190	190	162	133	114	103	97	92	85	80	75	68	60	52	47	43	39	33	30	26	0
.10	0	0	0	0	0	0	2	17	51	100	149	177	186	169	140	119	106	99	93	86	81	75	69	61	52	47	43	39	33	30	26	0
.20	0	0	0	0	0	0	1	12	38	79	126	160	181	173	147	124	109	101	95	88	81	76	69	62	53	48	44	39	33	30	26	0
.30	0	0	0	0	0	0	0	1	8	28	62	105	141	176	165	141	120	107	99	91	84	78	71	64	56	49	45	41	33	31	26	0
.40	0	0	0	0	0	0	0	1	6	20	48	86	123	172	172	146	125	111	101	92	85	79	72	65	56	50	45	41	34	31	26	0
.50	0	0	0	0	0	0	0	0	4	15	37	70	105	157	167	151	130	114	104	94	87	79	73	66	57	50	46	42	34	31	27	0
.75	0	0	0	0	0	0	0	0	0	1	6	17	37	91	139	157	150	134	119	103	93	84	76	69	62	54	48	44	35	32	27	0
1.0	0	0	0	0	0	0	0	0	0	0	1	3	9	40	91	135	153	149	135	113	99	88	79	72	65	57	50	45	37	32	27	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5	24	59	101	132	144	130	107	90	80	73	65	57	51	41	34	29	6
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	25	55	106	138	130	105	89	79	72	65	57	45	37	30	15
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	36	81	133	133	104	88	79	71	64	50	41	31	21
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5	24	74	128	122	103	88	78	70	55	45	32	23

RAINFALL TYPE = III   * * * TC = 0.5 HR * * *   SHEET 5 OF 10

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution—continued

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
IA/P = 0.10															* * * TC = 1.0 HR * * *																	
0.0	15	19	24	32	37	44	54	71	98	136	181	227	264	297	270	215	164	128	103	78	64	52	43	36	31	26	23	21	16	14	12	0
.10	13	17	22	28	31	35	41	49	64	87	120	161	205	273	289	254	201	155	122	90	71	56	45	38	33	28	24	21	17	14	12	0
.20	13	16	21	27	29	33	39	46	58	77	10S	142	184	257	285	263	214	167	130	95	74	57	46	39	33	28	24	22	17	14	12	0
.30	12	16	20	26	28	31	36	42	53	69	93	126	165	240	279	268	225	178	139	100	77	59	47	39	34	29	25	22	17	15	12	1
.40	11	14	13	23	25	27	30	34	40	48	62	83	112	185	251	276	256	213	163	118	87	65	50	41	35	30	26	23	18	15	12	1
.50	11	13	17	22	24	26	29	32	37	45	56	74	99	167	235	270	261	223	179	126	92	67	51	42	36	31	26	23	18	15	12	1
.75	8	10	13	17	19	19	21	23	25	28	31	36	44	72	122	186	239	258	243	189	136	90	62	48	40	34	29	25	20	16	12	2
1.0	6	9	11	14	15	17	18	20	21	23	25	23	32	46	75	124	185	234	253	226	170	110	71	53	43	37	31	27	21	16	13	4
1.5	4	6	8	10	11	12	13	14	15	16	17	19	21	25	32	46	74	119	170	230	239	179	108	70	52	43	36	31	23	18	13	7
2.0	2	3	4	6	7	7	8	9	10	10	11	12	13	16	18	22	28	38	58	111	179	228	185	116	75	54	44	37	27	21	14	9
2.5	1	1	2	4	4	5	5	6	6	7	8	8	9	11	13	15	18	22	28	46	87	167	219	176	113	73	54	43	31	23	15	10
3.0	0	0	1	2	2	2	3	3	3	4	4	5	5	7	8	10	12	14	16	21	32	68	156	210	179	120	78	56	37	27	16	11
IA/P = 0.30															* * * TC = 1.0 HR * * *																	
0.0	0	0	0	0	0	1	5	13	30	57	95	141	186	243	249	213	174	142	119	97	83	70	60	53	46	39	35	31	25	22	18	0
.10	0	0	0	0	0	1	3	10	23	46	79	120	164	230	245	221	183	150	125	101	85	72	61	53	46	40	35	31	25	22	18	0
.20	0	0	0	0	0	0	1	3	7	18	36	65	102	183	233	241	210	174	144	112	92	76	64	56	48	42	36	33	26	22	19	1
.30	0	0	0	0	0	0	1	2	6	14	29	53	86	163	221	237	217	183	151	117	95	78	65	56	49	42	37	33	26	22	19	1
.40	0	0	0	0	0	0	0	0	1	4	11	23	43	107	180	225	233	207	175	133	105	84	68	59	51	44	38	34	27	23	19	1
.50	0	0	0	0	0	0	0	0	1	3	8	18	34	91	162	214	230	213	183	139	109	86	70	60	52	45	39	34	27	23	19	1
.75	0	0	0	0	0	0	0	0	0	0	1	4	9	33	82	145	156	218	211	174	135	101	77	64	56	48	42	37	29	24	19	3
1.0	0	0	0	0	0	0	0	0	0	0	0	1	2	11	37	85	144	192	214	199	160	116	85	69	59	51	44	38	30	25	20	5
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	23	56	104	174	203	177	123	89	71	60	52	45	35	27	21	11
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	3	24	73	139	194	169	120	87	70	59	51	39	30	22	14
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	15	51	127	186	162	117	86	69	59	44	34	23	16
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	35	117	180	164	122	90	71	51	39	25	16
IA/P = 0.50															* * * TC = 1.0 HR * * *																	
0.0	0	0	0	0	0	0	0	2	6	17	34	57	83	127	151	142	130	118	109	99	91	83	75	68	59	52	47	43	35	31	27	1
.10	0	0	0	0	0	0	0	1	5	13	27	47	71	117	146	144	133	121	112	101	92	84	76	68	60	53	48	43	35	32	27	1
.20	0	0	0	0	0	0	0	1	3	10	21	38	60	106	138	143	135	124	114	102	94	85	76	69	61	54	48	44	35	32	27	1
.30	0	0	0	0	0	0	0	0	0	2	7	16	31	73	114	139	142	132	121	108	98	88	79	71	64	56	50	45	36	32	27	1
.40	0	0	0	0	0	0	0	0	0	2	5	13	25	62	104	133	140	134	124	110	99	89	80	72	04	56	50	45	37	32	27	1
.50	0	0	0	0	0	0	0	0	0	0	1	4	10	35	74	112	134	138	131	117	104	93	82	74	67	59	52	47	38	33	28	2
.75	0	0	0	0	0	0	0	0	0	0	0	2	5	19	43	84	115	131	134	123	110	97	85	77	69	61	54	48	39	33	28	3
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	21	49.	84	113	129	132	119	103	89	80	72	64	56	50	41	34	28	5
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	21	46	75	113	128	120	102	88	79	71	63	56	45	37	29	12
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	14	42	82	119	124	104	50	80	72	64	50	41	31	20
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	29	75	117	121	102	89	79	71	56	45	32	23
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	28	80	113	118	101	88	79	63	49	34	24

RAINFALL TYPE = III   * * * TC = 1.0 HR * * *   SHEET 7 OF 10

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution—continued

TRVL TIME (hr)	HYDROGRAPH TIME (HOURS) 11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
IA/P = 0.10												* * * TC = 0.75 HR * * *																				
0.0	17	22	28	39	45	56	73	104	151	215	281	328	343	310	228	163	121	94	77	63	53	45	39	34	29	25	22	20	15	14	11	0
.10	17	21	27	37	42	51	66	91	131	187	250	302	336	319	247	179	131	101	82	65	55	46	40	35	29	25	22	20	16	14	11	0
.20	15	19	24	31	35	40	48	60	81	114	163	221	275	328	298	229	167	124	96	73	60	49	42	36	31	26	23	20	16	14	12	0
.30	14	18	23	30	33	38	44	55	72	100	142	194	248	320	305	245	182	135	103	76	62	50	42	37	31	27	23	21	16	14	12	0
.40	13	16	21	26	29	32	36	41	50	65	88	124	171	268	313	288	228	170	127	88	68	54	44	38	33	28	24	21	17	14	12	0
.50	12	16	20	25	28	30	34	39	46	59	78	109	150	244	306	294	242	184	138	94	71	56	45	39	34	28	24	22	17	14	12	0
.75	10	13	16	21	23	25	27	30	33	38	46	58	77	140	221	277	287	248	197	133	92	66	50	42	36	31	26	23	18	15	12	1
1.0	8	10	13	16	18	19	21	23	25	27	30	34	39	61	109	181	249	280	265	198	134	85	58	46	40	34	29	25	20	16	12	2
1.5	5	7	9	12	13	14	15	16	17	19	20	22	24	30	40	63	106	167	225	261	226	147	84	58	46	39	34	29	22	17	13	5
2.0	2	4	5	7	8	9	9	10	11	12	13	14	16	18	22	26	34	50	80	155	226	246	158	91	61	47	40	34	25	20	14	9
2.5	1	2	3	5	5	6	6	7	8	8	9	10	11	13	15	18	21	26	34	62	120	209	234	151	90	60	47	39	29	22	15	10
3.0	0	1	2	3	3	3	4	4	5	5	6	7	7	9	10	12	15	17	21	29	50	113	209	224	144	88	59	46	33	25	16	10
IA/P = 0.30												* * * TC = 0.75 HR * * *																				
0.0	0	0	0	0	1	3	8	24	58	113	182	243	283	287	233	178	139	114	98	83	73	64	56	50	43	37	33	30	24	21	18	0
.10	0	0	0	0	0	2	6	18	45	91	151	212	259	284	245	191	149	120	102	85	75	65	57	50	43	37	33	30	24	21	18	0
.20	0	0	0	0	0	0	1	5	14	35	72	125	183	263	277	230	180	142	116	93	80	68	59	52	45	39	34	31	25	22	18	0
.30	0	0	0	0	0	0	1	3	10	26	57	102	156	245	270	240	192	151	122	96	82	69	60	53	46	39	35	31	25	22	18	0
.40	0	0	0	0	0	0	0	1	2	8	20	45	83	182	252	264	226	181	144	108	89	73	62	55	48	41	36	32	26	22	19	0
.50	0	0	0	0	0	0	0	0	2	6	15	35	67	158	235	259	235	192	153	113	92	75	63	56	49	42	36	33	26	22	19	1
.75	0	0	0	0	0	0	0	0	0	0	1	2	7	35	100	178	232	242	217	163	121	90	71	61	54	47	40	35	28	23	19	2
1.0	0	0	0	0	0	0	0	0	0	0	0	1	4	21	68	140	205	236	229	181	135	97	74	63	55	48	41	36	29	24	19	3
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	13	42	94	155	221	212	158	103	77	64	56	49	42	33	26	20	10
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	15	42	113	184	209	151	101	76	63	55	48	36	29	21	14
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	27	80	168	199	146	100	75	63	55	41	32	22	15
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	10	56	154	191	151	105	78	64	48	37	24	16
IA/P = 0.50												* * * TC = 0.75 HR * * *																				
0.0	0	0	0	0	0	0	0	3	13	37	71	108	140	167	156	136	120	108	101	92	85	78	72	64	56	50	45	41	34	31	26	0
.10	0	0	0	0	0	0	0	2	10	28	57	91	124	163	158	140	124	111	103	94	86	79	72	65	57	50	46	41	34	31	27	0
.20	0	0	0	0	0	0	0	0	1	7	21	45	76	135	159	153	136	120	109	98	90	82	74	67	59	52	47	43	35	31	27	0
.30	0	0	0.	0	0	0	0	0	1	5	15	35	62	121	157	157	140	124	112	100	91	83	75	68	60	52	47	43	35	31	27	0
.40	0	0	0	0	0	0	0	0	1	3	11	27	50	107	146	154	143	128	115	102	93	84	76	69	61	53	48	43	35	32	27	0
.50	0	0	0	0	0	0	0	0	0	0	2	8	21	66	118	148	152	139	125	108	97	87	78	71	63	55	49	45	36	32	27	1
.75	0	0	0	0	0	0	0	0	0	0	1	4	10	38	82	122	142	144	133	116	103	91	80	73	66	58	51	46	37	32	28	1
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	5	24	60	102	132	142	133	116	99	86	77	70	62	55	49	40	33	28	3
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	25	55	91	128	136	119	98	85	77	69	62	54	44	36	29	11
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	25	66	109	131	116	97	85	76	69	61	48	39	30	18
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	16	47	100	127	114	96	84	75	68	53	43	31	22
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	10	44	102	124	112	95	83	75	59	47	33	23

RAINFALL TYPE = III   * * * TC = 0.75 HR * * *   SHEET 6 OF 10

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution—continued

TRVL TIME (hr)	HYDROGRAPH TIME (HOURS) 11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
IA/P = 0.10 * * * TC = 1.25 HR * * *																																
0.0	13	17	22	28	32	37	44	56	75	100	133	170	206	255	264	236	194	155	125	95	75	59	47	39	33	28	24	22	17	14	12	1
.10	13	16	21	27	30	34	41	51	67	89	118	152	188	243	261	243	204	164	133	100	79	61	48	40	34	29	25	22	17	15	12	1
.20	11	15	15	24	26	29	33	38	47	60	79	105	137	203	248	256	232	193	156	115	88	67	51	42	35	30	26	23	18	15	12	1
.30	11	14	18	23	25	27	31	36	43	54	71	94	122	187	237	254	238	203	165	121	92	69	53	43	36	30	26	23	18	15	12	1
.40	10	12	16	20	22	24	26	29	34	40	49	64	84	140	201	242	250	228	193	142	106	77	57	46	38	32	27	24	19	15	12	1
.50	9	12	15	19	21	23	25	28	32	37	45	58	75	126	186	232	248	233	201	150	111	80	58	47	39	33	28	24	19	15	12	2
.75	8	11	14	17	19	20	22	24	27	31	36	44	56	93	144	196	230	237	220	174	131	91	64	50	41	34	29	25	20	16	12	2
1.0	6	8	11	14	15	16	17	19	20	22	25	28	32	46	74	119	170	213	234	218	174	120	79	58	46	38	32	27	21	17	13	4
1.5	3	5	7	9	9	10	11	12	13	14	16	17	18	22	27	38	58	91	134	197	223	194	128	83	60	47	39	33	24	19	14	7
2.0	2	3	4	6	6	7	8	8	9	10	11	12	13	15	18	22	27	38	57	106	166	214	184	124	82	59	47	39	28	21	14	9
2.5	1	1	2	3	3	4	4	5	5	6	7	7	8	10	11	13	16	19	24	38	68	137	206	186	130	87	62	48	33	24	15	10
3.0	0	0	1	2	2	2	2	3	3	4	4	5	5	6	8	9	11	13	16	21	31	66	145	200	178	126	85	61	39	28	17	11
IA/P = 0.10 * * * TC = 1.25 HR * * *																																
0.0	0	0	0	0	0	1	2	7	18	35	61	94	130	192	222	218	191	161	136	110	93	77	65	56	48	41	36	32	26	22	19	1
.10	0	0	0	0	0	1	2	5	14	28	50	79	113	177	214	219	197	168	142	115	96	79	66	57	49	42	37	33	26	22	19	1
.20	0	0	0	0	0	0	1	4	10	22	41	66	97	162	205	217	202	175	149	119	99	81	67	58	50	43	37	33	26	23	19	1
.30	0	0	0	0	0	0	0	1	3	8	17	33	55	115	174	208	214	195	168	133	108	87	70	60	52	45	39	34	27	23	19	2
.40	0	0	0	0	0	0	0	1	2	6	14	26	45	100	159	200	212	199	174	138	112	89	72	61	53	45	39	35	28	23	19	2
.50	0	0	0	0	0	0	0	0	1	2	5	11	21	60	116	170	203	209	192	156	124	97	76	64	55	47	41	36	29	24	19	3
.75	0	0	0	0	0	0	0	0	0	1	2	5	11	35	79	131	175	201	201	172	140	107	82	67	58	50	43	37	30	24	20	4
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	23	57	105	153	186	198	172	131	95	75	63	54	47	41	32	25	20	7
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	24	54	96	157	189	174	127	94	74	63	54	46	35	28	21	12
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	24	69	127	182	167	124	93	74	62	53	40	31	22	15
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	10	35	99	176	167	129	96	76	63	46	35	23	16
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	34	107	171	161	125	94	75	53	40	25	17
IA/P = 0.10 * * * TC = 1.25 HR * * *																																
0.0	0	0	0	0	0	0	0	1	4	10	21	36	55	95	123	138	131	122	114	104	95	86	78	70	62	55	49	44	36	32	27	1
.10	0	0	0	0	0	0	0	0	1	3	8	16	30	66	102	125	134	128	120	108	99	89	80	73	65	57	51	46	37	32	28	2
.20	0	0	0	0	0	0	0	0	0	2	6	13	24	57	94	120	132	130	122	110	101	91	81	73	66	58	51	46	37	33	28	2
.30	0	0	0	0	0	0	0	0	0	1	4	10	19	49	85	113	131	131	124	112	102	92	82	74	66	58	52	47	38	33	28	2
.40	0	0	0	0	0	0	0	0	0	1	3	8	16	42	76	106	126	130	125	114	104	93	83	75	67	59	52	47	38	33	28	3
.50	0	0	0	0	0	0	0	0	0	0	1	2	6	22	51	84	112	127	129	120	109	97	86	77	70	61	54	49	40	33	28	4
.75	0	0	0	0	0	0	0	0	0	0	0	1	3	12	32	61	91	113	125	123	114	101	89	80	72	64	56	50	41	34	28	5
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	22	47	76	102	122	124	111	96	85	76	69	61	54	43	36	29	9
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	22	43	82	111	121	109	95	84	76	68	60	48	39	30	16
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	9	30	63	104	118	107	94	83	75	67	53	43	31	21
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	20	58	104	116	106	93	83	74	59	47	33	23
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	19	64	104	114	104	92	82	65	52	35	24

RAINFALL TYPE = III * * * TC = 1.25 HR * * *

SHEET 8 OF 10

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution—continued

HYDROGRAPH TIME(HOURS)

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13 0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
IA/P = 0.10														* * * TC = 0.2 HR * * *																		
0.0	12	15	19	25	27	31	37	45	57	75	97	122	151	203	231	238	213	182	150	115	91	70	54	44	37	30	26	23	18	15	12	1
.10	10	13	17	21	23	26	29	34	41	52	67	87	111	165	210	233	227	205	173	131	102	77	58	46	39	32	27	24	19	15	12	1
.20	10	13	16	21	23	25	28	32	38	47	61	78	100	152	199	228	231	210	181	138	107	80	59	47	39	33	28	24	19	15	12	2
.30	9	12	15	20	22	24	27	30	36	44	55	70	90	139	188	221	228	215	188	145	112	83	61	48	40	33	28	24	19	15	12	2
.40	8	11	14	18	19	21	23	25	29	33	40	50	64	103	152	196	223	226	208	166	127	92	66	52	42	35	30	25	20	16	13	2
.50	8	10	13	17	18	20	22	24	27	31	37	46	58	93	140	186	216	224	212	173	133	96	69	53	43	36	30	26	20	16	13	3
.75	6	8	11	14	15	16	18	19	21	23	26	31	36	55	87	130	173	205	217	202	165	119	81	60	48	39	33	28	21	17	13	4
1.0	5	6	8	11	12	13	14	15	16	18	19	21	24	31	46	71	109	151	189	214	200	153	102	72	55	44	37	31	23	18	13	6
1.5	3	4	5	7	8	9	10	11	11	12	14	15	16	19	24	31	45	69	103	159	207	207	147	99	71	54	44	36	26	20	14	8
2.0	1	2	3	5	5	6	6	7	8	9	9	10	11	13	16	19	23	31	45	81	132	189	199	142	97	69	53	43	30	22	15	10
2.5	0	1	1	2	3	3	4	4	4	5	6	6	7	8	10	12	14	17	20	31	53	107	179	194	147	102	73	55	36	26	16	10
3.0	0	0	1	1	1	2	2	2	3	3	3	4	4	5	7	8	10	11	13	18	26	51	116	178	188	142	100	71	43	30	18	11
IA/P = 0.30														* * * TC = 0.2 HR * * *																		
0.0	0	0	0	0	0	1	2	5	11	22	38	59	84	138	180	200	195	176	154	125	105	86	71	60	52	44	39	34	27	23	19	2
.10	0	0	0	0	0	0	0	1	4	9	18	31	50	99	149	184	198	190	170	139	115	93	75	63	55	47	40	35	28	23	19	3
.20	0	0	0	0	0	0	0	1	3	7	14	25	41	86	137	176	195	192	175	144	119	95	76	64	55	47	41	36	28	24	19	3
.30	0	0	0	0	0	0	0	0	1	2	5	11	21	53	100	147	181	194	187	159	130	103	81	68	58	50	43	37	29	24	19	4
.40	0	0	0	0	0	0	0	0	0	2	4	9	17	45	88	136	172	192	189	164	135	106	83	69	59	51	43	38	30	24	20	4
.50	0	0	0	0	0	0	0	0	0	0	1	3	7	23	56	101	145	177	190	177	149	116	89	73	62	53	45	39	31	25	20	6
.75	0	0	0	0	0	0	0	0	0	0	0	1	3	13	35	71	113	151	176	184	162	128	97	77	65	55	48	41	32	26	20	7
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	24	53	92	132	174	182	153	114	88	72	61	52	45	34	27	21	10
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	9	24	50	103	152	175	148	112	87	71	60	51	38	30	22	14
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	6	24	62	127	170	151	116	89	73	61	45	34	23	16
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	16	58	131	165	146	113	88	72	52	39	25	16
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	16	67	134	161	142	111	87	60	44	27	17
IA/P = 0.50														* * * TC = 0.2 HR * * *																		
0.0	0	0	0	0	0	0	0	1	2	6	13	22	34	65	94	114	129	122	117	108	100	91	81	74	66	58	52	46	38	33	28	2
.10	0	0	0	0	0	0	0	0	0	2	5	10	18	42	73	99	116	126	121	112	104	94	84	76	68	60	53	48	39	33	28	3
.20	0	0	0	0	0	0	0	0	0	1	4	8	15	36	65	93	112	123	122	113	105	95	85	77	69	61	54	48	39	34	28	4
.30	0	0	0	0	0	0	0	0	0	0	1	3	6	20	44	72	97	114	122	118	109	99	88	79	71	63	56	50	41	34	28	5
.40	0	0	0	0	0	0	0	0	0	0	1	2	5	16	38	65	91	110	121	119	110	100	89	80	72	64	57	51	41	34	29	5
.50	0	0	0	0	0	0	0	0	0	0	0	1	4	13	33	59	85	105	118	120	112	101	90	81	73	65	57	51	41	35	29	6
.75	0	0	0	0	0	0	0	0	0	0	0	1	2	7	20	41	66	89	107	118	115	105	93	83	75	67	60	53	43	35	29	8
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	14	31	53	78	106	117	113	100	89	80	72	64	57	46	37	30	12
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	14	29	60	91	115	111	99	88	79	71	63	50	41	31	18
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	20	45	85	113	109	98	87	78	70	56	45	32	22
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	13	42	87	111	107	97	86	78	62	49	34	24
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	39	82	109	109	98	87	70	56	37	25

RAINFALL TYPE = III

* * * TC = 0.2 HR * * *

SHEET 9 OF 10

Exhibit 5-III: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution—continued

HYDROGRAPH TIME(HOURS)

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
IA/P = 0.10												* * * TC = 2.0 HR * * *						IA/P = 0.10														
0.0	9	11	15	19	21	23	27	31	39	48	60	75	91	129	164	187	200	191	178	147	119	92	69	55	45	37	31	26	20	16	13	3
.10	8	10	13	17	18	20	22	25	29	36	44	55	68	101	139	170	189	197	188	163	132	101	75	59	47	39	33	28	21	17	13	3
.20	7	10	13	16	18	19	21	24	28	33	40	50	62	93	129	162	185	195	190	168	137	104	77	60	48	40	33	28	21	17	13	4
.30	7	9	12	15	17	18	20	23	26	31	37	46	56	85	120	154	179	193	191	172	142	108	80	62	49	41	34	29	21	17	13	4
.40	6	8	11	14	15	16	18	19	22	25	29	34	42	64	94	129	161	183	192	183	157	120	87	66	53	43	36	30	22	17	13	5
.50	6	8	10	13	14	16	17	19	21	23	27	32	38	58	87	121	153	177	191	185	162	124	90	68	54	44	36	31	22	18	13	5
.75	5	6	8	11	12	13	14	15	16	18	20	23	26	37	55	81	113	144	169	186	179	147	106	78	61	49	40	33	24	19	14	6
1.0	4	5	7	9	10	11	12	13	14	15	17	18	20	27	38	56	82	113	143	176	185	164	121	88	67	53	43	36	26	20	14	7
1.5	2	3	4	5	6	7	7	8	9	10	10	11	12	15	18	23	31	45	65	106	148	180	166	126	92	69	55	44	31	23	15	9
2.0	1	1	2	3	4	4	5	5	6	6	7	8	8	10	12	14	18	23	31	52	87	139	176	160	122	90	68	54	36	26	16	10
2.5	0	0	1	2	2	2	2	3	3	4	4	5	5	6	7	9	11	13	16	22	36	71	132	172	161	126	94	71	45	31	18	11
3.0	0	0	0	1	1	1	1	1	2	2	2	3	3	4	5	6	7	9	10	14	19	35	78	136	168	156	123	92	54	36	20	11
IA/P = 0.10												* * * TC = 2.0 HR * * *						IA/P = 0.10														
0.0	0	0	0	0	0	0	1	2	6	11	18	29	41	75	111	140	159	170	163	145	124	103	84	70	60	51	44	39	30	25	20	4
.10	0	0	0	0	0	0	0	1	2	4	9	15	24	50	84	118	145	160	167	155	134	110	89	74	63	54	46	40	31	25	20	5
.20	0	0	0	0	0	0	0	0	1	3	7	12	20	43	76	110	138	157	165	157	138	113	91	75	64	55	47	41	32	26	20	5
.30	0	0	0	0	0	0	0	0	1	3	5	10	17	38	68	101	131	152	164	159	141	116	93	77	65	56	48	41	32	26	20	6
.40	0	0	0	0	0	0	0	0	0	1	2	4	8	22	45	76	109	137	155	163	151	125	99	81	68	58	50	43	33	26	20	7
.50	0	0	0	0	0	0	0	0	0	1	1	3	7	18	39	69	101	130	151	162	153	128	101	83	69	59	51	44	34	27	20	7
.75	0	0	0	0	0	0	0	0	0	0	0	1	2	6	17	36	63	93	122	151	158	143	114	92	76	64	55	47	36	28	21	9
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	18	37	63	93	132	157	153	125	100	82	68	58	50	38	29	21	11
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	12	26	59	100	142	154	128	102	83	70	59	44	34	23	15
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	18	43	93	142	150	125	101	82	69	50	38	24	1b
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	12	41	98	141	147	122	99	81	58	43	26	17
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	38	92	135	144	124	102	69	50	30	18
IA/P = 0.10												* * * TC = 2.0 HR * * *						IA/P = 0.10														
0.0	0	0	0	0	0	0	0	0	1	3	6	11	16	33	54	75	91	102	114	114	103	96	87	79	72	64	57	51	41	35	29	6
.10	0	0	0	0	0	0	0	0	1	2	5	9	14	29	49	70	87	99	106	113	104	97	88	80	72	65	58	52	42	35	29	6
.20	0	0	0	0	0	0	0	0	0	1	2	4	7	18	34	54	74	90	101	112	107	99	90	82	75	67	60	53	43	36	29	8
.30	0	0	0	0	0	0	0	0	0	0	1	3	6	15	30	49	69	86	98	111	108	100	91	83	75	68	60	54	43	36	29	8
.40	0	0	0	0	0	0	0	0	0	0	0	1	2	8	19	35	55	73	89	105	110	103	94	86	78	70	62	56	45	37	30	10
.50	0	0	0	0	0	0	0	0	0	0	0	1	2	6	16	31	50	69	85	102	109	104	95	86	78	71	63	56	45	37	30	11
.75	0	0	0	0	0	0	0	0	0	0	0	0	1	4	10	21	37	55	73	93	107	107	97	89	81	73	65	58	47	38	30	13
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	15	29	46	72	93	107	103	94	86	78	70	62	50	40	31	16
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	7	15	34	59	89	105	101	93	85	77	69	55	44	32	21
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	10	25	55	90	104	100	92	84	76	61	49	34	23
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	24	59	91	103	99	91	83	68	54	36	25
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	28	62	91	101	98	90	75	60	39	26

RAINFALL TYPE = III   * * * TC = 2.0 HR * * *   SHEET 10 OF 10

CHAPTER 3 – OPEN CHANNELS

3.1 GENERAL

Open channels are a natural or constructed conveyance for water in which the water surface is exposed to the atmosphere, and the gravity force component in the direction of motion is the driving force. The principles of open channel flow hydraulics are applicable to all drainage facilities including culverts and storm drains.

The two basic types of open channels encountered in the City of Fort Smith are natural channels and drainage ditches.

Natural channels are:

- undisturbed channels with their size and shape determined by natural forces,
- usually compound in cross section with a main channel for conveying low flows and a floodplain to transport flood flows, and
- usually shaped in cross section and plan form by the long-term history of sediment load and water discharge that they experience.

Drainage ditches are artificial channels which are:

- constructed channels with regular geometric cross sections, and
- unlined or lined with artificial or natural material to protect against erosion.

Although the principles of open-channel flow are the same regardless of the channel type, stream channels and drainage ditches will be treated separately in this chapter as needed.

3.2 DESIGN CRITERIA

Open channels shall be designed according to the criteria listed below.

3.2.1 *Natural Channels*

The following criteria apply to natural channels and may be revised as approved by the Engineering Department:

- If approved by the Engineering Department, natural channels (unaltered, with existing trees and vegetation, not relocated or channelized) may be used in new developments providing the channel will carry the design storm runoff without erosion problems and sufficient land is included in a drainage easement. No clearing shall be allowed within the drainage easement. Natural channels may only be used where all the criteria for natural channels can be met. Drainage ditches or underground enclosed storm sewers must be used where criteria cannot be met.

- Natural channels must have a minimum freeboard of 1 ft for the 25-yr storm event.
- 10-year flows greater than 50 cfs may be carried in natural channels. 10-year flows less than 50 cfs must be contained in an underground enclosed storm sewer.
- The hydraulic effects of drainageway encroachments shall be evaluated over a full range of frequency-based peak discharges from the 10-yr through 100-yr recurrence intervals in areas where houses or other structures are present.
- Where encroachments are necessary (such as street crossings, utility crossings, storm sewer outfalls, etc.), streambank stabilization shall be provided and shall include both upstream and downstream banks and the local site.
- For natural channels less than 500 feet in length, and not located within a regulatory floodplain, single-section analysis (Manning's Equation) may be used for design, provided that the channel does not discharge to a culvert, storm drain, or other obstruction. All other channels must be designed using step-backwater analysis.
- The maximum water surface elevation for the 100-year storm event shall be 1 foot lower than the floor elevation of adjacent buildings or structures.

3.2.2 *Drainage Ditches*

The following criteria apply to drainage ditches and may be revised as approved by the Engineering Department:

- Drainage ditches may only be used in locations where all criteria for drainage ditches can be met. Underground enclosed storm sewer must be used where criteria cannot be met.
- 10-year flows greater than 50 cfs may be carried in drainage ditches. 10-year flows less than 50 cfs must be contained in an underground enclosed storm sewer, with the following exception: 10-year flows less than 50 cfs may be carried in Open Channel Systems designed to treat the required Water Quality Volume, per Chapter 5 – Post Construction Stormwater Management.
- For roadside drainage ditches, a minimum 10 foot wide clear zone must be provided between the back of curb/edge of pavement and the top edge of side slope or channel wall.

- Drainage ditches used for side lot drainage shall be concrete-lined. Concrete-lined drainage ditches must extend to the rear of the lot.

- Drainage ditches must have a trapezoidal cross section with a bottom width that is equal to or exceeds 3 times the depth of flow for the design storm. The minimum bottom width shall be 4 feet.

- Side slopes for concrete-lined drainage ditches shall be 1V:2H or flatter. Side slopes for earthen drainage ditches shall be 1V:4H or flatter.

- For concrete-lined drainage ditches, the top edge of concrete lining shall extend to the original ground level or to a point where an earthen slope can be constructed on a grade of 1V:4H or flatter. The design flow, plus freeboard, must be contained within the concrete-lined drainage ditch.

- Earthen drainage ditches may be used where the velocities from a 25-yr storm are less than 6 ft/s. All earthen drainage ditches shall be seeded or sodded immediately after their construction, and adequate measures shall be taken to prevent erosion. Special protections (such as headwalls, rip rap, grouted rip rap, gabions, etc.) will be required in all locations (such as bends, junctions, inlets or outlets of storm sewers, etc.) where erosion is likely.

- The minimum longitudinal grade for concrete-lined drainage ditches shall be 0.30%. The minimum longitudinal grade for earthen drainage ditches shall be 0.50%.

- Vertical wall concrete channels may be used in lieu of concrete drainage ditches. The minimum bottom width shall be 4 feet. Vertical wall concrete channels greater than 2 feet in depth must be fenced in on all sides. The fence shall be a minimum of 4 feet high and shall be chain link or other approved type. The fence shall have, as a minimum, 10 foot wide gates accessible by easement, placed no more than 400 feet apart. Where fence is required, a one foot wide, 6" thick, concrete lip shall be constructed adjacent to the tops of the vertical walls. Fence posts shall be set along the centerline of the one foot wide concrete lip.

- All drainage ditches and channels shall be located in street right-of-way or a public drainage easement.

- All drainage ditches and channels shall have a minimum freeboard of 1 ft.

- For drainage ditches and channels less than 500 feet in length, and not located within a regulatory floodplain, single-section analysis (Manning's Equation) may be used for design, provided that the ditch or channel does not discharge to a culvert, storm drain, or other obstruction. All other ditches and channels must be designed using step-backwater analysis.

- The maximum water surface elevation for the 100-year storm event shall be 1 foot lower than the floor elevation of adjacent buildings or structures.

3.3 OPEN CHANNEL FLOW

Design analysis of both natural and artificial channels proceeds according to the basic principles of open-channel flow (see References *(3)*, *(7)*). The basic principles of fluid mechanics—continuity, momentum and energy—can be applied to open-channel flow with the additional complication that the position of the free surface is usually one of the unknown variables. The determination of this unknown is one of the principal (or primary) objectives of open-channel flow analysis. The following equations are the most commonly used to analyze open channel flow:

3.3.1 *Continuity Equation*

The continuity equation is the statement of conservation of mass in fluid mechanics. For the special case of one-dimensional, steady flow of an incompressible fluid, it assumes the simple form:

$$Q = A_1 V_1 = A_2 V_2 \tag{3.1}$$

where:

Q = discharge, ft^3/s
A = cross-sectional area of flow, ft^2
V = mean cross-sectional velocity, ft/s (which is perpendicular to the cross section)

The subscripts 1 and 2 refer to successive cross sections along the flow path.

3.3.2 *Manning's Equation*

For a given depth of flow in an open channel with a steady, uniform flow, the mean velocity, V, can be computed with Manning's equation:

$$V = (1.486/n) R^{2/3} S^{1/2} \tag{3.2}$$

where:

V = velocity, ft/s
n = Manning's roughness coefficient
R = hydraulic radius = A/P, ft
P = wetted perimeter, ft
S = slope of the energy gradeline, ft/ft (Note: For steady uniform flow, S = channel slope, ft/ft)

The selection of Manning's n is generally based on observation; however, considerable experience is essential in selecting appropriate n values. The selection of Manning's n is discussed in Section 3.4.2. The range of n values for various types of channels and floodplains is given in Table 3-1.

The continuity equation can be combined with Manning's equation to obtain the steady, uniform flow discharge as:

$$Q = (1.486/n)AR^{2/3}S^{1/2} \tag{3.3}$$

The conveyance represents the carrying capacity of a stream cross section based upon its geometry and roughness characteristics alone and is independent of the streambed slope.

The concept of channel conveyance is useful when computing the distribution of overbank flood flows in the stream cross section and the flow distribution through the opening in a proposed stream crossing. It is also used to determine the velocity distribution coefficient, a (see Equation 3.4).

For a given channel geometry, slope and roughness and a specified value of discharge Q, a unique value of depth occurs in steady, uniform flow. It is called normal depth and is computed from Equation 3.3 by expressing the area and hydraulic radius in terms of depth. The resulting equation may require a trial-and-error solution.

TABLE 3-1. Values of Manning's Roughness Coefficient *n* (Uniform Flow)

Type of Channel and Description	Minimum	Normal	Maximum
EXCAVATED OR DREDGED			
1. Earth, straight and uniform			
a. Clean, recently completed	0.016	0.018	0.020
b. Clean, after weathering	0.018	0.022	0.025
c. Gravel, uniform section, clean	0.022	0.025	0.030
d. With short grass, few weeds	0.022	0.027	0.033
2. Earth, winding and sluggish			
a. No vegetation	0.023	0.025	0.030
b. Grass, some weeds	0.025	0.030	0.033
c. Dense weeds or aquatic plans in deep channels	0.030	0.035	0.040
d. Earth bottom and rubble sides	0.025	0.030	0.035
e. Stony bottom and weedy sides	0.025	0.035	0.045
f. Cobble bottom and clean sides	0.030	0.040	0.050
3. Dragline-excavated or dredged			
a. No vegetation	0.025	0.028	0.033
b. Light brush on banks	0.035	0.050	0.060

4. Rock cuts			
a. Smooth and uniform	0.025	0.035	0.040
b. Jagged and irregular	0.035	0.040	0.050
5. Channels not maintained, weeds, and brush uncut			
a. Dense weeds, high as flow depth	0.050	0.080	0.120
b. Clean bottom, brush on sides	0.040	0.050	0.080
c. Same, highest stage of flow	0.045	0.070	0.110
d. Dense brush, high stage	0.080	0.100	0.140
NATURAL STREAMS			
1. Minor streams (top width at flood stage <100 ft)			
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 more stones/weeds	0.030	0.035	0.040
3) Clean, winding, some pools/shoals	0.033	0.040	0.045
4) Same as above, but some weeds/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
2. Floodplains			
a. Pasture, no brush			
1) Short grass	0.025	0.030	0.035
2) High grass	0.030	0.035	0.050
b. Cultivated area			
1) No crop	0.020	0.030	0.040
2) Mature row crops	0.025	0.035	0.045
3) Mature field crops	0.030	0.040	0.050
c. Brush			
1) Scattered brush, heavy weeds	0.035	0.050	0.070
2) Light brush and trees, in winter	0.035	0.050	0.060
3) Light brush and trees, in summer	0.040	0.050	0.080
4) Medium to dense brush, in winter	0.045	0.070	0.110
5) Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1) Dense willows, summer, straight	0.110	0.150	0.200
2) Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3) Same as above, but with heavy growth of sprouts	0.050	0.060	0.080

4) Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5) Same as above, but with flood stage reaching branches	0.100	0.120	0.160
3. Major Streams (top width at flood stage >100 ft)			
a. Regular section with no boulders or brush	0.025	—	0.060
b. Irregular and rough section	0.035	—	0.100

Source: Reference *(3)*.

If the normal depth is greater than critical depth, the slope is classified as a mild slope while, on a steep slope, the normal depth is less than critical depth. Thus, uniform flow is subcritical on a mild slope and supercritical on a steep slope.

3.3.3 *Velocity Distribution Coefficient*

The flow velocity may not be uniform in a channel cross section due to the presence of free surface, friction along the channel boundary, and change in alignment and cross section. As a result of nonuniform distribution of velocities in a channel section, the velocity head of an open channel is usually greater than the average velocity head computed as $(Q/A_t)^2/2g$. A weighted average value of the velocity head is obtained by multiplying the average velocity head, above, by a velocity distribution coefficient, α, defined as:

$$\alpha = \frac{\sum_{i=1}^{n}(K_i^3/A_i^2)}{(K_t^3/A_t^2)} \tag{3.4}$$

where:

K_i = conveyance in subsection (see Equation 3.5), ft^3/s
K_t = total conveyance in section (see Equation 3.5), ft^3/s
A_i = cross-sectional area of subsection, ft^2
A_t = total cross-sectional area of section, ft^2
n = number of subsections

3.3.4 *Conveyance*

In channel analysis, it is often convenient to group the channel cross section properties in a single term called the channel conveyance K:

$$K = (1.486/n)AR^{2/3} \tag{3.5}$$

and then Equation 3.3 can be written as:

$$Q = KS^{1/2} \tag{3.6}$$

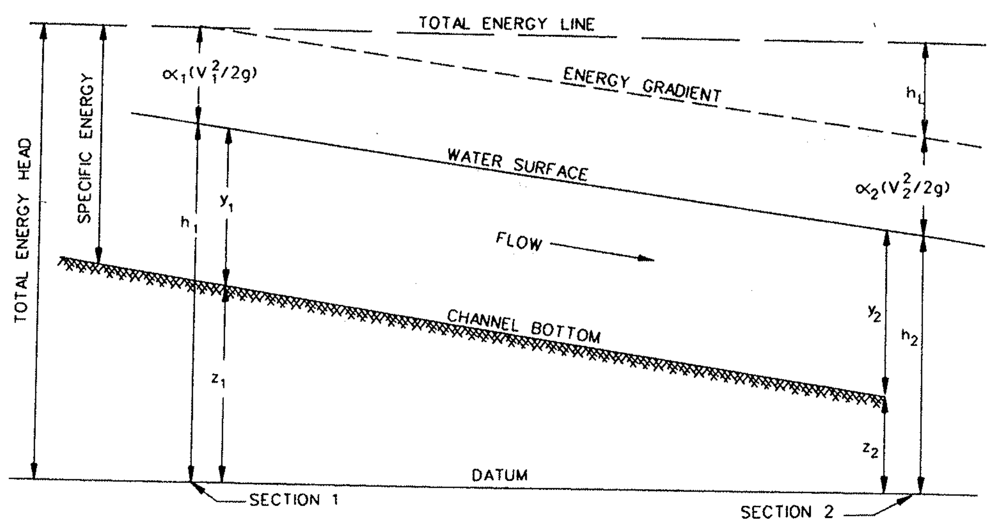
3.3.5 *Energy Equation*

The energy equation expresses conservation of energy in open channel flow expressed as energy per unit weight of fluid, which has dimensions of length and is therefore called energy head. The energy head is composed of potential energy head (elevation head), pressure head, and kinetic energy head (velocity head). These energy heads are scalar quantities that give the total energy head at any cross section when added. Written between an upstream open channel cross section designated 1 and a downstream cross section designated 2 (see Figure 3-1), the energy equation is:

$$h_1 + \alpha_1 \left(V_1^2/2g\right) = h_2 + \alpha_2 \left(V_2^2/2g\right) + h_L \tag{3.7}$$

where:

h_1, h_2 = the upstream and downstream stages, respectively, ft
α_1, α_2 = the upstream and downstream velocity distribution coefficients, respectively
V = mean velocity, ft/s
h_L = head loss due to local cross-sectional changes (minor loss) and boundary resistance, ft



Source: Reference *(5)*.

FIGURE 3-1. Terms in the Energy Equation

The stage, h, is the sum of the elevation head, z, at the channel bottom and the pressure head or depth of flow, y; i.e., $h = z + y$. The terms in the energy equation are illustrated graphically in Figure 3-1. The energy equation states that the total energy head at an upstream cross section is equal to the energy head at a downstream section plus the intervening energy head loss. The energy equation can only be applied between two cross sections at which the streamlines are nearly straight and parallel so that vertical accelerations can be neglected.

3.4 HYDRAULIC ANALYSIS

The hydraulic analysis of a channel determines the depth and velocity at which a given discharge will flow in a channel of known geometry, roughness and slope. The depth and velocity of flow are necessary for the design or analysis of roadway drainage structures.

The step-backwater method shall be used to compute the complete water surface profile in a stream reach or drainage ditch. Occasionally, the designer may need to use a more detailed method of analysis than the computation of a water surface profile using the step-backwater method. Special analysis techniques include two-dimensional analysis, water and sediment routing, and unsteady flow analysis.

3.4.1 *Cross Sections*

Cross-sectional geometry of streams is defined by coordinates of lateral distance and ground elevation that locate individual ground points. The cross section is taken normal to the flow direction along a single, straight line where possible but, in wide floodplains or bends, it may be necessary to use a section along intersecting straight lines; i.e., a "dog-leg" section. It is especially important to make a plot of the cross section to reveal any inconsistencies or errors.

Cross sections should be located to be representative of the subreaches between them. Stream locations with major breaks in bed profile, abrupt changes in roughness or shape, control sections such as free overfalls, bends and contractions, or other abrupt changes in channel slope or conveyance will require cross sections taken at shorter intervals to better model the change in conveyance.

Cross sections should be subdivided with vertical boundaries where there are abrupt lateral changes in geometry and/or roughness as for overbank flows. The conveyances of each subsection are computed separately to determine the flow distribution and are then added to determine the total flow conveyance. The subsection divisions must be chosen carefully so that the distribution of flow or conveyance is nearly uniform in each subsection (Reference *(4)*). Selection of cross sections and the vertical subdivision of a cross section are shown in Figure 3-2.

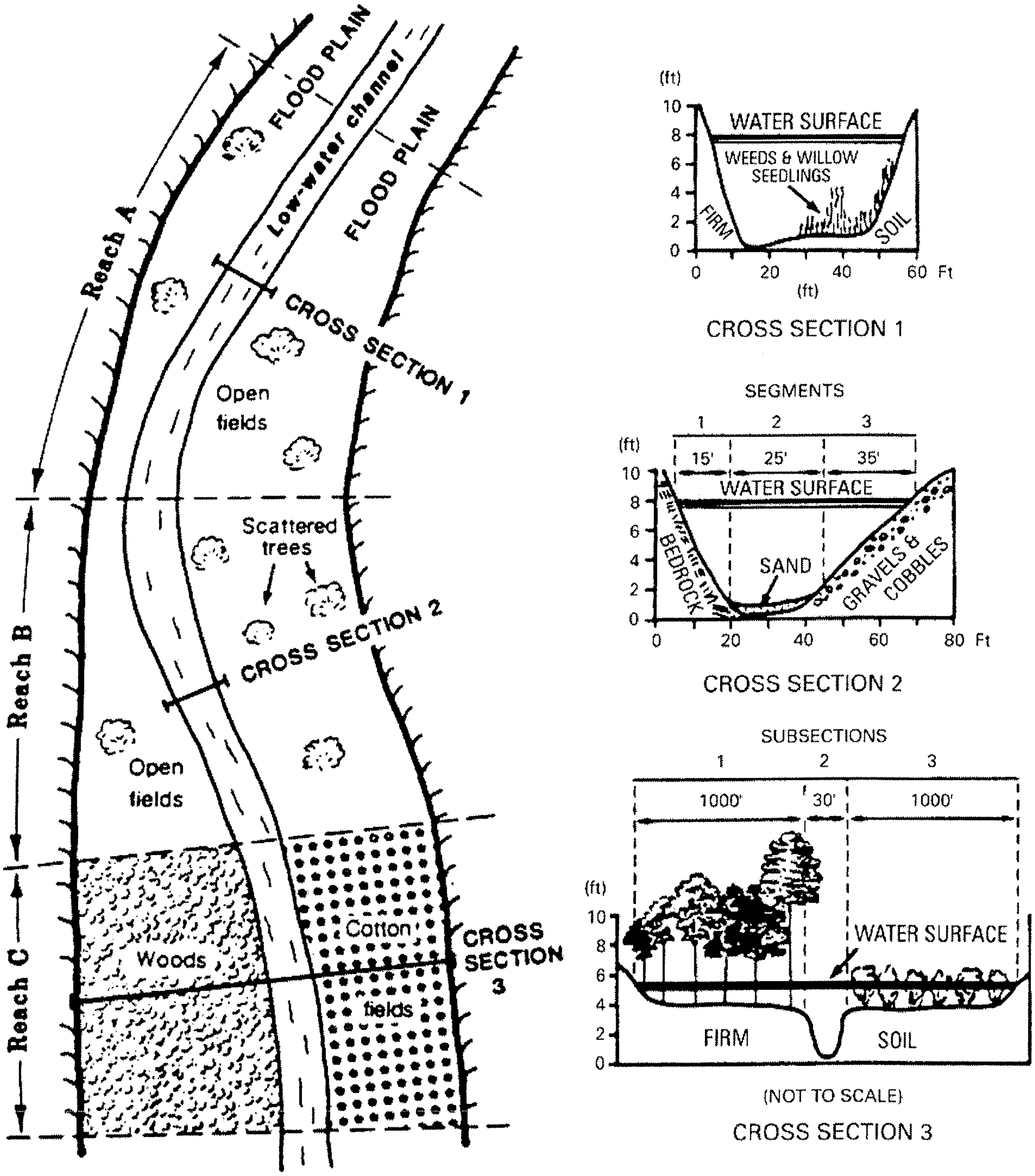


FIGURE 3-2. Hypothetical Cross Section Showing Reaches, Segments and Subsections Used in Assigning *n* Values

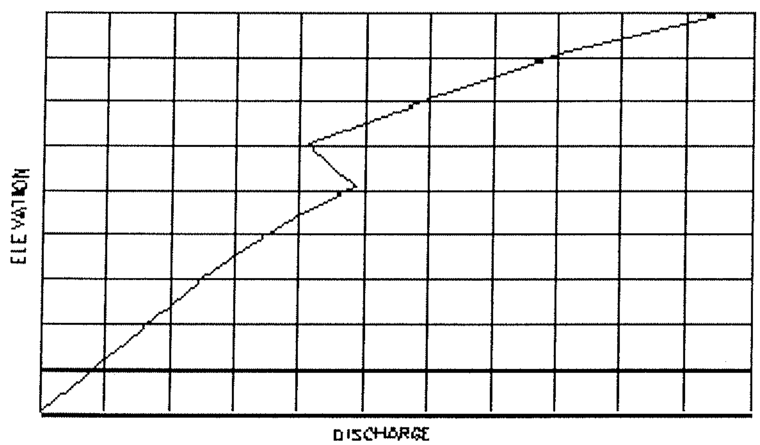
Source: Reference *(1)*.

3.4.2 *Manning's n Value Selection*

Manning's n is affected by many factors and its selection in natural channels depends heavily on engineering experience. Pictures of channels and floodplains for which the discharge has been measured and Manning's n has been calculated are very useful (see References *(1)*, *(2)*). For situations lying outside the engineer's experience, a more regimented approach is presented in Reference *(1)*. Once the Manning's n values have been selected, it is highly recommended that they be verified or calibrated with historical high-water marks and/or gaged streamflow data.

3.4.3 *Switchback Phenomenon*

If the cross section is improperly subdivided, the mathematics of Manning's equation causes a switchback. A switchback results where the calculated discharge decreases with an associated increase in elevation. This occurs when, with a minor increase in water depth, there is a large increase of wetted perimeter. Simultaneously, there is a corresponding small increase in cross-sectional area that causes a net decrease in the hydraulic radius from the value it had for a lesser water depth. With the combination of the lower hydraulic radius and the slightly larger cross-sectional area, a discharge is computed that is lower than the discharge based upon the lower water depth. More subdivisions within such cross sections should be used to avoid the switchback.



Switchback Phenomenon

This phenomenon can occur in any type of conveyance computation, including the step-backwater method. Computer logic can be seriously confused if a switchback were to occur in any cross section being used in a step-backwater program. For this reason, the cross section should always be subdivided with respect to both vegetation and geometric changes. Note that the actual n-value itself may be the same in adjacent subsections.

3.4.4 *Single-Section Analysis*

The single-section analysis method (slope-area method) is simply a solution of Manning's equation for the normal depth of flow given the discharge and cross section properties including geometry, slope, and roughness. It implicitly assumes the existence of steady, uniform flow; however, uniform flow rarely exists in either artificial or natural stream channels. Nevertheless, the single-section method is often used to design artificial channels for uniform flow as a first approximation and to develop a stage-discharge rating curve in a stream channel for tailwater determination at a culvert or storm drain outfall.

3.4.5 *Step-Backwater Analysis*

Step-backwater analysis is useful for determining unrestricted water surface profiles where a roadway crossing is planned and for analyzing how far upstream the water surface elevations are affected by a culvert or bridge. Because the calculations involved in this analysis are tedious and repetitive, it is recommended that a computer program such as the FHWA/USGS program WSPRO *(6)* or USACE HEC-RAS *(9–11)* be used. Special analysis techniques should be considered for complex situations where a step-backwater analysis might not give the desired level of accuracy.

The WSPRO program has been designed to provide a water surface profile for six major types of open channel flow situations:

- unconstricted flow,
- single-opening bridge,
- bridge opening(s) with spur dikes,
- single-opening embankment overflow, and
- multiple alternatives for a single site and multiple openings.

The HEC-RAS program, developed by USACE, is widely used for calculating water surface profiles for steady, gradually varied flow in a natural or constructed channel. Both subcritical and supercritical flow profiles can be calculated. The effects of bridges, culverts, weirs and structures in the floodplain may be also considered in the computations. These programs are also designed for application in floodplain management and flood insurance studies.

3.4.5.1 Step-Backwater Methodology

The computation of water surface profiles by WSPRO and HEC-RAS is based on the standard-step method in which the stream reach of interest is divided into a number of subreaches by cross sections spaced such that the flow is gradually varied in each subreach. The energy equation is then solved in a step-wise fashion for the stage at one cross section based on the stage at the previous cross section.

The method requires definition of the geometry and roughness of each cross section as discussed in Section 3.4.1. Manning's n values can vary both horizontally across the section and vertically. Expansion and contraction head loss coefficients, variable main channel and overbank flow lengths, and the method of averaging the slope of the energy grade line can all be specified.

To amplify on the methodology, the energy equation is repeated from Section 3.3.5:

$$h_1 + \alpha_1 (V_1^2/2g) = h_2 + \alpha_2 (V_2^2/2g) + h_L \tag{3.8}$$

where:

h_1, h_2 = the upstream and downstream stages, respectively, ft
α_1, α_2 = the upstream and downstream velocity distribution coefficients, respectively
V = mean velocity, ft/s
h_L = head loss due to local cross-sectional changes (minor loss) and boundary resistance, ft

The stage h is the sum of the elevation head z at the channel bottom and the pressure head, or depth of flow, y; i.e., $h = z + y$. The energy equation is solved between successive stream reaches with nearly uniform roughness, slope and cross-sectional properties.
The total head loss (h_L) is calculated from:

$$h_L = K_m \left|[(\alpha_1 V_1^2/2g) - (\alpha_2 V_2^2/2g)]\right| + \bar{S}_f L \tag{3.9}$$

where:

K_m = expansion or contraction loss coefficient
$\bar{S}_f$ = the mean slope of the energy grade line evaluated from Manning's equation and a selected averaging technique, ft/ft
L = discharge-weighted or conveyance-weighted reach length, ft

These equations are solved numerically in a step-by-step procedure called the Standard-Step Method from one cross section to the next.

The loss coefficient K_m is used to calculate the expansion or contraction loss between cross sections. Typical values for K_m are 0.1 for a gradual contraction, 0.3 for a sudden contraction, 0.3 for a gradual expansion and 0.5 for a sudden expansion. The default values of the minor loss coefficient K_m are 0.0 and 0.1 for contractions and 0.5 and 0.3 for expansions in WSPRO and HEC-RAS, respectively. Refer to the HEC-RAS *Hydraulic Reference Manual (10)* for guidance on selecting expansion and contraction loss coefficients.

WSPRO calculates a conveyance-weighted reach length, L, as:

$$L = [(L_{lob}K_{lob} + L_{ch}K_{ch} + L_{rob}K_{rob})/(K_{lob} + K_{ch} + K_{rob})] \quad (3.10)$$

where:

L_{lob}, L_{ch}, L_{rob} = flow distance between cross sections in the left overbank, main channel and right overbank, respectively, ft

K_{lob}, K_{ch}, K_{rob}= conveyance in the left overbank, main channel and right overbank, respectively, of the cross section with the unknown water surface elevation

HEC-RAS calculates a discharge-weighted reach length, L, as:

$$L = [(L_{lob}\bar{Q}_{lob} + L_{ch}\bar{Q}_{ch} + L_{rob}\bar{Q}_{rob})/(\bar{Q}_{lob} + \bar{Q}_{ch} + \bar{Q}_{rob})] \quad (3.11)$$

where:

L_{lob}, L_{ch}, L_{rob} = flow distance between cross sections in the left overbank, main channel and right overbank, respectively, ft

$\bar{Q}_{lob}$, $\bar{Q}_{ch}$, $\bar{Q}_{rob}$ = arithmetic average of flows between cross section for the left overbank, main channel and right overbank, respectively, ft^3/s

WSPRO and HEC-RAS allow the user the following options for determining the friction slope, $\bar{S}_f$:

- Average conveyance equation:

$$\bar{S}_f = [(Q_u + Q_d)/(K_u + K_d)]^2 \quad (3.12)$$

- Average friction slope equation:

$$\bar{S}_f = (S_{fu} + S_{fd})/2 \quad (3.13)$$

- Geometric mean friction slope equation:

$$\bar{S}_f = (S_{fu}S_{fd})^{1/2} \quad (3.14)$$

- Harmonic mean friction slope equation:

$$\bar{S}_f = (2S_{fu}S_{fd})/(S_{fu} + S_{fd}) \quad (3.15)$$

where:

Q_u, Q_d = discharge at the upstream and downstream cross sections, respectively, ft^3/s

K_u, K_d = conveyance at the upstream and downstream cross sections, respectively, ft^3/s

S_{fu}, S_{fd} = friction slope at the upstream and downstream cross sections, respectively, ft/ft

The default option is the geometric mean friction slope equation in WSPRO and the average conveyance equation in HEC-RAS.

3.4.5.2 Profile Computation

Water surface profile computation requires a beginning value of elevation or depth (boundary condition) and proceeds upstream for subcritical flow and downstream for supercritical flow. In the case of supercritical flow, critical depth is often the boundary condition at the control section but, in subcritical flow, uniform flow and normal depth may be the boundary condition. The starting depth in this case can either be found by the single-section method (slope-area method) or by computing the water surface profile upstream to the desired location for several starting depths and the same discharge. These profiles should converge toward the desired normal depth at the control section to establish one point on the stage-discharge relation. If the several profiles do not converge, then the stream reach may need to be extended downstream, or a shorter cross section interval should be used, or the range of starting water surface elevations should be adjusted. In any case, a plot of the convergence profiles can be a very useful tool in such an analysis (see Figure 3-3).

Given a long enough stream reach, the water surface profile computed by step-backwater will converge to normal depth at some point upstream for subcritical flow. Establishment of the upstream and downstream boundaries of the stream reach is required to define the limits of data collection and subsequent analysis. Calculations must begin sufficiently far downstream to assure accurate results at the structure site, and continued a sufficient distance upstream to accurately determine the impact of the structure on upstream water surface profiles (see Figure 3-4).

USACE *(8)* developed equations for determining upstream and downstream reach lengths as follows:

$$Ldn = 8000\ (HD^{0.8}/S) \tag{3.16}$$

$$Lu = 10{,}000\ [(HD^{0.6})(HL^{0.5})]/S \tag{3.17}$$

where:

- *Ldn* = downstream study length (along main channel), ft (for normal depth starting conditions)
- *Lu* = estimated upstream study length (along main channel), ft (required for convergence of the modified profile to within 0.1 ft of the base profile)
- *HD* = average hydraulic depth (1% chance event flow area divided by the top width), ft
- *S* = average reach slope, ft/mi
- *HL* = headloss ranging between 0.5 ft and 5 ft at the channel crossing structure for the 1% chance flood, ft

References *(4)* and *(8)* are very valuable sources of additional guidance on the practical application of the step-backwater method to highway drainage problems involving open channels. These references contain more specific guidance on cross section determination, location and spacing and stream reach determination. Reference *(8)* investigates the accuracy and reliability of water surface profiles related to n-value determination and the survey or mapping technology used to determine the cross section coordinate geometry.

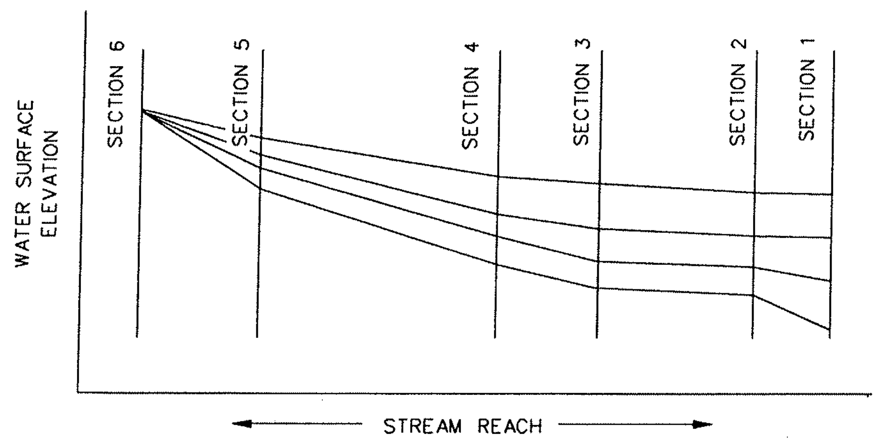


FIGURE 3-3. Profile Convergence Pattern Backwater Computation

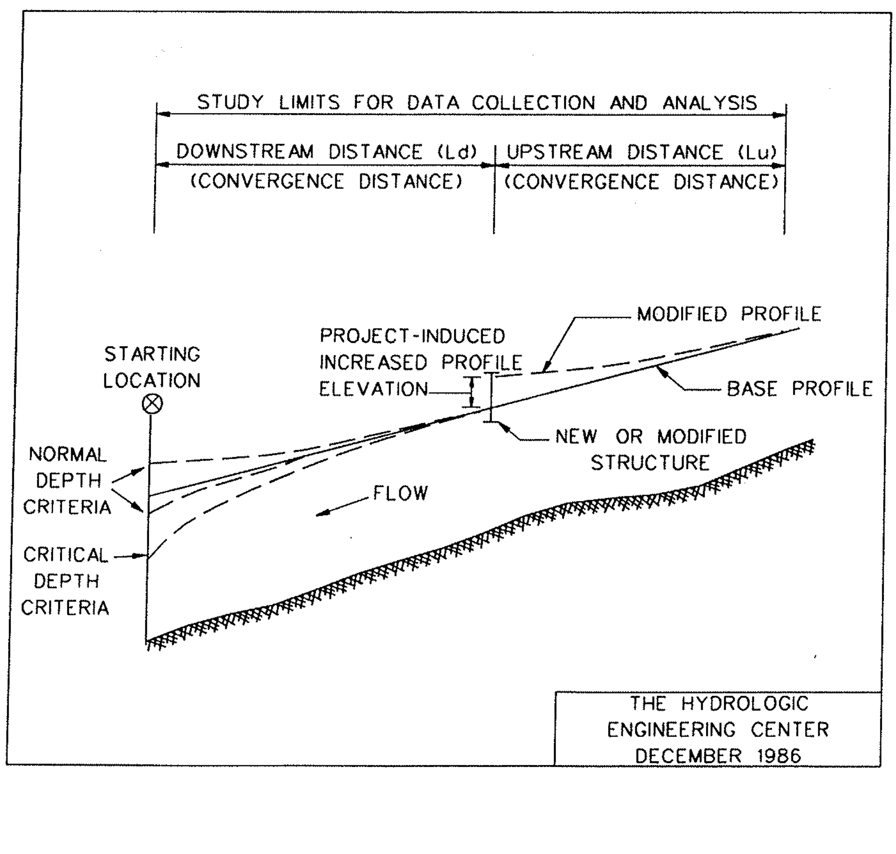


FIGURE 3-4. Profile Study Limits

Source: Reference *(8)*.

3.4.5.3 Computation Procedure

A sample procedure is taken from Reference *(12)*. An example problem using this procedure is provided in Appendix 3A.

A convenient form for use in calculating water surface profiles is shown in Figure 3-5. In summary, Columns 2 and 4 through 12 are devoted to solving Manning's equation to obtain the energy loss due to friction; Columns 13 and 14 contain calculations for the velocity distribution across the section; Columns 15 through 17 contain the average kinetic energy; Column 18 contains calculations for "other losses" (expansion and contraction losses due to interchanges between kinetic potential energies as the water flows); and Column 19 contains the computed change in water surface elevation. Conservation of energy is accounted for by proceeding from section to section down the computation form.

Column 1 CROSS SECTION NO., is the cross section identification number. Miles upstream from the mouth are recommended.

Column 2 ASSUMED, is the assumed water surface elevation that must agree with the resulting, computed water surface elevation within + 0.05 ft, or some allowable tolerance, for trial calculations to be successful.

Column 3 COMPUTED, is the rating curve value for the first section but, thereafter, is the value calculated by adding *WS* to the computed water surface elevation for the previous cross section.

Column 4 A, is the cross section area. If the section is complex and has been subdivided into several parts (e.g., left overbank, channel and right overbank), use one line of the form for each subsection and sum to get A_t, the total area of cross section.

Column 5 R, is the hydraulic radius. Use the same procedure as for Column 4 if section is complex, but do not sum subsection values.

Column 6 $R^{2/3}$, is 2/3 power of hydraulic radius.

Column 7 n, is Manning roughness coefficient.

Column 8 K, is conveyance and is defined as $(C_m A R^{2/3}/n)$ where C_m is 1.486. If the cross section is complex, sum subsection K values to get K_t.

Cross Section No.	Water Surface Elevation		Area	Hydraulic Radius (R)	$R^{2/3}$	n	K	$\overline{K_t}$	1000 $\bar{S}_f$	L	h_f	K^3/A^2	α	V	$\alpha V^2/2g$	$(\alpha V^2/2g)$	h_o	Water Surface Elevation
	Assumed	Computed																
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)

FIGURE 3-5. Water Surface Profile Form

Column 9 $\overline{K}_t$, is average conveyance for the reach, and is calculated by $0.5(K_{td} + K_{tu})$ where subscripts D and U refer to downstream and upstream ends of the reach, respectively.

Column 10 $\overline{S}_f$, is the average slope through the reach determined by $(Q/\overline{K}_t)^2$.

Column 11 L, is the discharge-weighted or conveyance-weighted reach length.

Column 12 h_f, is energy loss due to friction through the reach and is calculated by $h_f = (Q/\overline{K}_t)^2 L = \overline{S}_f L$.

Column 13 $S(K^3/A^2)$, is part of the expression relating distributed flow velocity to an average value. If the section is complex, calculate one of these values for each subsection and sum all subsection values to get a total. If one subsection is used, Column 13 is not needed and Column 14 equals one.

Column 14 α, is the velocity distribution coefficient and is calculated by $S(K^3/A^2)/(K_t^3/A_t^2)$ where the numerator is the sum of values in Column 13 and the denominator is calculated from K_t and A_t.

Column 15 V, is the average velocity and is calculated by Q/A_t.

Column 16 $\alpha V^2/2g$, is the average velocity head corrected for flow distribution.

Column 17 $D(\alpha V^2/2g)$, is the difference between velocity heads at the downstream and upstream sections.

Column 18 h_o, is "other losses," and is calculated by multiplying either the expansion or contraction coefficient, K_m, times the absolute value of Column 17. A contraction occurs whenever the velocity head downstream is greater than the velocity head upstream. Likewise, an expansion occurs when the velocity head upstream is greater than the velocity head downstream.

Column 19 DWS, is the change in water surface elevation from the previous cross section. It is the algebraic sum of Columns 12, 17, and 18.

3.4.6 *Special Analysis Techniques*

Open channel flow problems sometimes arise that require a more detailed analysis than the computation of a water surface profile using the Standard-Step Method or the Direct-Step Method. More detailed analysis techniques include two-dimensional analysis, water and sediment routing and unsteady flow analysis. Several computer programs are available for these more detailed analysis techniques. HEC-RAS can be used to analyze one-dimensional, unsteady flow. Both RMA2 and FESWMS-2DH can be used to analyze two-dimensional, unsteady flow, and are also commonly used for 2-D modeling. The BRI-STARS model can be used to study complicated sedimentation problems.

3.5 REFERENCES

(1) Arcement, G. J., Jr., and V. R. Schneider. *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains.* FHWA-TS-84-204. FHWA, U.S. Department of Transportation, Washington, DC, 1984.

(2) Barnes, H. H., Jr. *Roughness Characteristics of Natural Channels.* U.S. Geological Survey Water Supply Paper 1849. U.S. Government Printing Office, Washington, DC, 1978.

(3) Chow, V. T. *Open Channel Hydraulics.* McGraw-Hill, New York, NY, 1959.

(4) Davidian, J. *Computation of Water Surface Profiles in Open Channels, Techniques of Water Resources Investigation.* Book 3, Chapter A15. U.S. Geological Survey, Washington, DC, 1984.

(5) Federal Highway Administration. *River Engineering for Highway Encroachments—Highways in the River Environment.* Hydraulic Design Series No. 6, FHWA-NHI-01-004. FHWA, U.S. Department of Transportation, Washington, DC, December 2001.

(6) Federal Highway Administration. *WSPRO User's Manual.* Version P60188, FHWA-SA-98-080. FHWA, U.S. Department of Transportation, Washington, DC, 1999.

(7) Henderson, F. M. *Open Channel Flow.* Macmillan, New York, 1966.*l of Hydraulic Engineering.* Vol. 11, No. 5, American Society of Civil Engineers, Reston, VA, May 1985.

(8) U.S. Army Corps of Engineers. *Accuracy of Computed Water Surface Profiles.* The Hydrologic Engineering Center, Davis, CA, December 1986.

(9) U.S. Army Corps of Engineers. *HEC-RAS, River Analysis System—Applications Guide.* Version 3.1. The Hydrologic Engineering Center, Davis, California, November 2002.

(10) U.S. Army Corps of Engineers. *HEC-RAS, River Analysis System, Hydraulic Reference Manual.* Version 3.1. The Hydrologic Engineering Center, Davis, CA, November 2002.

(11) U.S. Army Corps of Engineers. *HEC-RAS, River Analysis System, User's Manual.* Version 3.1. The Hydrologic Engineering Center, Davis, CA, November 2002.

(12) U.S. Army Corps of Engineers. Water Surface Profiles. *Hydrologic Engineering Methods for Water Resources Development.* Volume 6, HEC-IHD-0600. The Hydrologic Engineering Center, Davis, CA, July 1975.

APPENDIX 3A

EXAMPLE PROBLEM – STEP BACKWATER METHOD

The step-backwater procedure is illustrated in the following example.

Four cross sections along a reach are shown in Figures 3A-1 through 3A-4. Each cross section is separated by 500 ft and is subdivided according to geometry and roughness. The calculations shown in Figure 3A-5 represent one set of water-surface calculations. An explanation of Figure 3A-5 is in Section 3.4.5.3.

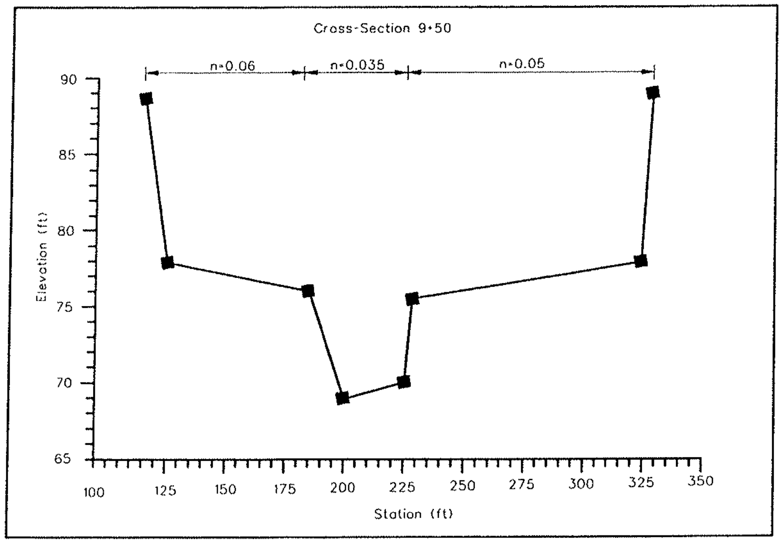


FIGURE 3A-1. Cross Section at Station 9+50 (farthest downstream)

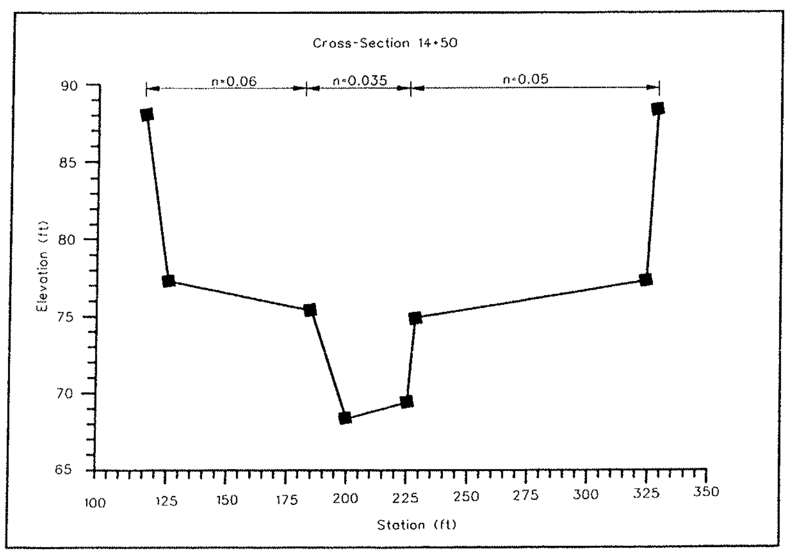


FIGURE 3A-2. Cross Section at Station 14+50

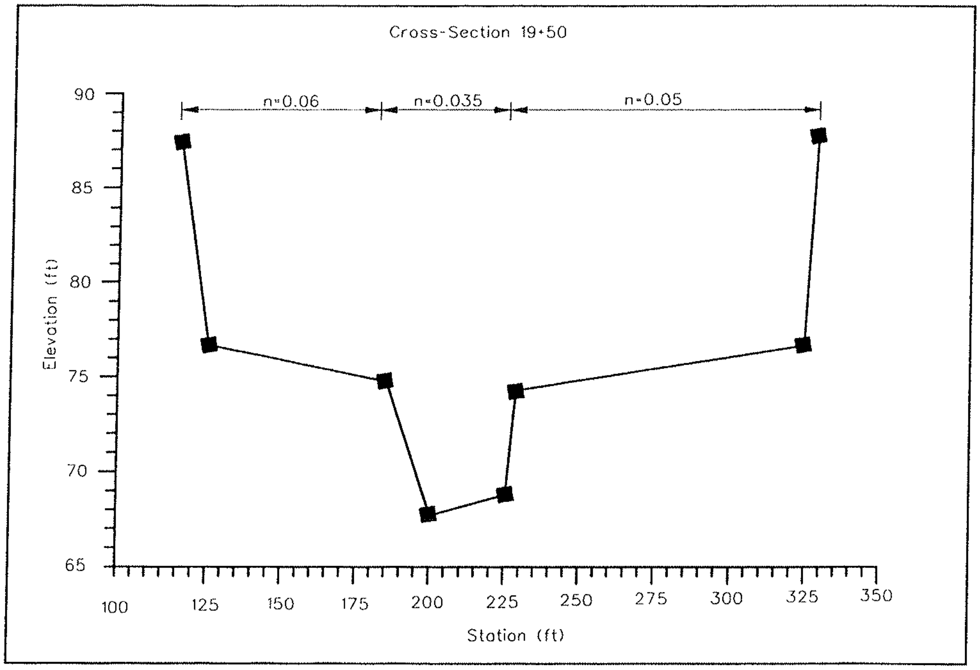


FIGURE 3A-3. Cross Section at Station 19+50

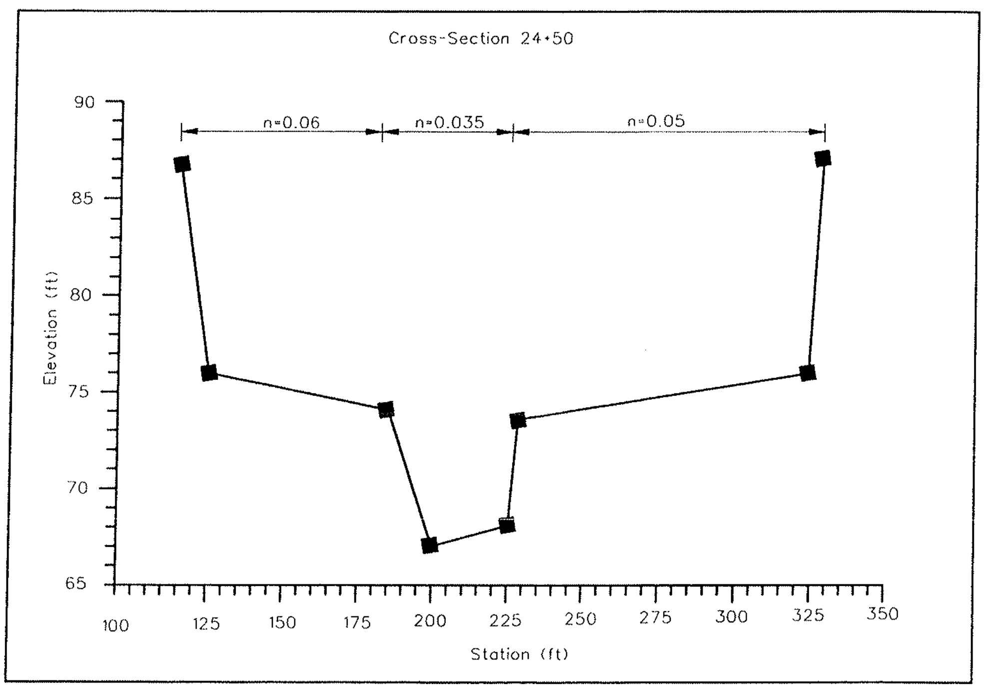


FIGURE 3A-4. Cross Section at Station 24+50 (farthest upstream)

FIGURE 3A-5. Example, Water Surface Profile Form

Cross Section No.	Water Surface Elevation		Area	R	$R^{2/3}$	n	K	$\overline{K}_t$	$1000S_f$	L	h_f	(K^3/A^2)	α	V	$\alpha V^2/2g$	$(\alpha V^2/2g)$	h_o	Water Surface Elevation
	Assumed	Computed																
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
9+50		79.30																
			191	2.23	1.71	0.060	8089					14,508,342						
			506	8.20	4.07	0.035	87437					2,610,866,774						
			146	1.28	1.18	0.050	5120					6,296,572						
			843				100646	100646	0.40	0	0	2,631,671,688	1.83	2.4	0.16	0.00	0.000	0.00
14+50	79.50	79.50																
			192	2.23	1.71	0.060	8131					14,582,414						
			507	8.20	4.07	0.035	87610					2,616,044,470						
			147	1.28	1.18	0.050	5155					6,339,446						
			846				100896	100771	0.40	500	0.20	2,636,966,330	1.84	2.4	0.16	0.00	0.000	0.20
19+50	80.22	79.71																
			227	2.66	1.92	0.060	10794					24,408,001						
			531	8.60	4.20	0.035	94688					3,010,888,601						
			196	1.71	1.43	0.050	8330					15,045,638						
			954				113812	107354	0.35	500	0.18	3,050,342,240	1.88	2.1	0.13	0.03	0.000	0.21
	79.66	79.87																
			182	2.17	1.68	0.060	7573					13,109,976						
			501	8.10	4.03	0.035	85722					2,509,603,778						
			136	1.18	1.12	0.050	4527					5,015,791						
			819				97822	99359	0.41	500	0.21	2,527,729,545	1.81	2.5	0.17	-0.04	0.000	0.17
24+50	79.89	79.30																
			130	1.87	1.52	0.060	4894					6,935,484						
			403	7.61	3.87	0.035	66217					1,787,681,873						
			84	0.95	0.97	0.050	2422					2,012,523						
			617				73532	93672	0.46	500	0	1,796,629,880	1.72	3.3	0.28	0.11	0.000	0.00

Q=2013 ft^3/s

CHAPTER 4 – CULVERTS

4.1 GENERAL

The function of a drainage culvert is to pass the design storm flow under a roadway or railroad without causing excessive backwater and without creating excessive downstream velocities. The designer shall keep energy losses and discharge velocities within reasonable limits when selecting a structure.

Culvert flow may be separated into two major types of flow, inlet or outlet control. Generally, the hydraulic control in a culvert will be at the culvert outlet if the culvert slope is less than the critical slope. Inlet control usually governs if the culvert slope is greater than critical slope. Other factors that determine how the culvert performs can be listed as physical makeup of the structure and the leading and trailing water surface profiles.

4.1.1 *Outlet Control*

For outlet control, factors such as type of opening, cross sectional area, barrel slope, barrel length, barrel roughness, and head losses due to tailwater are predominant in controlling the headwater of the culvert. These separately or conjointly create physical resistance that retards the flow of water. As the resistance accumulates, the flow begins to slow and increase in depth. At some point, when the resistance mounts, the water may cease to flow freely and back up in the structure and flood the upstream drainage basin.

4.1.2 *Inlet Control*

For inlet control, the entrance characteristics of the culvert are such that the entrance head losses are predominant in determining the headwater of the culvert. The barrel will carry water through the culvert faster than the water can enter the culvert.

4.2 DESIGN CRITERIA

- Design criteria for culverts shall include the following:
- Maximum headwater depth for the design storm shall be 1feet lower than the top of roadway or top of curb.
- Maximum headwater depth for the 100-year storm event shall be 1 foot lower than the floor elevation of adjacent buildings or structures.
- Minimum culvert diameter for round pipe shall be 15 inches. Minimum culvert size for arch pipe or elliptical pipe shall be 15 inch equivalent. Minimum size for box culverts shall be 4 feet by 4 feet.
- The minimum allowable fill or cover shall be 12 inches above the top of the culvert. For culverts under roadways there shall also be a minimum clearance of 6 inches from the top

of the culvert to the bottom of the pavement base. Special box culverts designed to carry traffic on the top slab do not have to meet minimum allowable fill requirements.

- As a minimum, pipe culverts shall be constructed of Class III, Reinforced Concrete Pipe. Box Culverts shall also be constructed of reinforced concrete. All culverts shall be constructed with reinforced concrete headwalls/endwalls, slope walls, or flared-end sections at the inlet and outlet.

- Culvert length and slope shall be chosen to approximate existing topography.

- Culvert invert shall be aligned with the channel bottom and the skew angle of the stream.

- Tailwater depth may be calculated with Manning's Equation (Section 3.3.2) if Step Backwater Analysis is not required for the downstream channel. If the headwater elevation for a nearby downstream culvert or storm drain is greater than the normal depth for the channel, a Step Backwater Analysis shall be required.

- Energy dissipators will be required at culvert outlets in earthen channels when the discharge velocity exceeds 6 ft/s. Energy dissipators shall be designed in accordance with the *Hydraulic Design of Stilling Basins and Energy Dissipators (2)*, developed by the U.S. Bureau of Reclamation.

4.3 DESIGN PROCEDURE

The computations involved in selecting the smallest feasible culvert which can be used without exceeding the design headwater elevation can be summarized on the tabulation sheet from Reference *(1)*. The tabulation sheet is shown in Appendix 4A.

After the initial data has been entered into the tabulation sheet, the initial trial size must be entered in Column 1. The square feet of opening for the initial trial size may be estimated by dividing the design discharge by 10. After the initial trial size has been selected, the following procedure may be used to complete the culvert design:

4.3.1 *Step 1 – Perform Outlet Control Calculations*

Column 1: Enter the span and height dimensions (or diameter of pipe) of culvert.

Column 2: Enter the type of structure and design of entrance.

Column 3: Enter the design discharge or quotient of design discharge divided by the applicable denominator.

Column 4: Enter the Entrance Loss Coefficient, K_e. A list of Entrance Loss Coefficients from Reference *(3)* can be found in Appendix 4B, Outlet Control Data.

Column 5: Enter the head from the applicable outlet control nomograph. Nomographs from Reference *(3)* can be found in Appendix 4B, Outlet Control Data.

Column 6: Enter the critical depth from the appropriate nomograph. Nomographs from Reference *(3)* can be found in Appendix 4B, Outlet Control Data.

Column 7: For tailwater elevations less than the top of the culvert at the outlet, headwater is found by solving for h_o using the following equation:

$$h_o = (d_c + D)/2 \tag{4.1}$$

where:
h_o = vertical distance in feet from culvert invert at outlet to the hydraulic grade line in feet
d_c = critical depth in feet
D = height of culvert opening in feet

Column 8: Enter the tailwater elevation from initial data shown at the top of the tabulation sheet.

Column 9: Enter the product of the culvert length times the slope.

Column 10: Headwater elevation required for culvert to pass flow in outlet control (HW_o) is computed by the following equation:

$$HW_o = H + h_o - LS \tag{4.2}$$

Note: Use TW elevation in lieu of h_o where TW > h_o.

Additional trials may be required to determine the minimum barrel size. Space for additional trials is provided on the tabulation sheet.

4.3.2 *Step 2 – Perform Inlet Control Calculations*

Column 11: Enter ratio of headwater to height of structure from the applicable inlet control nomograph. Nomographs from Reference *(3)* can be found in Appendix 4C, Inlet Control Data.

Column 12: HW is derived by multiplying Column 11 by the height (or diameter) of culvert.

4.3.3 *Step 3 – Choose Controlling Headwater, Calculate Velocity*

Column 13: Enter the greater of the two headwaters (Column 10 or 12).

Column 14: If inlet control governs, outlet velocity equals Q/A, where A is defined by the cross sectional area of normal depth of flow in the culvert barrel. A hydraulic elements chart from Reference *(1)* has been included in Appendix 4D to assist in estimating normal depth of flow and velocity. Manning's Equation (3.3.2) may also be used.

If outlet control governs, outlet velocity equals Q/A, where A is the cross sectional area of flow in the culvert barrel at the outlet, based on the following:

- Critical depth if the tailwater is below critical depth.
- The tailwater depth if the tailwater is between critical depth and the top of the barrel.
- The height of the barrel if the tailwater is above the top of the barrel.

Columns 15 & 16: These columns are self-explanatory.

4.4 EXAMPLE PROBLEM

An example problem from Reference *(1)* is shown below:

Given:	Design Discharge	= 1,000 cfs
	Slope of Stream Bed	= 0.071 ft/ft
	Allowable Headwater Elevation	= 200.0
	Elevation of Outlet Invert	= 182.9
	Culvert Length	= 100 ft

Downstream channel approximates an 8 ft wide trapezoidal channel with 2:1 side slopes, Manning "n" of 0.03 and a slope of 0.071 feet per foot.

Required: Design a culvert that will adequately convey the design discharge. The culvert should have the smallest possible barrel to pass design Q without exceeding AHW elevation. Try a reinforced box culvert with "n" = 0.012.

Solution: The solution is shown on the tabulation sheet from Reference *(1)* in Appendix 4E, Example Problem.

4.5 REFERENCES

(1) Arkansas State Highway and Transportation Department. *Drainage Manual.* July 1982.

(2) Bureau of Reclamation. *Hydraulic Design of Stilling Basins and Energy Dissipators.* Engineering Monograph No. 25. U.S. Department of Interior, Bureau of Reclamation, Washington, DC. Third printing. 1974.

(3) Federal Highway Administration. *Hydraulic Design of Highway Culverts.* Hydraulic Design Series No. 5, FHWA-NHI-01-020. FHWA, U.S. Department of Transportation, Washington, DC, September 2001.

APPENDIX 4A

CULVERT TABULATION SHEET
(Source: AHTD)

FI RE 4A-1. Culvert Tabulation Sheet

FORM HYD 4-1

CULVERT COMPUTATIONS
(SQUARE AND BEVELED EDGES)

DESIGNER: ________

PROJECT: ________

DATE: ________

HYDROLOGIC AND CHANNEL INFORMATION

HYDROLOGY

Q_1 ____ = ____ cfs

Q_2 ____ = ____ cfs

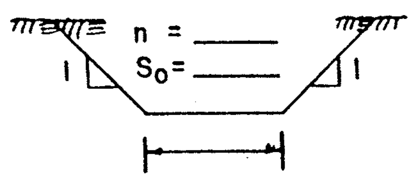
STREAM DATA

Tw_1 = ____

Tw_2 = ____

n = ____

S_o = ____



OUTLET CHANNEL
(APPROX. DIMENSIONS)

SKETCH

STATION: ________

FIN. GR. EL. ____

HW_f EL. ____

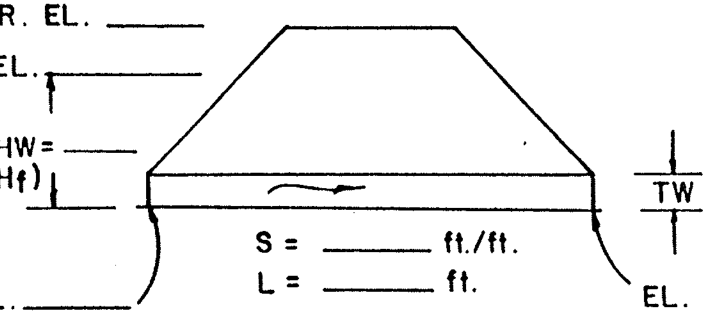
AHW = ____
(H_f)

H_f EL. ____

S = ____ ft./ft.

L = ____ ft.

EL. ____



TRIAL NO.	SIZE	STRUCTURE TYPE & ENTRANCE DESIGN	Q	HEADWATER COMPUTATION									CONTROL-LING HW	OUTLET VELOCITY ft./sec.	COST	COMMENTS
			Q/NB	OUTLET CONTROL							INLET CONT.					
			$\frac{Q}{NBD^{3/2}}$	(a) K_e	H	(b) d_c	(c) h_o	(d) TW	LS	(e) HW_o	(f) $\frac{HW}{D}$	HW				
0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16

(a) Entrance loss coefficient, Refer to Table 4-10, page 4-60.
(b) "d_c" cannot exceed D.
(c) $h_o = \frac{d_c + D}{2}$ or TW, whichever is larger.
(d) TW = d_n in natural channel, or other downstream control.
(e) $HW_o = H + H_o - LS$
(f) Use Chart 4-7, page 4-64, for Conventional face.
Use Chart 4-8, page 4-65, for Beveled Edge.

APPENDIX 4B

OUTLET CONTROL DATA
(Source: FHWA)

Outlet Control, Full or Partly Full Entrance head loss

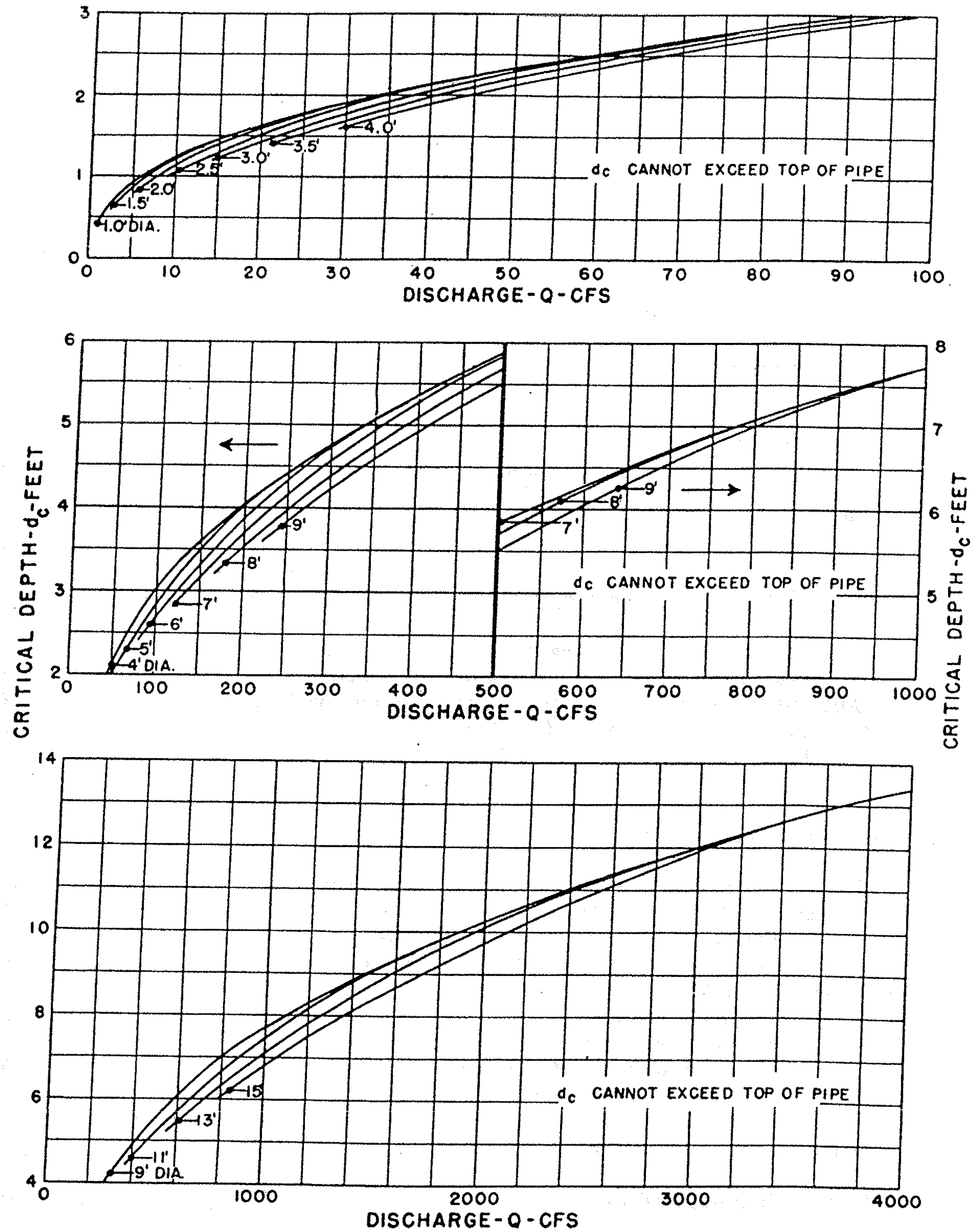
$$H_e = k_e \left(\frac{V^2}{2g} \right)$$

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side-or slope-tapered inlet	0.2

*Note: "End Section conforming to fill slope," made of either metal or concrete, 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. These latter sections can be

TABLE 4B-1. Entrance Loss Coefficients

CHART 4



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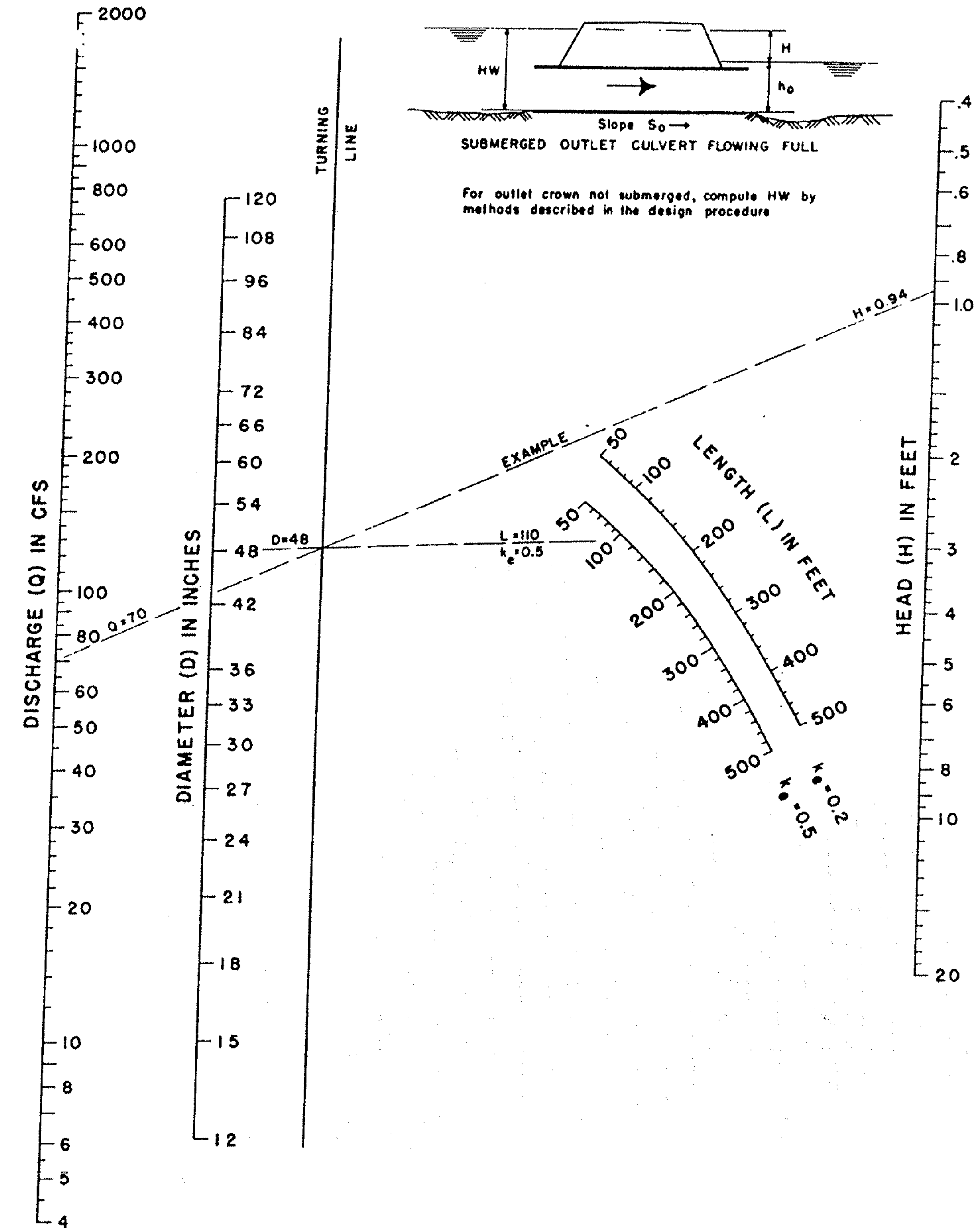
JAN. 1964

CRITICAL DEPTH
CIRCULAR PIPE

FIGURE 4B-1. Critical Depth, Circular Pipe

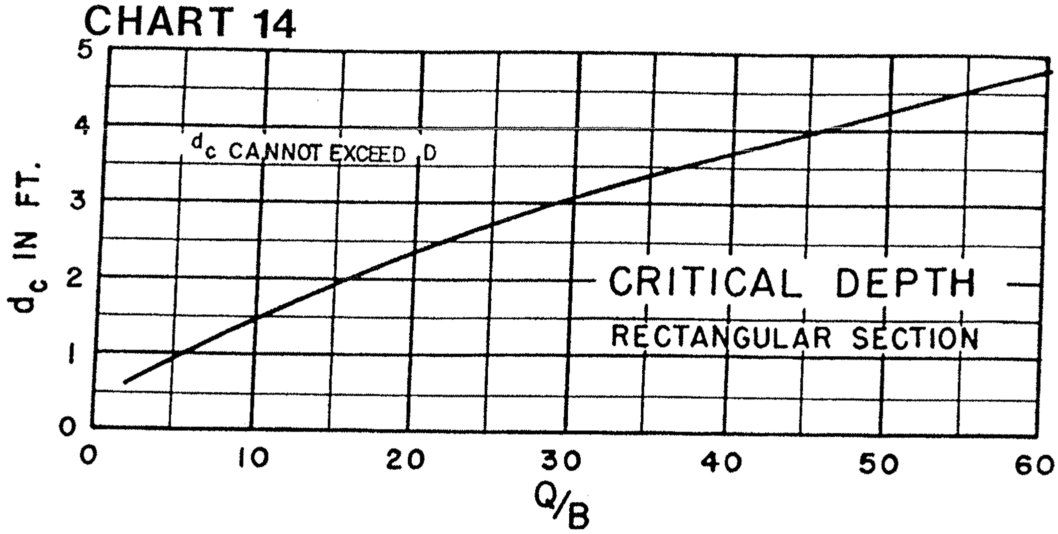


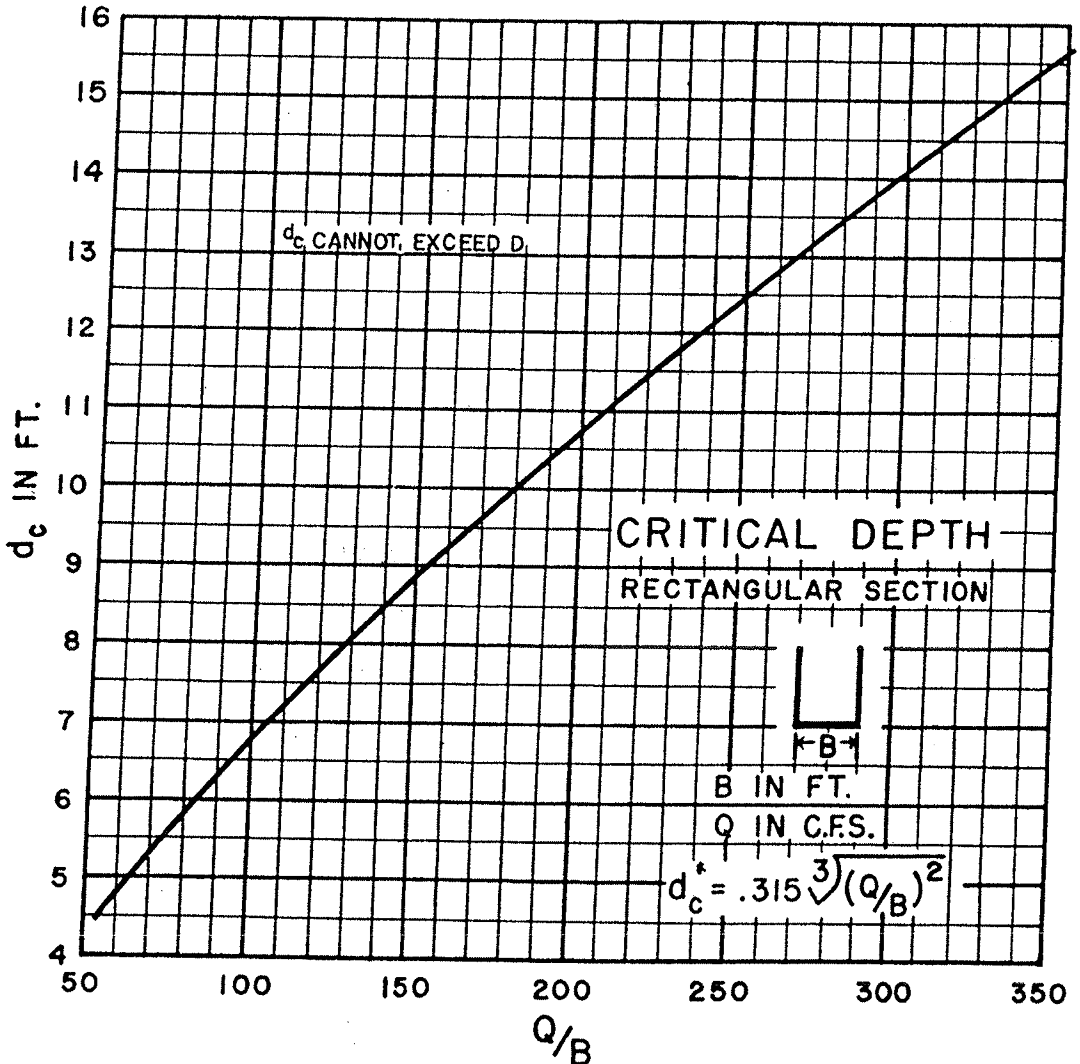
HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

FIGURE 4B-2. Head for Concrete Pipe Culverts Flowing Full

CHART 14

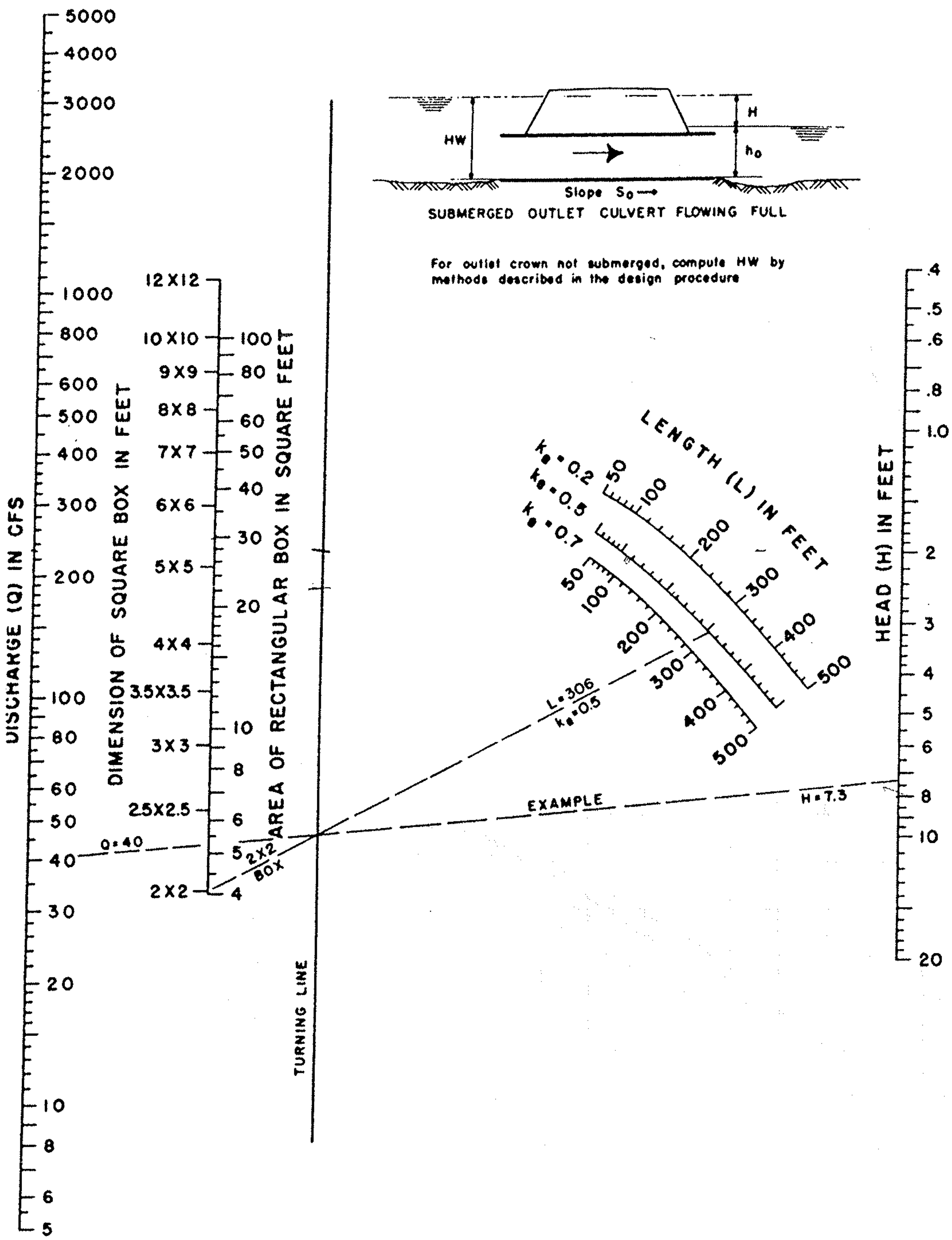




BUREAU OF PUBLIC ROADS JAN. 1963

CRITICAL DEPTH
RECTANGULAR SECTION

FIGURE 4B-3. Critical Depth, Rectangular Section



HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0.012

AU OF PUBLIC ROADS JAN. 1963

FIGURE 4B-4. Head for Concrete Box Culverts Flowing Full

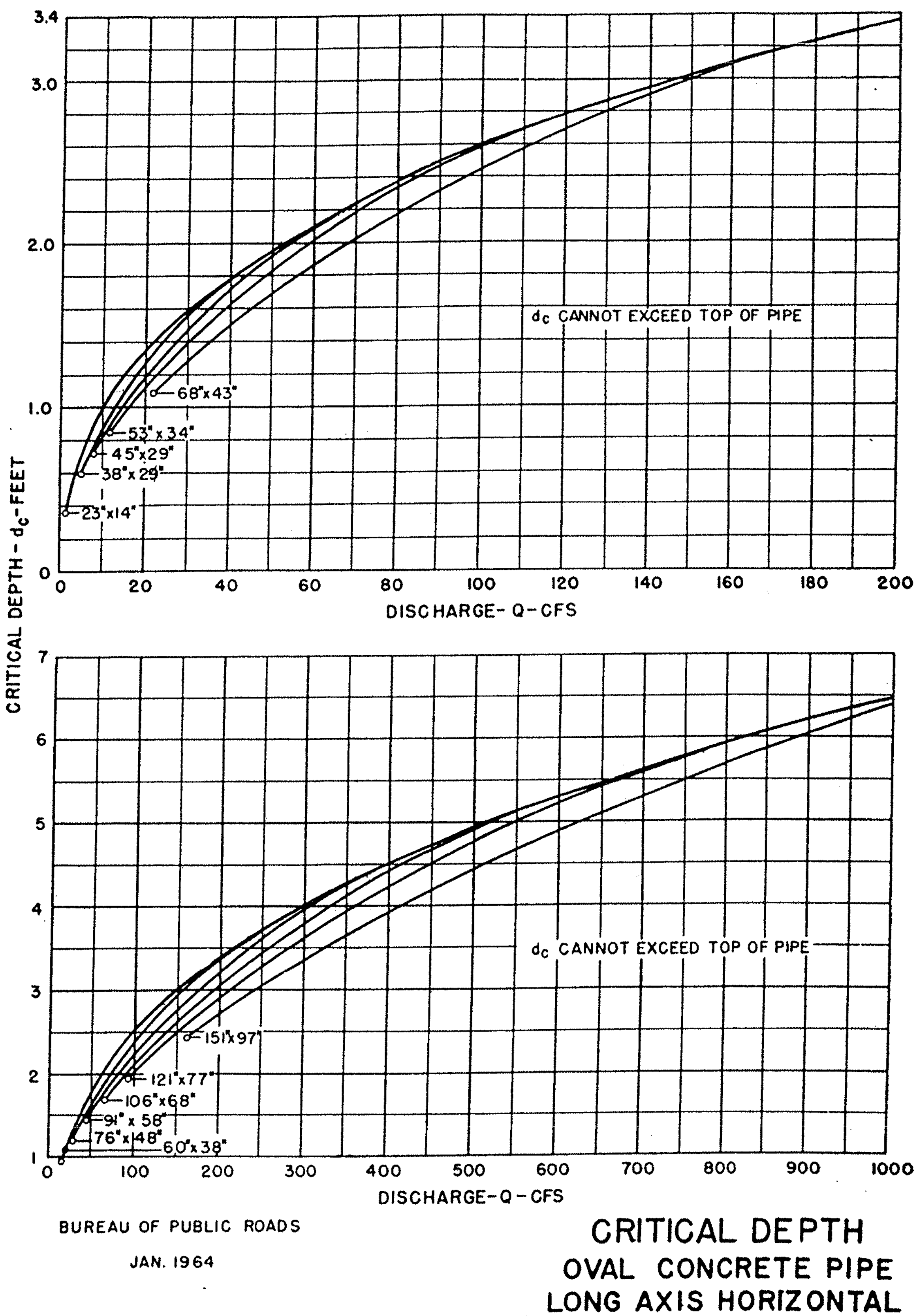


FIGURE 4B-5. Critical Depth, Oval Concrete Pipe, Long Axis Horizontal

CHART 32

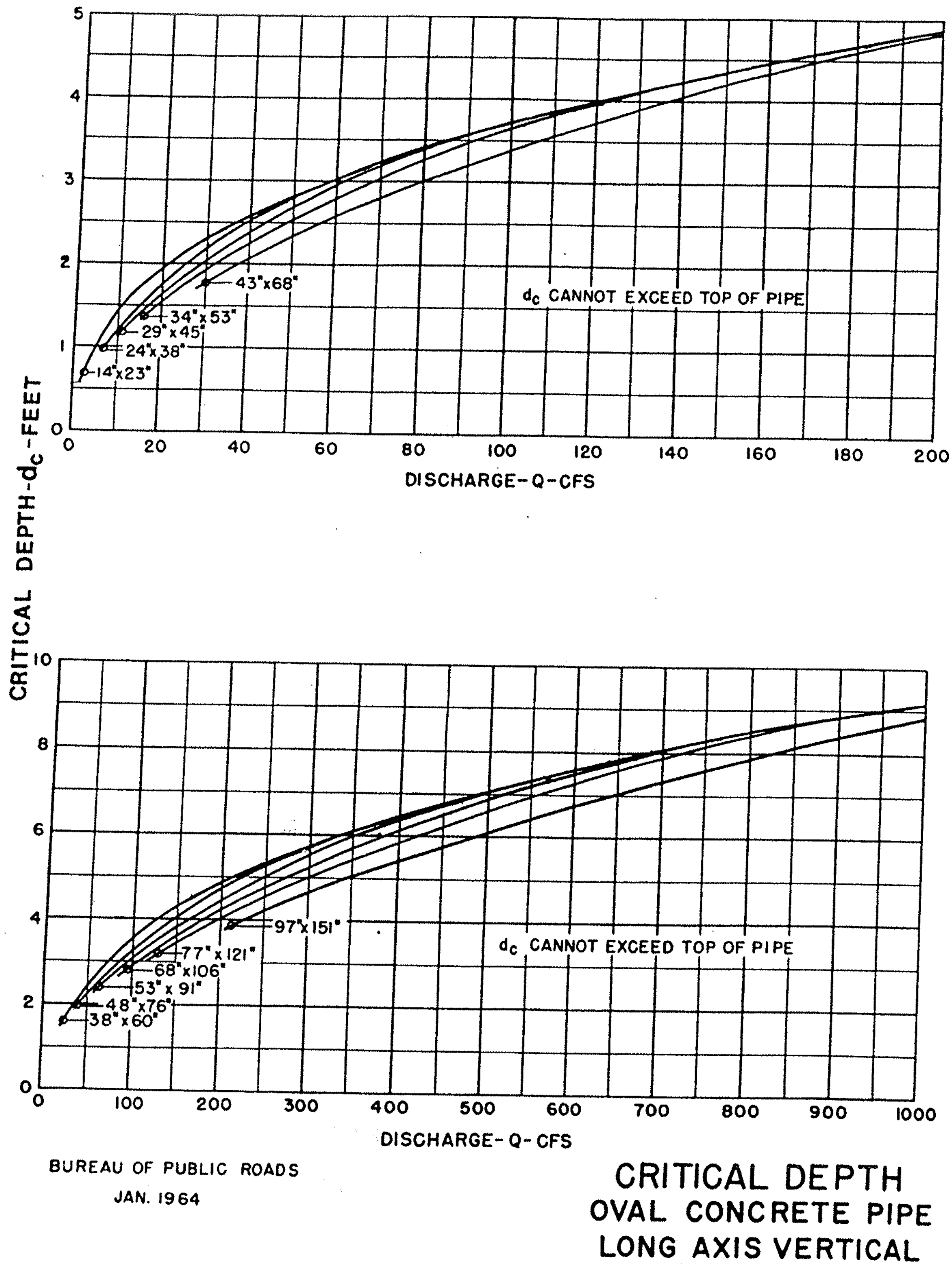


FIGURE 4B-6. Critical Depth, Oval Concrete Pipe, Long Axis Vertical

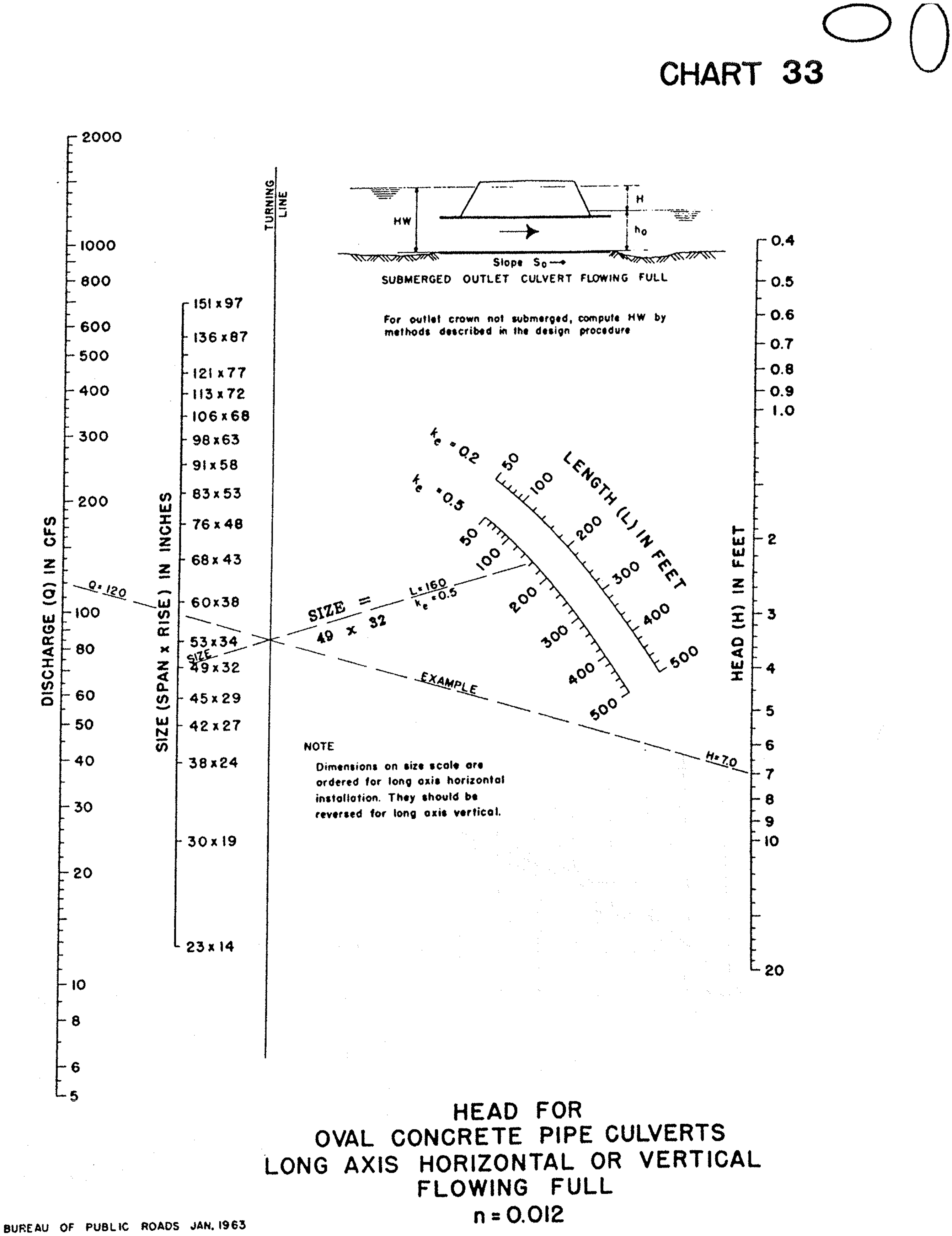


FIGURE 4B-7. Head for Oval Concrete Pipe Culverts, Long Axis Horizontal or Vertical Flowing Full

APPENDIX 4C

INLET CONTROL DATA
(Source: FHWA)

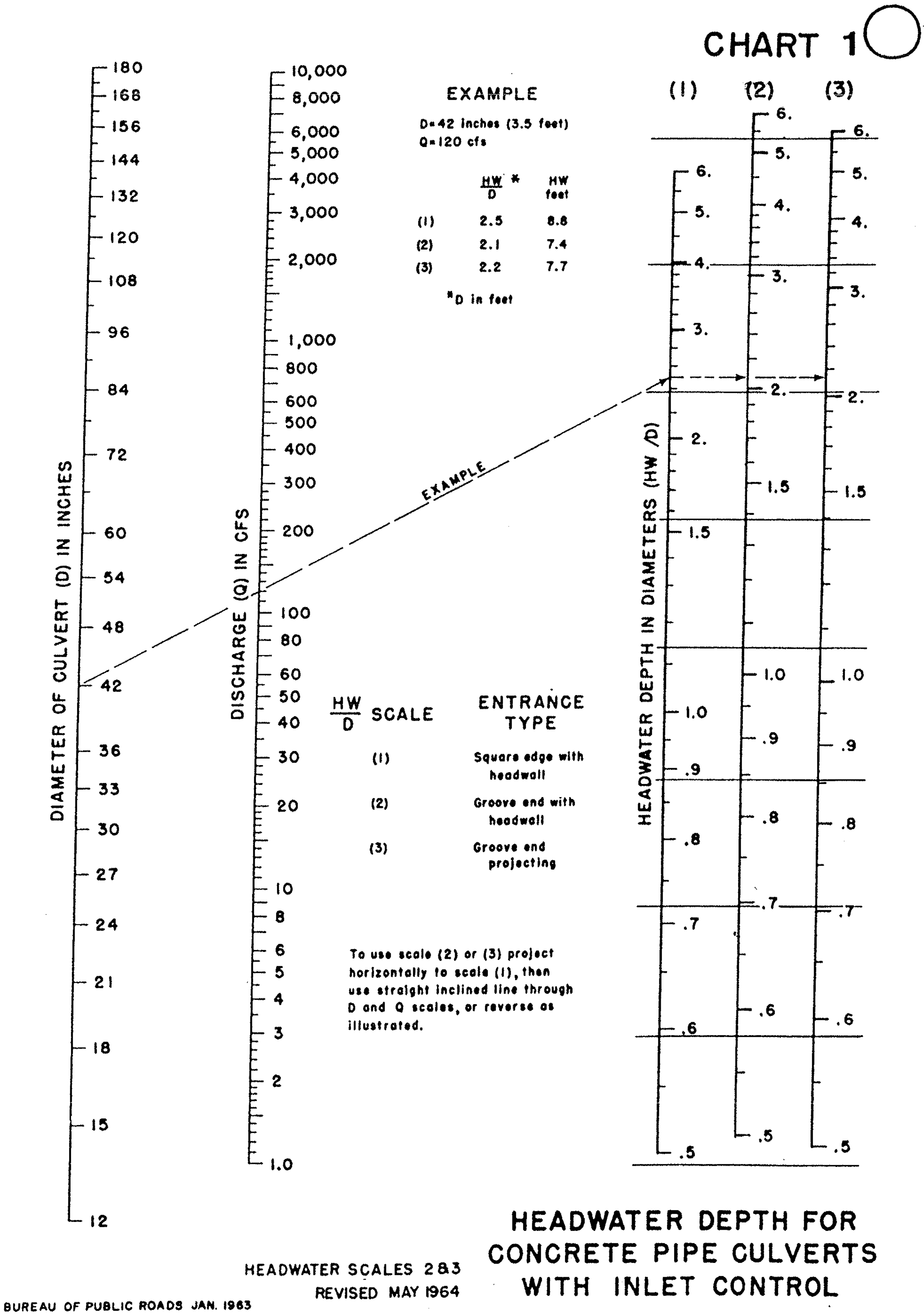


FIGURE 4C-1. Headwater Depth for Concrete Pipe Culverts With Inlet Control

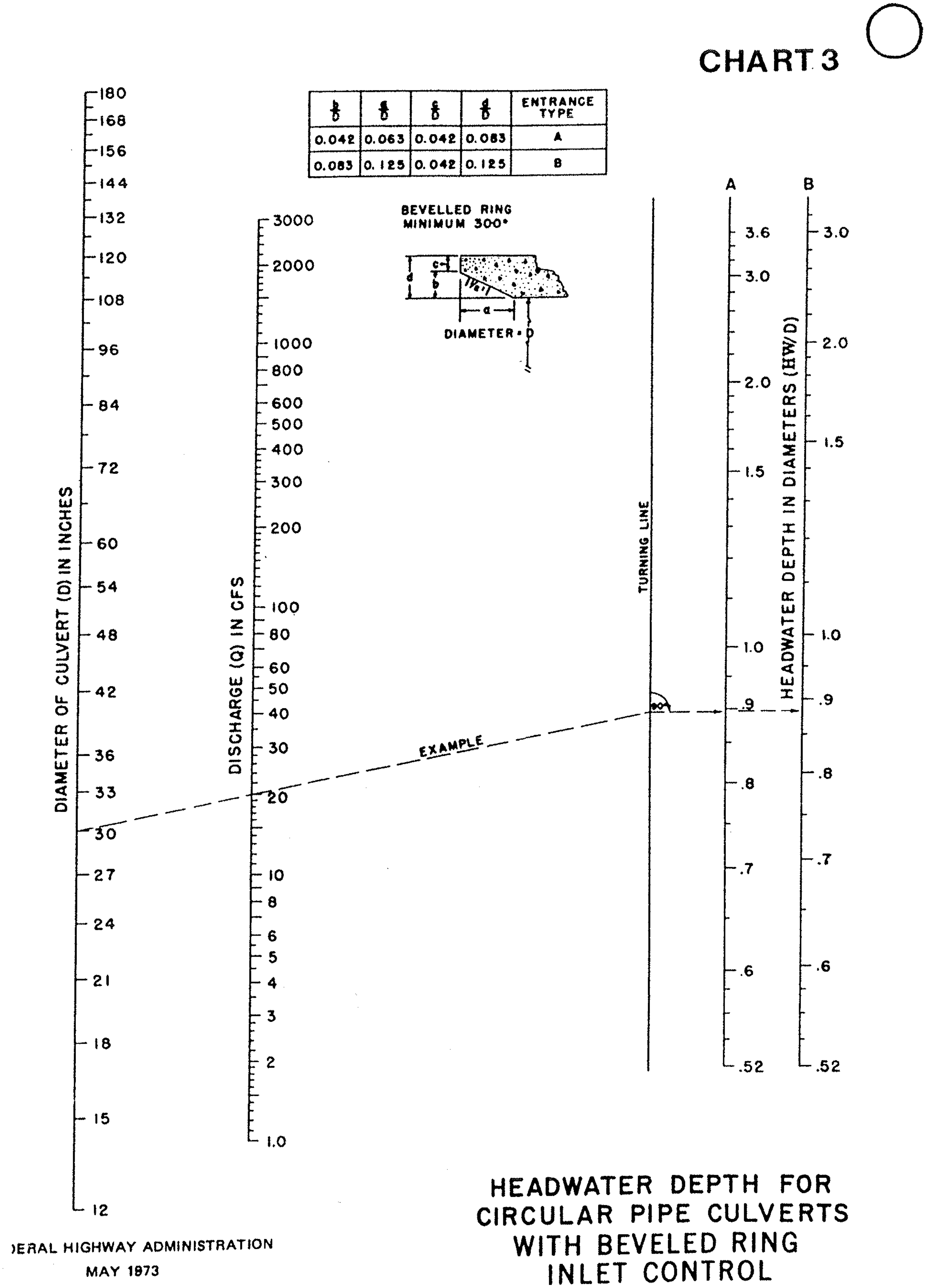


FIGURE 4C-2. Headwater Depth for Circular Pipe Culverts With Beveled Ring Inlet Control

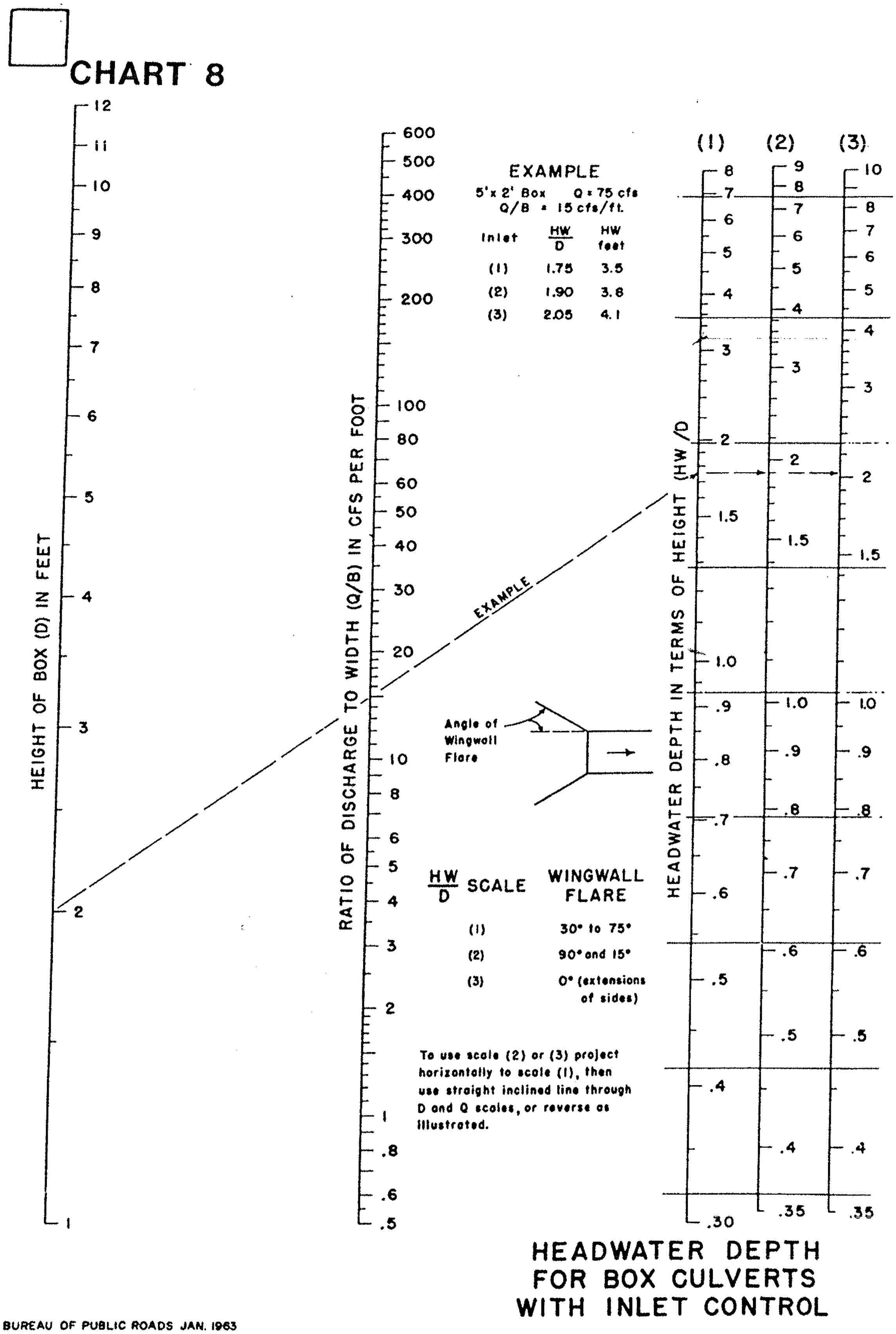


FIGURE 4C-3. Headwater Depth for Box Culverts With Inlet Control

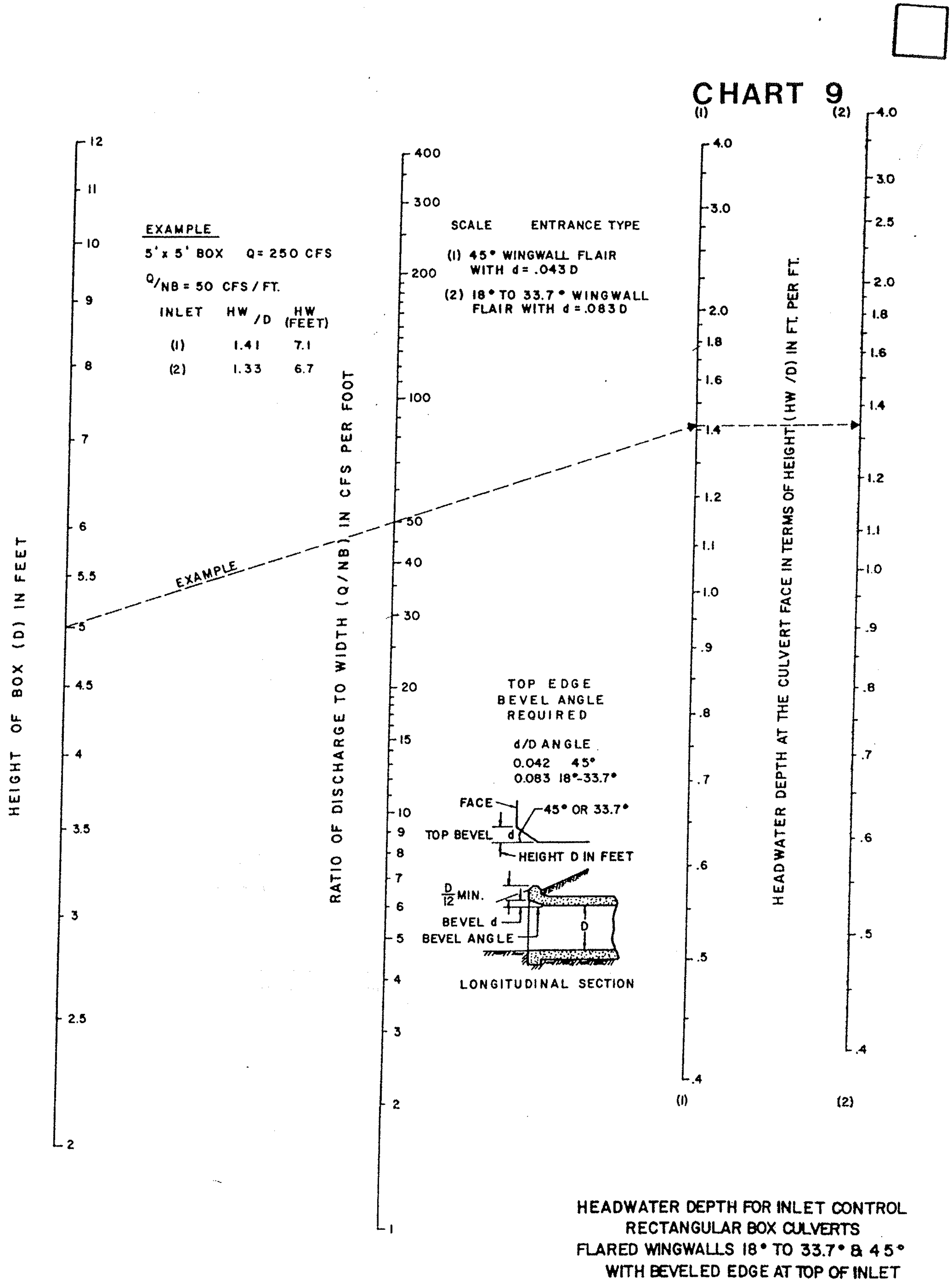


FIGURE 4C-4. Headwater Depth for Inlet Control, Rectangular Box Culverts, Flared Wingwalls 18° to 33.7° & 45° With Beveled Edge at Top of Inlet

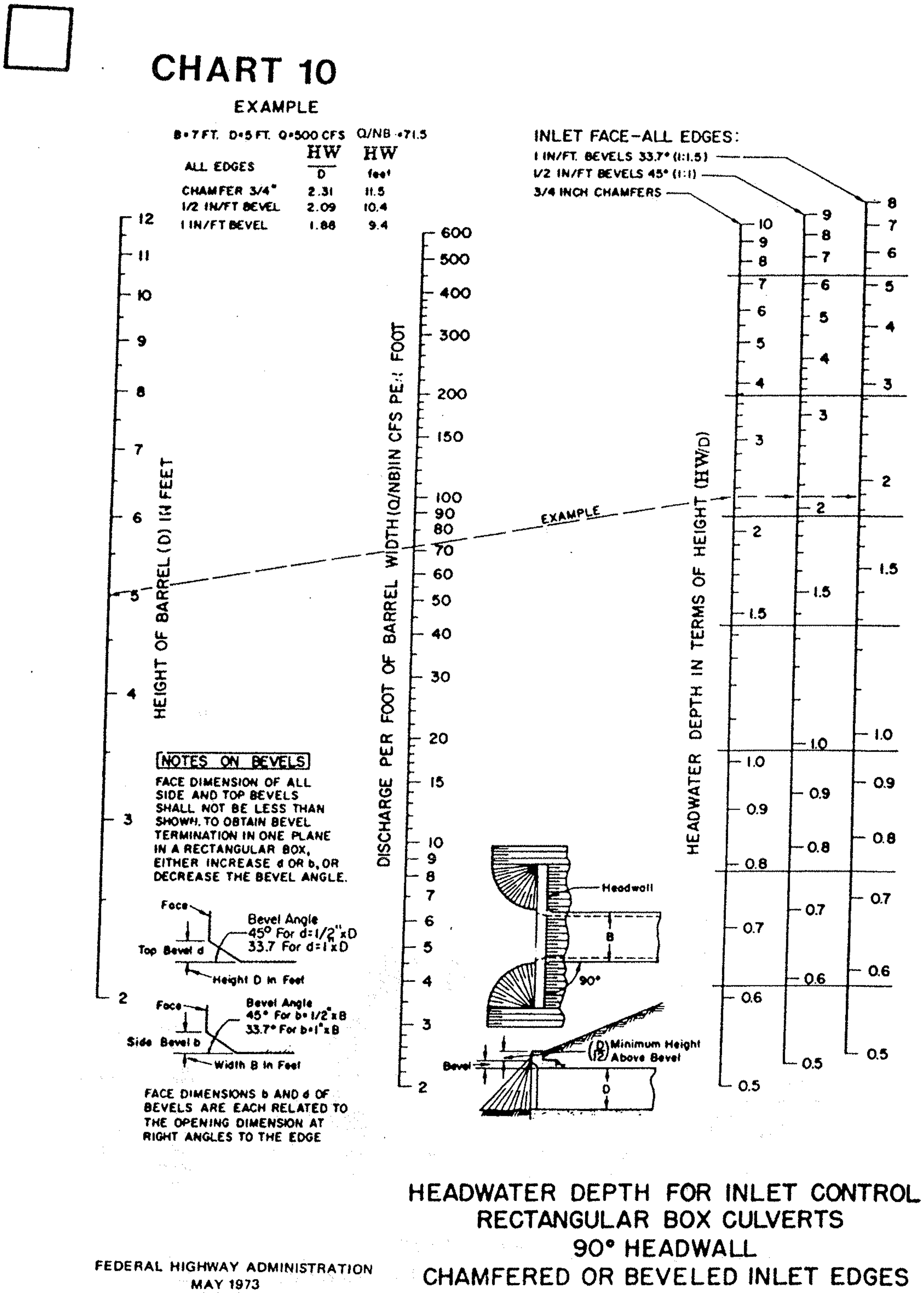


FIGURE 4C-5. Headwater Depth for Inlet Control, Rectangular Box Culverts, 90° Headwall Chamfered or Beveled Inlet Edges

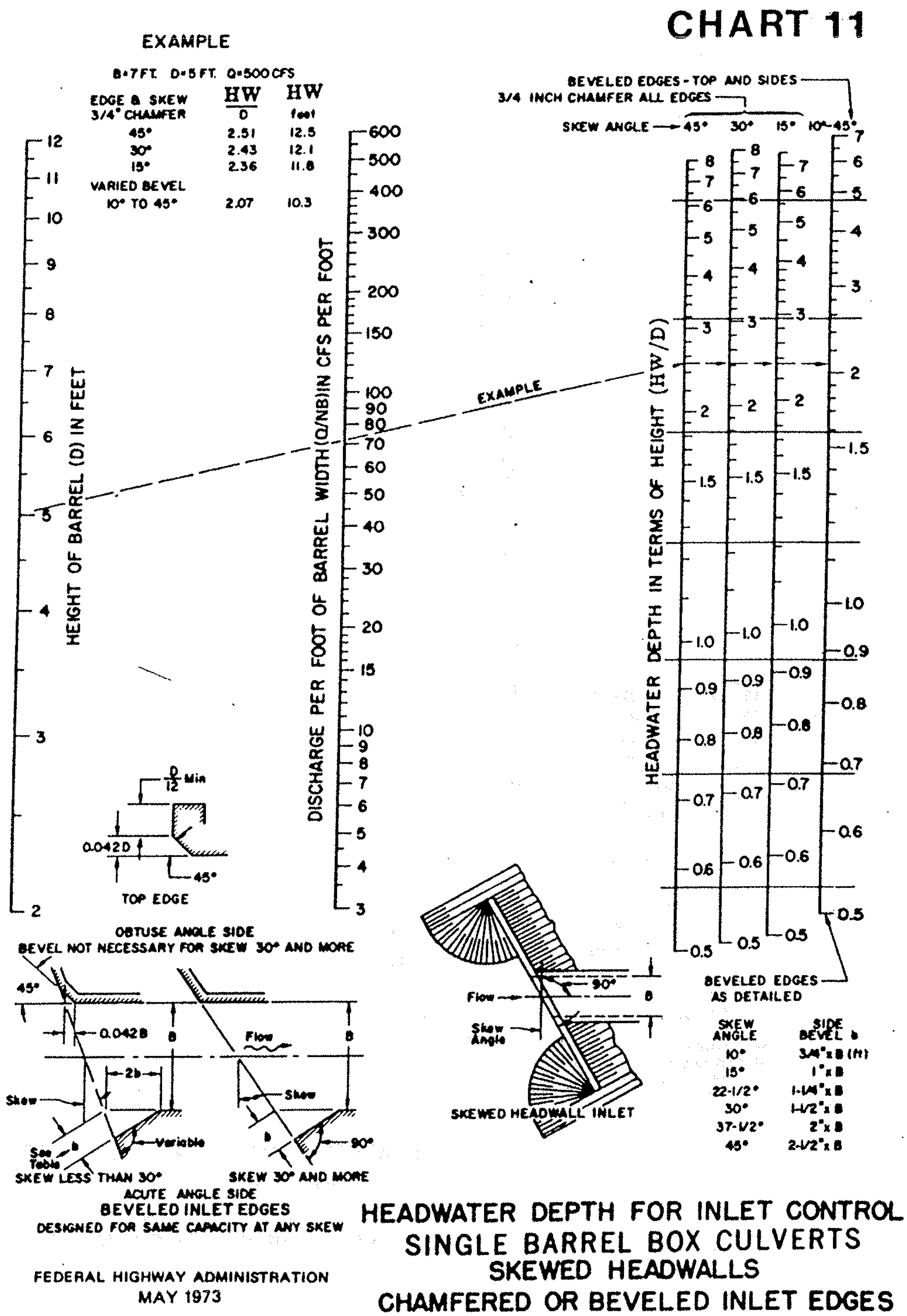


FIGURE 4C-6. Headwater Depth for Inlet Control, Single Barrel Box Culverts, Skewed Headwalls, Chamfered or Beveled Inlet Edges

CHART 12

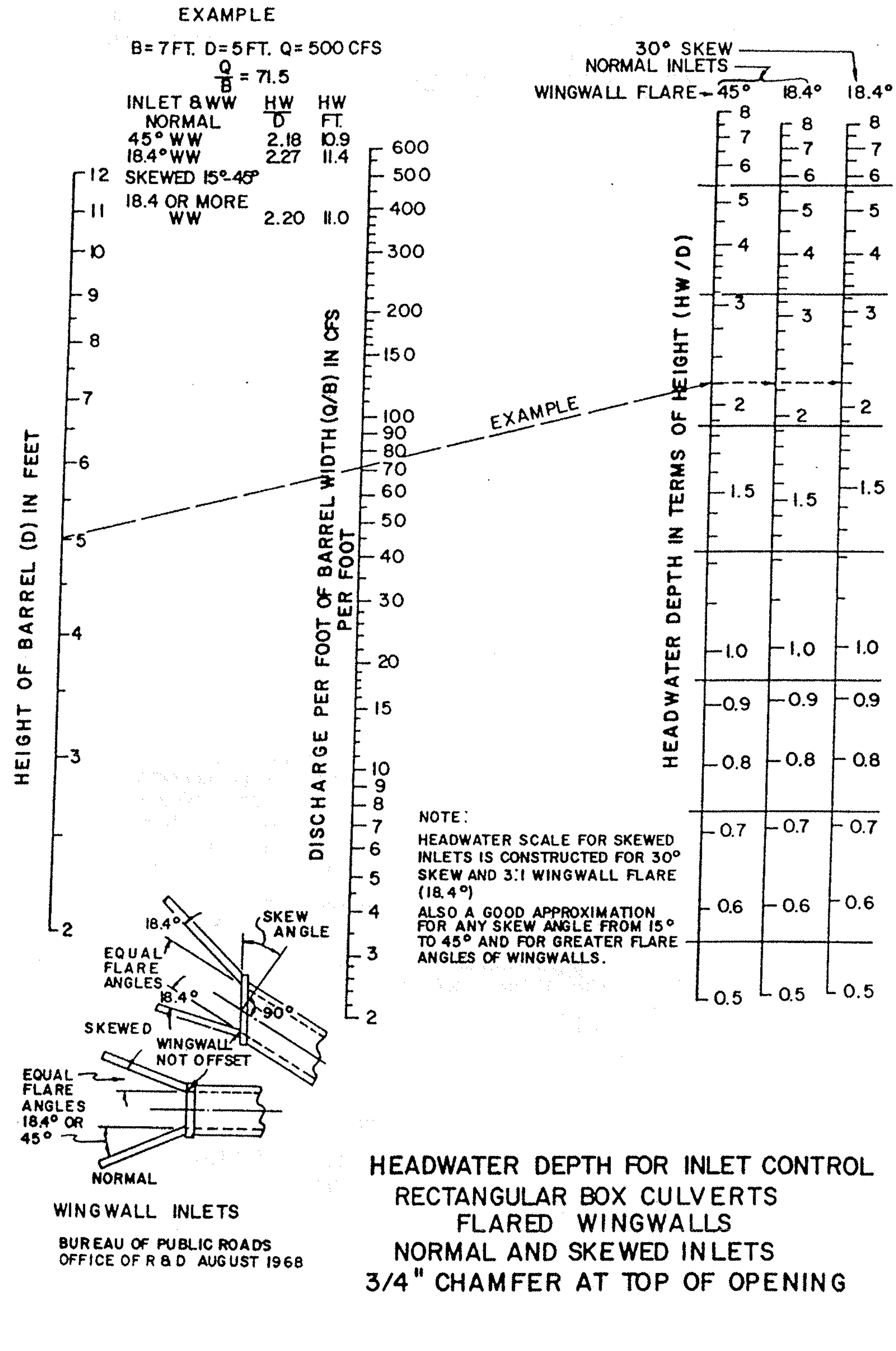


FIGURE 4C-7. Headwater Depth for Inlet Control, Rectangular Box Culverts, Flared Wingwalls, Normal and Skewed Inlets, ¾" Chamfer at Top of Opening

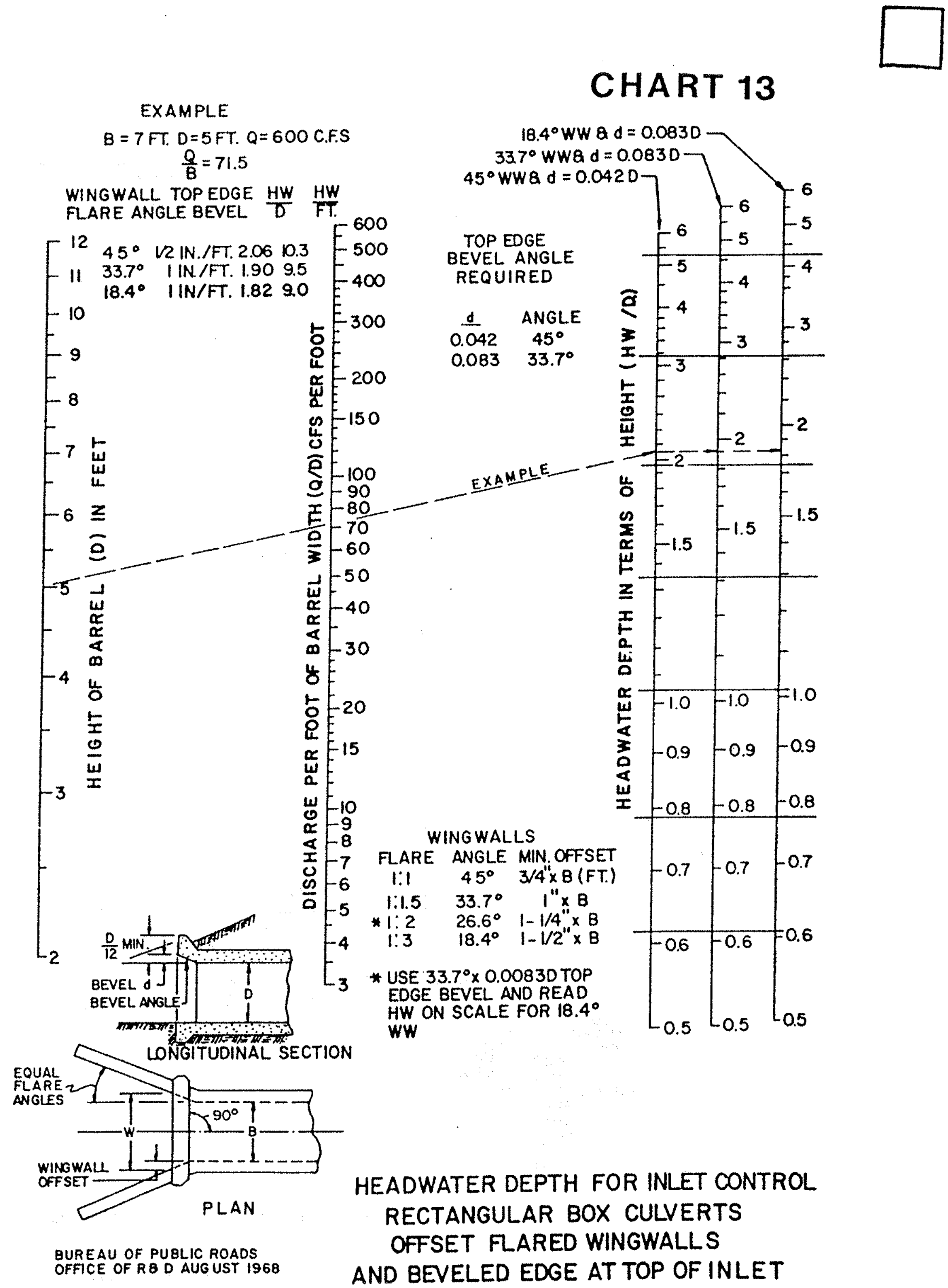


FIGURE 4C-8. Headwater Depth for Inlet Control, Rectangular Box Culverts, Offset Flared Wingwalls and Beveled Edge at Top of Inlet

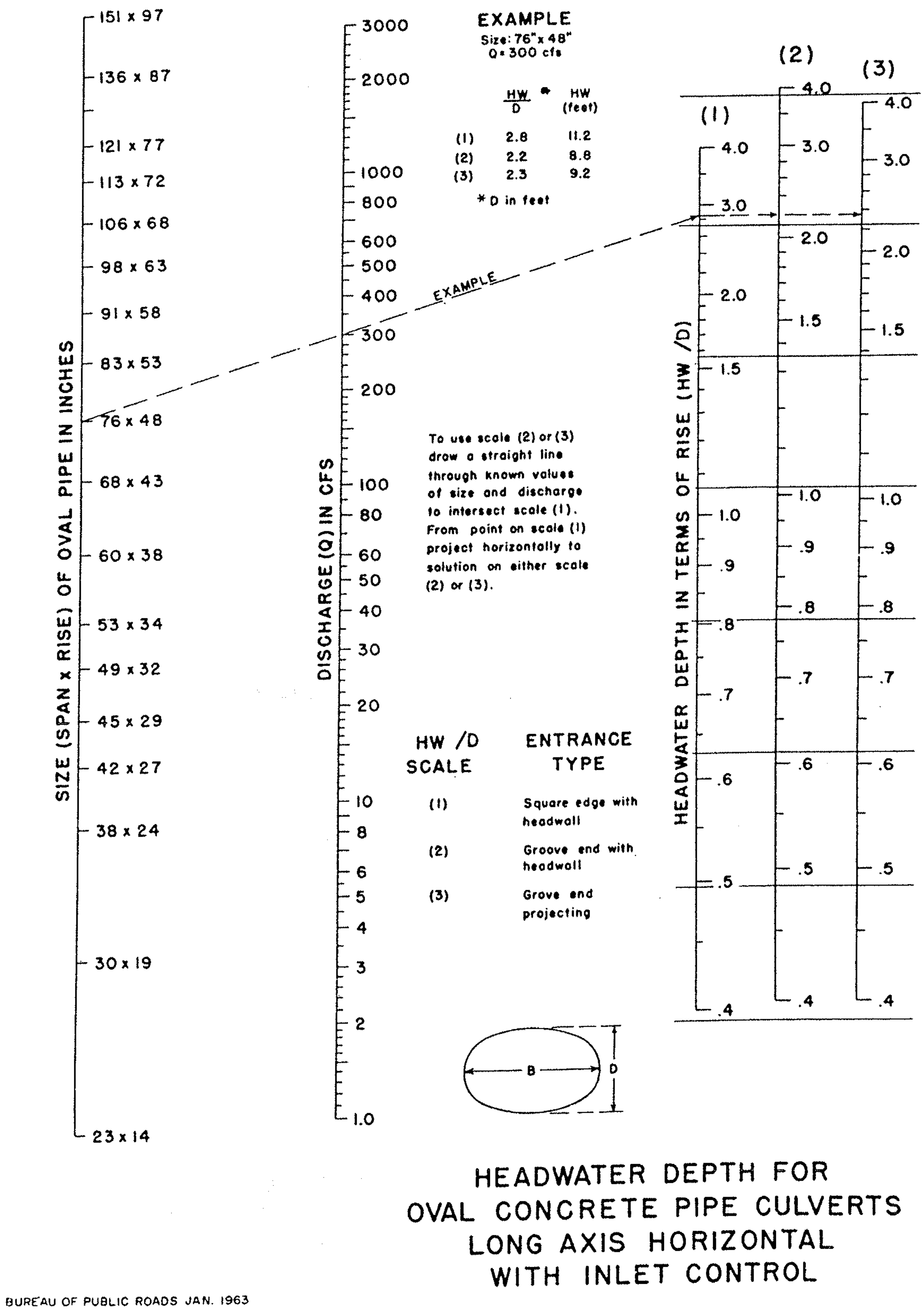
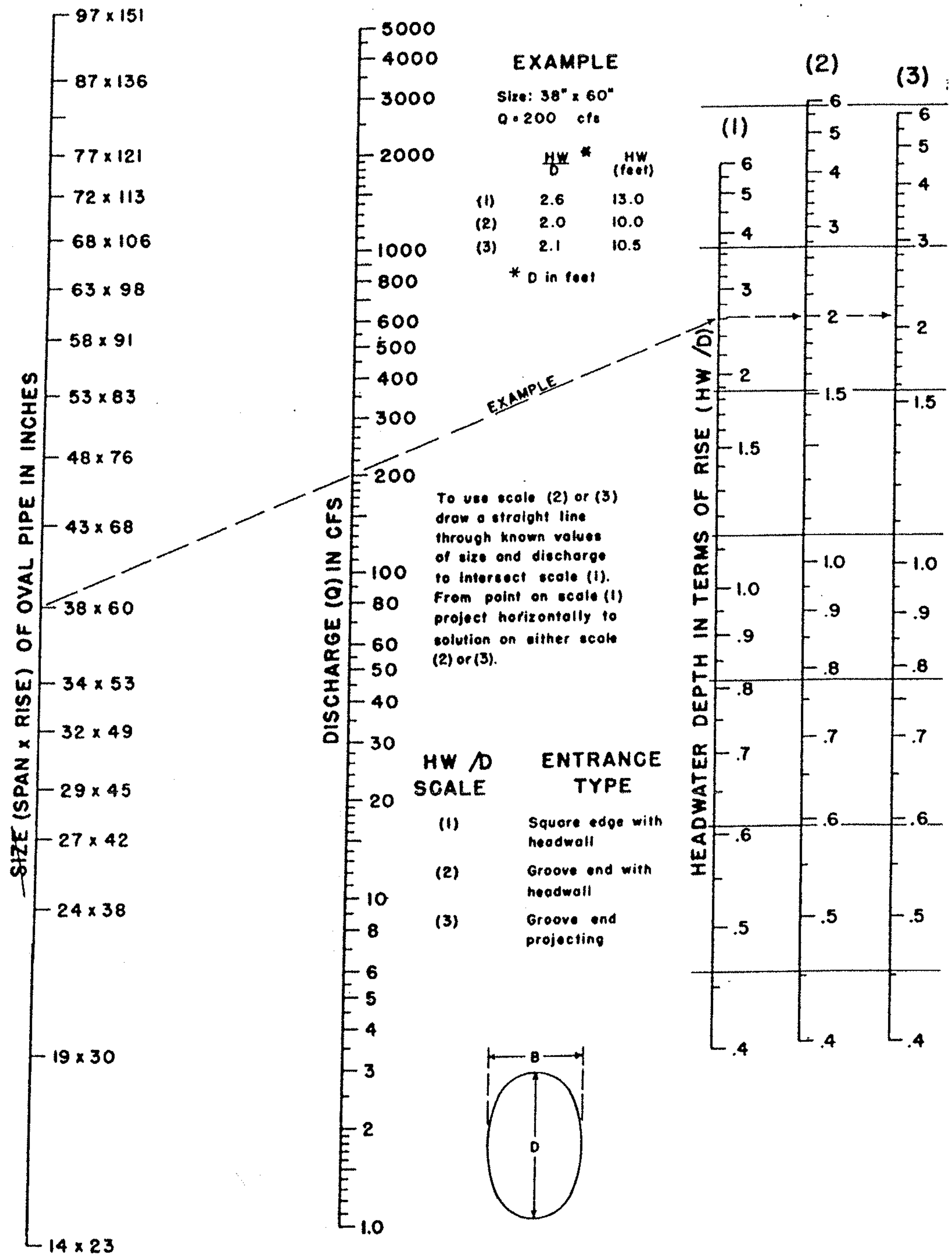


FIGURE 4C-9. Headwater Depth for Oval Concrete Pipe Culverts, Long Axis Horizontal With Inlet Control

CHART 30



HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS VERTICAL
WITH INLET CONTROL

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FIGURE 4C-10. Headwater Depth for Oval Concrete Pipe Culverts, Long Axis Vertical With Inlet Control

APPENDIX 4D

HYDRAULIC ELEMENTS CHART
(Source: AHTD)

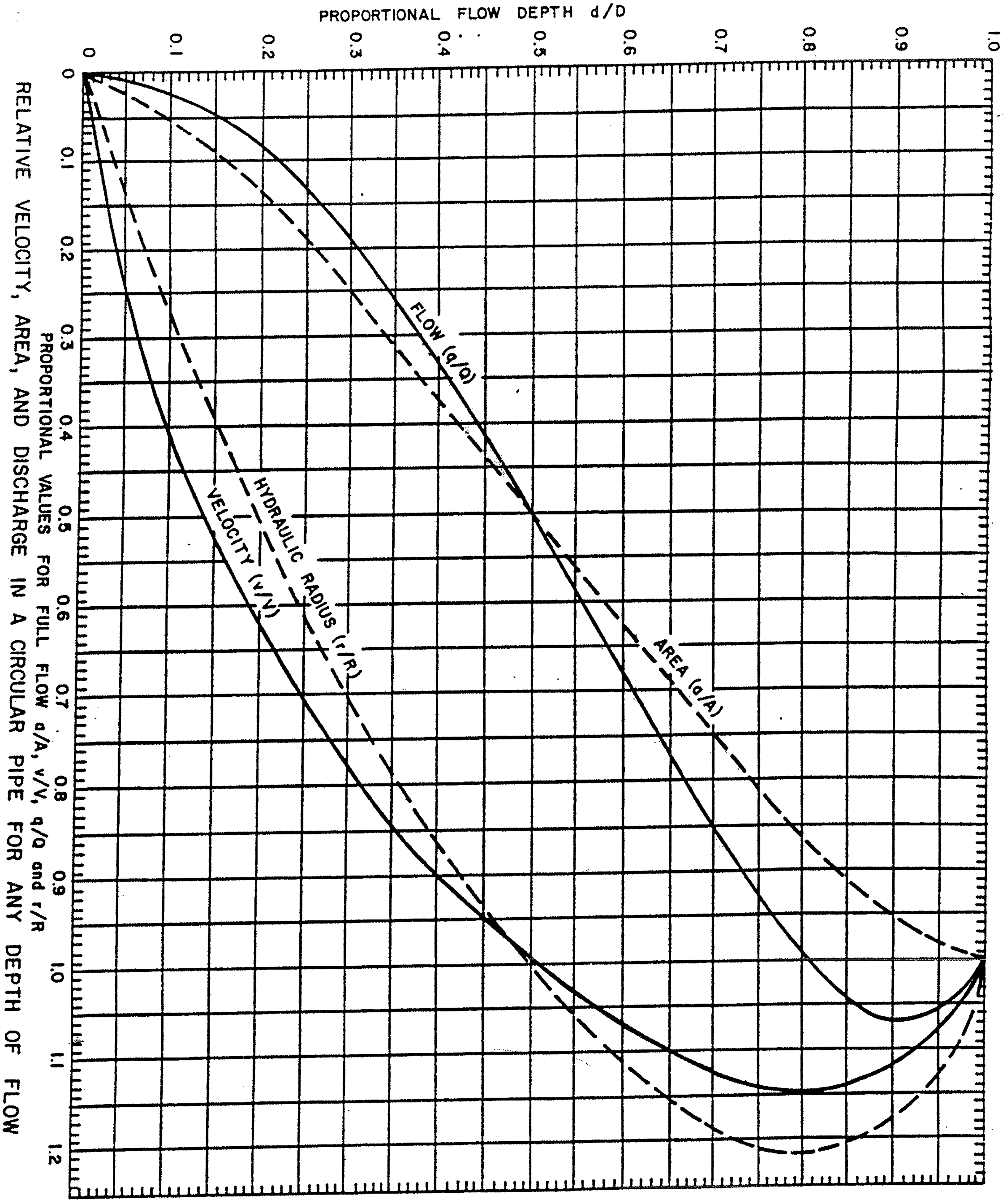


FIGURE 4D-1. Hydraulic Elements Chart.

APPENDIX 4E

EXAMPLE PROBLEM
(Source: AHTD)

FIGUR -1. Example Problem, Culvert Tabulation Sheet

FORM HYD 4-1

PROJECT: *Example No.1*

CULVERT COMPUTATIONS
(SQUARE AND BEVELED EDGES)

DESIGNER: *HYD*

DATE: *11-11-81*

HYDROLOGIC AND CHANNEL INFORMATION

HYDROLOGY

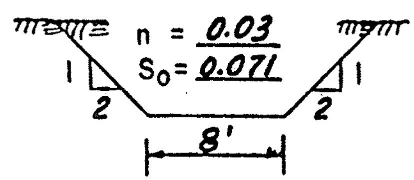
Q_1 *50* = *1000* cfs

Q_2 ____ = ____ cfs

STREAM DATA

Tw_1 = *3.2*

Tw_2 = ____



OUTLET CHANNEL
(APPROX. DIMENSIONS)

SKETCH

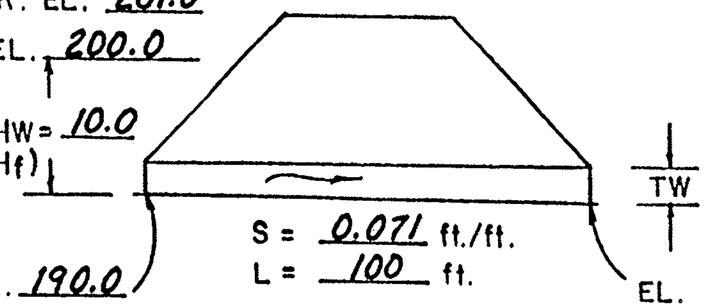
STATION: *30+00*

FIN. GR. EL. *201.0*

HW_f EL. *200.0*

AHW = *10.0*
(H_f)

H_f EL. *190.0*



S = *0.071* ft./ft.

L = *100* ft.

EL. *182.9*

TRIAL NO.	SIZE	STRUCTURE TYPE & ENTRANCE DESIGN	Q/NB $\frac{Q}{NBD^{3/2}}$	HEADWATER COMPUTATION — OUTLET CONTROL (a) K_e	H	(b) d_c	(c) h_o	(d) TW	LS	(e) HW_o	INLET CONT. $\frac{HW}{D}$ (f)	HW	CONTROL-LING HW	OUTLET VELOCITY ft./sec.	COST	COMMENTS
0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1	*8x6*	*SQ. EDGE*	*125*	*0.4*	*11.0*	*>6*	*6*	*3.2*	*7.1*	*9.9*						*CLOSE TO AHW - TRY 7'x6'*
2	*7x6*	*"*	*143*	*0.4*	*15.0*	*>6*	*6*	*3.2*	*7.1*	*13.9*						*EXCEEDS AHW - CHECK TRIAL 1 FOR BEVELED EDGE*
3	*8x6*	*BEVELED*	*125 / 8.5*	*0.2*	*9.5*	*>6*	*6*	*3.2*	*7.1*	*8.4*	*3.1*	*18.6*	*18.6*	*21*		*LOWERED HW_o 1.5'. H_f EXCEEDS AHW - TRY SIDE-TAPERE*

(a) Entrance loss coefficient, Refer to Table 4-10, page 4-60.
(b) "d_c" cannot exceed D.
(c) h_o = $\frac{D}{2}$ or TW, whichever is larger.
(d) TW = d_n in natural channel, or other downstream control.
(e) HW_o = H + H_o − LS
(f) Use Chart 4-7, page 4-64, for Conventional
Use Chart 4-8, page 4-65 for Beveled E

CHAPTER 5 – POST CONSTRUCTION STORMWATER MANAGEMENT

5.1 GENERAL

Land development projects and associated increases in impervious cover alter the hydrologic response of local watersheds and increase storm water runoff rates and volumes, flooding, stream channel erosion, and sediment transport and deposition; This storm water runoff contributes to increased quantities of water-borne pollutants, and; Storm water runoff, soil erosion, and nonpoint source pollution can be controlled and minimized through the regulation of storm water runoff from development sites. For these reasons, the Arkansas Department of Environmental Quality (ADEQ), under regulations administered by the United States Environmental Protection Agency (EPA) requires the City of Fort Smith to meet certain requirements as established in the National Pollutant Discharge Elimination System (NPDES), Phase II, for Small Municipal Separate Storm Sewer Systems (MS4's).

5.1.1 *Detention Required*

If hydrologic and hydraulic studies reveal that the post-development runoff for a proposed development or redevelopment project one acre or more in size will exceed the pre-development runoff, and the existing drainage system is not adequate to carry the post-development runoff, then the proposed development or redevelopment project shall not be permitted unless one or more of the following mitigation measures are used: onsite detention, offsite or regional detention, or improvements to the existing drainage system.

All detention facilities shall be designed to limit the peak storm water discharge rate of the 10-, 25-, 50-, and 100-year storm frequencies after development to pre-development flow rates.

5.1.1.1 Acceptable Detention Practices

Only stormwater ponds and wetlands shall be allowed for publicly owned detention, i.e. within residential subdivisions and developments (see Section 5.9). Other methods of detention such as infiltration trenches, infiltration basin, etc., will not be allowed for publicly owned detention and are discouraged for privately owned detention. If other methods are proposed, proper documentation of soil data, percolation, geological features, etc., will be needed for review and consideration.

5.1.1.2 Parking Lot Detention

Privately owned detention is permitted in parking lots to maximum depths of 6 inches. In no case shall the maximum limits of ponding be designed closer than 10 feet from a structure unless waterproofing of the structure and pedestrian accessibility are properly documented and approved. The minimum freeboard and the maximum ponding elevation to the lowest sill or floor elevation shall be 2 feet.

5.1.2 *Stormwater Treatment Required*

Development and redevelopment projects one or more acres in size (or less than an acre if part of a larger common plan of development), that will increase the impervious area onsite, shall not be permitted without employing Stormwater Treatment Practices (STP's) to address the water quality of the surface waters being discharged from the site. All STP's or systems of STP's utilized to address water quality shall be required to capture and treat the Water Quality Volume (WQ_v). The WQ_v shall be equal to:

$$WQ_v = (P_1)(R_v)(A)/12 \tag{5.1}$$

Where:
WQ_v = Water Quality Volume (acre-ft)
P_1 = The First One Inch (1.0") of Direct Runoff
R_v = Runoff Coefficient
A = Site Area (acres)

$$R_v = 0.05+0.009I \tag{5.2}$$

Where:
I = Site Impervious Cover (%)

The WQ_v shall be based on the impervious cover of the proposed site. Offsite existing impervious areas may be excluded from the calculation of the water quality volume requirements.

5.1.2.1 Acceptable Stormwater Treatment Practices (STPs)

All acceptable STP's shall be designed to capture and treat the WQ_v with a goal of at least 80% removal of total suspended solids (TSS) from post-construction discharges (See Table 5.2 for Pollutant Removal Percentages). STP's that meet these requirements can be divided into five basic groups – Stormwater Ponds, Wetlands, Infiltration Systems, Filtering Systems, and Open Channel Systems. When properly designed, the following STP's shall be considered sufficient to meet the requirements above:

Group 1: Stormwater Ponds

Stormwater ponds are practices that have a combination of a permanent pool, extended detention or shallow marsh equivalent to the entire WQ_v. Design variants include:

- Micropool Extended Detention Pond
- Wet Pond
- Wet Extended Detention Pond
- Multiple Pond System
- "Pocket" Pond

Group 2: Wetlands

Stormwater wetlands are practices that create shallow marsh areas to treat urban stormwater and often incorporate small permanent pools and/or extended detention storage to achieve the full WQ_v. Design variants include:

- Shallow Wetland
- ED Shallow Wetland
- Pond/Wetland System
- "Pocket" Wetland

Group 3: Infiltration Systems

Stormwater infiltration practices capture and temporarily store the WQ_v before allowing it to infiltrate into the soil. Design variants include:

- Infiltration Trench
- Infiltration Basin

Group 4: Filtering Systems

Stormwater filtering system capture and temporarily store the WQ_v and pass it through a filter bed of sand, organic matter, soil or other media. Filtered runoff may be collected and returned to the conveyance system, or allowed to partially exfiltrate into the soil. Design variants include:

- Surface Sand Filter
- Underground Sand Filter
- Perimeter Sand Filter
- Organic Filter
- Bioretention

Group 5: Open Channel Systems

Open channel systems are vegetated open channels that are explicitly designed to capture and treat the full WQ_v within dry or wet cells formed by checkdams or other means. Design variants include:

- Dry Swale
- Wet Swale
- Grass Channels

5.1.2.2 Sub-Standard Storm Water Treatment Practices

Many current and future stormwater management structures may not meet the performance criteria specified in Section 5.1.2.1 above to qualify to be used as "stand-alone" practices for full

WQ_v treatment. Reasons for this include poor longevity, poor performance, inability to decrease TSS by 80%, or inadequate testing. Some of these practices include:

- Dry Extended Detention Ponds
- Catch Basin Inserts
- Water Quality Inlets and Oil/Grit Separators
- Hydro-Dynamic Structures
- Filter Strips
- Deep Sump Catch Basins
- Dry Wells
- On-Line Storage in the Storm Drain Network

In some cases, these practices are appropriately used for pretreatment, as part of an overall STP system, or may be applied in redevelopment situations on a case-by-case basis where other practices are not feasible. New structural BMP designs are continually being developed, including many proprietary designs. All current and future structural practice design variants should fit in one of the five STP groups referenced above if the intent is to use them independently to treat the full WQ_v. Current or new STP design variants cannot be accepted for inclusion on the list until independent pollutant removal performance and monitoring data determine that they can meet the 80% TSS removal target and that the new STPs conform with local and/or State criteria for treatment, maintenance, and environmental impact.

5.1.2.3 Stormwater Hot Spots

Stormwater hot spots are areas where land use or activities generate highly contaminated runoff, with concentrations of pollutants in excess of those typically found in stormwater. A greater level of stormwater treatment is needed at hot spot sites to prevent pollutant washoff after construction. This typically involves preparing and implementing a ***stormwater pollution prevention plan*** (SWPPP) that involves a series of operational practices at the site that reduces the generation of pollutants by preventing contact with rainfall.

For the purposes of this document, stormwater hot spots shall be classified as industrial facilities that:

- have Standard Industrial Classification (SIC) codes listed in "40 CFR 122.26(b)(14) Subpart (i) – (xi)"
- and, are required to submit applications for a storm water permit to the Arkansas Department of Environmental Quality (ADEQ).

A copy of "40 CFR 122.26(b)(14) Subpart (i) – (xi)" can be found in Appendix 5A.

5.1.3 *Variances*

Criteria for differential runoff and detention guidelines are set out in the following in an attempt to decrease the possible effects of development on downstream properties due to increased runoff and pollutants. Variances to the requirements in this chapter may be granted by the Engineering Department if it is determined that detention would be ineffective to prevent flooding or would aggravate the flooding conditions. Variances to the detention requirements do not relieve the developer/owner of any water quality requirements. However, reductions in the required WQ_v are possible with the use of storm water credits (See Section 5.9).

5.1.4 *Verification of Adequacy*

Projects shall provide documented verification of adequacy according to the scope and complexity of design. Documentation must have original signature and be certified as-built by the same Arkansas Registered Professional Engineer, if feasible.

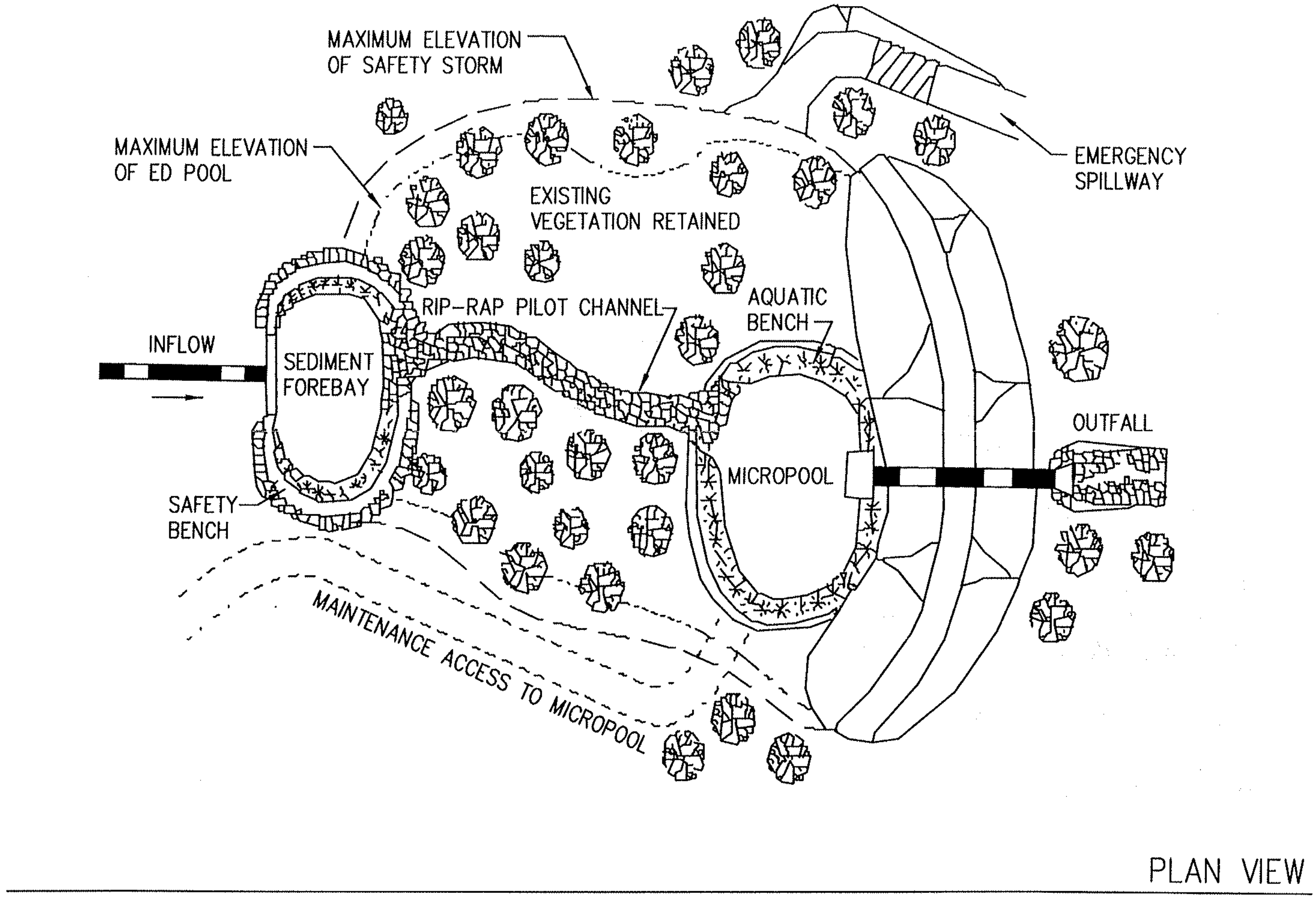
5.2 DESIGN CRITERIA – STORMWATER PONDS

Stormwater ponds are practices that have a combination of a permanent pool, extended detention or shallow marsh equivalent to the entire WQ_v. Design variants include:

- Micropool Extended Detention Pond (Figure 5-1)
- Wet Pond (Figure 5-2)
- Wet Extended Detention Pond (Figure 5-3)
- Multiple Pond System (Figure 5-4)
- "Pocket" Pond (Figure 5-5)

The term "pocket" refers to a pond or wetland that has such a small contributing drainage area that little or no baseflow is available to sustain water elevations during dry weather. Instead, water elevations are heavily influenced and, in some cases, maintained by a locally high water table.

Stormwater ponds may be used in residential, private, commercial, and industrial subdivisions and developments to meet the detention and WQ_v requirements.



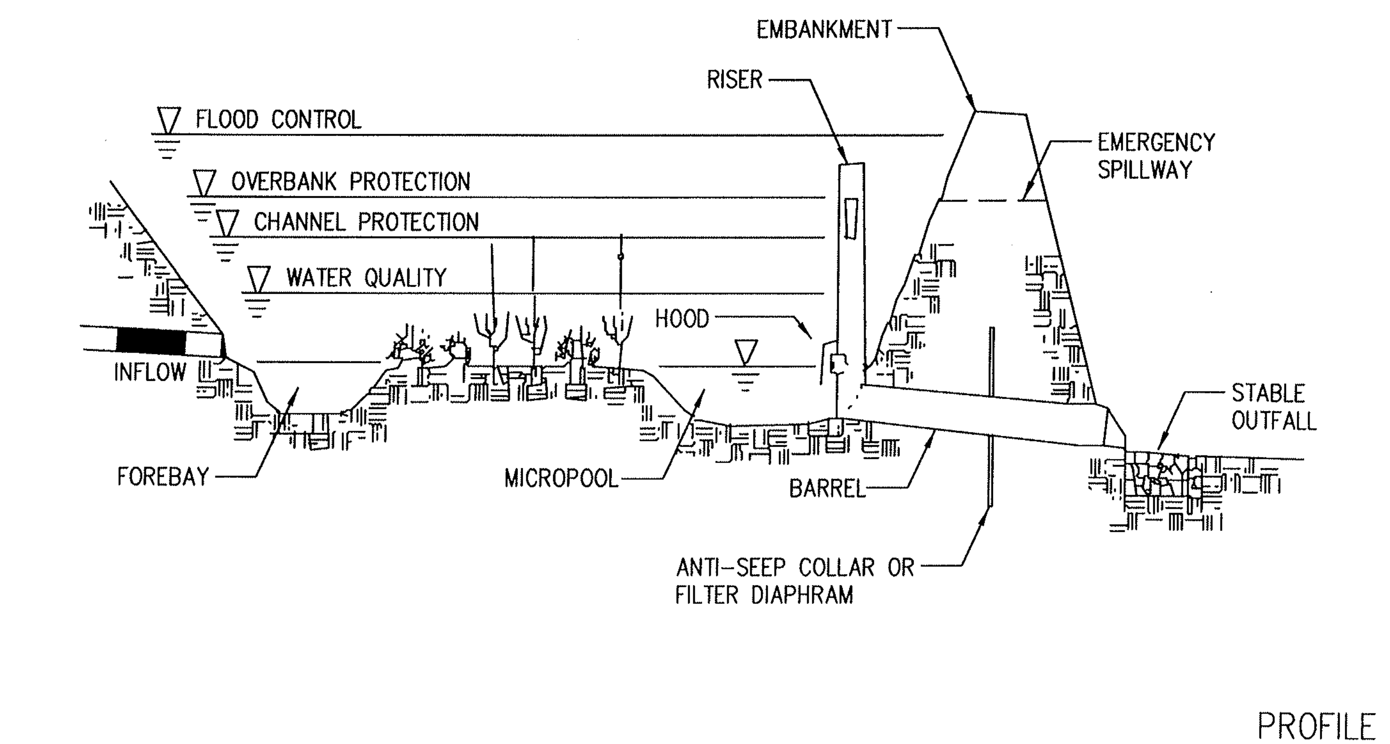


FIGURE 5-1. Micropool Extended Detention Pond

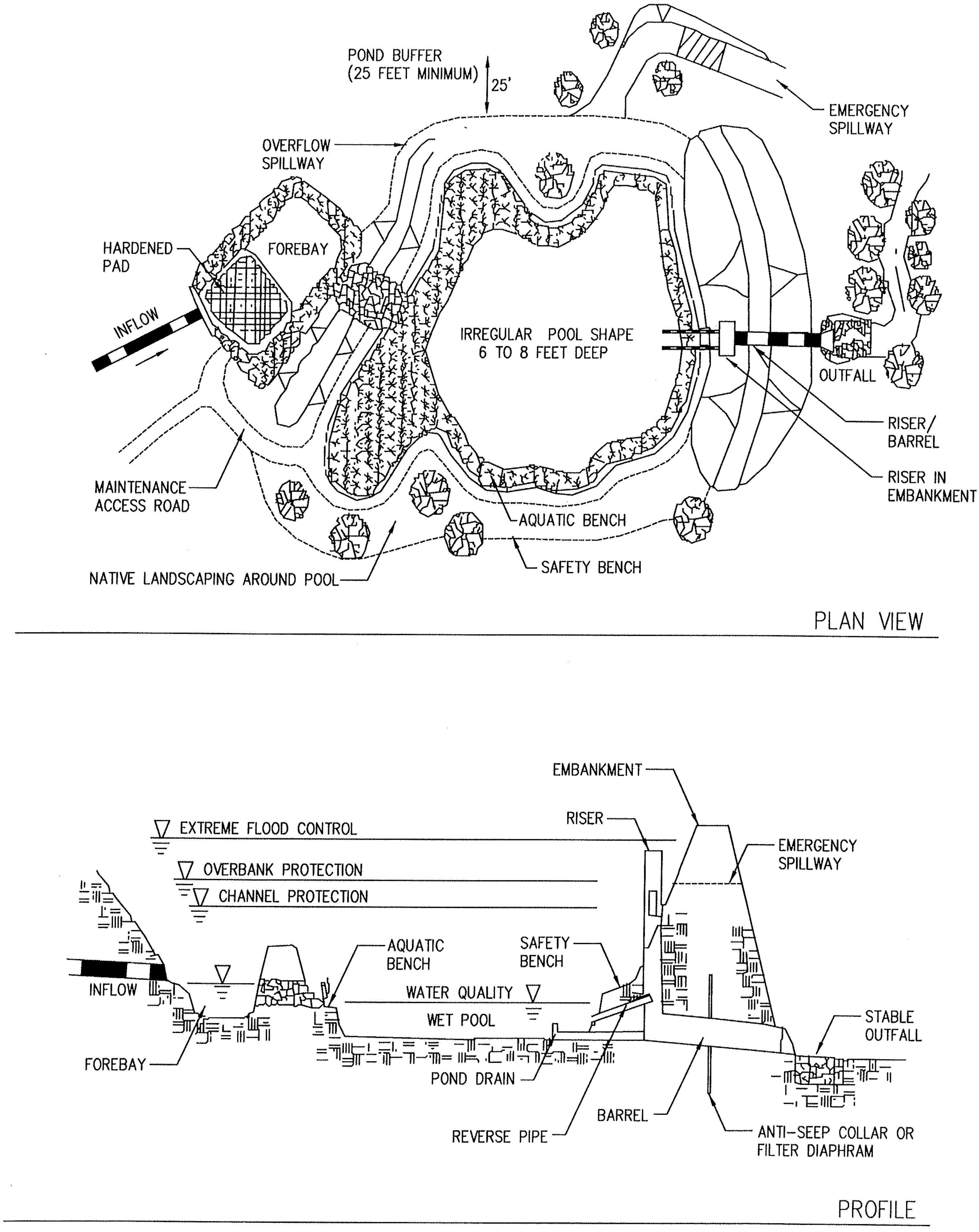
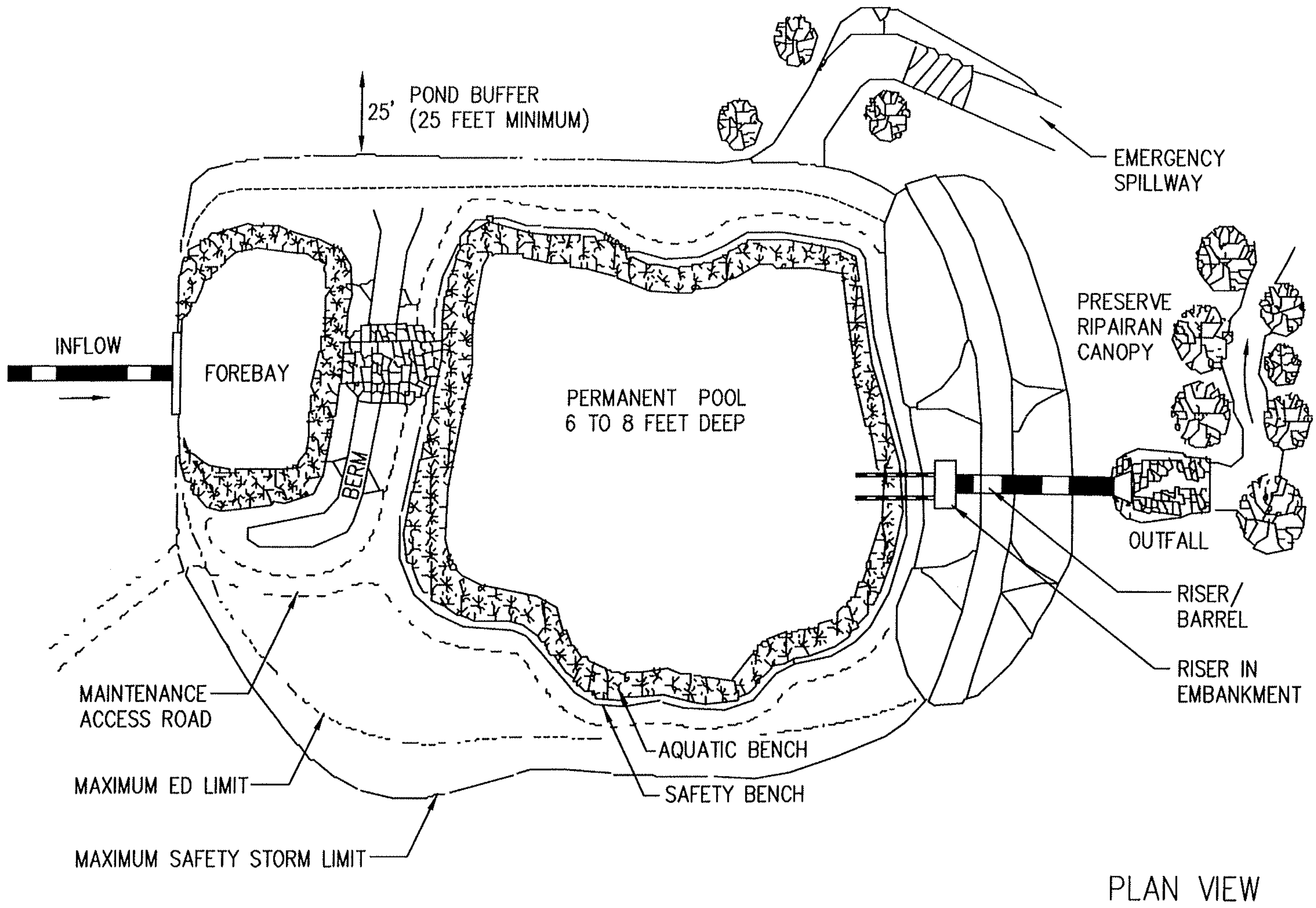


FIGURE 5-1. Micropool Extended Detention Pond



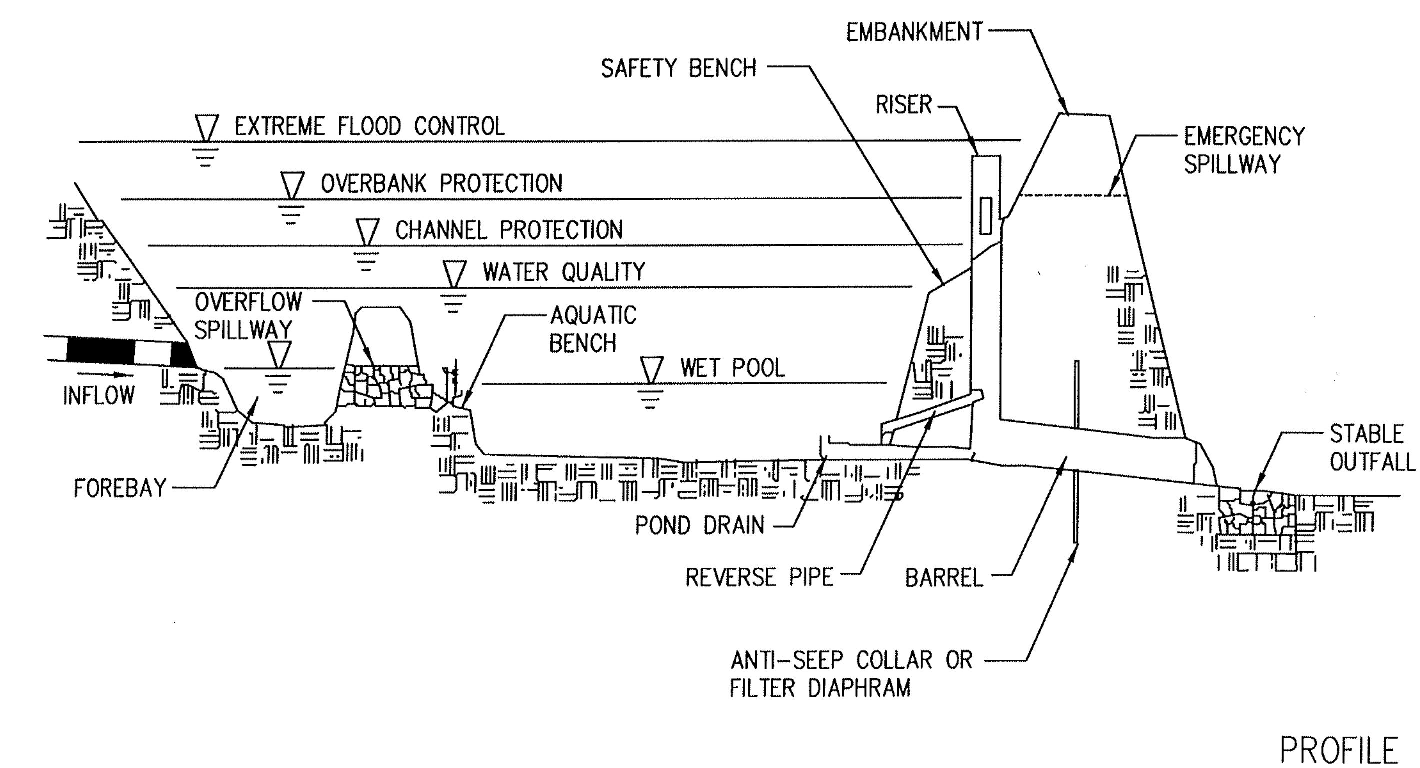


FIGURE 5-3. Wet Extended Detention Pond

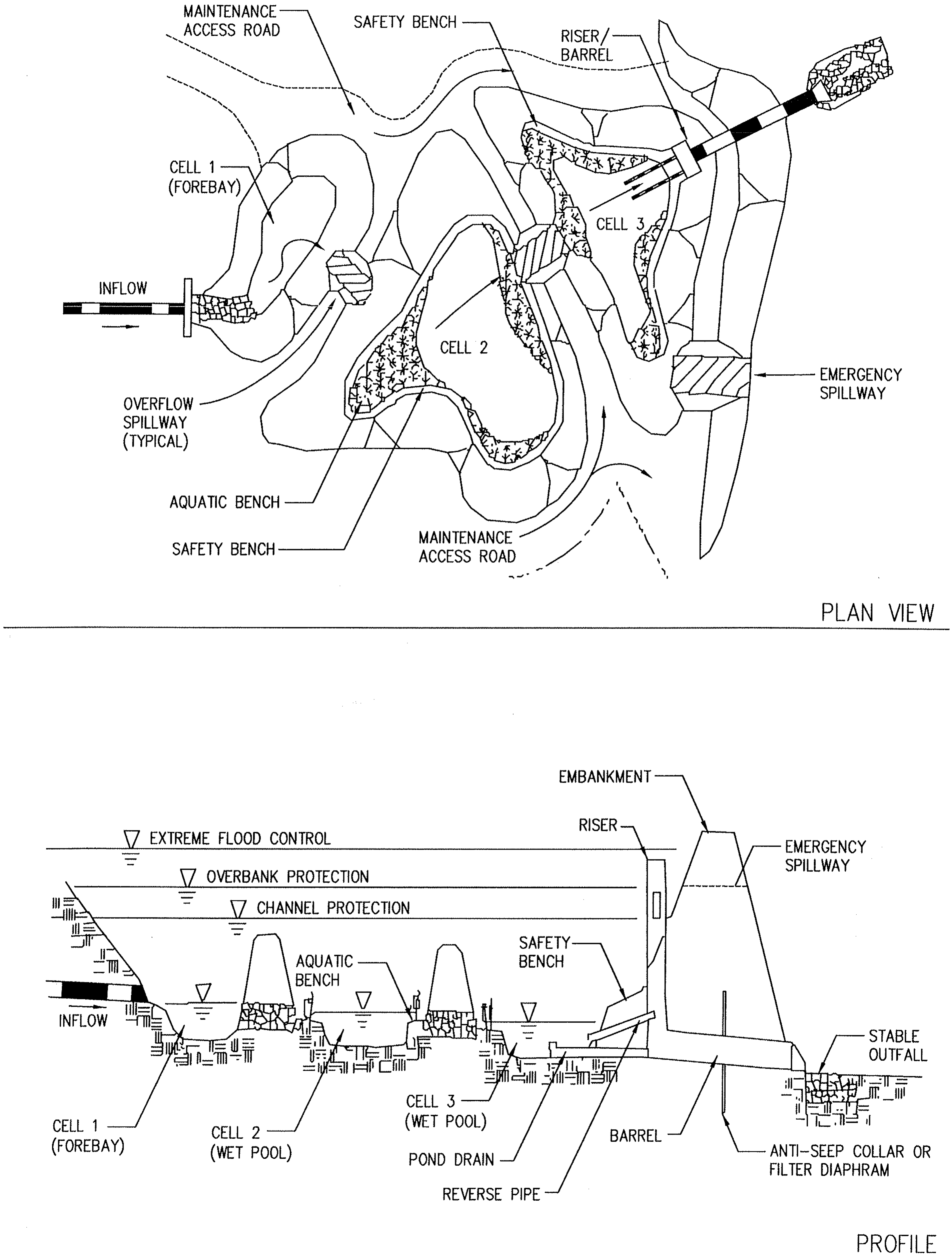
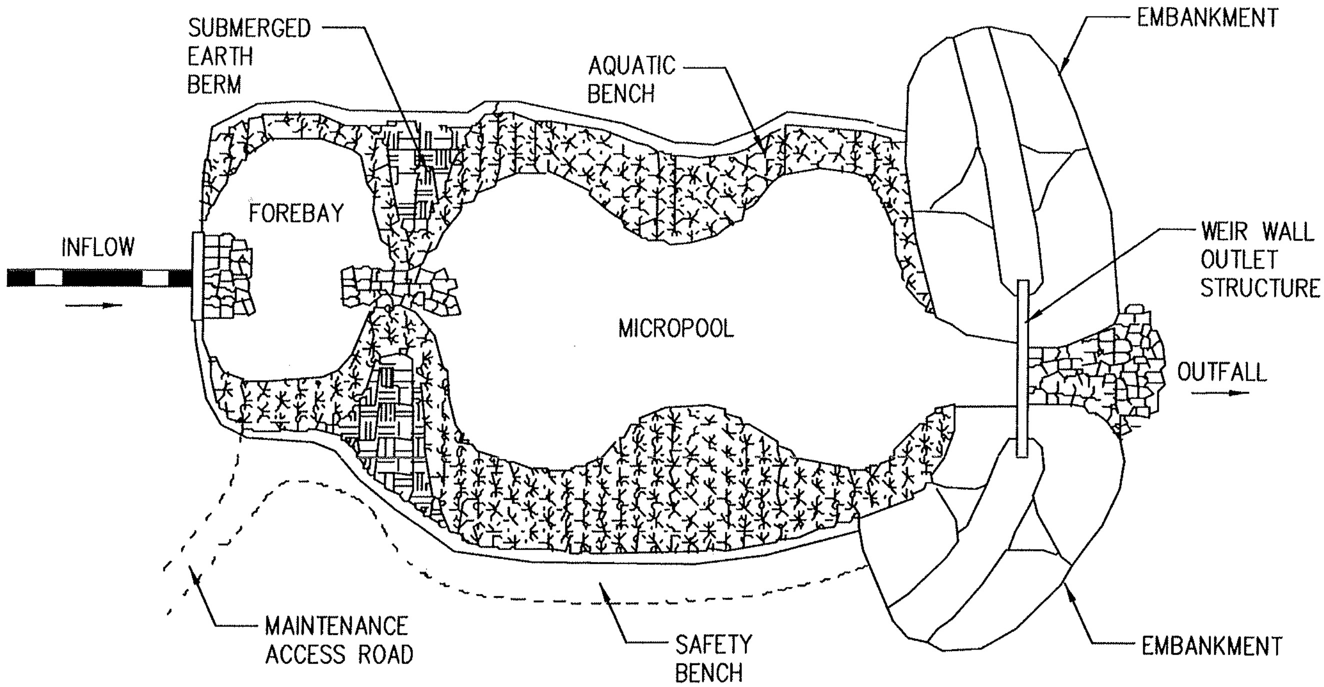
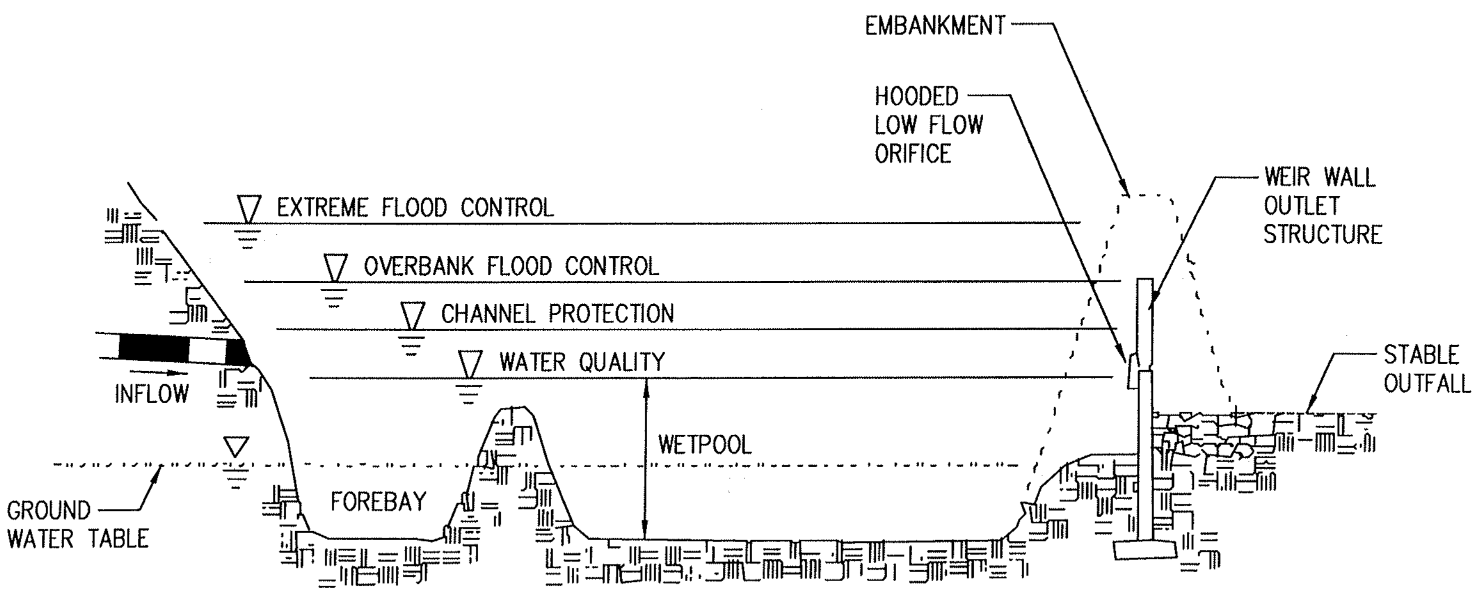


FIGURE 5-4. Multiple Pond System



PLAN VIEW



PROFILE

FIGURE 5-5. Pocket Pond

Dry extended detention ponds that have no permanent pool are not considered an acceptable "stand-alone" option for meeting WQ_v. However, with the approval of the Engineering Department, they may be used in conjunction with other STP's to meet the WQ_v requirement. They may also be used to meet the detention requirement.

5.2.1 *Feasibility Criteria*

When used to meet water quality requirements, a minimum contributing drainage area of ten acres or more is preferred for stormwater ponds, unless groundwater can be confirmed as the primary water source (i.e., pocket ponds).

Stormwater ponds shall not be located within jurisdictional waters, including wetlands.

Stormwater ponds shall be located within the parcel limits of the project under consideration, except as specified below. No stormwater ponds will be permitted within public road rights-of-way. Location of stormwater ponds immediately upstream or downstream of the project will be considered by special request if proper documentation is submitted with reference to practicality, feasibility, and proof of ownership or right-of-way use of the area proposed.

In no case shall the limits of maximum ponding for a stormwater pond be closer than 25 feet horizontally from any structure.

5.2.1.1 Safe Dam Act

National responsibility for the promotion and coordination of dam safety lies with FEMA. State responsibility for administration of the provisions of the Federal Dam Safety Act is given by Title 15, Chapter 22 of the Arkansas State Code. Rules and regulations relating to applicable dams are promulgated by the Arkansas Soil and Water Conservation Commission (ASWCC).

All dams within the state of Arkansas, except those that meet certain exemptions, must have a construction and operation permit from ASWCC. Under the ASWCC criteria, a dam is exempt from the regulations if it is less than 25 ft in height or has a normal storage volume less than 50 ac•ft. The ASWCC also allows an exemption if the crest height of the dam is below the ordinary high water mark of the stream at that location. However, smaller dams may also be required to meet the dam safety regulations as well. If persons downstream feel that their life or their property is endangered by a dam, they can petition the ASWCC for the dam safety regulations to be enforced *(2)*. Consult Reference *(2)* for more information on dam safety regulations, design criteria, and hazard classifications. Any questions regarding permits, exemptions, design criteria, or compliance with dam safety regulations should be directed to the ASWCC.

Dams which are greater than 10 feet in height but do not fall into State or Federal requirement categories shall be designed in accordance with the latest edition of the SCS Technical Release No. 60, "Earth Dams and Reservoirs," as Class "C" structures *(1), (8)*.

An analysis shall be furnished of any soil proposed for use in earthen dam construction. Borings of the foundation for an earthen dam may be requested by the Engineering Department. Earthen dam structures, of any height, shall be designed by a Professional Engineer registered to practice in the state of Arkansas.

5.2.1.2 Freeboard Criteria

All stormwater ponds shall have a minimum freeboard of one foot.

5.2.1.3 Minimum Geometric Criteria

The minimum length to width ratio for stormwater ponds is 1.5:1 (i.e., length relative to width). Long flow paths and irregular shapes are recommended.

5.2.1.4 Pond Benches

The perimeter of all deep pool areas (four feet or greater in depth) shall be surrounded by two benches:

- A safety bench that extends 15 feet outward from the normal water edge to the toe of the pond side slope. The maximum slope of the safety bench shall be 6%.
- An aquatic bench that extends up to 15 feet inward from the normal shoreline and has a maximum depth of eighteen inches below the normal pool water surface elevation.

5.2.1.5 Safety Features

Side slopes to the pond shall not exceed 3:1 (h:v), and shall terminate on a safety bench. Both the safety bench and the aquatic bench may be landscaped to prevent access to the pool. The bench requirement may be waived if slopes are 4:1 or flatter.

The principal spillway opening shall not permit access by small children, and endwalls above pipe outfalls greater than 24 inches in diameter shall be fenced to prevent a hazard.

5.2.2 *Detention Criteria*

When used to meet detention requirements, stormwater ponds shall be designed to limit the peak storm water discharge rate of the 10-, 25-, 50-, and 100-year storm frequencies after development to pre-development flow rates.

5.2.2.1 Volume of Detention

Volumes of detention shall be evaluated according to the following methods:

- Volumes of stormwater ponds with total drainage areas of 20 acres or less may be evaluated by the "Modified Rational Hydrograph Method."

- For basins with total drainage areas larger than 20 acres, the Owner's Engineer shall submit the proposed method of evaluation for the sizing of the stormwater pond to the Engineering Department. The method will be evaluated for professional acceptance, applicability, and reliability by the Engineering Department. No detailed review for projects larger than 20 acres will be rendered before the method of evaluation of the detention/retention basin is approved.

- The computed hydraulic detention volume shall be increased by 25 percent as a factor of safety and to provide for sediment storage. The Engineering Department may reduce this requirement depending on the development characteristics and stream stability of upstream tributary areas.

5.2.2.2 Routing Method

The hydrograph routing method used shall be the Modified Puls Method.

5.2.2.3 Stormwater Pond Design Procedure (Modified Rational Method)

1. Compute pre-development and post-development site characteristics:

- Drainage Area
- Composite Runoff Coefficient
- Time of Concentration

2. Determine rainfall intensity for pre-development conditions (10- through 100-year storm).

3. Compute pre-development peak runoff rates using Rational Formula. These flow rates will be the maximum allowable release rates from the detention basin.

4. Determine inflow hydrograph using Modified Rational Method (see example problem and Figure 5-6 in section 5.2.2.4).

5. Find estimated detention volume using Modified Rational Method

6. Size Stormwater Pond based on estimated required volume. Develop stage-storage curve for the detention basin.

7. Size release structure based on allowable release flow. Develop stage-discharge curve for the release structure.

8. Route the inflow hydrographs (developed using the Modified Rational Method for the 10- through 100-year storms) through the stormwater pond using Modified Puls Method.

9. Check routed hydrographs to ensure flows do not exceed pre-development peaks. Adjust stormwater pond and release structure if necessary.

5.2.2.4 Example Problem – Modified Rational Method

The following example problem describes the general procedure to complete a design of a stormwater pond using the Modified Rational Method. The values and information provided in this example do not represent actual data for the City of Fort Smith but are only provided to illustrate the procedure.

<u>Given:</u> A 10-acre site currently agricultural use is to be developed for townhouses. The entire area is the drainage area of the proposed stormwater pond.

<u>Determine:</u> Maximum release rate and required detention storage.

<u>Solution:</u>

Step 1: Determine 100-year peak runoff rate prior to site development. This is the maximum release rate from site after development.

NOTE: Where a stormwater pond is being designed to provide detention for both its drainage area and a bypass area, the maximum release rate is equal to the peak runoff rate prior to site development for the total of the areas minus the peak runoff rate after development for the bypass area. This rate for the bypass area will vary with the duration being considered.

Present Conditions $Q = CiA$ (See Section 2.4.1) (5.9)

$C = 0.30$
$T_c = 20$ minutes
$i_{100} = 7.0$ in./hr
$Q_{100} = 0.30(7.0)(10) = 21.0$ cfs (Maximum Release Rate)

Step 2: Determine inflow hydrograph for storms of various durations in order to determine maximum volume required with release rate determined in Step 1.

NOTE: Incrementally increase durations by 10 minutes to determine maximum required volume. The duration with a peak inflow less than the maximum release rate or where required storage is less than storage for the prior duration is the last increment.

Future Conditions (Townhouses)

$C = 0.80$
$T_c = 15$ minutes
$i_{100} = 7.7$ in./hr
$Q_{100} = 0.80(7.7)(10) = 61.6$ cfs

Check various duration storms.

20 min	$i = 7.0$	$Q_{in} = 0.80\ (7.0)\ (10) = 56.0$ cfs
30 min	$i = 5.8$	$Q_{in} = 0.80\ (5.8)\ (10) = 46.4$ cfs
40 min	$i = 5.0$	$Q_{in} = 0.80\ (5.0)\ (10) = 40.0$ cfs
50 min	$i = 4.4$	$Q_{in} = 0.80\ (4.4)\ (10) = 35.2$ cfs
60 min	$i = 4.0$	$Q_{in} = 0.80\ (4.0)\ (10) = 32.0$ cfs
70 min	$i = 3.7$	$Q_{in} = 0.80\ (3.7)\ (10) = 29.6$ cfs
80 min	$i = 3.4$	$Q_{in} = 0.80\ (3.4)\ (10) = 27.2$ cfs
90 min	$i = 3.1$	$Q_{in} = 0.80\ (3.1)\ (10) = 24.8$ cfs

The Maximum Storage Volume in cubic feet (cf) is determined by deducting the volume of runoff released during the time of inflow from the total outflow for each storm duration.

$$V = (time \times Q_{in} \times 60\ s/min) - (0.5 \times (time + T_c) \times Q_{out} \times 60\ s/min) \qquad (5.10)$$

15 min Storm Inflow 15 (61.6) (60) = 55,440 cf
Outflow 0.5 (30)(21.0)(60) = 18,900 cf

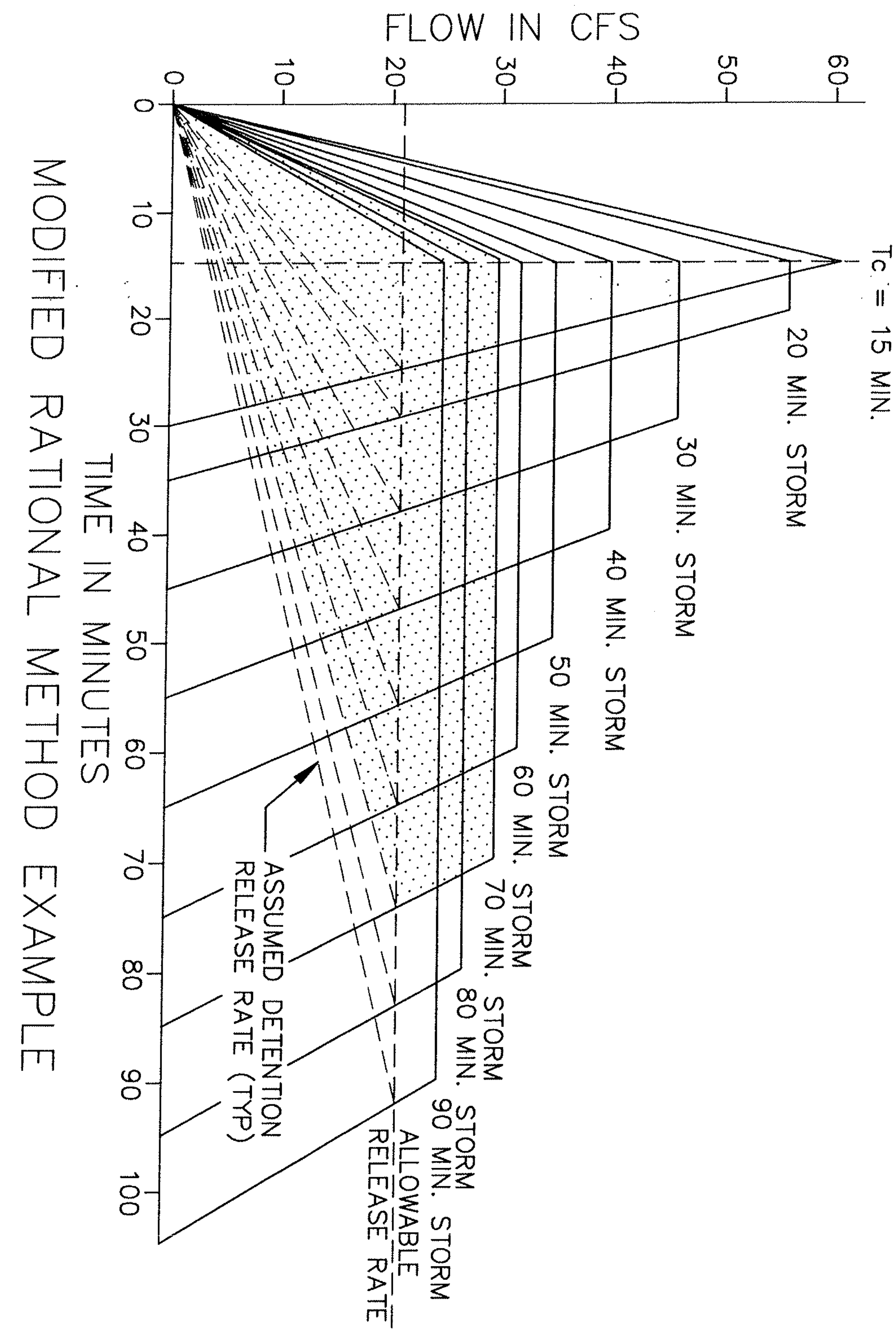
20 min Storm Inflow 20 (56.0) (60) = 67,200 cf
Outflow 0.5 (35)(21.0)(60) = 22,050 cf
Storage 45,150 cf

30 min Storm Inflow 30 (46.4) (60) = 83,520 cf
Outflow 0.5 (45)(21.0)(60) = 28,350 cf
Storage 55,170 cf

40 min Storm Inflow 40 (40.0) (60) = 96,000 cf
Outflow 0.5 (55)(21.0)(60) = 34,650 cf
Storage 61,350 cf

50 min Storm Inflow 50 (35.2) (60) = 105,600 cf
Outflow 0.5 (65)(21.0)(60) = 40,950 cf
Storage 64,650 cf

FIGURE 5-6. Example Problem, Modified Rational Method.



CONCEPT OF DETENTION POND MODIFIED RATIONAL METHOD EXAMPLE STANDARD DETAIL CITY OF FORT SMITH, ARKANSAS	The City of Fort Smith ARKANSAS	Project: Date: FEN. 2008 Scale: NONE Drawn By: RBR

60 min Storm Inflow 60 (32.0) (60) = 115,200 cf
Outflow 0.5 (75)(21.0)(60) = 47,250 cf
Storage 67,950 cf

70 min Storm Inflow 70 (29.6) (60) = 124,320 cf
Outflow 0.5 (85)(21.0)(60) = 53,550 cf
Storage 70,770 cf

80 min Storm Inflow 80 (27.2) (60) = 130,560 cf
Outflow 0.5 (95)(21.0)(60) = 59,850 cf
Storage 70,710 cf

90 min Storm Inflow 90 (24.8) (60) = 133,920 cf
Outflow 0.5 (105)(21.0)(60) = 66,150 cf
Storage 67,770 cf

Step 3: Route design storm hydrograph through the stormwater pond using the Modified Puls Routing Method or another approved method, based on final stormwater pond and release structure design. Computer programs to accomplish this are readily available.

5.2.2.5 Stormwater Detention Analysis Software

The City will allow the use of the following software or an acceptable equal approved by the Engineering Department for the analysis of storm water detention facilities: HEC-HMS, HEC-1, PondPack.

5.2.3 *Outlet Works*

Stormwater ponds shall be provided with effective outlet works. Safety considerations shall be an integral part of the design of all outlet works. Plan view and sections of the structure with adequate details shall be included in the plans.

The riser structure selected shall have documented evidence that it will control the 10-, 25-, 50-, and 100-year storm events. Generally, the full range of frequency control is achieved by selecting the 100-year and an intermediate frequency, such as the 10-year flood. Documented evidence shall also be provided that the riser will control the WQ_v if the stormwater pond is used to meet this requirement. The riser shall also be located within the embankment for maintenance access, safety and aesthetics. Access to the riser is to be provided by lockable manhole covers (the principal spillway opening can be "fenced" with pipe or rebar at 8 inch intervals for safety purposes). The principal spillway shall also be equipped with a trash rack that provides access for maintenance.

A non-clogging low flow orifice must be provided for the WQ_v. The low flow orifice shall have a minimum diameter of 3 inches, and shall be adequately protected from clogging by an acceptable external trash rack. The preferred method is a submerged reverse-slope pipe that extends downward from the riser to an inflow point one foot below the normal pool elevation. Alternative methods are to employ a broad crested rectangular weir or a V-notch weir protected by a half-round CMP that extends at least 12 inches below the normal pool. Horizontal perforated pipe protected by geotextile and gravel shall not be used. Vertical perforated pipes shall not be used.

The emergency spillway may either be combined with the outlet works or be a separate structure or channel meeting the following criteria:

- Emergency spillways shall be designed so that their crest elevation is 0.5 feet or more above the maximum water surface elevation in the detention facility attained by the 100-year storm event *(1)*.

- In cases where the emergency spillway is not regulated by either State or Federal agencies, the emergency spillway shall be designed to pass the 100-year storm with 1 foot of freeboard (or as designated) from the design stage to the top of dam, assuming zero available storage in the basin and zero flow through the primary outlet *(1)*.

Each stormwater pond shall have a drain pipe that can completely or partially drain the pond. The drain pipe shall have an elbow within the pond to prevent sediment deposition, and a diameter capable of draining the pond within 24 hours. The pond drain should be sized one pipe size greater than the calculated design diameter. Care shall be exercised during pond drawdowns to prevent downstream discharge of sediments or anoxic water and rapid drawdown. The Engineering Department shall be notified before draining a pond.

The pond drain shall be equipped with an adjustable valve (typically a handwheel activated knife or gate valve). Valve controls shall be located inside of the riser at a point where they (a) will not normally be inundated and (b) can be operated in a safe manner. To prevent vandalism, the handwheel should be chained to a ringbolt, or other fixed object

Sharp-crested weir flow equations for no end contractions, two end contractions and submerged discharge conditions are presented below, followed by equations for broad-crested weirs, V-notch weirs, and orifices, or combinations of these facilities. If culverts are used as outlets works, procedures presented in the Culverts Chapter should be used to develop stage-discharge data. When analyzing release rates, the tailwater influence of the principal spillway culvert on the control structure (orifice and/or weirs) must be considered to determine the effective head on each opening. Slotted riser pipe outlet facilities shall not be used.

5.2.3.1 Sharp-Crested Weirs

A sharp-crested weir with no end contractions is illustrated in Figure 5-7. The discharge equation for this configuration is *(4)*:

$$Q = [3.27 + 0.4(H/H_c)]\ LH^{1.5} \tag{5.3}$$

where:
Q = discharge, ft^3/s
H = head above weir crest excluding velocity head, ft
H_c = height of weir crest above channel bottom, ft
L = horizontal weir length, ft

A sharp-crested weir with two end contractions is illustrated in Figures 5-7 and 5-8. The discharge equation for this configuration is *(4)*:

$$Q = [3.27 + 0.4(H/H_c)]\ (L - 0.2H)\ H^{1.5} \tag{5.4}$$

where: Variables are the same as Equation 5.1.

A sharp-crested weir will be affected by submergence where the tailwater rises above the weir-crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is *(3)*:

$$Q_S = Q_f(1 - (H_2/H_1)^{1.5})^{0.385} \tag{5.5}$$

where:
Q_s = submergence flow, ft^3/s
Q_f = free flow, ft^3/s
H_1 = upstream head above crest, ft
H_2 = downstream head above crest, ft

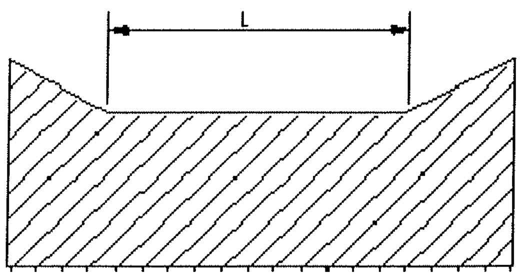


FIGURE 5-7.
Sharp-Crested Weir (No End Contractions)

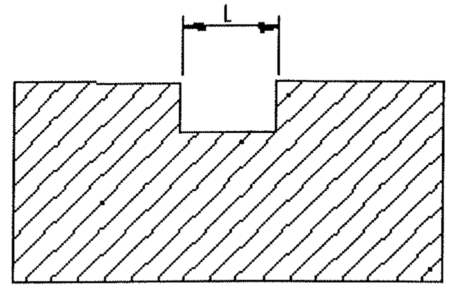


FIGURE 5-8.
Sharp-Crested Weir (Two End Contractions)

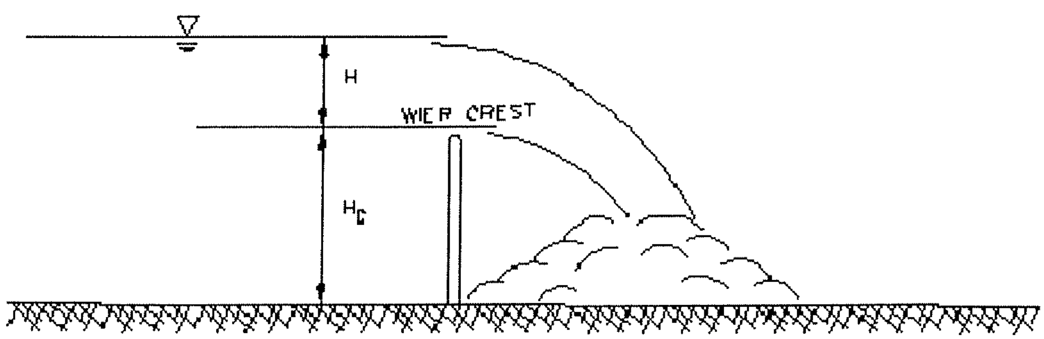


FIGURE 5-9. Sharp-Crested Weir and Head

5.2.3.2 Broad-Crested Weirs

The equation generally used for the broad-crested weir is *(3)*:

$$Q = CLH^{1.5} \tag{5.6}$$

where:

Q = discharge, ft^3/s
C = broad-crested weir coefficient
L = broad-crested weir length, ft
H = head above weir crest, ft

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Additional information on C values as a function of weir crest breadth and head is given in Table 5-1.

TABLE 5-1. Broad-Crested Weir Coefficient *C* Values as a Function of Weir Crest Breadth and Head (ft)

Measured Head, H[1] (ft)	Breadth of the Crest of Weir (ft)										
	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.27	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

[1]Measured at least 2.5*H* upstream of the weir.

Source: Reference *(3)*.

5.2.3.3 V-Notch Weirs

The discharge through a V-notch weir can be calculated from the following equation *(3)*:

$$Q = 2.5\ tan(q/2)H^{2.5} \tag{5.7}$$

where:
Q = discharge, ft^3/s
q = angle of V-notch, degrees
H = head on apex of notch, ft

5.2.3.4 Orifices

Pipes smaller than 12 in. may be analyzed as a submerged orifice if H/D is greater than 1.5. For square-edged entrance conditions:

$$Q = 0.6A(2gH)^{0.5} \tag{5.8}$$

where:
Q = discharge, ft^3/s
A = cross-section area of pipe, ft^2
g = acceleration due to gravity, 32.2 ft/s^2
D = diameter of pipe, ft
H = head on pipe, from the center of pipe to the water surface, ft *

* Where the tailwater is higher than the center of the opening, the head is calculated as the difference in water surface elevations.

5.2.4 *Discharge Systems*

For site-specific runoff, the effectiveness of local stormwater ponds used for detention can be acknowledged in the design of any onsite downstream drainage facilities, assuming that the stormwater ponds comply with all criteria and that they are properly constructed and maintained.

In the case of regional stormwater ponds, sizing of the system below the control structure shall be for the total improved peak runoff tributary to the structure, with no allowance for detention unless approved in writing by the Engineering Department.

In the event the Engineer desires to incorporate the flow reduction benefits of existing upstream stormwater ponds, the following field investigations and hydrologic analysis shall be required:

- A field survey of the existing physical characteristics of both the outlet structure and ponding volume. Any departure from the original engineer's design must be accounted for. If a dual use for the stormwater pond exists, then this also must be accounted for.

- A comprehensive hydrologic analysis that simulates the attenuation of the contributing area ponds. This should not be limited to a linear additive analysis, but rather should consist of a network of hydrographs that considers incremental timing of discharge and potential coincidence of outlet peaks.

Please note that under no circumstances will the previously approved construction plans of the upstream pond or ponds suffice as an adequate analysis. While the responsibility of the individual site or subdivision plans rests with the Engineer of Record, any subsequent engineering analysis must assure that all the incorporated ponds work collectively.

5.2.5 *Conveyance Criteria*

Conveyance shall be provided which does not cause erosion. Primary outlet works, emergency spillways, and conveyance system entrances to stormwater ponds shall be equipped with energy dissipating devices as necessary to limit erosion on receiving channels *(1)*.

5.2.5.1 Inlet Protection

A forebay shall be provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the pond. Inlet areas shall be protected to reduce erosion.

5.2.5.2 Adequate Outfall Protection

Outfalls shall be constructed such that they do not increase erosion or have undue influence on the downstream geomorphology of the stream.

Flared pipe sections that discharge at or near the stream invert or into a step-pool arrangement shall be used at the spillway outlet.

The channel immediately below the pond outfall shall be modified to prevent erosion and conform to natural dimensions in the shortest possible distance, typically by use of large rip-rap placed over filter cloth.

A stilling basin or outlet protection shall be used to reduce flow velocities from the principal spillway to non-erosive velocities.

If a pond daylights to a channel with dry weather flow, care should be taken to minimize tree clearing along the downstream channel, and to reestablish a forested riparian zone in the shortest possible distance. Excessive use of rip-rap should be avoided to reduce stream warming.

5.2.5.3 Pond Liners

When a pond is located in karst topography, gravelly sands or fractured bedrock, a liner may be needed to sustain a permanent pool of water. If geotechnical tests confirm the need for a liner, acceptable options include: (a) 6 to 12 inches of clay soil (minimum 15% passing the #200 sieve and a minimum permeability of 1×10^{-5} cm/sec), (b) a 30 ml poly-liner (c) bentonite, or (d) use of chemical additives (see NRCS Agricultural Handbook No. 387, dated 1971, or Engineering Field Manual).

5.2.6 *Water Quality Criteria*

5.2.6.1 Pretreatment Criteria

Each stormwater pond used to meet water quality requirements shall have a sediment forebay or equivalent upstream pretreatment. The forebay shall consist of a separate cell, formed by an acceptable barrier.

The forebay shall be 4 to 6 feet deep. It shall be sized to contain 0.1 inches of runoff per impervious acre of contributing drainage. The forebay storage volume counts toward the total WQ_v requirement. Exit velocities from the forebay shall be non-erosive.

Direct maintenance access for appropriate equipment shall be provided to the forebay.
The bottom of the forebay may be hardened (e.g., using concrete, paver blocks, etc.) to make sediment removal easier.

A fixed vertical sediment depth marker should be installed in the forebay to measure sediment deposition over time.

5.2.6.2 Treatment Criteria

Stormwater ponds used to meet water quality requirement shall be designed to capture and treat the computed WQ_v through any combination of permanent pool, extended detention (ED) or wetland. Stormwater ponds shall release the WQ_v over a minimum period of 24-hours and within a maximum of 72-hours.

It is generally desirable to provide water quality treatment off-line when topography, head and space permit (e.g., apart from stormwater quantity storage).

Water quality storage can be provided in multiple cells. Performance is enhanced when multiple treatment pathways are provided by using multiple cells, longer flowpaths, high surface area to volume ratios, complex microtopography, and/or redundant treatment methods (combinations of pool, ED, and wetland).

If a micropool extended detention pond is constructed, the micropool shall be sized to contain 0.1 inches per impervious acre of contributing drainage.

5.2.7 *Landscaping Criteria*

5.2.7.1 Landscaping Plan

A landscaping plan for a stormwater pond and its buffer shall be prepared to indicate how aquatic and terrestrial areas will be vegetatively stabilized and established.

Wherever possible, wetland plants should be encouraged in a pond design, either along the aquatic bench (fringe wetlands), the safety bench and side slopes (ED wetlands) or within shallow areas of the pool itself.

The best elevations for establishing wetland plants, either through transplantation or volunteer colonization, are within six inches (plus or minus) of the normal pool.

The soils of a pond buffer are often severely compacted during the construction process to ensure stability. The density of these compacted soils is so great that it effectively prevents root penetration, and therefore, may lead to premature mortality or loss of vigor. Consequently, it is advisable to excavate large and deep holes around the proposed planting sites, and backfill these with uncompacted topsoil.

As a rule of thumb, planting holes should be 3 times deeper and wider than the diameter of the rootball (of balled and burlap stock), and 5 times deeper and wider for container grown stock. This practice should enable the stock to develop unconfined root systems. Avoid species that require full shade, are susceptible to winterkill, or are prone to wind damage. Extra mulching around the base of the tree or shrub is strongly recommended as a means of conserving moisture and suppressing weeds.

5.2.7.2 Pond Buffers and Setbacks

Pond buffers can be important in providing ample space for access and safety. The buffer can be planted or left in trees to discourage resident goose populations.

A pond buffer shall be provided that extends 25 feet outward from the maximum water surface elevation of the pond. The pond buffer shall be contiguous with other buffer areas, that are required by existing regulations (e.g., stream buffers). An additional setback may be provided to permanent structures.

Woody vegetation may not be planted on or allowed to grow within 15 feet of the toe of the embankment and 25 feet from the principal spillway structure.

Existing trees should be preserved in the buffer area during construction. It is desirable to locate forest conservation areas adjacent to ponds. To discourage resident geese populations, the buffer can be planted with trees, shrubs and native ground covers.

5.2.8 *Ownership of Stormwater Ponds*

Ownership of stormwater ponds in residential subdivisions accepted by the City shall be vested in the City of Fort Smith with the filing of the final plat. The Developer shall warrant the operation of the drainage system for 2 years after acceptance by the City by a Maintenance Bond provided by the Developer's Contractor or the Developer. The bond shall be required to be extended until 2 years after all phases of the subdivision or development that substantially drain into the stormwater pond are completed.

Ownership of stormwater ponds in commercial, industrial, private subdivisions, and non-residential areas shall be vested in the property owner.

5.2.9 *Maintenance of Stormwater Ponds*

When ownership of a stormwater pond is not vested in the City of Fort Smith, the maintenance responsibility for a pond and its buffer shall be vested with a responsible party by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval or the permitting process.

Stormwater ponds, when required, are to be built in conjunction with storm sewer installation and/or grading. Since these facilities are intended to control increased runoff, they must be partially or fully operational soon after the clearing of the vegetation. Silt and debris connected with construction activities shall be removed periodically from the detention area and control structure to maintain the facility's storage capacity.

Maintenance of stormwater ponds is divided into two components – short term maintenance and long term maintenance. Requirements for both are discussed in the following sections. Requirements for maintenance access are also discussed below.

5.2.9.1 Short Term Maintenance

For public stormwater ponds, short term or annual maintenance is the responsibility of the developer or property owners' association for two years after acceptance of the final plat or filing of the last subdivision phase that substantially adds storm water to a stormwater pond. The items considered short term maintenance are as follows:

- Sediment removal
- Outlet cleaning
- Mowing
- Herbicide Spraying
- Litter Control

5.2.9.2 Long Term Maintenance

Long term maintenance includes removal of sediment from the basin and outlet structure. Studies show that to be needed once every 5 to 10 years. Sediment removal in the forebay shall occur when 50% of the total forebay capacity has been lost. Where the City has accepted the stormwater pond, the City is responsible for long term maintenance. Where basins are not accepted by the City, the property owner is responsible for the long term maintenance.

5.2.9.3 Maintenance Access

A maintenance right of way or easement shall extend to the stormwater pond from a public road. Maintenance access shall be at least 20 feet wide; have a maximum slope of no more than 15%; and should be appropriately stabilized to withstand maintenance equipment and vehicles. The maintenance access shall extend to the forebay, safety bench, riser, and outlet and be designed to allow vehicles to turn around.

5.3 DESIGN CRITERIA – WETLANDS

Stormwater wetlands are practices that create shallow marsh areas to treat urban stormwater and often incorporate small permanent pools and/or extended detention storage to achieve the full WQ_v. Design variants include:

- Shallow Wetland (Figure 5-10)
- ED Shallow Wetland (Figure 5-11)
- Pond/Wetland System (Figure 5-12)
- "Pocket" Wetland (Figure 5-13)

Stormwater wetlands may be used in residential, private, commercial, and industrial subdivisions and developments to meet the detention and WQ_v requirements.

All of the pond criteria presented in 5.2 DESIGN CRITERIA – STORMWATER PONDS also apply to the design of stormwater wetlands. Additional criteria that govern the geometry and establishment of created wetlands are presented in this section.

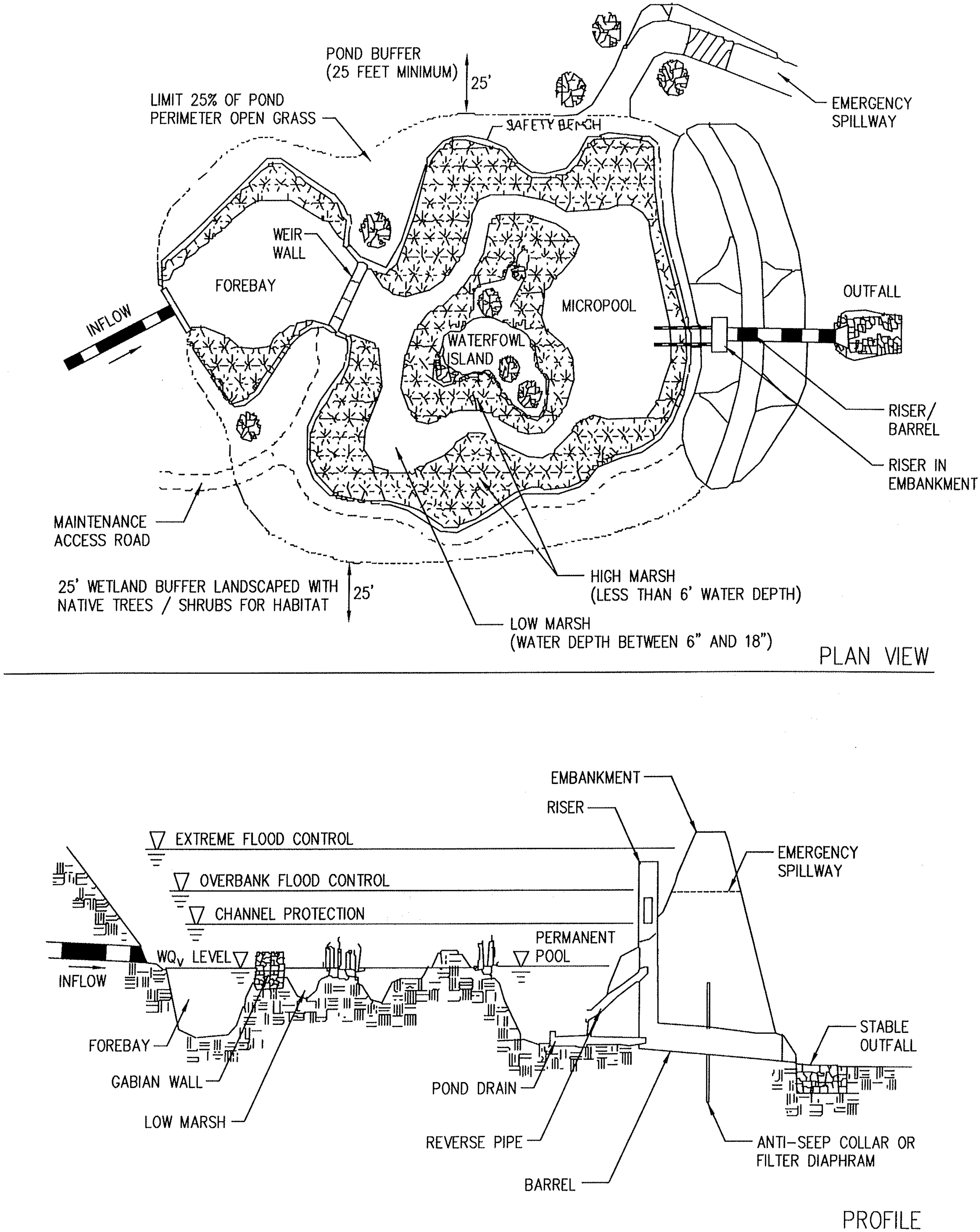


FIGURE 5-10. Shallow Wetland

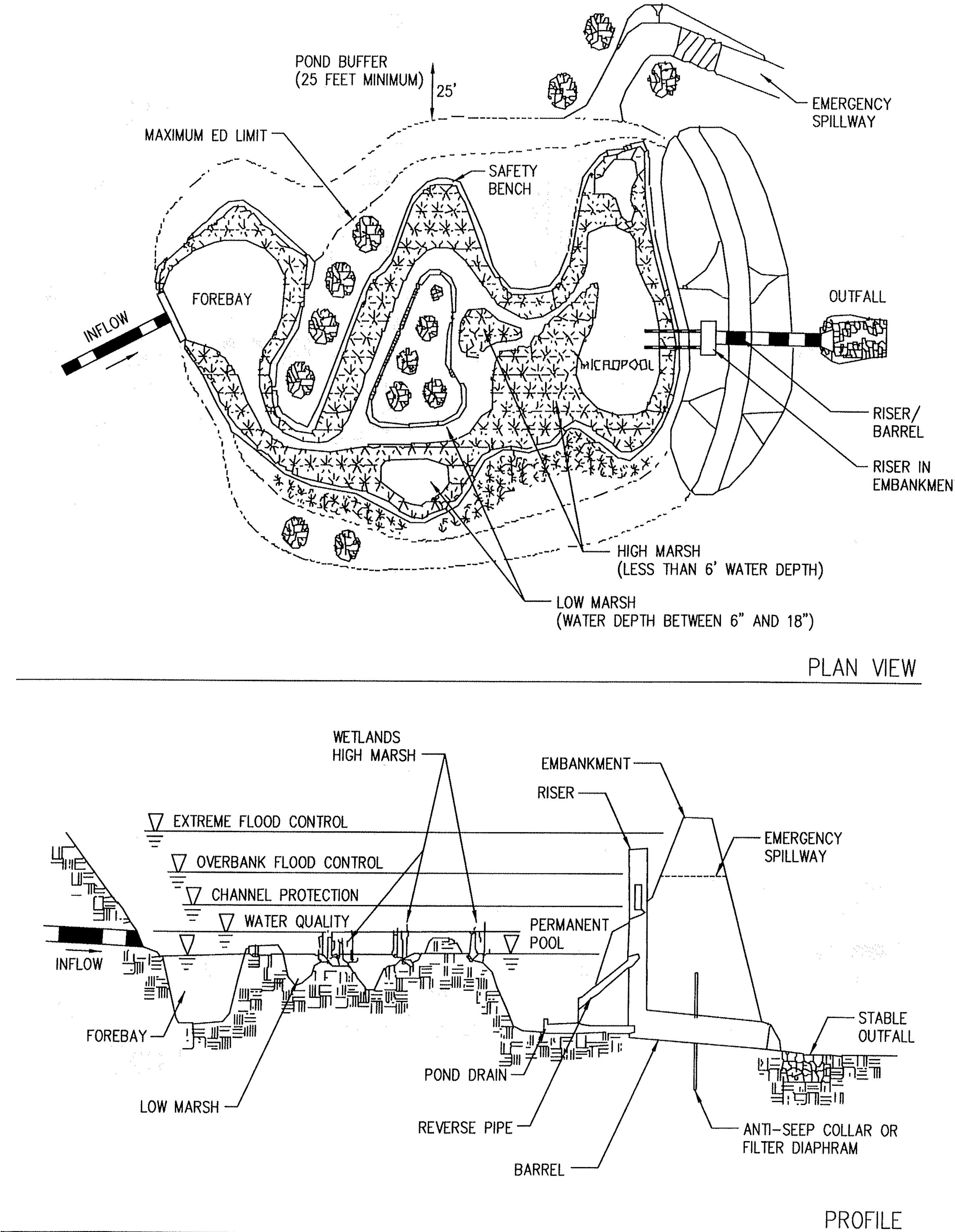
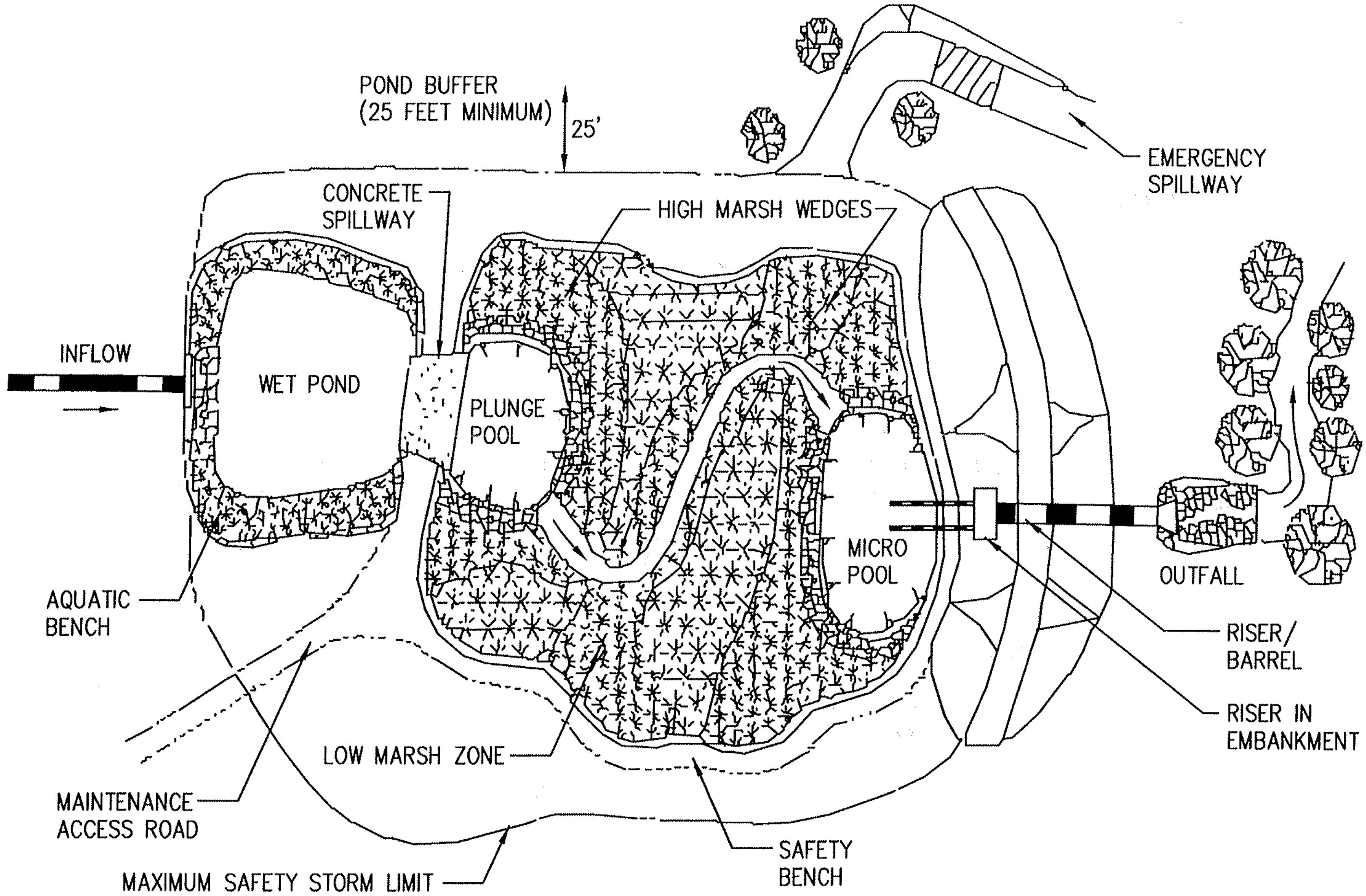
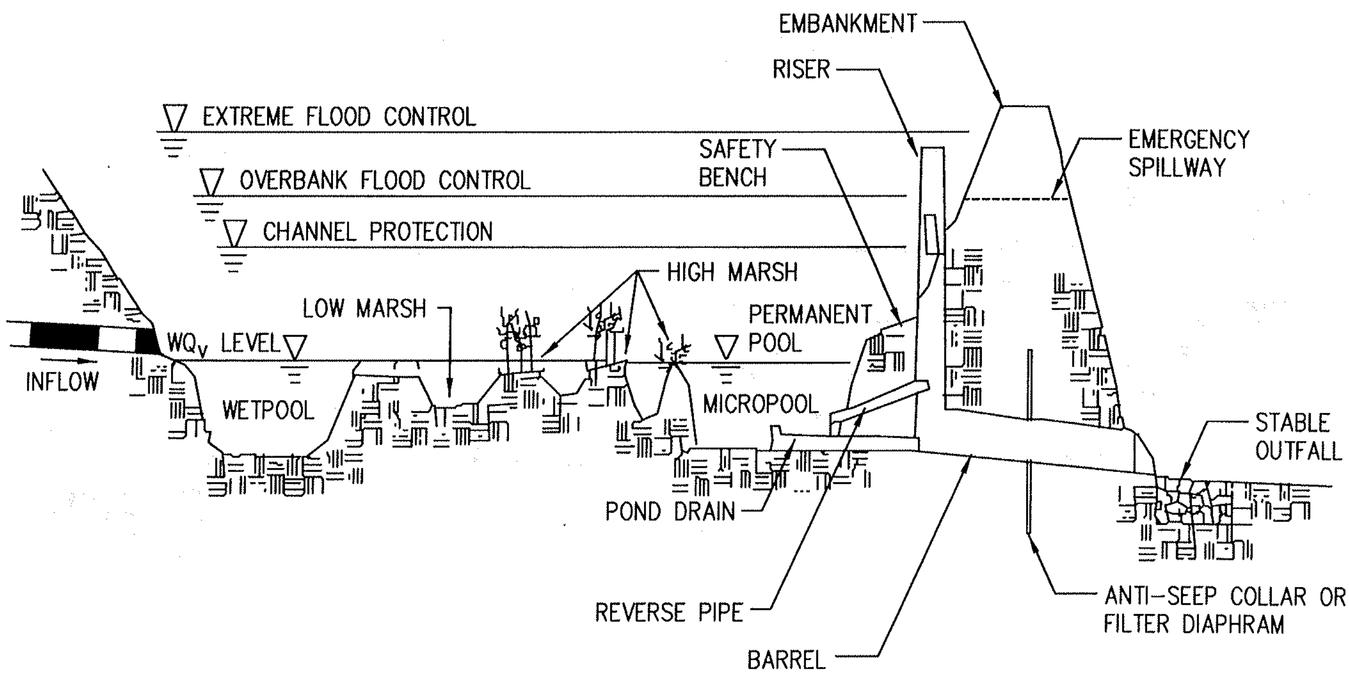


FIGURE 5-11. Extended Detention Shallow Wetland

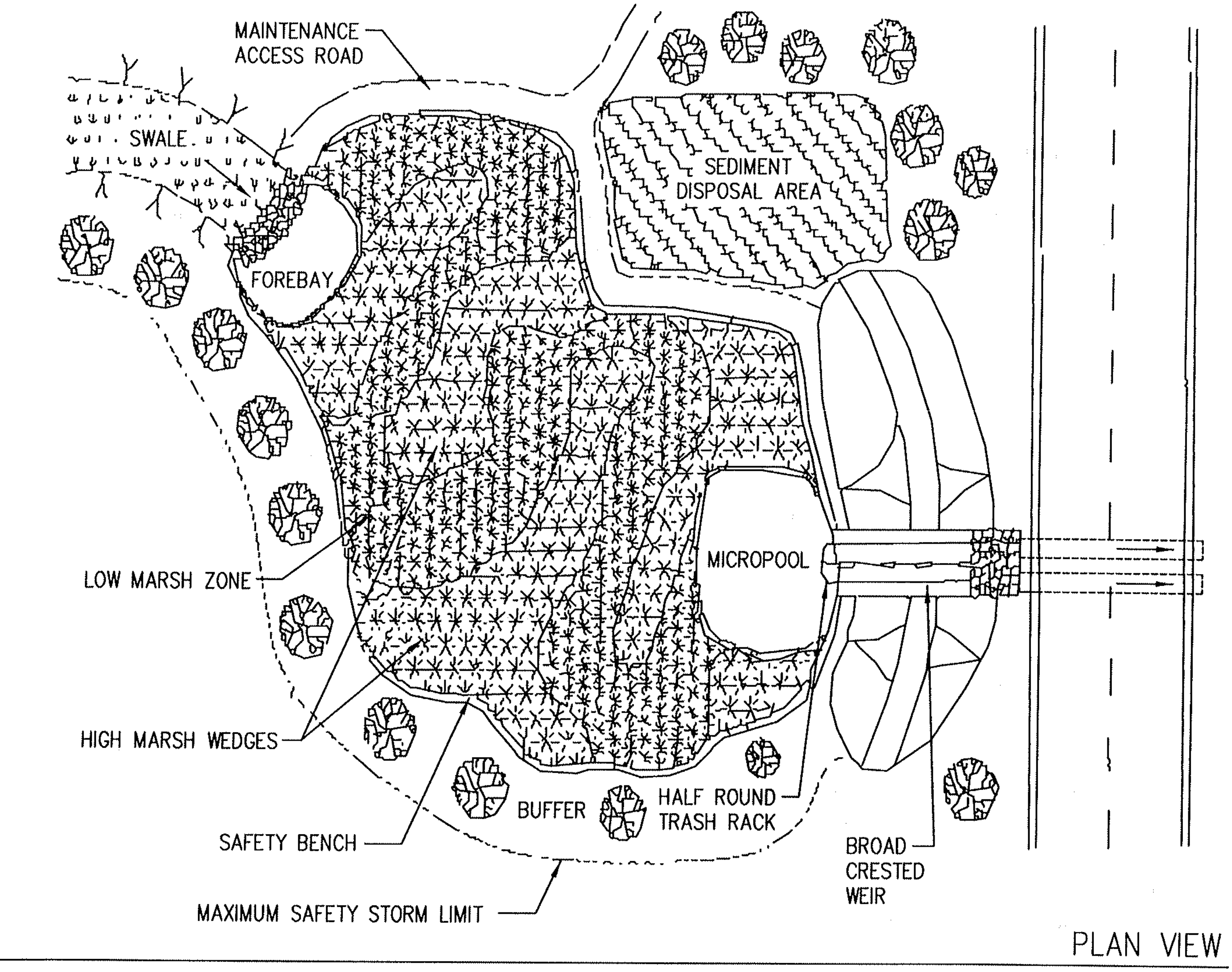


PLAN VIEW



PROFILE

FIGURE 5-12. Pond/Wetland System



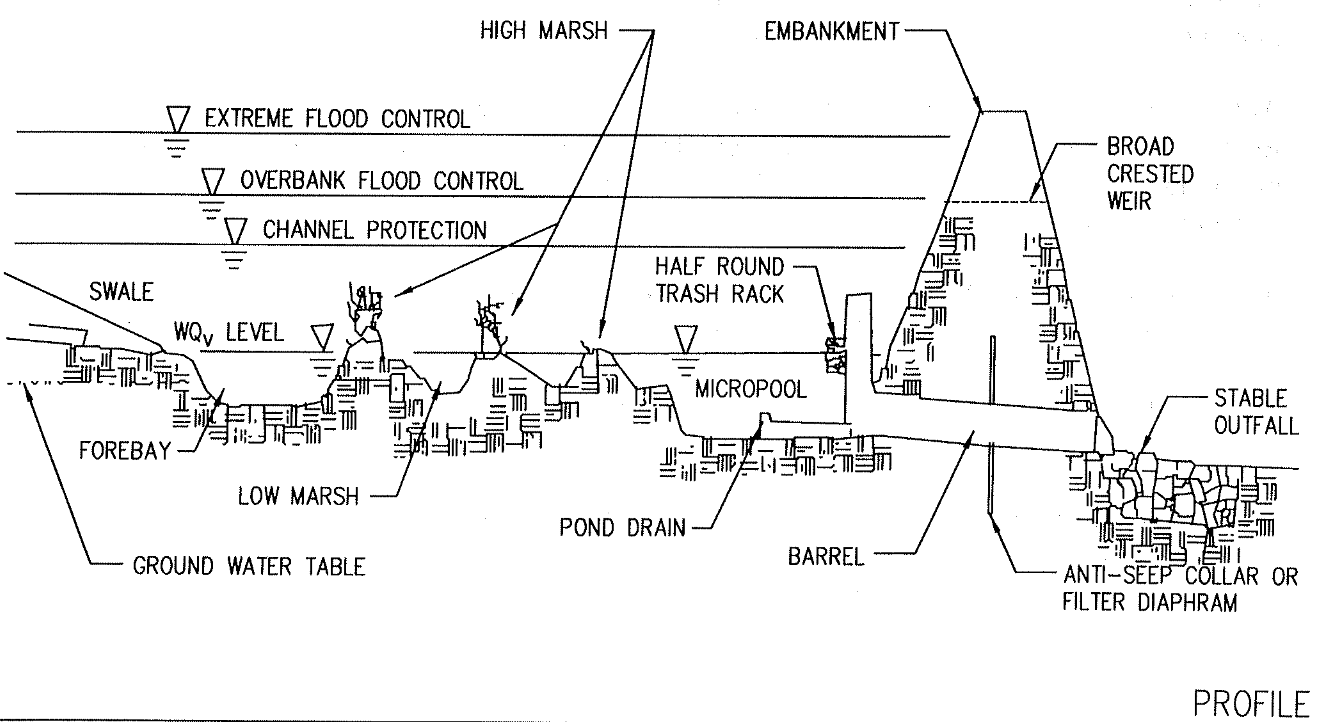


FIGURE 5-13. "Pocket" Wetland

5.3.1 *Feasability Criteria*

A water balance shall be performed to demonstrate that a stormwater wetland can withstand a significant drought at summer evaporation rates without completely drawing down.

Stormwater wetlands shall not be located within existing jurisdictional wetlands.

5.3.2 *Conveyance Criteria*

Flowpaths from the inflow points to the outflow points of stormwater wetlands shall be maximized. A minimum flowpath of 2:1 (length to relative width) shall be provided across the stormwater wetland. This path may be achieved by constructing internal berms (e.g., high marsh wedges or rock filter cells).

Microtopography is encouraged to enhance wetland diversity.

5.3.3 *Pretreatment Criteria*

Sediment regulation is critical to sustain stormwater wetlands. Consequently, a forebay shall be located at the inlet, and a micropool shall be located at the outlet. For forebay design criteria, consult 5.2 DESIGN CRITERIA – STORMWATER PONDS.

A micropool three to six feet deep shall be used to protect the low flow pipe from clogging and prevent sediment resuspension.

5.3.4 *Treatment Criteria*

The surface area of the entire stormwater wetland shall be at least one percent of the contributing drainage area (1.5% for shallow marsh design).

At least 25% of the WQ_v shall be in deepwater zones with a depth greater than four feet. The forebay and micropool may meet this criteria. In addition, the deepwater zones serve to keep mosquito populations in check by providing habitat for fish and other pond life that prey on mosquito larvae.

A minimum of 35% of the total surface area can have a depth of six inches or less, and at least 65% of the total surface area shall be shallower than 18 inches.

The bed of the wetland shall be graded to create maximum internal flow path and microtopography.

If extended detention is utilized in a stormwater wetland, the WQ_v -ED volume shall not comprise more than 50% of the total WQ_V, and its maximum water surface elevation shall not extend more than three feet above the permanent pool.

To promote greater nitrogen removal, rock beds may be used as a medium for growth of wetland plants. The rock should be one to three inches in diameter, placed up to the normal pool elevation, and open to flow-through from either direction.

5.3.5 *Landscaping Criteria*

A landscaping plan shall be provided that indicates the methods used to establish and maintain wetland coverage. Minimum elements of a plan include: delineation of pondscaping zones, selection of corresponding plant species, planting plan, sequence for preparing wetland bed (including soil amendments, if needed) and sources of plant material.

Structures such as fascines, coconut rolls, straw bales, or filter fence can be used to create shallow marsh cells in high energy areas of the stormwater wetland.

The landscaping plan should provide elements that promote greater wildlife and waterfowl use within the wetland and buffers.

A wetland plant buffer shall extend 25 feet outward from the maximum water surface elevation, with an additional 15 foot setback to structures.

5.3.6 *Wetland Establishment*

The most common and reliable technique for establishing an emergent wetland community in a stormwater wetland is to transplant nursery stock obtained from aquatic plant nurseries. The following guidance is suggested when transplants are used to establish a wetland.

Plant only during the transplanting window. Wetland plants need a full growing season to build root reserves needed to get through the winter. If at all possible, plants should be ordered at least three months in advance to ensure the availability of the desirable species.

The optimal depth requirements for several common species of emergent wetland plants are often six inches or less.

To add diversity to the wetland, five to seven species of emergent wetland plants should be planted.

No more than half the wetland surface area needs to be planted. If the appropriate planting depths are achieved, the entire wetland should be colonized within three years.

The wetland area should be subdivided into separate planting zones of more or less constant depth.

One plant species should be planted within each flagged planting zone, based on approximate depth requirements.

Individual plants should be planted 18 inches on center in clumps.

Post-nursery care of wetland plants is very important in the interval between delivery of the plants and their subsequent planting, as they are prone to dessication. Stock should be frequently watered and shaded while on-site.

A wet hydroseed mix should be used to establish permanent vegetative cover in the buffer outside the permanent pool. For rapid germination, scarify the soil to ½ inch prior to hydroseeding. Alternatively, grass species can be used as a temporary cover for the wet species.

Because most stormwater wetlands are excavated to deep subsoils, they often lack the nutrients and organic matter needed to support vigorous growth of wetland plants. At these sites, three to six inches of topsoil or wetland mulch should be added to all depth zones in the wetland from one foot below the normal pool to six inches above. Wetland mulch is preferable to topsoil if it is available.

The stormwater wetland should be staked at the onset of the planting season. Depths in the wetland should be measured to the nearest inch to confirm original planting zones. At this time, it may be necessary to modify the pondscaping plan to reflect altered depths or the availability of wetland plant stock. Surveyed planting zones should be marked on an "as-built" or design plan, and located in the field using stakes or flags. The wetland drain should be fully opened at least three days prior to the planting dates (which should coincide with the delivery date for the wetland plant stock).

Wetland mulch is another technique to establish a wetland plant community which utilizes the seedbank of wetland soils to provide the propagules for marsh development. The majority of the seedbank is contained within the upper six inches of the donor wetland soils. The mulch is best collected at the end of the growing season. Best results are obtained when the mulch is spread three to six inches deep over the high marsh and semi-wet zones of the wetland (-6 inches to +6 inches relative to the normal pool).

In some cases, the use of "volunteer wetlands," allowing cattails and phragmites to colonize may be appropriate.

Donor soils for wetland mulch should not be removed from natural wetlands.

5.3.7 *Ownership of Wetlands*

Ownership of stormwater wetlands in residential subdivisions accepted by the City shall be vested in the City of Fort Smith with the filing of the final plat. The Developer shall warrant the operation of the drainage system for 2 years after acceptance by the City by a Maintenance Bond provided by the Developer's Contractor or the Developer. The bond shall be required to be extended until 2 years after all phases of the subdivision or development that substantially drain into the stormwater wetland are completed.

Ownership of stormwater wetlands in commercial, industrial, private subdivisions, and non-residential areas shall be vested in the property owner.

5.3.8 *Maintenance of Wetlands*

Stormwater wetlands shall be required to meet all the maintenance requirements found in Section 5.2.9 *Maintenance of Stormwater Ponds.* In addition, stormwater wetlands shall also be required to meet the criteria below.

5.3.8.1 Minimum Coverage

If a minimum coverage of 50% is not achieved in the planted wetland zones after the second growing season, a reinforcement planting will be required.

5.4 DESIGN CRITERIA – STORMWATER INFILTRATION

Stormwater infiltration practices capture and temporarily store the WQ_v before allowing it to infiltrate into the soil over a two day period. Design variants include:

- Infiltration Trench (Figure 5-14)
- Infiltration Basin (Figure 5-15)

Extraordinary care must be taken to assure that long-term infiltration rates are achieved through post construction inspection and long-term maintenance.

Stormwater infiltration practices may be used in private, commercial, and industrial subdivisions and developments to meet the WQ_v requirement. In certain limited cases, with proper documentation, they may also be used in private, commercial, and industrial subdivisions and developments to meet the detention requirement.

5.4.1 *Feasibility Criteria*

To be suitable for infiltration, underlying soils must have an infiltration rate (f_c) of 0.52 inches per hour or greater, as initially determined from NRCS soil textural classification, and subsequently confirmed by field geotechnical tests. The minimum geotechnical testing is one test hole per 5000 sf, with a minimum of two borings per facility (taken within the proposed limits of the facility).

Soils shall also have a clay content of less than 20% and a silt/clay content of less than 40%.

Infiltration cannot be located on slopes greater than 6% or within fill soils.

To protect groundwater from possible contamination, runoff from designated hotspot land uses or activities cannot be infiltrated.

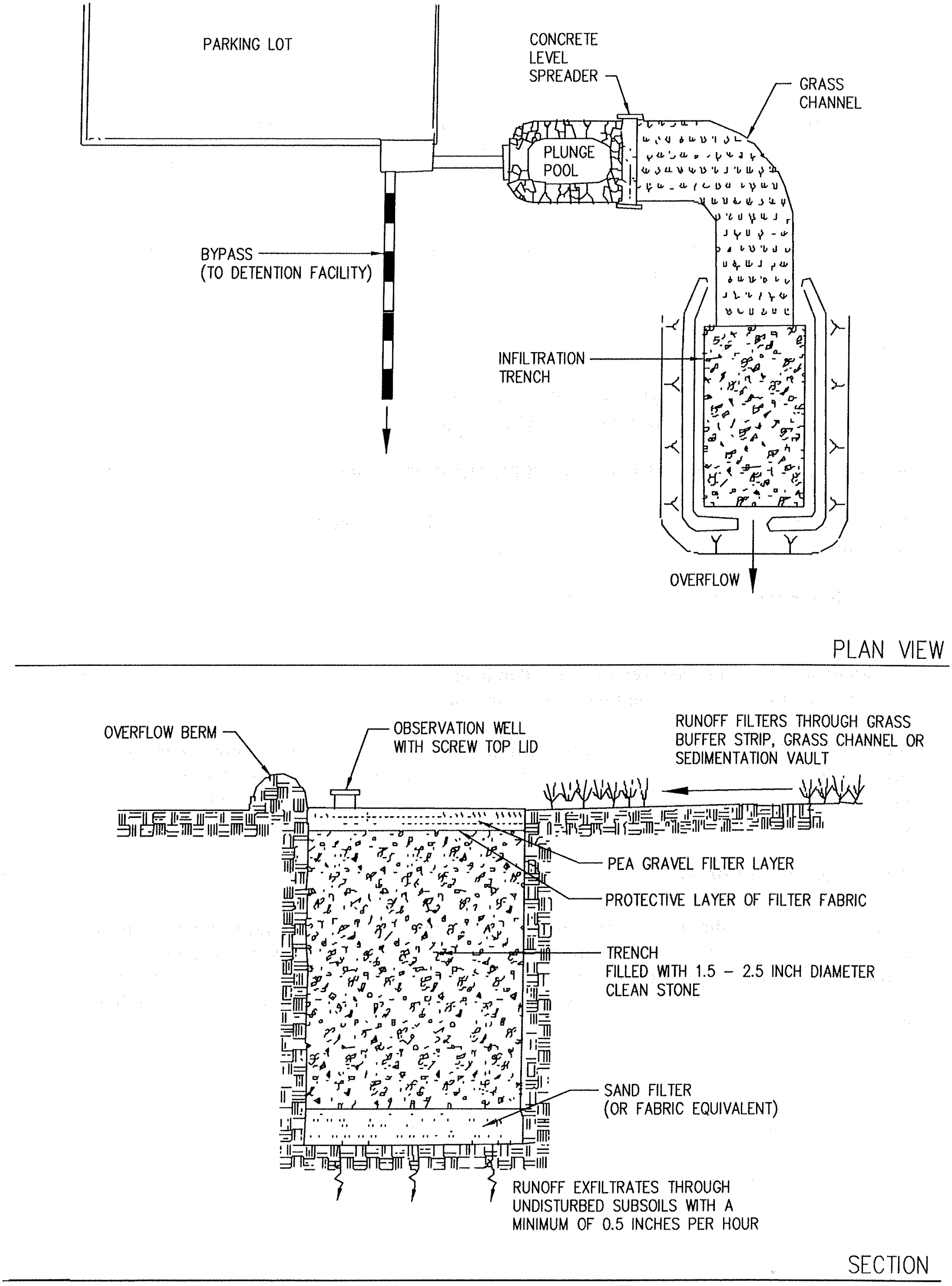
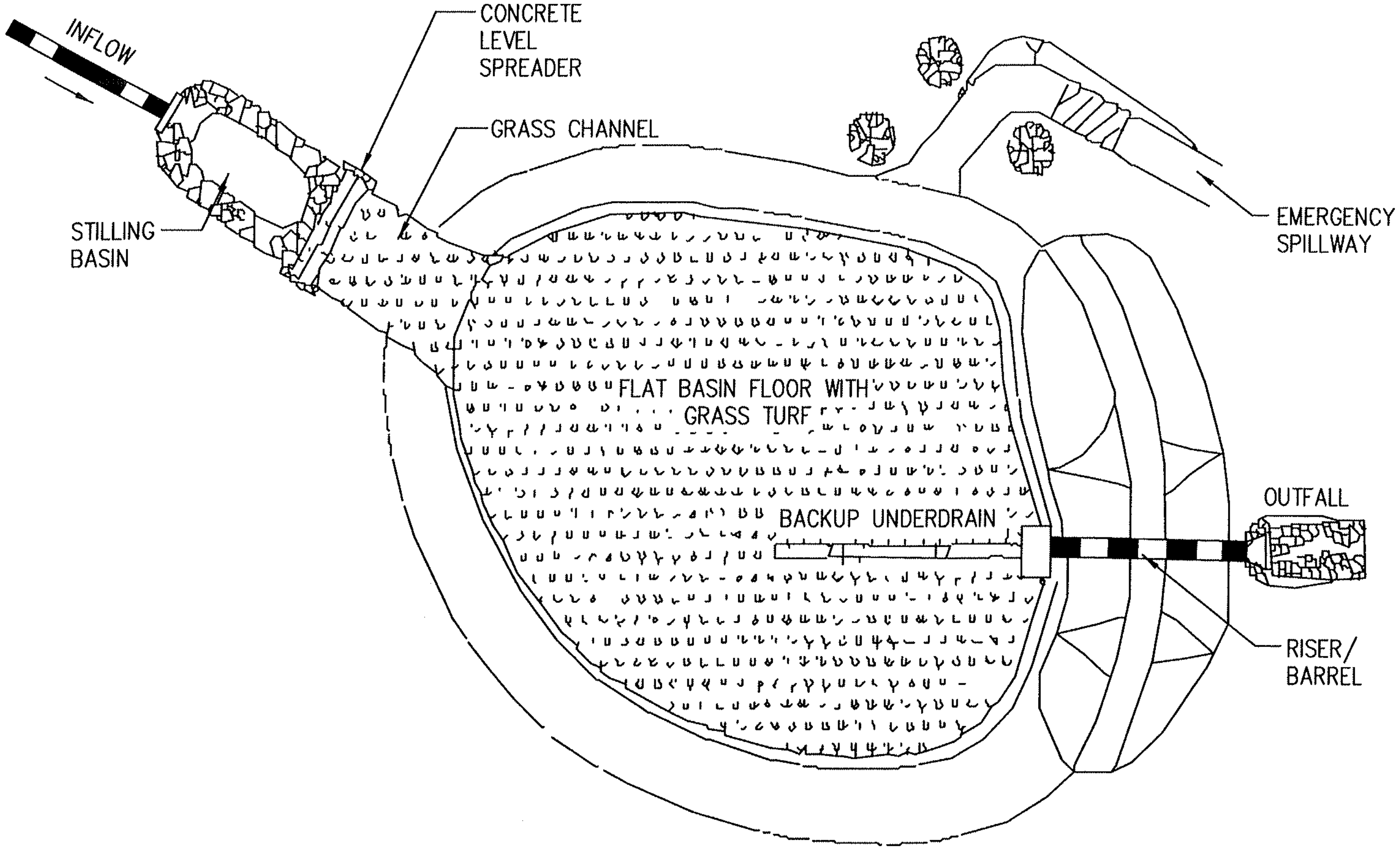
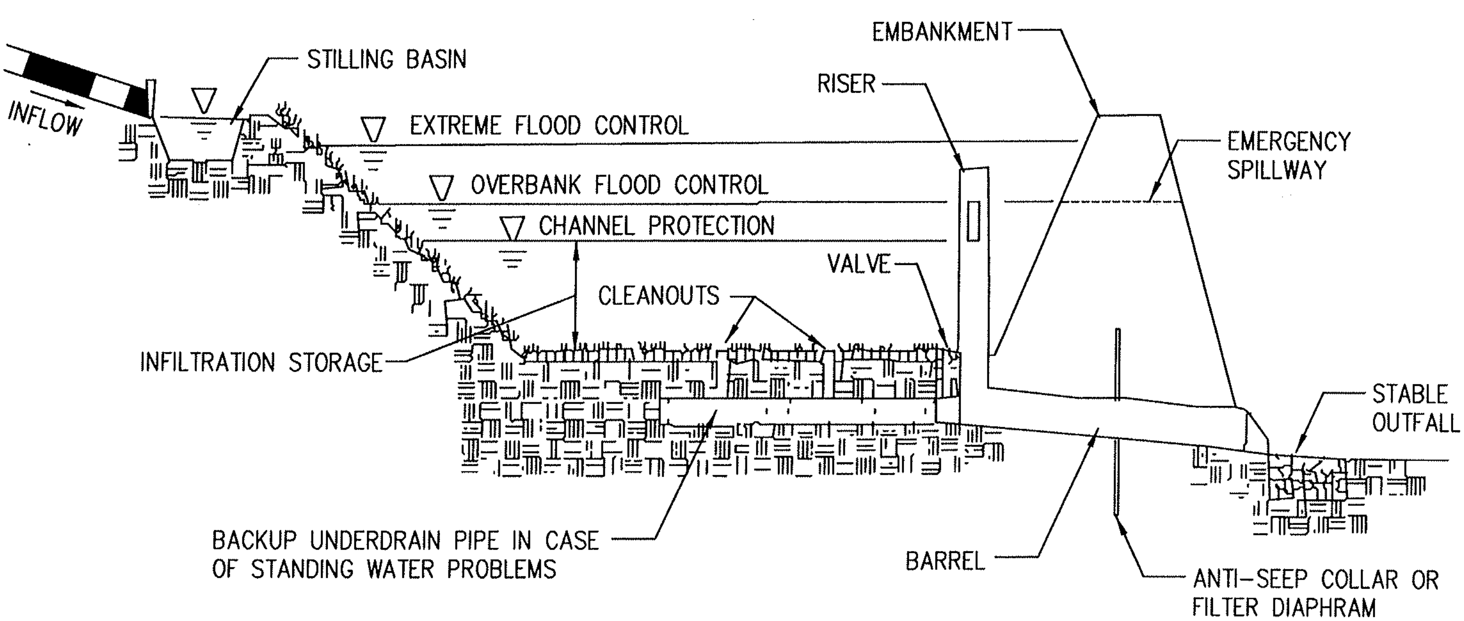


FIGURE 5–14. Infiltration Trench



PLAN VIEW



PROFILE

FIGURE 5-15. Infiltration Basin

The bottom of the infiltration facility shall be separated by at least four feet vertically from the seasonally high water table or bedrock layer, as documented by on-site soil testing.

Infiltration facilities can be located at least 100 feet horizontally from any water supply well.

Infiltration practices cannot be placed in locations that cause water problems to downgrade properties. Infiltration facilities must be setback at least 25 feet down-gradient from structures.

The maximum contributing area to an individual infiltration practice shall be less than 5 acres.

5.4.2 *Conveyance Criteria*

The overland flow path of surface runoff exceeding the capacity of the infiltration system can be evaluated to preclude erosive concentrated flow during the overbank events. If computed flow velocities exceed the non-erosive threshold, a overflow channel shall be provided to a stabilized water course.

All infiltration systems should be designed to fully de-water the entire WQ_v within 48 hours after the storm event.

If runoff is delivered by a storm drain pipe or along the main conveyance system, the infiltration practice must be designed as an off-line practice. Pretreatment shall be provided for storm drain pipes systems discharging directly to infiltration systems.

Adequate stormwater outfalls shall be provided for the overflow associated with the ten year design storm event (non-erosive velocities on the down-slope).

5.4.3 *Pretreatment Criteria*

5.4.3.1 Pretreatment Volume

A minimum pretreatment volume of at least 25% of the WQ_v must be provided prior to entry to an infiltration facility, and can be provided in the form of a sedimentation basin, sump pit, grass channel, plunge pool or other measure.

Exit velocities from pretreatment chambers shall be non-erosive (5 fps) during the two year design storm. If the f_c for the underlying soils is greater than 2.00 inches per hour, 50% of the WQ_v shall be treated by another method prior to entry into an infiltration facility.

5.4.3.2 Pretreatment Techniques to Prevent Clogging

Each infiltration system can have redundant methods to protect the long term integrity of the infiltration rate. Three or more of the following techniques must be installed in every facility:

- grass channel

- grass filter strip (minimum 20 feet and only if sheet flow is established and maintained)
- bottom sand layer
- upper filter fabric layer
- use of washed bank run gravel as aggregate

The sides of infiltration practices shall be lined with an acceptable filter fabric that prevents soil piping.

5.4.4 *Treatment Criteria*

Infiltration practices shall be designed to exfiltrate the entire WQ_v through the floor of each practice.

Infiltration practices are best used in conjunction with other practices, and often a stormwater pond is still needed downstream to meet the detention requirement.

A porosity value (V_v/V_t) of 0.32 can be used to design stone reservoirs for infiltration practices.

5.4.5 *Landscaping Criteria*

A dense and vigorous vegetative cover shall be established over the contributing pervious drainage areas before runoff can be accepted into the facility. Infiltration trenches shall not be constructed until all of the contributing drainage area has been completely stabilized.

5.4.6 *Ownership of Stormwater Infiltration*

Ownership of stormwater infiltration practices in commercial, industrial, private subdivisions, and non-residential areas shall be vested in the property owner.

Stormwater infiltration practices may not be used in residential subdivisions.

5.4.7 *Maintenance of Stormwater Infiltration*

The maintenance responsibility for a stormwater infiltration system shall be vested with a responsible party by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval or the permitting process.

Infiltration practices must never serve as a sediment control device during site construction phase. In addition, the Erosion and Sediment Control plan for the site shall clearly indicate how sediment entry will be prevented from entering the infiltration site. Normally, this is done by using diversion berms around the perimeter of the infiltration practice, along with immediate vegetative stabilization and/or mulching.

An observation well shall be installed in every infiltration trench, consisting of an anchored six-inch diameter perforated PVC pipe with a lockable cap installed flush with the ground surface.

Direct access shall be provided to infiltration practices for maintenance and rehabilitation. If a stone reservoir or perforated pipe is used to temporarily store runoff prior to infiltration, the practice shall not be covered by an impermeable surface.

Infiltration designs shall include dewatering methods in the event of failure. This can be accomplished with underdrain pipe systems that accomodate drawdown.

5.5 DESIGN CRITERIA – STORMWATER FILTERING SYSTEMS

Stormwater filtering system capture and temporarily store the WQ_v and pass it through a filter bed of sand, organic matter, soil or other media. Filtered runoff may be collected and returned to the conveyance system, or allowed to partially exfiltrate into the soil. Design variants include:

- Surface Sand Filter (Figure 5-16)
- Underground Sand Filter (Figure 5-17)
- Perimeter Sand Filter (Figure 5-18)
- Organic Filter (Figure 5-19)
- Bioretention (Figure 5-20)

Stormwater filtering systems may be used in private, commercial, and industrial subdivisions and developments to meet the WQ_v requirement. Filtering systems shall not be designed to provide the detention requirement. Filtering practices shall be combined with a separate facility to provide detention.

5.5.1 *Feasibility Criteria*

Most stormwater filters normally require two to six feet of head. The perimeter sand filter (Figure 5-18), however, can be designed to function with as little as one foot of head.

The maximum contributing area to an individual stormwater filtering system shall be less than 10 acres.

Sand and organic filtering systems are generally applied to land uses with a high percentage of impervious surfaces. Sites with imperviousness less than 75% shall require full sedimentation pretreatment techniques.

5.5.2 *Conveyance Criteria*

If runoff is delivered by a storm drain pipe or is along the main conveyance system, the filtering practice shall be designed off-line.

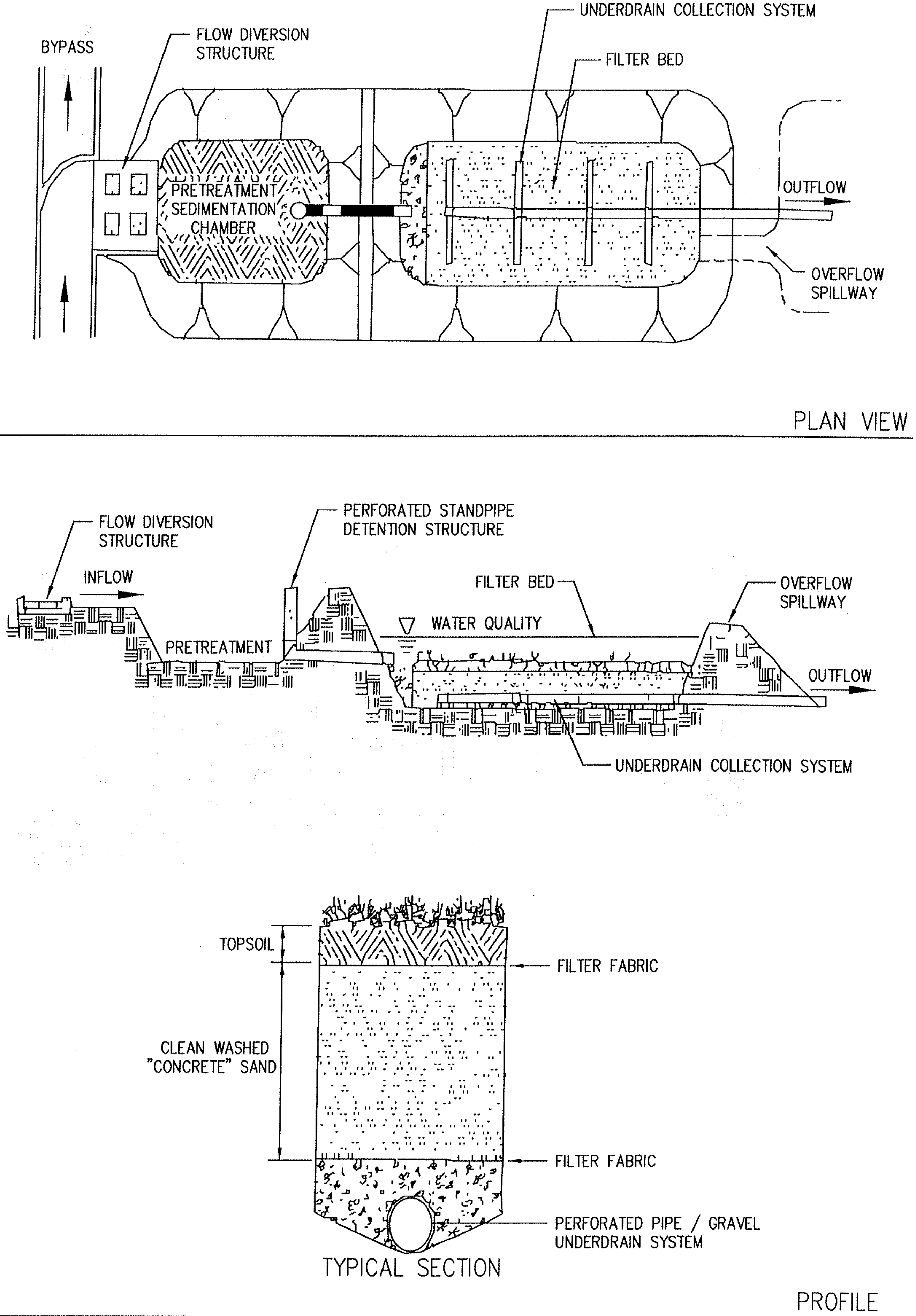


FIGURE 5–16. Surface Sand Filter

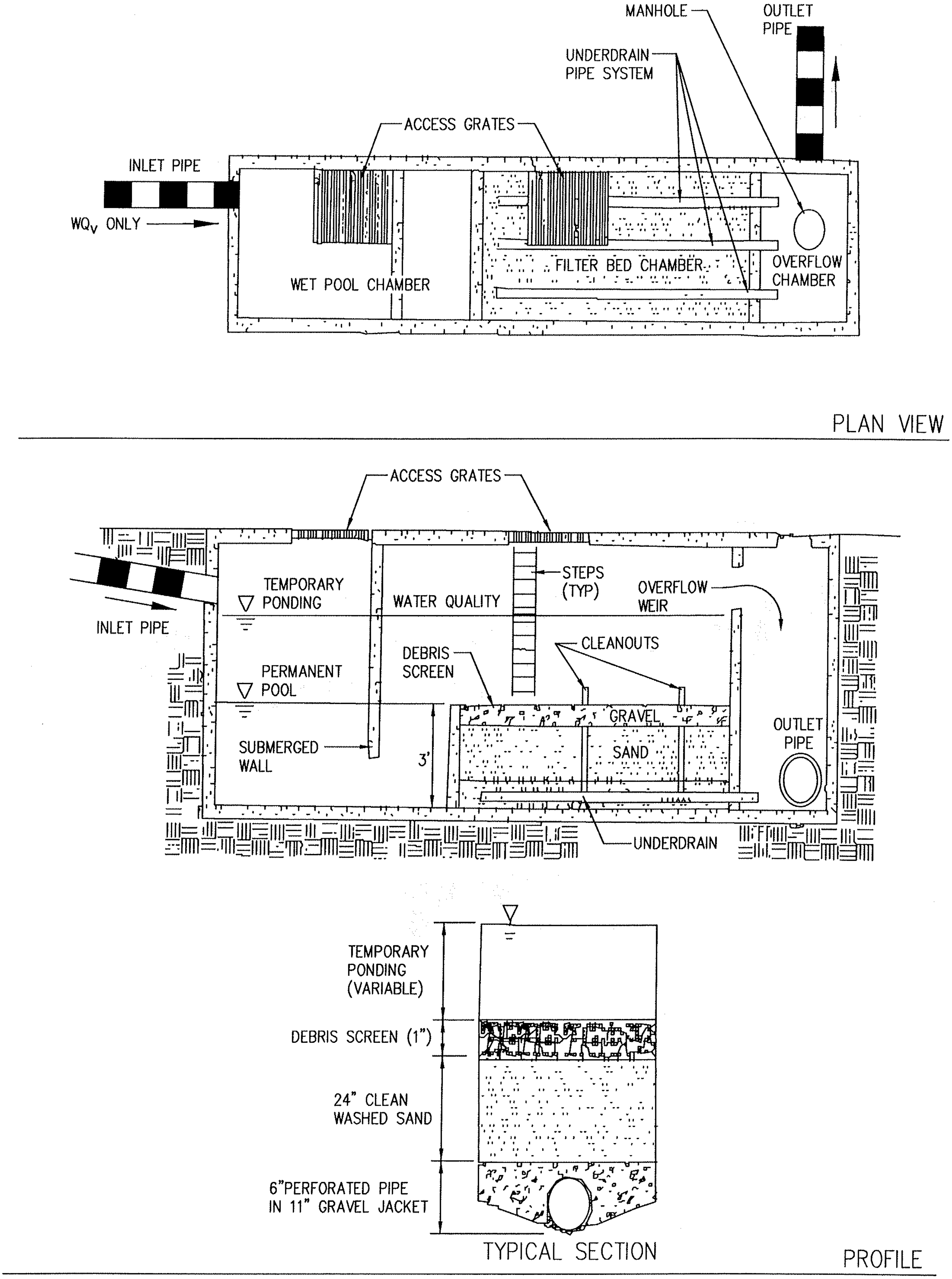


FIGURE 5-17. Underground Sand Filter

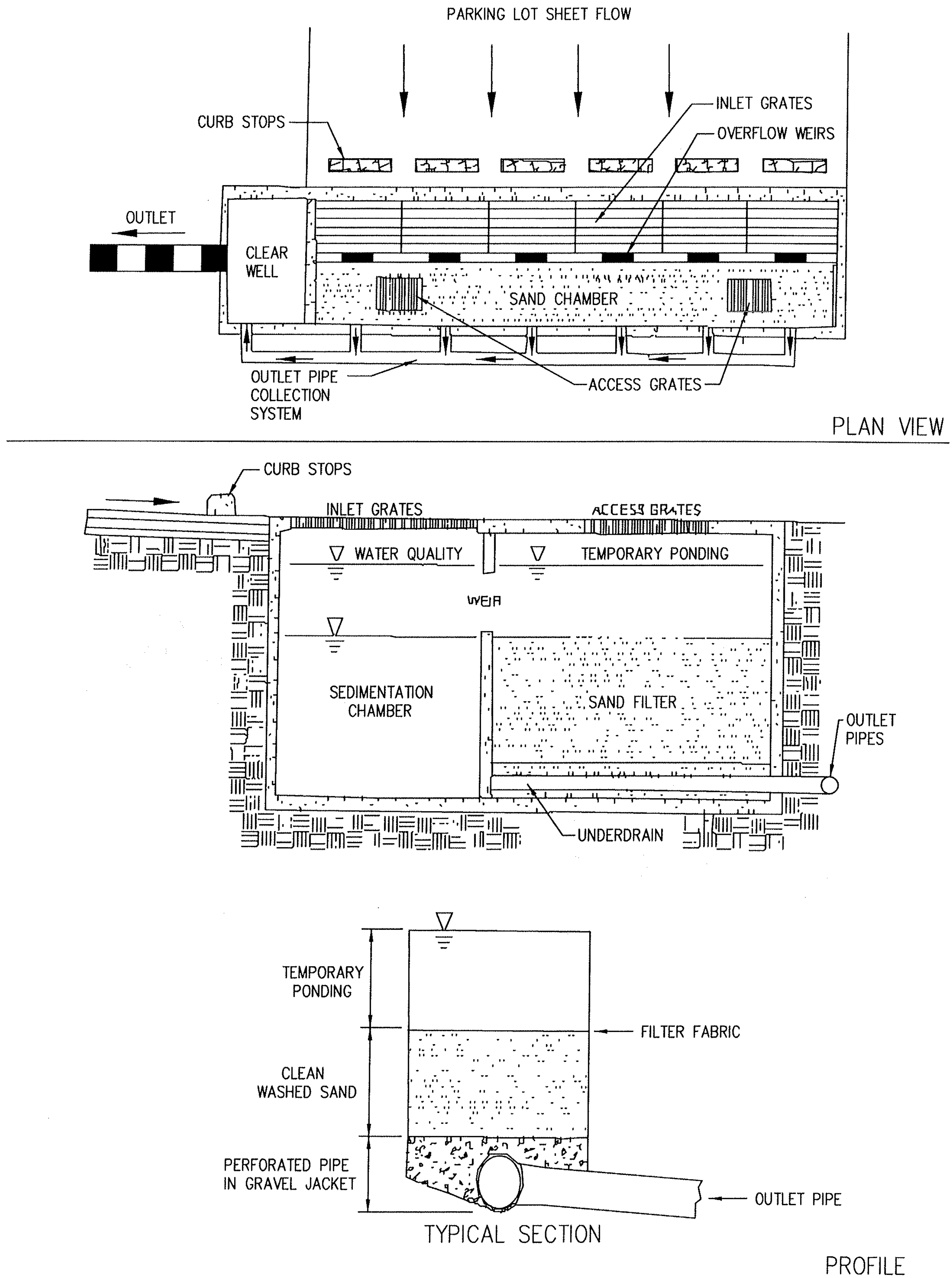


FIGURE 5–18. Perimeter Sand Filter

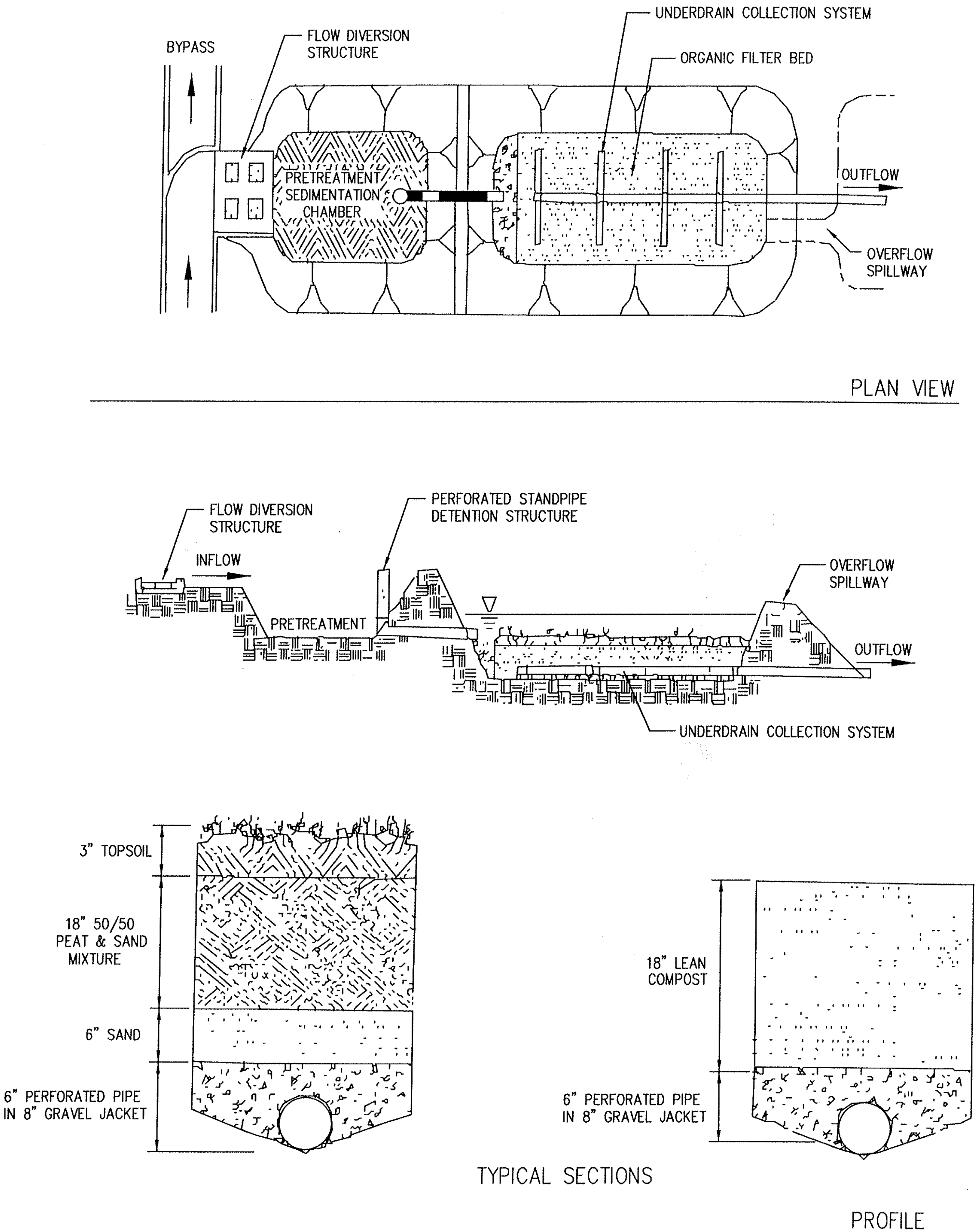


FIGURE 5-19. Organic Filter

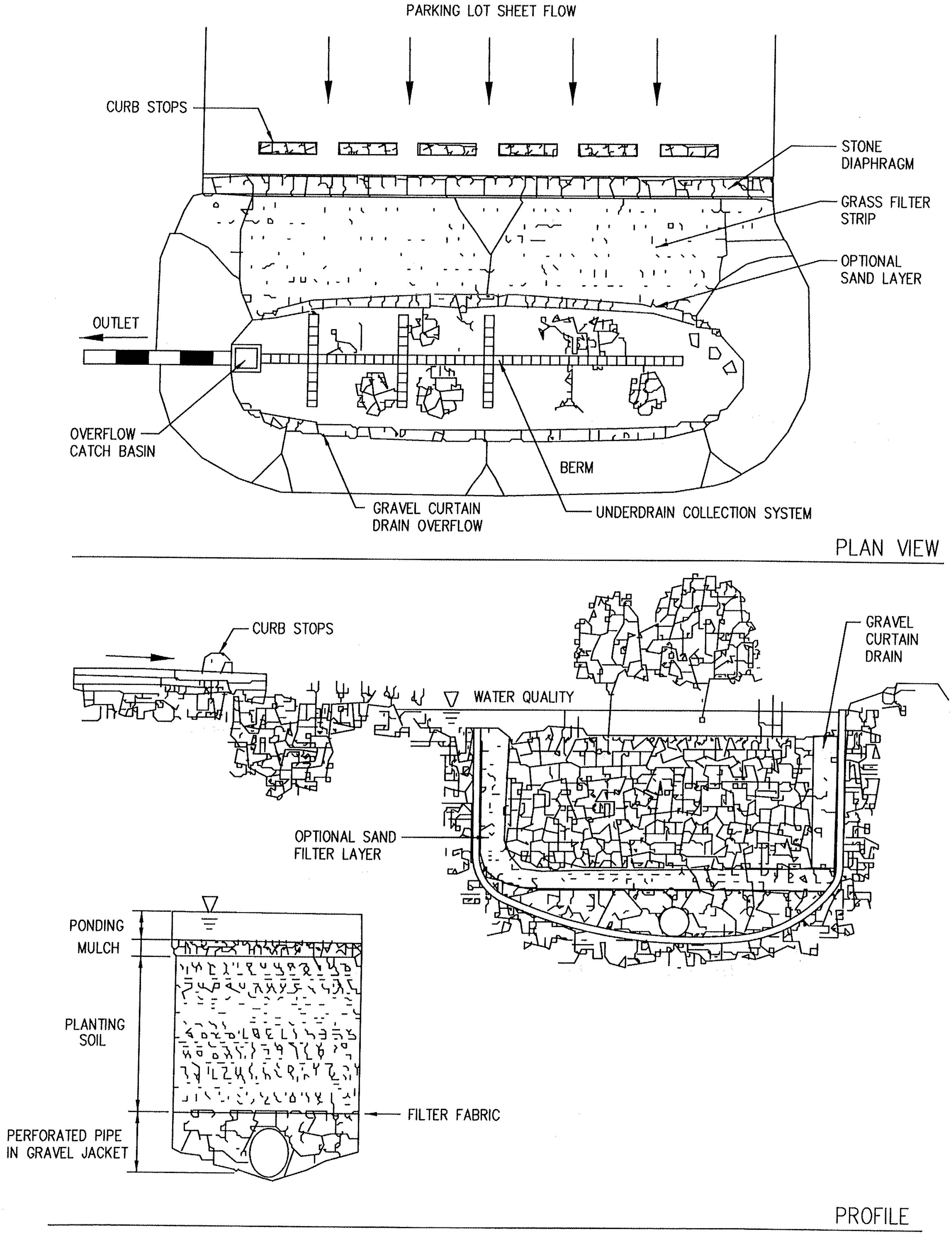


FIGURE 5-20. Bioretention

5.5.3 *Pretreatment Criteria*

Dry or wet pretreatment shall be provided prior to filter media equivalent to at least 25% of the computed WQ_V. The typical method is a sedimentation basin that has a length to width ratio of 2:1. The Camp-Hazen equation is used to compute the required surface area for sand and organic filters requiring full sedimentation for pretreatment *(9)* as follows:

The required sedimentation basin area is computed using the following equation:

$$A_s = (Q_o/W) = Ln\ (1\text{-}E) \tag{5.9}$$

Where:

A_s = Sedimentation basin surface area (ft^2)
E = sediment trap efficiency (use 90%)
W = particle settling velocity (ft/sec)
use 0.0004 ft/sec for imperviousness (I) 75%
use 0.0033 ft/sec for I > 75%
Q_o = Discharge rate from basin = (WQ_v/24 hr)

Equation reduces to:

$$A_s = (0.066)\ (WQ_v)\ ft^2 \text{ for I } 75\% \tag{5.10}$$

$$A_s = (0.0081)\ (WQ_v)\ ft^2 \text{ for I} > 75\% \tag{5.11}$$

Adequate pretreatment for bioretention systems is provided when all of the following are provided: (a) grass filter strip below a level spreader, (b) gravel diaphragm and (c) a mulch layer. In this regard, bioretention systems are fundamentally different from other filtering practices.

5.5.4 *Treatment Criteria*

The entire treatment system (including pretreatment) shall temporarily hold at least 75% of the WQ_v prior to filtration.

The filter media shall consist of a medium sand (meeting ASTM C-33 concrete sand). Media used for organic filters may consist of peat/sand mix or leaf compost. Peat shall be a reed-sedge hemic peat.

The filter bed shall have a minimum depth of 18" with the following exception: The perimeter filter may have a minimum filter bed depth of 12".

The filter area for sand and organic filters shall be sized based on the principles of Darcy's Law. A coefficient of permeability (k) shall be used as follows:

- Sand: 3.5 ft/day *(5)*
- Peat: 2.0 ft/day *(7)*
- Leaf compost: 8.7 ft/day *(6)*
- Bioretention Soil: 0.5 ft/day *(6)*

Bioretention systems shall consist of the following treatment components: A four foot deep planting soil bed, a surface mulch layer, and a 6" deep surface ponding area.

The required filter bed area is computed using the following equation

$$A_f = (WQ_v)\,(d_f) \,/\, [\,(k)\,(h_f + d_f)\,(t_f)] \qquad (5.12)$$

Where:

A_f = Surface area of filter bed (ft^2)
d_f = filter bed depth (ft)
k = coefficient of permeability of filter media (ft/day)
h_f = average height of water above filter bed (ft)
t_f = design filter bed drain time (days)
(1.67 days or 40 hours is maximum for sand filters, 48 hours for bioretention)

5.5.5 *Landscaping Criteria*

A dense and vigorous vegetative cover shall be established over the contributing pervious drainage areas before runoff can be accepted into the facility.

Surface filters can have a grass cover to aid in the pollutant adsorption. The grass should be capable of withstanding frequent periods of inundation and drought.

Landscaping is critical to the performance and function of bioretention areas. Therefore, a landscaping plan must be provided for bioretention areas.

Planting recommendations for bioretention facilities are as follows:

- Native plant species should be specified over non-native species.
- Vegetation should be selected based on a specified zone of hydric tolerance.
- A selection of trees with an understory of shrubs and herbaceous materials should be provided.
- Woody vegetation should not be specified at inflow locations.
- Trees should be planted primarily along the perimeter of the facility.

5.5.6 *Ownership of Stormwater Filtering Systems*

Ownership of stormwater filtering systems in commercial, industrial, private subdivisions, and non-residential areas shall be vested in the property owner.

Stormwater filtering systems may not be used in residential subdivisions.

5.5.7 *Maintenance of Stormwater Filtering Systems*

The maintenance responsibility for a stormwater filtering system shall be vested with a responsible party by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval or the permitting process.

Sediment should be cleaned out of the sedimentation chamber when it accumulates to a depth of more than six inches. Vegetation within the sedimentation chamber shall be limited to a height of 18 inches. The sediment chamber outlet devices shall be cleaned/repaired when drawdown times exceed 36 hours. Trash and debris shall be removed as necessary.

Silt/sediment shall be removed from the filter bed when the accumulation exceeds one inch. When the filtering capacity of the filter diminishes substantially (i.e., when water ponds on the surface of the filter bed for more than 48 hours), the top few inches of discolored material shall be removed and shall be replaced with fresh material. The removed sediments should be disposed in an acceptable manner.

A stone drop of at least six inches shall be provided at the inlet of bioretention facilities (Figure 5-19) (pea gravel diaphragm). Areas devoid of mulch should be re-mulched on an annual basis. Dead or diseased plant material shall be replaced.

Direct maintenance access shall be provided to the pretreatment area and the filter bed.

5.6 DESIGN CRITERIA – OPEN CHANNEL SYSTEMS

Open channel systems are vegetated open channels that are explicitly designed to capture and treat the full WQ_v within dry or wet cells formed by checkdams or other means. Design variants include:

- Dry Swale (Figure 5-21)
- Wet Swale (Figure 5-22)
- Grass Channels (Figure 5-23)

Dry swales and grass channels may be used in residential, private, commercial, and industrial subdivisions and developments to meet the WQ_v requirement. Wet swales may only be used in private, commercial, and industrial subdivisions and developments. Open channel systems shall not be designed to provide the detention requirement. Open channel systems shall be combined with a separate facility to provide detention.

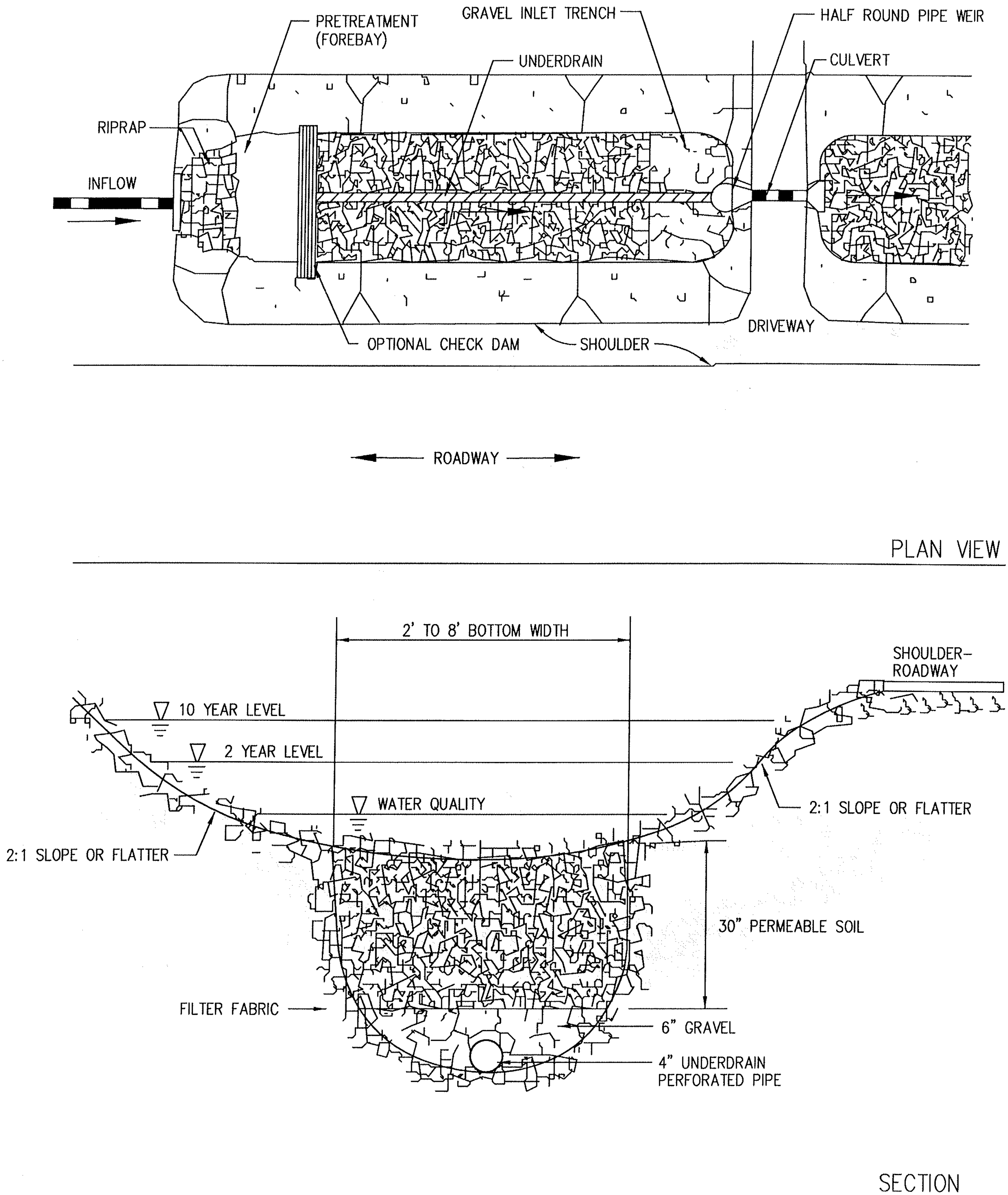


FIGURE 5-21. Dry Swale

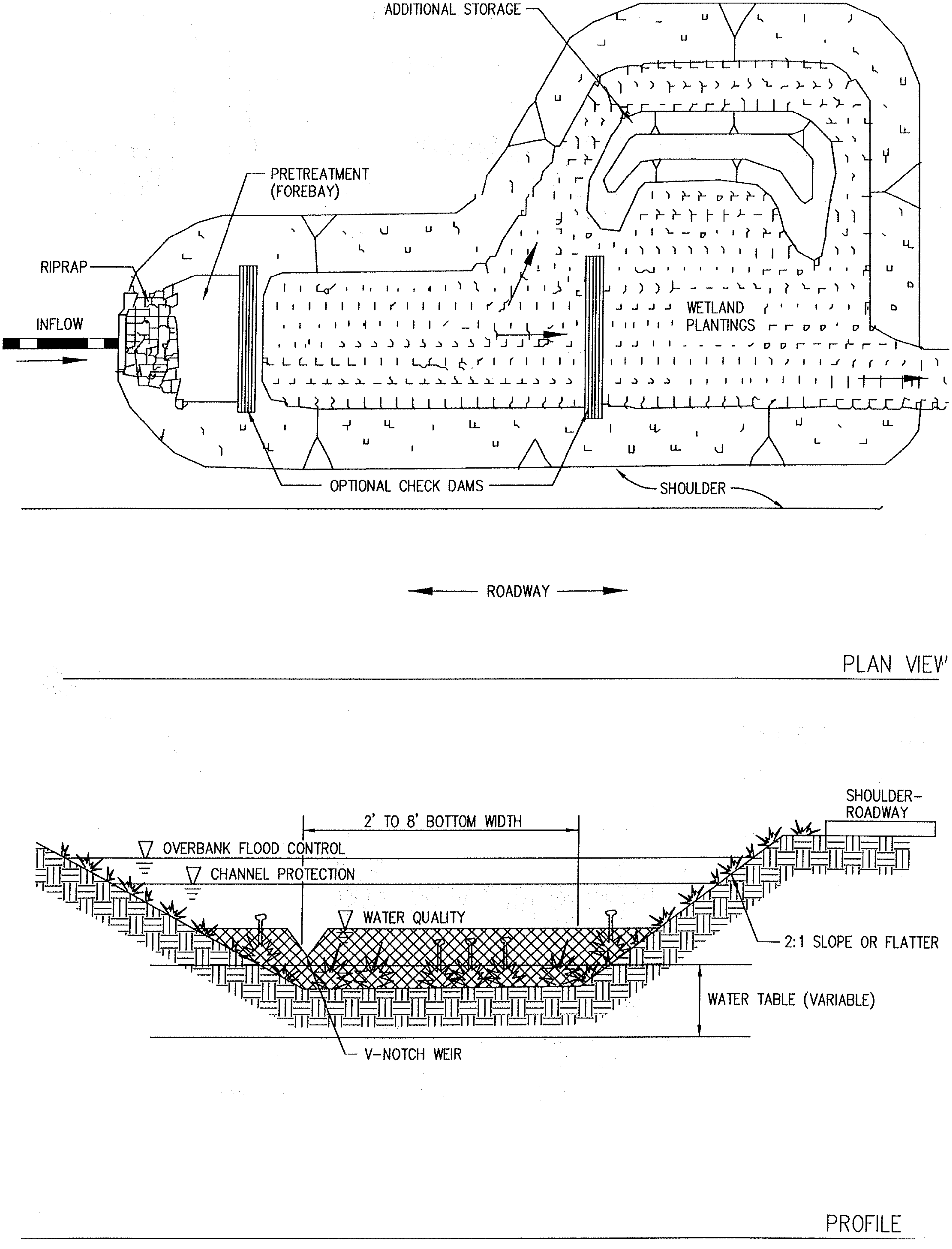


FIGURE 5-22. Wet Swale

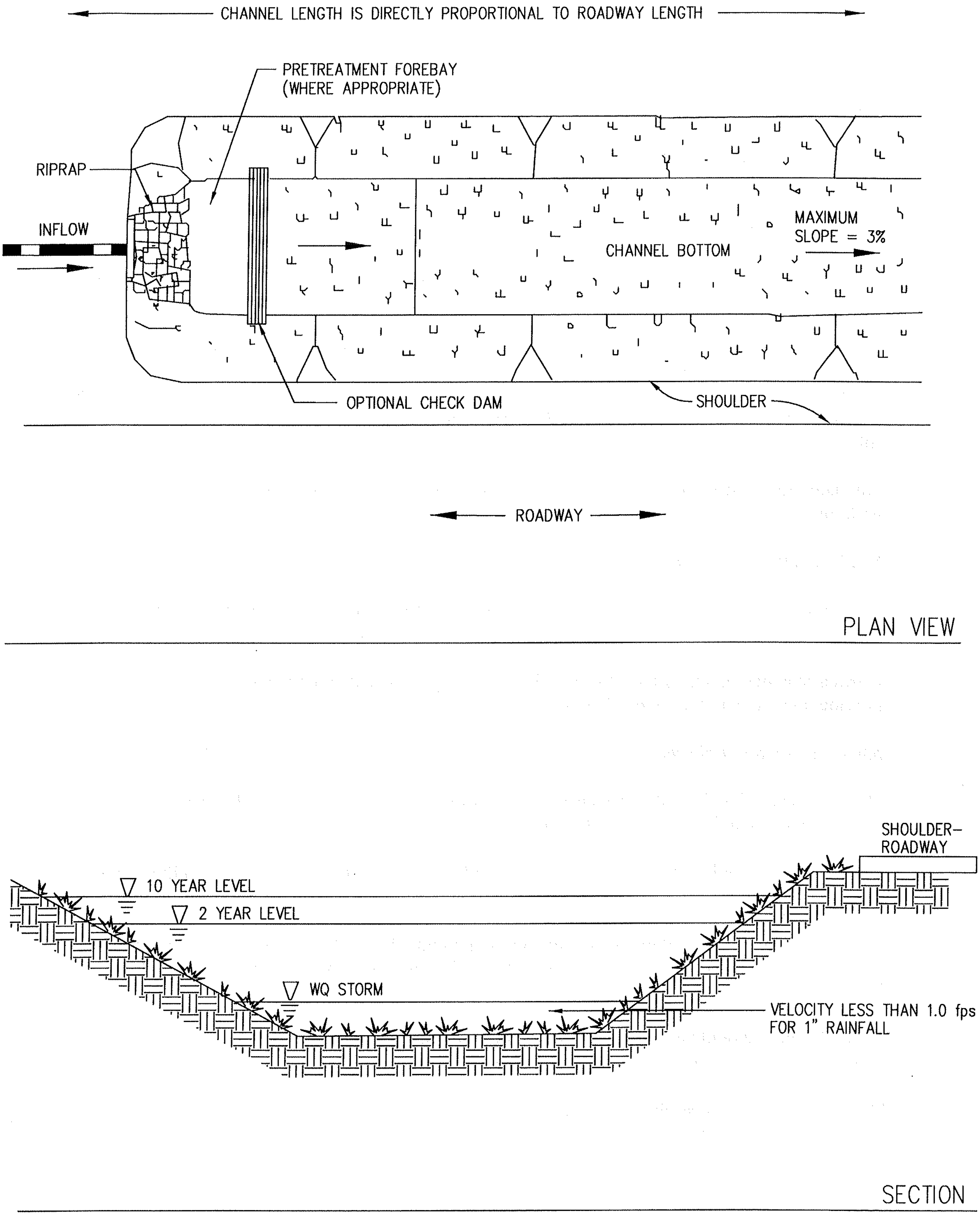


FIGURE 5-23. Grass Channels

5.6.2 *Conveyance Criteria*

The peak velocity for the 2 year storm must be non-erosive.

Open channels shall be designed to safely convey the ten year storm with a minimum of one (1.0') foot of freeboard.

Channels shall be designed with moderate side slopes for most conditions. Side slopes shall not be steeper than 4:1.

The maximum allowable temporary ponding time within a channel shall be less than 48 hours.

Open channel systems which directly receive runoff from impervious surfaces shall have a 6 inch drop onto a protected shelf (pea gravel diaphragm) to minimize the clogging potential of the inlet.

An underdrain system shall be provided for the dry swale to ensure a maximum ponding time of 48 hours.

5.6.3 *Pretreatment Criteria*

Pretreatment of 0.1 inch of runoff per impervious acre storage shall be provided. This storage is usually obtained by providing checkdams at pipe inlets and/or driveway crossings.

A pea gravel diaphragm and gentle side slopes shall be provided along the top of channels to provide pretreatment for lateral sheet flows.

5.6.4 *Treatment Criteria*

Dry and wet swales should be designed to temporarily store the WQ_v within the facility to be released over a maximum 48 hour duration.

Open channels should have a bottom width no wider than 8 feet to avoid potential gullying and channel braiding.

Dry and wet swales should maintain a maximum ponding depth of one foot at the "mid-point" of the channel, and a maximum depth of 18" at the end point of the channel (for storage of the WQ_v).

Grass channels should be designed to retain the water quality volume in the practice for a minimum of 10 minutes, with no greater than a 1.0 fps velocity.

Please note that the grass channel design is the only practice with a "rate-based" design. The designer determines the peak flow rate from the water quality storm event, and then uses

Manning's equation to ensure that the velocity required to retain flow can be achieved with the channel's cross section and slope.

5.6.5 *Landscaping Criteria*

Wet swales shall not be used for residential developments as they can create potential nuisance or mosquito breeding conditions.

Landscape design shall specify proper grass species and wetland plants based on specific site, soils and hydric conditions present along the channel.

5.6.6 *Ownership of Open Channel Systems*

Ownership of dry swales and grass channels in residential subdivisions accepted by the City shall be vested in the City of Fort Smith with the filing of the final plat. The Developer shall warrant the operation of the drainage system for 2 years after acceptance by the City by a Maintenance Bond provided by the Developer's Contractor or the Developer. The bond shall be required to be extended until 2 years after all phases of the subdivision or development that substantially drain to the dry swale or grass channel are completed.

Ownership of dry swales, grass channels, and wet swales in commercial, industrial, private subdivisions, and non-residential areas shall be vested in the property owner.

Wet swales may not be used in residential subdivisions.

5.6.7 *Maintenance of Open Channel Systems*

When ownership of an open channel system is not vested in the City of Fort Smith, the maintenance responsibility for the system shall be vested with a responsible party by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval or the permitting process.

Open channel systems and grass filter strips should be mowed as required during the growing season to maintain grass heights in the 4 to 6 inches range. Wet swales, employing wetland vegetation, do not require frequent mowing of the channel.

Sediment build-up within the bottom of the channel or filter strip should be removed when 25% of the original WQ_v volume has been exceeded.

5.7 DESIGN CRITERIA – SUBSTANDARD STP'S

Substandard STP's are not considered "stand alone" practices for stormwater treatment, and therefore, the acceptable STP's listed above must be considered first. However, substandard STP's may be used as pretreatment for one of the acceptable STP methodologies listed previously.

Site difficulties may prevent the use of acceptable STP's for treatment, especially with redevelopment projects. When site difficulties prevent the use of the acceptable STP's, combinations of substandard STP's may be utilized to form a "treatment train." This "treatment train" must be able to remove at least 80% of the TSS. Where appropriate, data must be submitted from the manufacturers of substandard STP's documenting the performance capabilities of the structures.

5.7.1 *Dry Extended Detention Ponds*

All of the pond criteria presented in 5.1 GENERAL and 5.2 DESIGN CRITERIA – STORMWATER PONDS also apply to the design of dry extended detention ponds.

5.7.2 *Deep Sump Catch Basins*

The sump shall be no shallower than 24 inches below the invert of the outlet pipe. The deep sump catch basin immediately upstream of the storm drain outfall must also have a hood inside the basin attached to the outlet.

5.7.3 *Other Substandard STP's*

Other substandard STP's shall be designed according to current engineering practice and according to the manufacturers' recommendations, as applicable. All accompanying data and calculations documenting reasonableness of design shall be submitted to the Engineering Department for review and approval.

5.7.4 *Ownership of Substandard STP's*

5.7.4.1 Ownership of Dry Extended Detention Ponds and Deep Sump Catch Basins

Ownership of dry extended detention ponds and deep sump catch basins in residential subdivisions accepted by the City shall be vested in the City of Fort Smith with the filing of the final plat. The Developer shall warrant the operation of the drainage system for 2 years after acceptance by the City by a Maintenance Bond provided by the Developer's Contractor or the Developer. The bond shall be required to be extended until 2 years after all phases of the subdivision or development that substantially drain to the dry extended detention pond or deep sump catch basin are completed.

Ownership of dry extended detention ponds and deep sump catch basins in commercial, industrial, private subdivisions, and non-residential areas shall be vested in the property owner.

5.7.4.2 Ownership of Other Substandard STP's

Public ownership of other substandard STP's within residential subdivisions shall be considered on a "case by case" basis by the Engineering Department. If approved, ownership shall be vested in the City of Fort Smith with the filing of the final plat. The Developer shall warrant the operation of the drainage system for 2 years after acceptance by the City by a Maintenance Bond provided by the Developer's Contractor or the Developer. The bond shall be required to be extended until 2 years after all phases of the subdivision or development that substantially drain to the STP are completed. If an STP is not approved for public ownership, it may not be used in a residential subdivision.

Ownership of other substandard STP's in commercial, industrial, private subdivisions, and non-residential areas shall be vested in the property owner.

5.7.5 *Maintenance of Substandard STP's*

5.7.5.1 Maintenance of Dry Extended Detention Basins

Dry extended detention basins shall be required to meet all the maintenance requirements found in Section 5.9.4 *Maintenance of Stormwater Ponds.*

5.7.5.2 Maintenance of Other Substandard STP's

When ownership of a substandard STP is not vested in the City of Fort Smith, the maintenance responsibility for the STP shall be vested with a responsible party by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval or the permitting process.

Maintenance requirements for substandard STP's shall be in accordance with manufacturer's recommendations or specifications established by design engineer if the manufacturer's recommendations are unavailable.

5.8 STP SCREENING MATRICES

This section presents matrices that can be used as a screening process for selecting the best STP or group of STPs for a development site. The matrices presented can be used to screen practices in a step-wise fashion. Screening factors include:

- Land Use
- Stormwater Management Capability
- Pollutant Removal

5.8.1 *Land Use*

This matrix (see Figure 5-24) allows the designer to make an initial screen of practices most appropriate for a given land use.

Rural. This column identifies STPs that are best suited to treat runoff in rural or very low density areas.

Residential. This column identifies the best treatment options in medium to high density residential developments.

Roads and Highways. This column identifies the best practices to treat runoff from major roadways and highway systems.

Commercial Development. This column identifies practices that are suitable for new commercial development

Hotspot Land Uses. This last column examines the capability of an STP to treat runoff from designated hotspots. An STP that receives hotspot runoff may have design restrictions, as noted.

Ultra-Urban Sites. This column identifies STPs that work well in the ultra-urban environment, where space is limited and original soils have been disturbed. These STPs are frequently used at redevelopment sites.

5.8.2 *Stormwater Management Capability*

This matrix (see Figure 5-25) examines the capability of each STP option to meet stormwater management criteria. It shows whether an STP can meet requirements for:

Water Quality. The matrix tells whether each practice can be used to provide water quality treatment effectively. For more detail, consult the Pollutant Removal matrix in section 5.8.3.

Recharge. The matrix indicates whether each practice can provide groundwater recharge, however, it should be noted that groundwater recharge is not a requirement.

Channel Protection. The matrix indicates whether the STP can typically provide channel protection storage, however, it should be noted that channel protection is not a requirement.

Quantity Control The matrix shows whether an STP can typically meet the overbank flooding criteria for the site. Again, the finding that a particular STP cannot meet the requirement does not necessarily mean that it should be eliminated from consideration, but rather is a reminder that more than one practice may be needed at a site (e.g., a bioretention area and a downstream stormwater detention pond).

FIGURE 5-24. STP Selection Matrix, Land Use.

STP GROUP	STP DESIGN	Rural	Residential	Roads and Highways	Commercial / High Density	Hotspots	Ultra Urban
Pond	Micropool ED	○	○	○	◗	①	●
	Wet Pond	○	○	○	◗	①	●
	Wet ED Pond	○	○	○	◗	①	●
	Multiple Pond	○	○	◗	◗	①	●
	Pocket Pond	○	◗	○	◗	●	●
Wetland	Shallow Marsh	○	○	◗	◗	①	●
	ED Wetland	○	○	◗	◗	①	●
	Pond/Wetland	○	○	●	◗	①	●
	Pocket Marsh	○	◗	○	◗	●	●
Infiltration	Infiltration Trench	◗	◗	○	○	●	◗
	Shallow T-Basin	◗	◗	◗	◗	●	◗
Filters	Surface Sand	●	◗	○	○	②	○
	Underground SF	●	●	◗	○	○	○
	Perimeter SF	●	●	◗	○	○	○
	Organic SF	●	◗	○	○	②	○
	Pocket Sand Filter	●	◗	○	○	②	○
	Bioretention	◗	◗	○	○	②	○
Open Channels	Dry Swale	○	◗	○	◗	②	◗
	Wet Swale	○	●	○	●	●	●
	Grass Channel	○	◗	○	◗	●	◗

○ Yes. Good option in most cases.

◗ Depends. Suitable under certain conditions, or may be used to treat a portion at the site.

● No. Seldom or never suitable.

① Acceptable option, but may require a pond liner to reduce risk of groundwater contamination.

② Acceptable option, if not designed as an exfilter.

Note: Infiltration practices, filtering practices, and wet swales may not be used in residential subdivsions or developments.

FIGURE 5-25. STP Selection Matrix, Stormwater Management Capability.

STP GROUP	STP DESIGN	WATER QUALITY?	RECHARGE?	CHANNEL PROTECTION?	FLOOD CONTROL?
Pond	Micropool ED	○	●	○	○
	Wet Pond	○	●	○	○
	Wet ED Pond	○	●	○	○
	Multiple Pond	○	●	○	○
	Pocket Pond	○	●	○	○
Wetland	Shallow Marsh	○	●	○	○
	ED Wetland	○	●	○	○
	Pond/Wetland	○	●	○	○
	Pocket Marsh	○	●	○	❷
Infiltration	Infiltration Trench	○	○	❷	❸
	Shallow I-Basin	○	○	❷	❸
Filters	Surface Sand	○	❶	❷	●
	Underground SF	○	●	●	●
	Perimeter SF	○	●	●	●
	Organic SF	○	❶	●	●
	Pocket Sand Filter	○	❶	●	●
	Bioretention	○	❶	❷	●
Open Channels	Dry Swale	○	❶	●	●
	Wet Swale	○	●	●	●
	Grass Channel	❷	❷	●	●

○ Practice generally meets this stormwater management goal.

● Practice can almost never be used to meet this goal.

❶ Provides recharge only if designed as an exfilter system.

❷ Practice may partially meet this goal, or under specific site and design conditions.

❸ Can be used to meet flood control in rare conditions, with very cobbly or highly infiltrative soils.

Note: Only stormwater ponds and wetlands may be used in residential subdivisions or developments for flood control. Only stormwater ponds, wetlands, dry swales, and grass channels may be used in residential subdivisions or developments for water quality.

5.8.3 *Pollutant Removal*

This matrix (see Table 5-2) examines the capability of each STP option to remove specific pollutants from stormwater runoff. The matrix includes data for:

- Total Suspended Solids
- Total Phosphorous
- Total Nitrogen
- Metals
- Bacteria

TABLE 5-2. STP Selection Matrix, Pollutant Removal Efficiencies.

STP Selection Matrix. Pollutant Removal (Acceptable STP's)					
STP Group	**TSS**	**TP**	**TN**	**Metals**[1]	**Bacteria**
Ponds	80	51	33	62	70
Wetlands	76	49	30	42	78[2]
Filters[3]	86	59	38	69	37[2]
Infiltration	95[2]	70	51	99[2]	N/A
Open Channels [4]	81	34[2]	84[2,5]	61	-25[2]
(Sub-Standard STP's)					
STP Group	**TSS**	**TP**	**TN**	**Metals**[1]	**Bacteria**
Dry Extended Detention Ponds	61	19	31	26-54	N/A
Deep Sump Catch Basins	32	N/A	N/A	N/A	N/A
Water Quality Inlets[7]	35	5	20	5	N/A
Hydrodynamic Structures[7]	21	17	5[6]	17	N/A
Filter Strips (75 ft width)	54	-25	-27[6]	47	N/A
Filter Strips (150 ft width)	84	40	20[6]	55	N/A

1: Average of zinc and copper. Zinc only for infiltration and sub-standard STP's.
2: Based on fewer than five data points.
3: Excludes vertical sand filters and filter strips.
4: Highest removal rates for dry swales

5: No data available for grass channels
6: Nitrate + Nitrite
7: Percentages will vary. Refer to manufacturer for specific removal percentages.
N/A: Not applicable. Data not available

5.9 STORMWATER CREDITS

The purpose of the stormwater credit system is to provide incentive to developers, engineers, and builders to implement better site design and locate new development in a manner that causes less impact to aquatic resources. By taking advantage of the credit system, developers and builders can reduce the stormwater management quality requirements. The credit system directly translates into cost savings to the developer by reducing the water quality volume that has to be captured and treated.

This section presents two broad types of credits: Site Design Credits and Watershed Credits. Site design credits act as incentives to encourage *Better Site Design* techniques by reducing required water quality volumes on site. Watershed credits are reductions or exemptions from stormwater management requirements to support watershed goals such as redevelopment or watershed zoning.

5.9.1 *Site Design Credits*

Site design credits allow developers to reduce or eliminate requirements for *Water Quality* in exchange for implementation of these non-structural site design elements. The credits are calculated as volumes that are based on the fraction of the total site area or site impervious area affected by the credit.

Specific design credits detailed in this section include the following:

- Conservation of Natural Areas
- Reforestation
- Rooftop Disconnection
- Non-Rooftop Disconnection
- Green Rooftops

5.9.1.1 Conservation of Natural Areas

This stormwater credit rewards protection of natural vegetation or critical resource areas on site. This credit may be given when natural areas are conserved at development sites, thereby retaining their pre development hydrologic and water quality characteristics. Examples of natural area conservation areas include:

- forest retention areas
- jurisdictional wetlands

- other lands in protective easement (floodplains, open space, steep slopes)

Under the credit, a designer can subtract conservation areas from total site area when computing the water quality volume.

The credit for the water quality volume can be based on the site area in natural conservation, such that:

$$C_{WQ} = (A_{NA}/A)(WQ_v) \tag{5.13}$$

Where:

C_{WQ} = Natural Area Credit for Water Quality (ac-ft)
A_{NA} = Natural Conservation Area (acres)
A = Total Site Area (acres)
WQ_v = Original Water Quality Volume (ac-ft)

The water quality volume can then be reduced by the value of C_{WQ}. The example in Figure 5-26 illustrates how this credit would be applied.

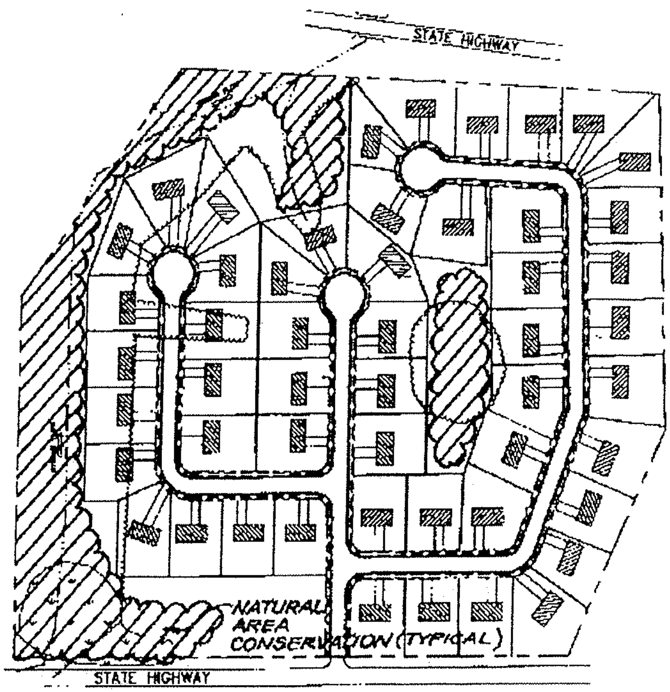
5.9.1.2 Reforestation

This credit is similar to the credit for *Conservation of Natural Areas*, except that it rewards active reforestation, rather than preservation of existing forest. This credit can apply to both *reforestation* and *afforestation.* The credit for afforestation shall be weighted higher because the afforestation implies a net increase of forest cover on the site, while reforestation only compensates for trees cleared on site.

A reforestation credit shall be applied where tree planting is used to supplement existing tree cover, or to compensate for forest cleared during development. The areas in reforestation and afforestation can be applied to water quality volumes.

In order to receive credit, the following criteria must be met:

- Tree species used for afforestation or reforestation shall be native to the City of Fort Smith, and selected from a list of approved species established by the Parks Department.
- Reforestation shall be guaranteed with a performance bond, letter of credit, or similar surety measure. The bond shall be returned after two successful growing seasons.
- Plantings shall be from nursery stock, at a minimum of 1.5" diameter at chest height.

Natural Area Conservation Credit Example	
Site Data: 51 Single Family Lots Area = 38 ac Conservation Area = 7.0 ac Original WQ_v = 1.08 ac-ft	

Computation of Stormwater Credits

C_{WQ} = (7.0 / 38)(1.08 ac-ft) = 0.20 ac-ft

WQ_v = 1.08 ac-ft – 0.20 ac-ft = 0.88 ac-ft

FIGURE 5-26. Example, Natural Area Conservation Credit.

The credit for the water quality volume can be expressed based on the area in reforestation and afforestation, such that:

$$C_{WQ} = (1.5A_A+0.5A_R)/A)(WQ_v) \tag{5.14}$$

Where:

C_{WQ} = Reforestation Credit for Water Quality (ac-ft)
A_A = Afforestation Area (acres)
A_R = Reforestation Area (acres)
A = Total Site Area (acres)
WQ_v = Original Water Quality Volume (ac-ft)

The water quality volume can then be reduced by the value of C_{WQ}. The example in Figure 5-27 illustrates how this credit would be applied.

5.9.1.3 Rooftop Disconnection

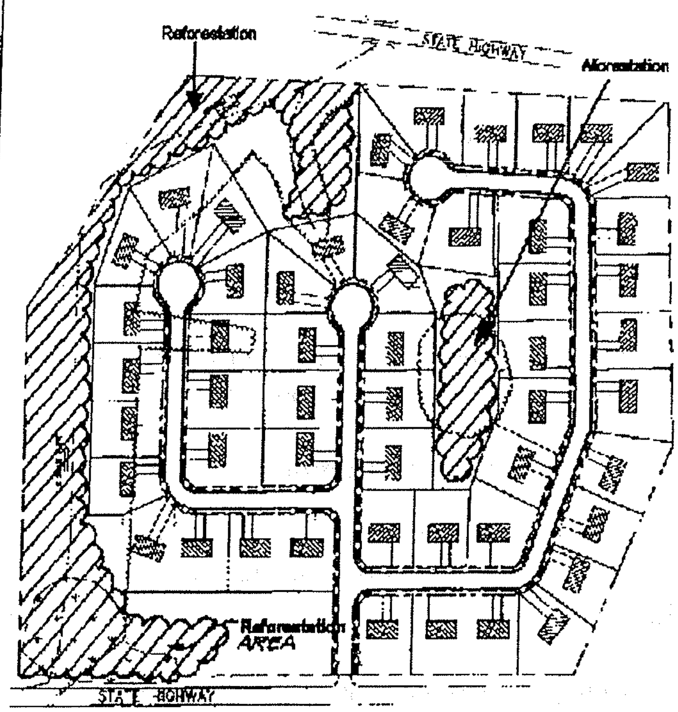
This credit can be applied to encourage disconnection of rooftops, thus promoting overland treatment of these surfaces. Credits can be applied to water quality requirements. In order to receive the credit, disconnections must meet the following criteria:

- The rooftop cannot be a designated hotspot.
- Disconnection must ensure no basement seepage.
- The contributing length of rooftop to a discharge location shall be 75 feet or less.
- The rooftop contributing area shall be no more than 1,000 sq. feet per disconnection.
- The length of the "disconnection" shall be equal to or greater than the contributing rooftop length.
- Disconnections will only be credited for residential lot sizes greater than 6000 sq. ft.
- The entire vegetative "disconnection" shall be on a slope less than or equal to 3.0%.
- The disconnection must drain continuously through a vegetated channel, swale, or through a filter strip to the property line or STP.
- Downspouts must be at least 10 feet away from the nearest impervious surface to discourage "re-connections."
- Disconnections are encouraged on relatively permeable soils (HSGs A and B) without soil testing.
- In less permeable soils (HSGs C and D), the water table and permeability shall be tested by a geotechnical engineer to determine if a spreading device is needed to provide sheetflow over grass surfaces. In some cases, dry wells, french drains or other temporary underground storage devices may be needed to compensate for a poor infiltration capability.
- For those rooftops draining directly to a stream buffer, one can only use either the rooftop disconnection credit or the stream buffer credit, not both.

Reforestation Credit Example

Site Data: 51 Single Family Lots
Area = 38 ac
Reforestation Area = 5.0 ac
Afforestation Area = 2.0 ac
Impervious Area = 13.8 ac

Original WQ_v = 1.08 ac-ft



Computation of Stormwater Credits

C_{WQ} = [((1.5 x 2.0 ac) + (0.5 x 5.0 ac)) / 38ac] x 1.08 ac-ft = 0.16 ac-ft

WQ_v = 1.08 ac-ft – 0.16 ac-ft = 0.92 ac-ft

FIGURE 5-27. Example, Reforestation Credit.

The water quality credit can be calculated with the following equation:

$$C = (A_{DR}/A_I)WQ_v \tag{5.15}$$

Where:

C = Rooftop Disconnection Credit (ac-ft)
A_{DR} = Disconnected Roof Area (acres)
A_I = Site Impervious Area (acres)
WQ_v = Original Water Quality Volume.

The water quality volume would both be reduced by the credit (C). The example in Figure 5-28 illustrates how this credit would be applied.

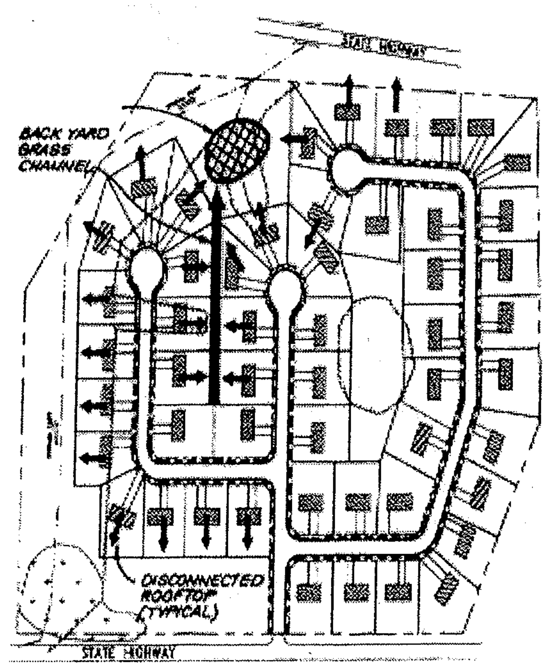
FIGURE 5-28. Example, Rooftop Disconnection Credit.

<table>
<tr><th colspan="2">Rooftop Disconnection Credit Example</th></tr>
<tr><td>Site Data: 51 Single Family Lots
Area = 38 ac
Original Impervious Area = 13.80 ac

# of Disconnected Rooftops = 22
Original WQ_v = 1.08 ac-ft

22 Lots Disconnected
(2500 sq ft each)

Net impervious area reduction

(22)(2500) / (43560) = 1.3 ac</td><td>
</td></tr>
<tr><td colspan="2">Computation of Stormwater Credits

C = (1.3 / 13.8)(1.08 ac-ft) = 0.10 ac-ft

WQ_v = 1.08 ac-ft – 0.10 ac-ft = 0.98 ac-ft</td></tr>
</table>

5.9.1.4 Non-Rooftop Disconnection

This credit is applied to credit disconnection of other impervious surfaces by encouraging drainage to overland treatment such as swales or filter strips. In order to receive the credit, disconnections must meet the following criteria:

- The maximum contributing impervious flow path length shall be 75 feet.
- Runoff cannot come from a designated hotspot.
- The disconnection must drain continuously through a vegetated channel, swale, or filter strip to the property line or STP.
- The length of the "disconnection" must be equal to or greater than the contributing length.
- The entire vegetative "disconnection" shall be on a slope less than or equal to 3.0%.
- The surface imperviousness area to any one discharge location cannot exceed 1,000 ft^2.
- Disconnections discharging over relatively permeable soils (HSGs A and B) do not require geotechnical testing.
- If the site has less impermeable soils (HSGs C and D), testing by a geotechnical engineer is needed to determine if a spreading device, such as a french drain, gravel trench or other temporary storage device is needed to compensate for poor infiltration capability.

The water quality credit can be calculated with the following equation:

$$C = (A_D/A_I)WQ_v \qquad (5.16)$$

Where:

C = Non-Rooftop Credit (ac-ft)
A_D = Disconnected Impervious Area (acres)
A = Total site area (acres)
A_I = Site Impervious Area (acres)
WQ_v = Original Water Quality Volume.

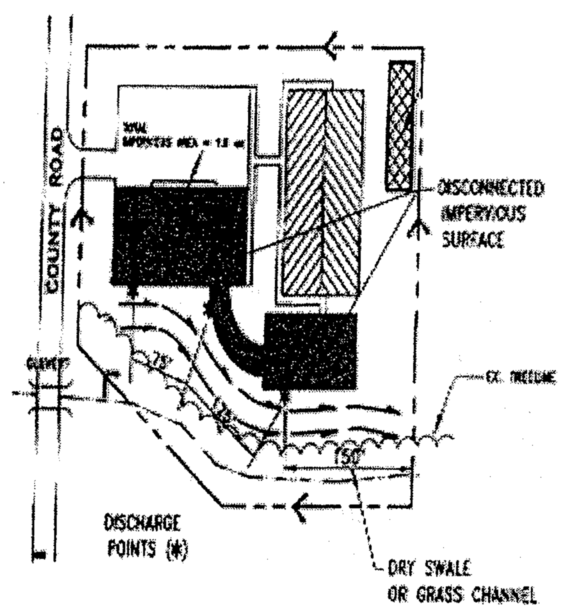
The water quality volume can then be reduced by the credit (C). The example in Figure 5-29 how this credit would be applied.

Non-Rooftop Disconnection Credit Example

Site Data: Community Center
Area = 3.0 ac
Original Impervious
Area = 1.9 ac = 63.3%

Original WQ_v = 6,752 ft^3

0.8 ac of surface imperviousness disconnected



Computation of Stormwater Credits

$C = (0.8 / 1.9)(6{,}752\ ft^3) = 2{,}843\ ft^3$

$WQ_v = 6{,}752\ ft^3 - 2{,}843\ ft^3 = 3{,}909\ ft^3$

FIGURE 5-29. Example, Non-Rooftop Disconnection Credit.

5.9.1.5 Green Rooftop Credit

The term "green rooftops" refers to a few practices that detain and treat stormwater runoff on rooftops using vegetation on the roof surface. Several different options exist, including variations on the type of vegetation used, and the specific design of the green roof. The criteria presented below are adapted from the Portland Stormwater Manual criteria for the Eco-Roof. In order to receive the credit, green rooftops must meet the following criteria:

- The system shall include a 6" soil bed, with a silt loam texture.
- The soil bed shall be underlain with a 2" gravel layer, and these two layers shall be separated by a layer of filter fabric.
- An impermeable layer shall be placed between the rooftop and the gravel layer.
- The roof shall have a maximum slope of 25%
- The roof shall be designed to hold an additional 25 lbs/sf, beyond minimum regional design criteria
- Vegetation shall be established within two growing seasons.
- Vegetation should require minimal fertilization, watering and pesticides.
- A 2" mulch layer shall be immediately placed above the soil layer to prevent erosion.
- The vegetation and mulch layer shall be maintained at least quarterly, removing dead vegetation and eroded mulch.
- If the rooftop is used as an amenity (e.g., a rooftop sitting area) as well as to detain stormwater, credit shall only be applied to pervious sections of the rooftop.
- The credit shall only apply for businesses where owners sign a maintenance agreement.

The water quality credit can be calculated with the following equation:

$$C = (A_{GR}/A_I)WQ_v \qquad (5.17)$$

Where:

C = Green Rooftop Credit (ac-ft)
A_{GR} = Green Rooftops (acres)
A_I = Site Impervious Area (acres)
WQ_v = Original Water Quality Volume (ac-ft)

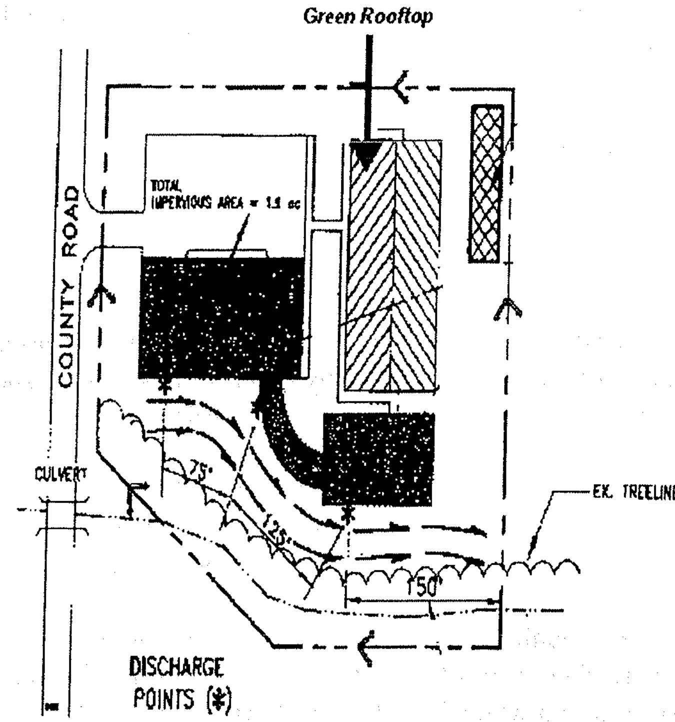
The water quality volume is then reduced by the credit, C. The example in Figure 5-30 illustrates how this credit would be applied.

Green Rooftop Credit Example

Site Data: Community Center
Area = 3.0 ac
Original Impervious
Area = 1.9 ac = 63.3%

Original $WQ_v = 6{,}752\ ft^3$

0.5 ac of green rooftops



Computation of Stormwater Credits

$C = (0.5 / 1.9)(6{,}752\ ft^3) = 1{,}777\ ft^3$

$WQ_v = 6{,}752\ ft^3 - 1{,}777\ ft^3 = 4{,}975\ ft^3$

FIGURE 5-30. Example, Green Rooftop Credit.

5.9.2 *Watershed Credits*

Watershed credits focus on the location of the development, rather than on the design of the site. They reward developers who locate in areas that result in less impact to water resources by encouraging development in already urbanized or highly degraded areas. Three watershed credits are presented in this section:

- Watershed Zoning
- Infill
- Redevelopment

5.9.2.1 Watershed Zoning

This credit reduces stormwater management requirements in developments that support a strategy of *watershed zoning* by locating in subwatersheds that are designated as *non-supporting*. A watershed Zoning Credit can be awarded to developments that meet the following criteria:

- Must be located in a subwatershed with greater than 25% impervious cover
- Shall not contribute more than 5% impervious cover to the subwatershed, or to the drainage of any 2nd order or larger stream.

The water quality credit received is a function of the impervious cover fraction in the subwatershed, times the water quality volume. This credit will not be given if the development is located in a watershed that discharges directly to a stream with a published Total Maximum Daily Load (TMDL). Furthermore, this credit will not be given if it is found that the development will impact a relatively high quality reach within the subwatershed.

5.9.2.2 Infill Credit

The infill credit acts as an incentive for developing infill lots, as opposed to greenfields away from existing development. Infill development results in lower infrastructure costs, fewer miles driven, and a net reduction in impervious cover creation when compared with greenfield development.

An infill credit can be applied to all sites that are built within the current sewer envelope, and are smaller than 5 acres for residential development, and 2 acres for commercial or industrial uses. Sites that meet these criteria can receive a 20% water quality credit. This credit will not be given if the development is located in a watershed that discharges directly to a stream with a published TMDL. Furthermore, this credit will not be given if it is found that the development will impact a relatively high quality reach within the subwatershed.

5.9.2.3 Redevelopment Credit

The redevelopment credit encourages development on sites that have previous commercial, industrial, or residential land use. The credit allows reduction in required treatment and management volumes, depending on the existing conditions at the site. For redevelopment projects, treatment is only required for the additional stormwater generated on site.

The redevelopment credit may be awarded for all redevelopment sites. The water quality credit is based on pre-developed impervious cover. The credit can be expressed as:

$C = I_P WQ_v$

Where:

C = Credit (ac-ft)

I_P = Pre-Developed Impervious Cover

This credit can then be subtracted from the water quality volume.

Redevelopment Credit Example
Consider a site with a pre-developed impervious cover of 25% and a water quality volume of 10,000 ft^3.
Redevelopment Credit Calculation $C = (10,000\ ft^3)(25\%) = 2,500\ ft^3$ $WQ_v = 10,000\ ft^3 - 2,500\ ft^3 = 7,500\ ft^3$

It should be noted that stormwater treatment is not required for redevelopment projects less than one acre in size or projects where the impervious area will not be increased.

5.10 REFERENCES

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(3) Brater, E. F. and H. W. King. *Handbook of Hydraulics.* 6th edition. McGraw Hill Book Company, New York, NY, 1976.

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(7) Galli, F. 1990. Peat-Sand Filters: A Proposed Stormwater Management Practice for Urban Areas. Metropolitan Washington Council of Governments. Washington, DC.

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(9) Washington State Department of Ecology (WSDE). 1992. Stormwater Management Manual for the Puget Sound Basin. Olympia, WA.

APPENDIX 5A

40 CFR 122.26(b)(14) Subpart (i) – (xi)

(Source: ADEQ)

INDUSTRIAL FACILITIES THAT MUST SUBMIT APPLICATIONS FOR STORM WATER PERMITS

40 CFR 122.26(B)(14) Subpart	**Description**	
(i)	Facilities subject to storm water effluent limitations guidelines, new source performance standards, or toxic pollutants effluent standards under 40 CFR, Subchapter N [except facilities which are exempt under category (xi)].	
(ii)	Facilities classified as:	
	SIC 24 (EXCEPT 2434)	Lumber and Wood Products
	SIC 26 (EXCEPT 265 and 267)	Paper and Allied Products
	SIC 28 (EXCEPT 283 and 285)	Chemicals and Allied Products
	SIC 29	Petroleum and Coal Products
	SIC 311	Leather Tanning and Finishing
	SIC 32 (except 323)	Stone, Clay and Glass Products
	SIC 33	Primary Metal Industries
	SIC 3441	Fabricated Structural Metal
	SIC 373	Ship and Boat Building and Repairing

40 CFR 122.26(B)(14) Subpart	Description
(iii)	Facilities classified as SIC 10 through 14, including active or inactive mining operations and oil and gas exploration, production, processing, or transmission facilities that discharge storm water contaminated by contact with, or that has come into contact with, any overburden, raw material, intermediate products, finished products, byproducts, or waste products located on the site of such operations. SIC 10 Metal Mining SIC 11 Anthracite Mining SIC 12 Coal Mining SIC 13 Oil and Gas Extraction SIC 14 Nonmetallic Minerals, except Fuels
(iv)	Hazardous waste treatment, storage, or disposal facilities, including those that are operating under interim status or a permit under Subtitle C of the Resource Conservation and Recovery Act (RCRA).
(v)	Landfills, land application sites, and open dumps that receive or have received any industrial wastes including those that are subject to regulation under subtitle D or RCRA.

40 CFR 122.26(B)(14) Subpart	Description
(vi)	Facilities involved in the recycling of material, including metal scrap yards, battery reclaimers, salvage yards, and automobile junkyards, including but limited to those classified as: SIC 5015 Motor Vehicle Parts, Used SIC 5093 Scrap and Waste Materials
(vii)	Steam electric power generating facilities, including coal-handling sites.
(viii)	Transportation facilities which have vehicle maintenance shops, equipment cleaning operations, or airport de-icing operations. Only those portions of the facility that are either involved in vehicle maintenance (including vehicle rehabilitation, mechanical repairs, fueling, and lubrication), equipment cleaning operations, or airport de-icing operations, or which are otherwise listed in another category, are included. SIC 40 Railroad Transportation SIC 41 Local Suburban Transit SIC 42 (except 4221-25) Motor Freight and Warehousing SIC 43 U.S. Postal Service SIC 44 Water Transportation SIC 45 Transportation by Air SIC 5171 Petroleum Bulk Stations and Terminals

40 CFR 122.26(B)(14) Subpart	Description
(ix)	Treatment works treating domestic sewage or any other sewage sludge or wastewater treatment device or system, used in the storage, treatment, recycling, and reclamations of municipal or domestic sewage, including lands dedicated to the disposal of the sewage sludge that are located within the confines of the facility, with a design flow of 1.0 million gallons per day or more, or required to have an approved pretreatment program under 40 CFR Part 403. Not included are farm lands, domestic gardens, or lands used for sludge management where sludge is beneficially reused and which are not physically located in the confines of the facility, or areas that are in compliance with Section 405 of the Clean Water Act.
(x)	Construction activity including clearing, grading, and excavation activities except operations that result in the disturbance of less than 5 acres of total land area and those that are not part of a larger common plan of development or sale. *

40 CFR 122.26(B)(14) Subpart	Description	
(xi)	Facilities under the following SICs [which are not otherwise included in categories (ii)-(x)], including only storm water discharges where material handling equipment or activities, raw materials, intermediate products, final products, waste materials, byproducts, or industrial machinery are exposed to storm water. *	
	SIC 20	Food and Kindred Products
	SIC 21	Tobacco Products
	SIC 22	Textile Mill Products
	SIC 23	Apparel and Other Textile Products
	SIC 2434	Wood Kitchen Cabinets
	SIC 25	Furniture and Fixtures
	SIC 265	Paperboard Containers and Boxes
	SIC 267	Converted Paper and Paper Board Products (except containers and boxes)
	SIC 27	Printing and Publishing
	SIC 283	Drugs
	SIC 285	Paints, Varnishes, Lacquer, Enamels
	SIC 30	Rubber and Misc. Plastics Products
	SIC 31 (except 311)	Leather and Leather Products
	SIC 323	Products of Purchased Glass
	SIC 34 (except 3441)	Fabricated Metal Products
	SIC 35	Industrial Machinery and Equipment, except Electrical
	SIC 36	Electronic and Other Electric Equipment
	SIC 37 (except 373)	Transportation Equipment

40 CFR 122.26(B)(14) Subpart	Description	
	SIC 38	Instruments and Related Products
	SIC 39	Miscellaneous Manufacturing Industries
	SIC 4221	Farm Products Warehousing and Storage
	SIC 4222	Refrigerated Warehousing and Storage
	SIC 4225	General Warehousing and Storage

Source: Federal Register, Volume 55, Number 222, Page 48065, November 16, 1990.

* On June 11, 1992, the U.S. Court of Appeals for the Ninth Circuit remanded the exemption for construction sites of less than five acres in Category (x) and for manufacturing facilities in category (xi) which do not have materials or activities exposed to storm water to the EPA for further rulemaking. (Nos. 90-70671 & 91-70200).

CHAPTER 6 – STORM DRAINS AND INLETS

6.1 GENERAL

Roadway storm drainage systems collect stormwater runoff and convey it through the street right-of-way in a manner that adequately drains the roadway and minimizes the potential for flooding and erosion to properties adjacent to the right-of-way. Storm drainage systems consist of curbs, gutters, storm drains, channels and culverts.

The design of a drainage system must address the needs of the traveling public and those of the local community through which it passes. The drainage system for a roadway traversing an urbanized region is more complex than for roadways traversing sparsely settled rural areas. This is often due to:

- the wide roadway sections, flat grades (both in longitudinal and transverse directions), shallow water courses and absence of side channels;

- the more costly property damages that may occur from ponding of water or from flow of water through builtup areas; and

- the fact that the roadway section must carry traffic but also act as a channel to convey the water to a disposal point. Unless proper precautions are taken, this flow of water along the roadway will interfere with or possibly halt the passage of traffic.

The most serious effects of an inadequate roadway drainage system are:

- damage to surrounding or adjacent property, resulting from water overflowing the roadway curbs and entering such property;

- risk and delay to traffic caused by excessive ponding in sags or excessive spread along the roadway; and

- weakening of base and subgrade due to saturation from frequent ponding of long duration.

6.2 DESIGN CRITERIA

Storm drainage systems shall be designed according to the criteria listed below:

6.2.1 *Design Frequency*

6.2.1.1 Roadway Drainage/Gutter Spread

The design flood frequency for roadway drainage is related to the allowable gutter spread on the pavement and the roadway classification. Gutter spread may not extend into the clear zone of the street for the required design frequency. Roadways within the City of Fort Smith must be designed according to the minimum clear zone and design frequency requirements for the given roadway classification shown in TABLE 6-1 below:

TABLE 6-1. Design Frequency for Roadway Drainage

Roadway Classification	Design Frequency	Clear Zone
Residential	10-year	Can't Overtop Centerline
Residential Collector	10-year	Center 12 ft
Residential Collector (Restricted Parking)	10-year	Center 12 ft
Major Collector	25-year	Center 12 ft
Minor Arterial	50-year	Center 24 ft
Major Arterial	50-year	Center 24 ft
Boulevard	50-year	12 ft Each Side
Industrial	25-year	Center 12 ft

6.2.1.1 Storm Drains

Storm drains within the City of Fort Smith shall be designed for the frequencies shown in TABLE 2-1.

6.2.2 *Criteria for Storm Drains*

Storm drains shall be designed in accordance with the following criteria:

- All 10-year flows less than 50 cfs must be contained in an underground enclosed storm drain, unless carried in an Open Channel System designed to treat the required WQ_v.

- Maximum spacing between inlets and/or junction boxes is 400 ft. Inlets and/or junction boxes shall be placed at all grade changes, changes of direction, changes in structure size, and locations where storm drains intersect. Curb inlets shall be constructed on both sides of a street in sump areas.

- Inlets located in sump areas shall be designed to operate with 50 % blockage.

- Generally, only curb-opening or area inlets shall be allowed. However, with prior approval of the Engineering Department, grated or combination inlets may be used for special situations where curb-opening or area inlets are not sufficient. If used, grated and combination inlets must be designed to operate with 50 % blockage. Slotted drain inlets shall not be allowed.

- Any concentration of surface flow in excess of 6.0 cfs for a 10-year storm shall be intercepted before crossing the back of curb and carried by storm drains. No storm drain will be allowed to discharge into the street.

- The hydraulic grade line shall be calculated for all storm drains. Minimum freeboard shall be 9 inches from the water surface to:

  - the gutter flow line for curb inlets,
  - the throat invert for area inlets, or
  - the rim elevation for junction boxes.

- Minimum storm drain diameter for round pipe shall be 15 inches. Minimum storm drain size for arch pipe or elliptical pipe shall be 15 inch equivalent. Minimum size for box culverts shall be 4 feet by 4 feet.

- The minimum allowable fill or cover shall be 12 inches above the top of the storm drain. For storm drains under roadways there shall also be a minimum clearance of 6 inches from the top of the storm drain to the bottom of the pavement base. Special reinforced concrete boxes designed to carry traffic on the top slab do not have to meet minimum allowable fill requirements.

- As a minimum, pipes shall be Class III, Reinforced Concrete Pipe. Boxes shall also be constructed of reinforced concrete. A reinforced concrete inlet box, headwall/endwall, slope wall, or flared-end section shall be constructed at the entrance and outfall of each storm drain.

- The minimum allowable velocity for storm drains shall be 3 ft/s, and the maximum allowable velocity shall be 15 ft/s. The minimum slope for storm drains shall be 0.30 %.

- Tailwater depth at outlet may be calculated with Manning's Equation (Section 3.3.2) if Step Backwater Analysis is not required for the downstream channel. If the headwater elevation for a nearby downstream culvert or storm drain is greater than the normal depth for the channel, a Step Backwater Analysis shall be required. (Regardless of the method used, the minimum tailwater elevation shall be the invert elevation plus 0.8 times the pipe diameter.)

- Energy dissipators will be required at storm drain outfalls in earthen channels when the discharge velocity exceeds 6 ft/s. Energy dissipators shall be designed in accordance with the *Hydraulic Design of Stilling Basins and Energy Dissipators (1)*, developed by the U.S. Bureau of Reclamation.

- Storm drain pipes and boxes should not decrease in size in a downstream direction regardless of the available pipe gradient.

- At inlets and junctions, where practicable, soffits of inflowing pipes and boxes shall be placed at or above the soffit elevation of the outflowing pipes and boxes. In no instance shall the inverts of inflowing pipes and boxes be placed below the invert elevation of outflowing pipes and boxes.

6.2.3 *Additional Criteria*

Storm drainage systems shall also be designed according to the criteria listed below:

- Swales shall not be permitted across through streets.

- The maximum water surface elevation for the 100-year storm event shall be 1 foot lower than the floor elevation of adjacent buildings or structures.

6.3 GUTTER FLOW CALCULATIONS

6.3.1 *Introduction*

Gutter flow calculations are necessary to relate the quantity of flow (Q) in the curbed channel to the spread of water on the shoulder, parking lane or pavement section. The nomograph on Figure 6-1 can be utilized to solve uniform cross slope channels, composite gutter sections and V-shape gutter sections. Figure 6-3 is also useful in solving composite gutter section problems. Computer programs, such as the HYDRAIN program *(4)*, are also useful for this computation and inlet capacity. Composite gutter sections have a greater hydraulic capacity for normal cross slopes than uniform gutter sections and are therefore preferred. Example problems for each gutter section are shown in the following sections.

6.3.2 *Manning's n for Pavements*

The roughness of the pavement surface affects water spread. The methods for determining spread provided in this chapter use Manning's roughness coefficient (n). Refer to Table 6-2 for recommended values.

6.3.3 *Uniform Cross Slope Procedure*

The nomograph in Figure 6-1 is used with the following procedures to find gutter capacity for uniform cross slopes:

CONDITION 1: Find spread, given gutter flow:

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), gutter flow (Q) and Manning's n.

TABLE 6-2. Manning's *n* for Street and Pavement Gutters

Type of Gutter or Pavement	Manning's n
Concrete Gutter, Broom finish	0.016
Asphalt Pavement, Rough texture	0.016
Concrete Pavement, Broom finish	0.016
For gutters with small slope, where sediment may accumulate, increase above n values by:	0.002

Source: Reference *(5)*.

Step 2 Draw a line between the S and S_x scales, and note where it intersects the turning line.

Step 3 Draw a line between the intersection point from Step 2 and the appropriate gutter flow value on the capacity scale. If Manning's n is 0.016, use Q from Step 1; if not, use the product of Q and n.

Step 4 Read the value of the spread (T) at the intersection of the line from Step 3 and the spread scale.

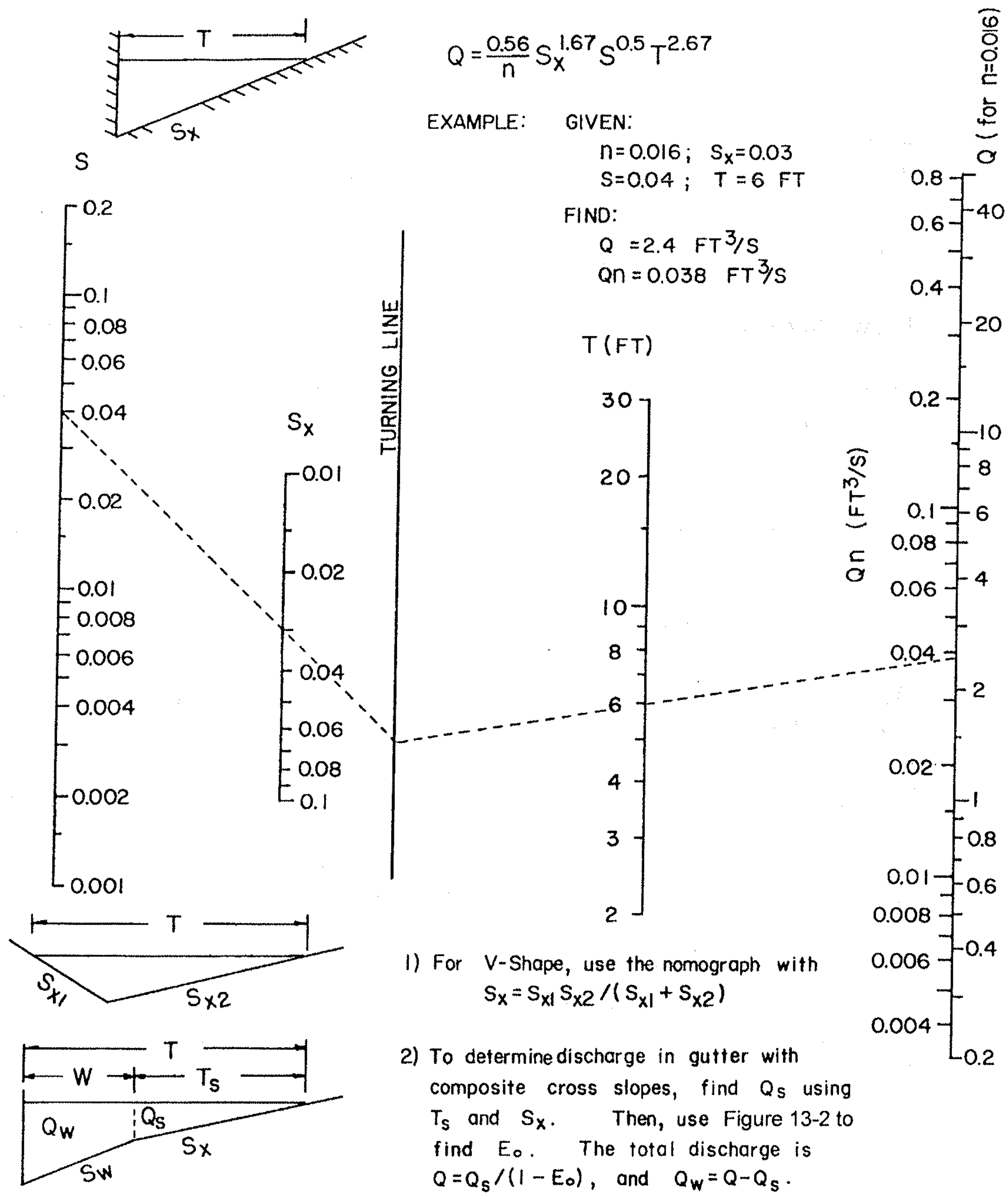
CONDITION 2: Find gutter flow, given spread:

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), spread (T), and Manning's n.

Step 2 Draw a line between the S and S_x scales, and note where it intersects the turning line.

Step 3 Draw a line between the intersection point from Step 2 and the appropriate value on the T scale. Read the value of Q or Qn from the intersection of that line on the capacity scale.

Step 4 For Manning's n values of 0.016, the gutter capacity (Q) from Step 3 is selected. For other Manning's n values, the gutter capacity times *n(Qn)* is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.



Flow in Triangular Gutter Sections - English Units

FIGURE 6-1. Flow in Triangular Gutter Sections

Source: HEC 22 *(6)*.

6.3.4 *Composite Gutter Sections Procedure*

Figure 6-2 can be used to find the flow in a gutter section with width (*W*) less than the total spread (*T*). Such calculations are generally used for evaluating composite gutter sections or frontal flow for grate inlets:

CONDITION 1: Find spread, given flow:

Step 1 Determine input parameters, including longitudinal slope (*S*), cross slope (S_x), depressed section slope (S_w), depressed section width (*W*), Manning's *n*, gutter flow (*Q*) and a trial value of the gutter capacity above the depressed section (Q_s). (Example: $S = 0.01$; $S_x = 0.02$; $S_w = 0.06$; $W = 2$ ft; $n = 0.016$; $Q = 2.0$ ft^3/s; try $Q_s = 0.7$ ft^3/s).

Step 2 Calculate the gutter flow in W (Q_w), using the equation:

$$Q_w = Q - Q_s \quad (Q_w = 2.0 - 0.7 = 1.3 \text{ ft}^3/\text{s}) \tag{6.1}$$

Step 3 Calculate the ratios Q_w/Q and S_w/S_x, and use Figure 6-2 to find an appropriate value of W/T. ($Q_w/Q = 1.3/2.0 = 0.65$; $S_w/S_x = 0.06/0.02 = 3$. From Figure 6-2, $W/T = 0.27$).

Step 4 Calculate the spread (*T*) by dividing the depressed section width (*W*) by the value of W/T from Step 3. ($T = 2.0/0.27 = 7.41$ ft).

Step 5 Find the spread above the depressed section (T_s) by subtracting *W* from the value of *T* obtained in Step 4. ($T_s = 7.41 - 2.0 = 5.41$ ft).

Step 6 Use the value of T_s from Step 5 and Manning's *n*, *S* and S_x to find the actual value of Q_s from Figure 6-1. (From Figure 6-1, $Q_s = 0.5$ ft^3/s).

Step 7 Compare the value of Q_s from Step 6 to the trial value from Step 1. If values are not comparable, select a new value of Q_s and return to Step 1:

(Compare 0.5 to 0.7 "no good." Try $Q_s = 0.8$; then $2.0 - 0.8 = 1.2$, and $1.2/2.0 = 0.6$. From Figure 13-2, $W/T = 0.23$; then $T = 2.0/0.23 = 8.7$ ft and $T_s = 8.7 - 2.0 = 6.7$ ft. From Figure 13-1, $Q_s = 0.8$ ft^3/s—OK).

ANSWER: Spread $T = 8.7$ ft

CONDITION 2: Find gutter flow, given spread:

Step 1 Determine input parameters, including spread (*T*), spread above the depressed section (T_s), cross slope (S_x), longitudinal slope (*S*), depressed section slope (S_w), depressed section width (*W*), Manning's n and depth of gutter flow (*d*):

EXAMPLE: Allowable spread, $T = 10$ ft; $W = 2$ ft; $T_s = 10.0 - 2.0 = 8$ ft; $S_x = 0.04$; S = 0.005 ft/ft; $S_w = 0.06$; $n = 0.016$; $d = 0.43$ ft.

Step 2 Use Figure 6-1 to determine the capacity of the gutter section above the depressed section (Q_S). Use the procedure for uniform cross slopes, Condition 2, substituting T_s for T. (From Figure 6-1, $Q_s = 3.0$ ft^3/s).

Step 3 Calculate the ratios W/T and S_w/S_x and, from Figure 6-2, find the appropriate value of E_o (the ratio of Q_w/Q). (W/T = 2.0/10.0 = 0.2; $S_w/S_x = 0.06/0.04 = 1.5$; from Figure 6-1, $E_o = 0.46$).

Step 4 Calculate the total gutter flow using the equation:

$$Q = Q_s/(1 - E_o) \tag{6.2}$$

where:

Q = gutter flow rate, ft^3/s

Q_s = flow capacity of the gutter section above the depressed section, ft^3/s

E_o = ratio of frontal flow to total gutter flow (Q_w/Q)

($Q = 3.0 / (1 - 0.46) = 5.6$ ft^3/s)

Step 5 Calculate the gutter flow width (W), using Equation 6.1:

($Q_w = Q - Q_s = 5.6 - 3.0 = 2.6$ ft^3/s)

Note: Figure 6-3 can also be used to calculate the flow in a composite gutter section.

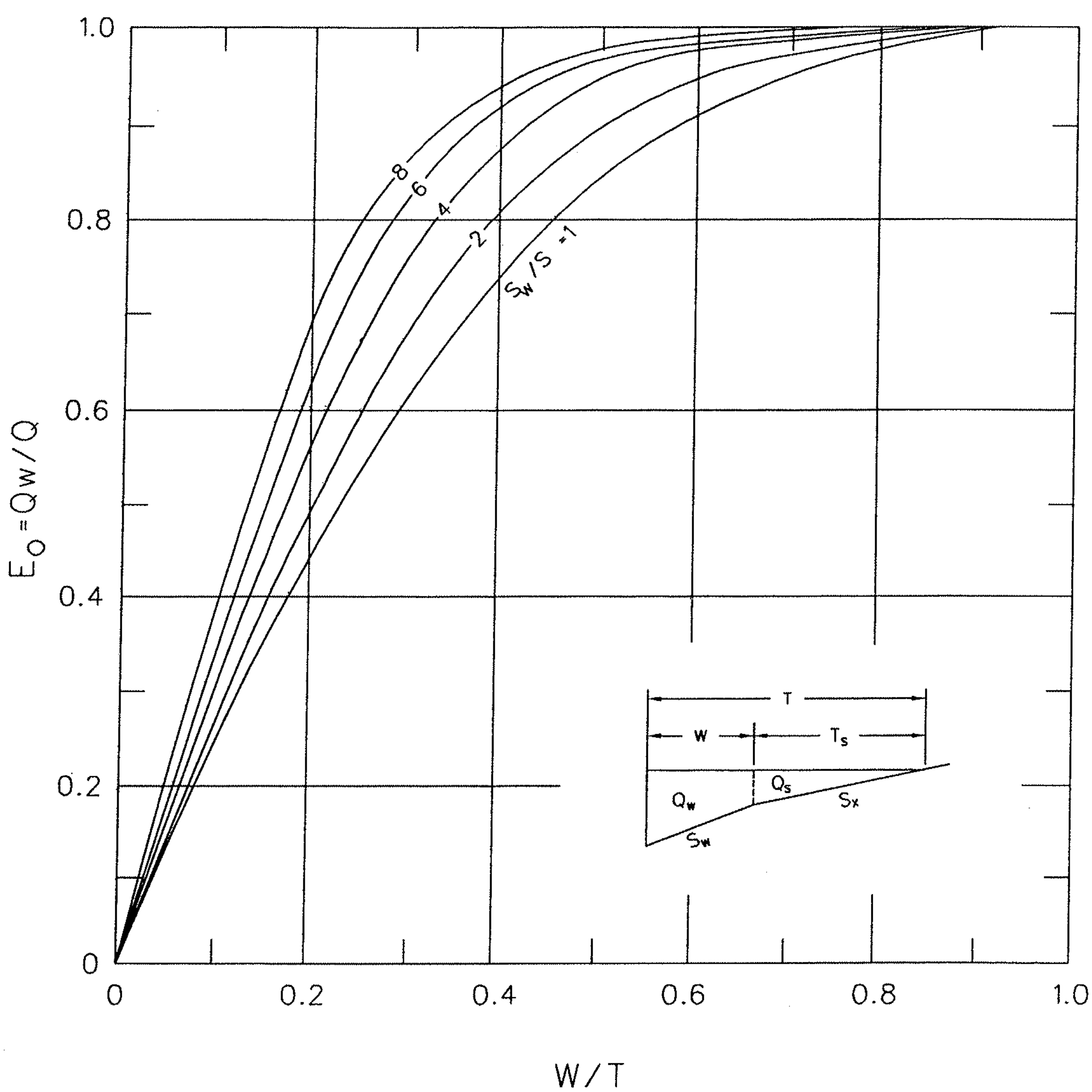


FIGURE 6-2. Ratio of Frontal Flow to Total Gutter Flow

Source: HEC 22 *(6)*.

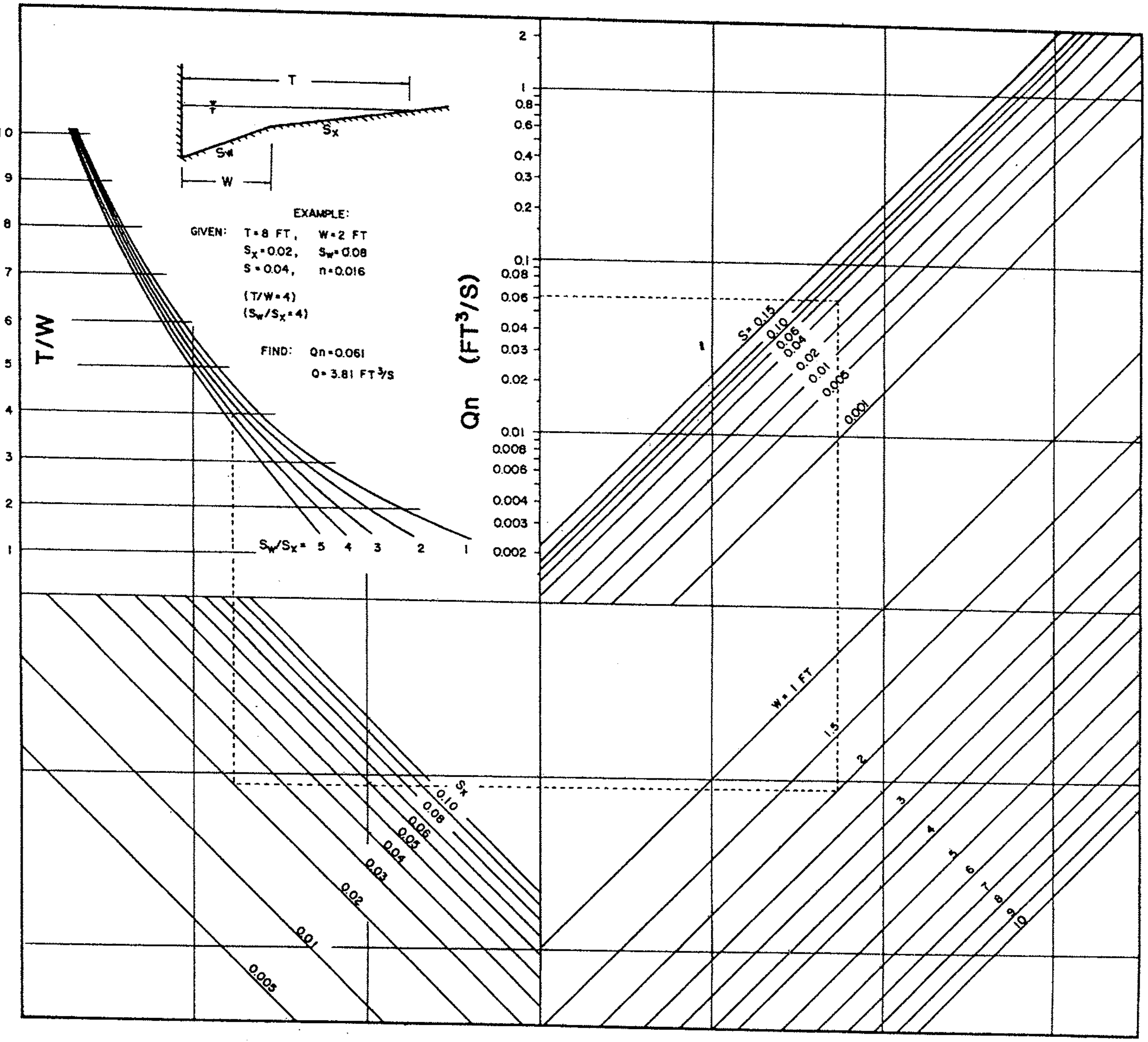


FIGURE 6-3. Flow in Composite Gutter Sections

Source: HEC 12 *(3)*.

6.3.5 *V-Type Gutter Sections (Procedures)*

Figure 6-1 can also be used to solve V-type channel problems. The spread (T) can be calculated for a given flow (Q), or the flow can be calculated for a given spread. This method can be used to calculate approximate flow conditions in the triangular channel adjacent to concrete median barriers. It assumes that the effective flow is confined to the V channel with spread T_1. Figure 6-4 illustrates the following procedure for a V-type gutter:

CONDITION 1: Given flow (Q), find spread (T):

Step 1 Determine input parameters, including longitudinal slope (S), cross slope $S_x = S_{x1}S_{x2}/(S_{x1} + S_{x2})$, Manning's n, total flow (Q). (Example: $S = 0.01$, $S_{x1} = 0.06$, $S_{x2} = 0.04$, $S_{x3} = 0.015$, $n = 0.016$, $Q = 2.0$ ft^3/s, shoulder width = 6 ft).

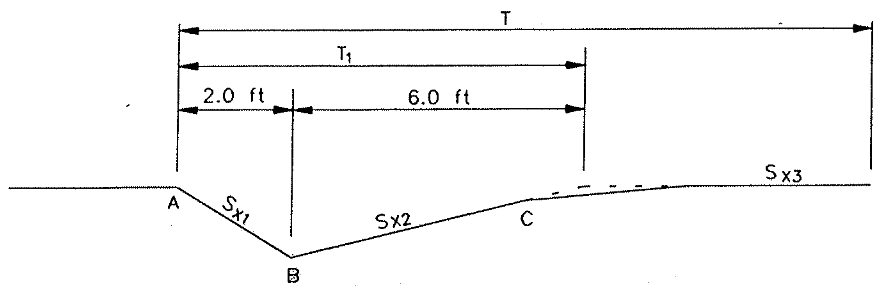


FIGURE 6-4. V-Type Gutter (Mountable Curb)

Step 2 Calculate S_x:

$S_x = S_{x1}S_{x2}/(S_{x1} + S_{x2})$ $\qquad$ $S_x = (0.06)(0.04)/(0.06 + 0.04) = 0.024$

Step 3 Solve for T_1 using the nomograph on Figure 6-1:

T_1 is a hypothetical width that is correct if it is contained within S_{x1} and S_{x2}. From the nomograph, $T_1 = 8.4$ ft; however, because the shoulder width of 6 ft is less than 8.4 ft, S_{x2} is 0.04, and the pavement cross slope S_{x3} is 0.015. T will actually be greater than 8.4 ft, 8.4 - 2.0 = 6.4 ft, which is > 4.0 ft; therefore, the spread is greater than 8.4 ft.

Step 4 To find the actual spread, solve for depth at Points B and C:

Point B: 6.4 ft @ 0.04 = 0.26 ft
Point C: 0.26 ft - (4.0 ft @ 0.04) = 0.1 ft

Step 5 Solve for the spread on the pavement. (Pavement cross slope = 0.015):

$T_{0.015} = 0.1/0.015 = 6.7$ ft

Step 6 Find the actual total spread (T): T = 6.0 + 6.7 = 12.7 ft

CONDITION 2: Given spread (T), find flow (Q):

Step 1 Determine input parameters such as longitudinal slope (S), cross slope ($S_x = S_{x1}S_{x2}/(S_{x1} + S_{x2})$, Manning's n and allowable spread. (Example: $n = 0.016$, $S = 0.015$, $S_{x1} = 0.06$, $S_{x2} = 0.04$, $T = 6.0$ ft).

Step 2 Calculate S_x:

$$S_x = S_{x1}S_{x2}/(S_{x1} + S_{x2}) = (0.06)(0.04)/(0.06 + 0.04) = 0.024$$

Step 3 Using Figure 6-1, solve for Q: For $T = 6.0$ ft, $Q = 1.1$ ft^3/s

The equation shown on Figure 6-1 can also be used.

6.4 INLETS

6.4.1 *General*

Inlets are drainage structures utilized to collect surface water through curb or other similar openings and convey it to storm drains or to culverts. If grated or combination inlets are allowed, they must be safe for pedestrian, bicycle, and wheelchair traffic.

6.4.2 *Inlet Locations*

Inlets are required at locations needed to collect runoff within the design controls specified in the design criteria (see Section 6.2). In addition, there are a number of locations where inlets may be necessary with little regard to contributing drainage area. These locations should be marked on the plans prior to any computations regarding discharge, water spread, inlet capacity or runby. Examples of such locations are as follows:

- sag points in the gutter grade;
- upstream of median breaks, entrance/exit ramp gores, cross walks and street intersections;
- immediately upstream and downstream of bridges;
- immediately upstream of cross slope reversals;

- on side streets at intersections;
- at the end of channels in cut sections; and
- behind curbs, shoulders or sidewalks to drain low areas.

Inlets should not be located in the path where pedestrians are likely to walk.

6.5 INLET SPACING

6.5.1 *General*

A number of inlets are required to collect runoff at locations with little regard for contributing drainage area as discussed in Section 6.4.2. These should be plotted on the plan first. Next, it is recommended to start locating inlets from the crest and working downgrade to the sag points. The location of the first inlet from the crest can be found by determining the length of pavement and the area back of the curb sloping toward the roadway that will generate the design runoff. The design runoff can be computed as the maximum allowable flow in the curbed channel that will meet the design criteria as specified in Section 6.2. Where the contributing drainage area consists of a strip of land parallel to and including a portion of the highway, the first inlet can be calculated as follows:

$$L = \frac{43{,}560\ Q_t}{CIW} \tag{6.3}$$

where:
L = distance from the crest, ft
Q_t = maximum allowable flow, ft^3/s
C = composite runoff coefficient for contributing drainage area
W = width of contributing drainage area, ft
I = rainfall intensity for design frequency, in./h

If the drainage area contributing to the first inlet from the crest is irregular in shape, trial and error will be necessary to match a design flow with the maximum allowable flow. Equation 6.3 is an alternative form of the Rational Equation.

To space successive downgrade inlets, it is necessary to compute the amount of flow that will be intercepted by the inlet (Q_i) and subtract it from the total gutter flow to compute the runby. The runby from the first inlet is added to the computed flow to the second inlet, the total of which must be less than the maximum allowable flow dictated by the criteria. Figure 6-8 (Section 6.5.4) is an inlet spacing computation sheet that can be utilized to record the spacing calculations.

Inlet interception capacity for all types of inlets has been investigated by FHWA. References *(6)* and *(4)* may be used to analyze the flow in gutters and the interception capacity of all types of inlets on continuous grades and sags. Both uniform and composite cross slopes can be analyzed.

6.5.2 *Curb Inlets on Grade*

Curb-opening inlets are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are relatively free of clogging tendencies and offer little interference to traffic operation. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

The length of a curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is expressed by:

$$L_T = KQ^{0.42}S^{0.3}(1/nS_x)^{0.6} \qquad (6.4)$$

where:
K = 0.6
L_T = curb-opening length required to intercept 100 percent of the gutter flow, ft

The efficiency of curb-opening inlets shorter than the length required for total interception is expressed by:

$$E = 1 - (1 - L/L_T)^{1.8} \qquad (6.5)$$

where:
L = curb-opening length, ft

Figure 6-5 is a nomograph for the solution of Equation 6.4, and Figure 6-6 provides a solution of Equation 6.5.

The length of inlet required for total interception by depressed curb-opening inlets or curb openings in depressed gutter sections can be found by the use of an equivalent cross slope, S_e, in Equation 6.4:

$$S_e = S_x + S_w E_o \qquad (6.6$$

where:
S_w = cross slope of the gutter measured from the cross slope of the pavement
= $(a/12W)$, ft/ft

a = gutter depression, in.
E_o = ratio of flow in the depressed section to total gutter flow. It is determined by the gutter configuration upstream of the inlet. Reference Figure 6-2 to determine E_o.

Note: S_e can be used to calculate the length of curb opening by substituting S_e for S_x in Equation 6.4.

Example Problem

The following example illustrates the use of this procedure:

Given: $S_x = 0.03$ ft/ft $S = 0.035$ ft/ft $n = 0.016$ $Q = 5$ ft^3/s

Find:

(1) Q_i for 10-ft curb-opening inlet, uniform cross slope

(2) Q_i for a depressed 10-ft curb-opening inlet with composite cross slope $a = 2$ in., $W = 2$ ft

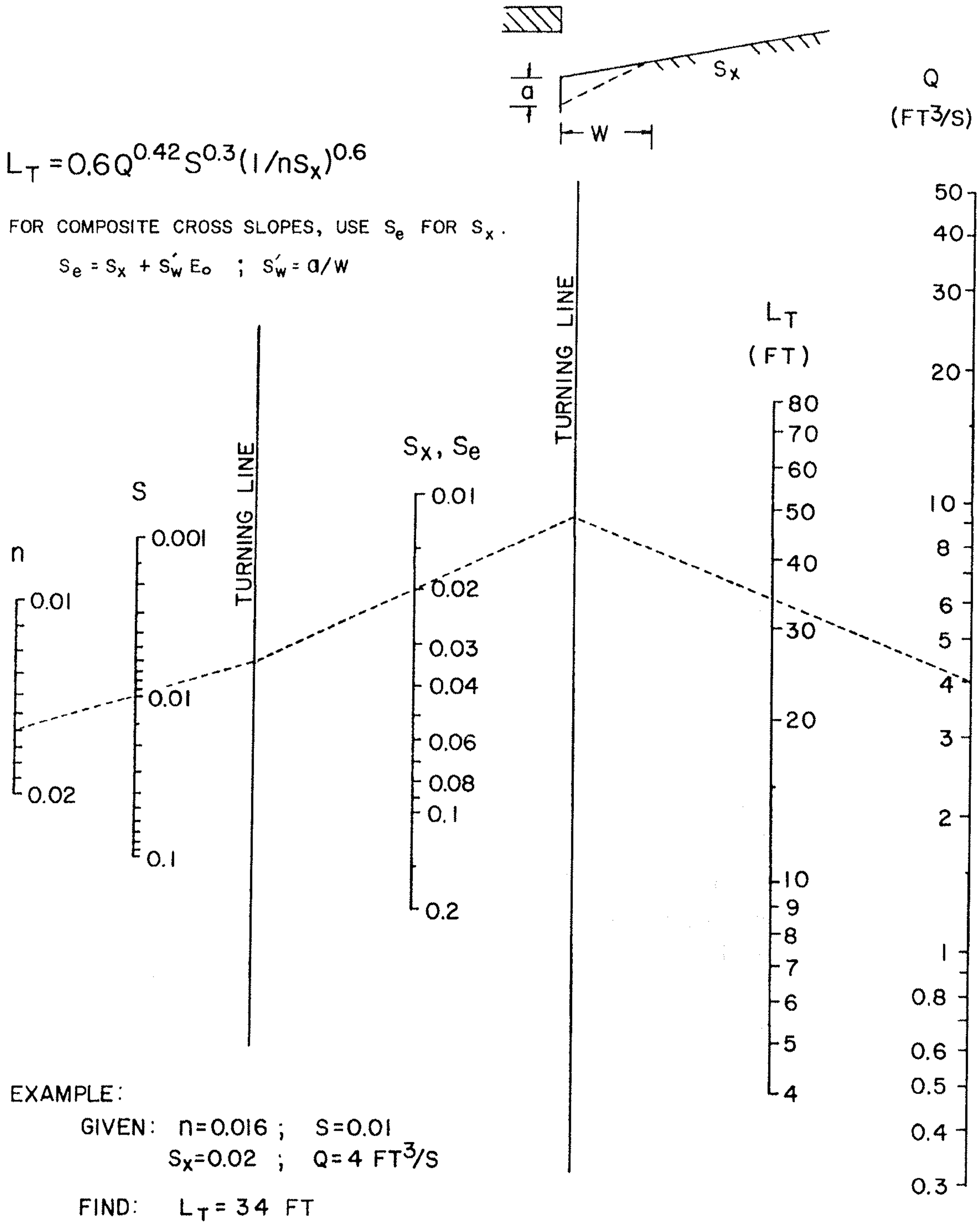
(3) Q_i for a depressed 10-ft curb-opening inlet with uniform cross slope

Solution:

(1) From Figure 6-1, $T = 8$ ft
From Figure 6-5, $L_T = 41$ ft
$L/L_T = 10/41 = 0.24$
From Figure 6-6, $E = 0.39$
$Q_i = EQ = (0.39)(5) = \underline{\underline{2 \text{ ft}^3/\text{s}}}$

(2) $Qn = (5)(0.016) = 0.08$ ft^3/s
$S_w/S_x = (0.03 + 0.085)/0.03 = 3.83$
From Figure 6-3, $T/W = 3.5$ and $T = 7$ ft
Then W/T (Depress) $= 2/7 = 0.29$
From Figure 6-2, $E_o = 0.74$
$S_e = S_x + S_w E_o = 0.03 + 0.085(0.74) = 0.09$
From Figure 6-5, $L_T = 21$ ft, then $L/L_T = 10/21 = 0.48$
From Figure 6-6, $E = 0.69$, then $Q_i = (0.69)(5) = \boldsymbol{3.5\ ft^3/s}$

(3) $S_w/S_x = 0.03/0.03 = 1$
$W/T = 2/8 = 0.25$
From Figure 6-2, $E_o = 0.53$
$S_e = 0.03 + (0.085)(0.53) = 0.075$
From Figure 6-5, $L_T = 25$ ft, then $L/L_T = 10/25 = 0.4$
From Figure 6-6, $E = 0.60$, then $Q_i = (0.6)(5) = \boldsymbol{3\ ft^3/s}$



Curb-opening & Slotted Drain Inlet Length for Total Interception - English Units

FIGURE 6-5. Curb-Opening Inlet Length for Total Interception

Source: HEC 22 *(6)*.

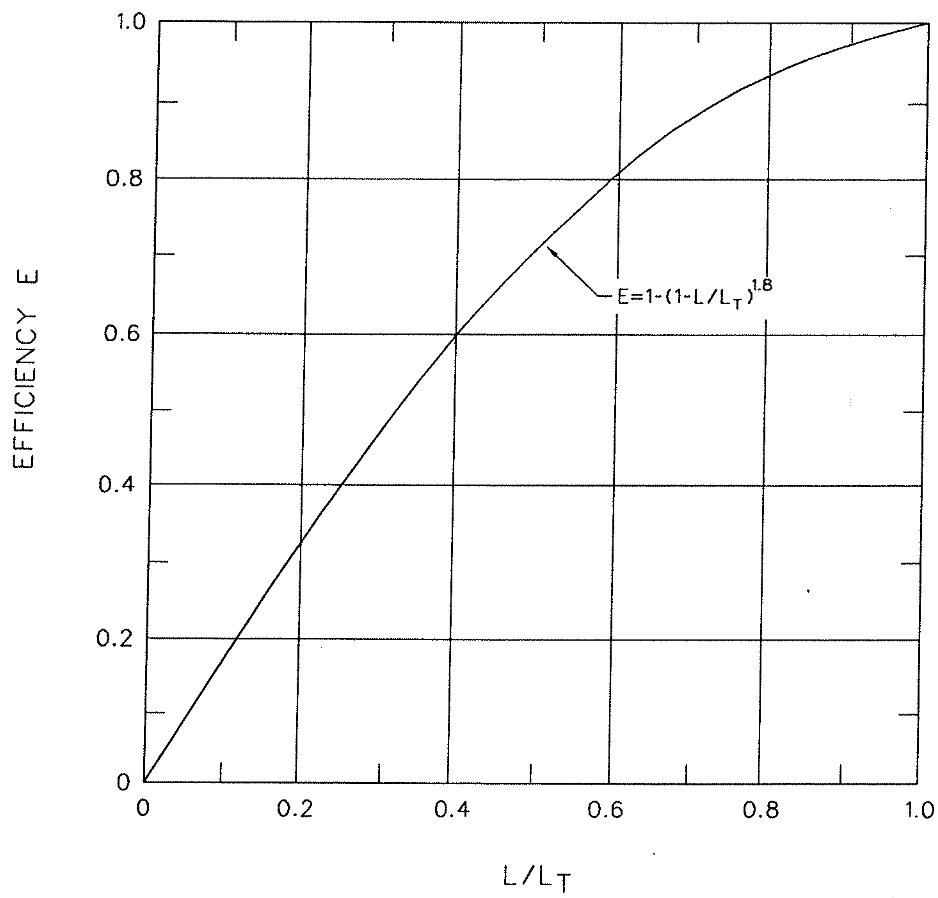


FIGURE 6-6. Curb-Opening and Slotted Drain Inlet Interception Efficiency

Source: HEC 22 *(6)*.

6.5.3 *Curb Inlets in Sag*

The capacity of a curb-opening inlet in a sag depends on water depth at the curb, the curb-opening length and the height of the curb opening. The inlet operates as a weir to depths equal to the curb-opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

The equation for the interception capacity of a depressed curb-opening inlet operating as a weir is:

$$Q_i = C_W (L + 1.8W)d^{1.5} \tag{6.7}$$

where:

C_W = 2.3
L = length of curb opening, ft
W = width of depression, ft
d = depth of water at curb measured from the normal cross slope gutter flow line, ft

See Figure 6-7 for a definition sketch.

The weir equation for curb-opening inlets without depression becomes:

$$Q_i = C_W L d^{1.5} \tag{6.8}$$

The depth limitation for operation as a weir becomes: $d \leq h$.

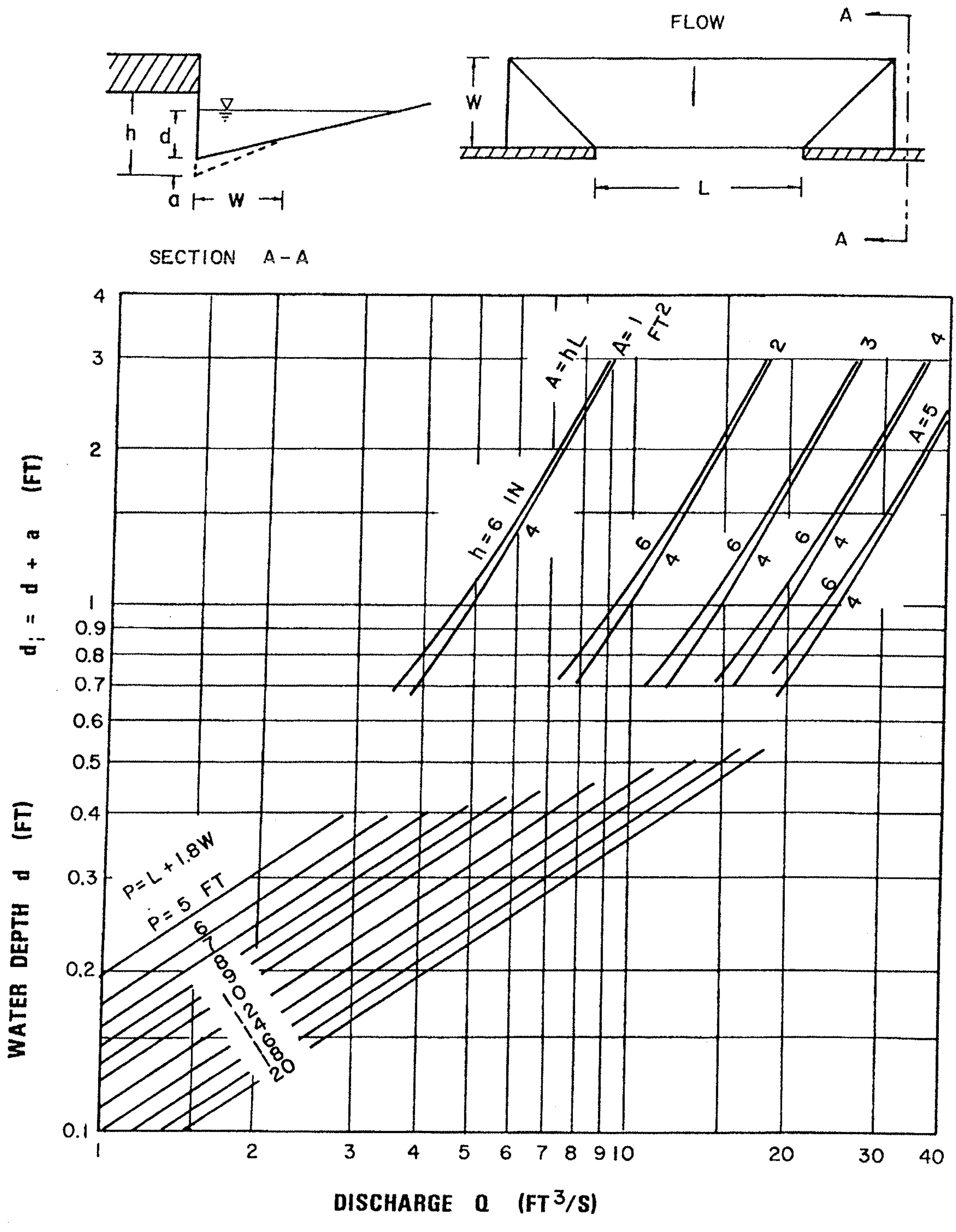
Curb-opening inlets operate as orifices at depths greater than approximately 1.4 × height of curb opening. The interception capacity can be computed by:

$$Q_i = C_o A[2g(d_i - h/2)]^{0.5} \tag{6.9}$$

where:

C_o = orifice coefficient (0.67)
h = height of curb-opening orifice, ft
A = clear area of opening, ft^2
d_i = depth at lip of curb opening, ft

The weir equations use an effective weir length and coefficient that is representative of the line of gutter transition to the depression. The user should be cautioned not to use the depth from the water surface to the depressed inlet throat, but to the undepressed depth (or more specifically, the



Depressed Curb-opening Inlet Capacity in Sump Locations - English Units

FIGURE 6-7. Depressed Curb-Opening Inlet Capacity in Sump Locations

Source: HEC 22 *(6)*.

depth at the beginning of the transition). Otherwise, the capacity for weir flow will be overestimated.

Note: Equation 6.9 is applicable to depressed and undepressed curb-opening inlets, and the depth at the inlet includes any gutter depression.

Example Problem

The following Example illustrates the use of this procedure:

Given: Curb-opening inlet in a sump location:

$L = 5$ ft $\quad h = 5$ in

(1) Undepressed curb opening:

$S_x = 0.05 \quad T = 8$ ft

(2) Depressed curb opening:

$S_x = 0.05 \quad W = 2$ ft
$a = 2$ in $\quad T = 8$ ft

Find: Q_i

Solution: (1) $d = TS_s = (8)(0.05) = 0.4$ ft $\quad d < h$; therefore, weir controls
$Q_i = C_W L d^{1.5} = (2.3)(5)(0.4)^{1.5} = 2.9$ ft^3/s

(2) $d = 0.4$ ft $< (1.4\ h) = 0.6$; therefore, weir controls
$P = L + 1.8W = 5 + 1.8(2) = 8.6$ ft
$Q_i = (2.3)(8.6)(0.4)^{1.5} = 5$ ft^3/s (Figure 6-7)

At $d = 0.4$ ft, the depressed curb-opening inlet has about 70 percent more capacity than an inlet without depression. In practice, the flow rate would be known and the depth at the curb would be unknown.

6.5.4 *Inlet Spacing Computations*

To design the location of the inlets for a given project, information such as a layout or plan sheet suitable for outlining drainage areas, road profiles, typical cross sections, grading cross sections, superelevation diagrams and contour maps are necessary. The inlet computation sheet, Figure 6-8, should be used to document the computations. A step-by-step procedure is as follows:

Step 1	Complete the blanks on top of the sheet to identify the job by S.P., route, date and your initials.
Step 2	Mark on the plan the location of inlets that are necessary even without considering any specific drainage area. See Section 6.4.2 for additional information.
Step 3	Start at one end of the job, at one high point and work towards the low point, then space from the other high point back to the same low point.
Step 4	Select a trial drainage area approximately 300 to 400 ft below the high point, and outline the area including any area that may come over the curb. (Use drainage area maps). Large areas of behind-the-curb drainage (producing 6 cfs or more during the design storm) should be intercepted before crossing the back of curb.
Step 5 (Col 1)	Describe the location of the proposed inlet by number and station in Columns 1 and 2.
(Col 5)	Identify the curb and gutter type in the Remarks, Column 19. A sketch of the cross section should be provided in the open area of the computation sheet.
Step 6 (Col 3)	Compute the drainage area in acres and enter in Column 3.
Step 7 (Col 4)	Select a *C* value from one of the tables in Chapter 2, Section 2.4, or compute a weighted value based on area and cover type.
Step 8 (Col 5)	Compute a time of concentration for the first inlet. This will be the travel time from the hydraulically most remote point in the drainage area to the inlet. See additional discussion in Chapter 2. The minimum time of concentration should be 5 minutes. Enter value in Column 5.
Step 9 (Col 6)	Using the Intensity-Duration-Frequency curves, select a rainfall intensity at the t_c for the design frequency. Enter in Column 6.
Step 10 (Col 7)	Calculate Q by multiplying Column 3 × Column 4 × Column 6. Enter in Column 7.
Step 11 (Col 8)	Determine the gutter slope at the inlet from the profile grade—check effect of superelevation. Enter in Column 8.

FIGURE 6-8. Inlet Spacing Computation Sheet

INLET COMPUTATION SHEET

DATE ________ SP ________ ROUTE ________

COMPUTED BY ________ SHEET ___ OF ___

LOCATION		GUTTER DISCHARGE Design Frequency ______					GUTTER DISCHARGE Allowable Spread ______								INLET DISCHARGE			REMARKS
INLET NO.	STAT.	DRAIN AREA "A" (ac)	RUNOFF COEF "C"	TIME OF CONC. "t_c" (min)	Rain Intensity "I" (in./h)	Q= CIA/360 (ft³/s)	GRADE "S_o" (ft/ft)	CROSS SLOPE "S_x" (ft/ft)	PREV RUNBY (ft³/s)	TOTAL GUTTER FLOW (ft³/s)	DEPTH "d" (ft)	GUTTER WIDTH "W" (ft)	SPREAD "T" (ft)	W/T	INLET TYPE	INTER-CEPT "Q_i" (ft³/s)	RUNBY "Q_r" (ft³/s)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19

Step 12 (Col 9) (Col 13)	Enter cross slope adjacent to inlet in Column 9 and gutter width in Column 13. Sketch composite cross slope with dimensions.
Step 13 (Col 11)	For the first inlet in a series (high point to low point), enter Column 7 in Column 11 since no previous runby has occurred yet.
Step 14 (Col 12) (Col 14)	Using Figure 6-1 or the available computer model, determine the spread T, enter in Column 14, and calculate the depth d at the curb by multiplying T times the cross slope(s) and enter in Column 12. Compare with the allowable spread as determined by the design criteria in Section 6.2. If Column 15 is less than the curb height and Column 14 is near the allowable spread, continue on to Step 16. If not OK, expand or decrease the drainage area to meet the criteria and repeat Steps 5 through 14. Continue these repetitions until Column 14 is near the allowable spread, then proceed to Step 15.
Step 15 (Col 15)	Calculate *W/T* and enter in Column 15.
Step 16 (Col 16)	Select the inlet type and dimensions and enter in Column 16.
Step 17 (Col 17)	Calculate the Q intercepted (Q_i) by the inlet and enter in Column 17. Use Figures 6-1 and 6-2 or 6-3 to define the flow in the gutter. Use Figures 6-5 and 6-6 to calculate Q_i for a curb-opening inlet. See Section 6.5.2 for a curb-opening inlet example.
Step 18 (Col 18)	Calculate the runby by subtracting Column 17 from Column 11, and enter into Column 18 and Column 10 on the next line if an additional inlet exists downstream.
Step 19 (Col 1-4)	Proceed to the next inlet downgrade. Select an area approximately 300 to 400 ft below the first inlet as a first trial. Repeat Steps 5 through 7 considering only the area between the inlets.
Step 20 (Col 5)	Compute a time of concentration for the second inlet downgrade based on the area between the two inlets.
Step 21 (Col 6)	Determine the intensity based on the time of concentration determined in Step 19 and enter in Column 6.

Step 22 (Col 7) Determine the discharge from this area by multiplying Column 3 × Column 4 × Column 6. Enter the discharge in Column 7.

Step 23 (Col 11) Determine total gutter flow by adding Column 7 and Column 10 and enter in Column 11. Column 10 is the same as Column 18 from the previous line.

Step 24 (Col 12) (Col 14) Determine "*T*" based on total gutter flow (Column 11) by using Figure 6-1 or 6-3 and enter in Column 14. (If "*T*" in Column 14 exceeds the allowable spread, reduce the area and repeat Steps 19 through 24. If "*T*" in Column 14 is substantially less than the allowable spread, increase the area and repeat Steps 19 through 24).

Step 25 (Col 16) Select inlet type and dimensions and enter in Column 16.

Step 26 (Col 17) Determine Q_i and enter in Column 17—see instruction in Step 17.

Step 27 (Col 18) Calculate the runby by subtracting Column 17 from Column 7 and enter in Column 16. This completes the spacing design for this inlet.

Step 28 Go back to Step 19 and repeat Step 19 through Step 27 for each subsequent inlet. If the drainage area and weighted "*C*" values are similar between each inlet, the values from the previous inlet location can be reused. If they are significantly different, recomputation will be required.

6.6 STORM DRAINS

6.6.1 *Introduction*

After the preliminary locations of inlets, connecting pipes and outfalls with tailwaters have been determined, the next logical step is the computation of the rate of discharge to be carried by each reach of the storm drain, and the determination of the size and gradient of pipe required to convey this discharge. This is done by starting at the upstream reach, calculating the discharge and sizing the pipe, then proceeding downstream, reach by reach, to the point where the storm drain connects with other drains or the outfall.

The rate of discharge at any point in the storm drain is not necessarily the sum of the inlet flow rates of all inlets above that section of storm drain. It is generally less than this total. The time of concentration is most influential and, as the time of concentration grows larger, the rainfall intensity to be used in the design grows smaller. In some cases, where a relatively large drainage area with a short time of concentration is added to the system, the peak flow may be larger using the shorter time even though the entire drainage area is not contributing. The prudent designer will be alert for unusual conditions and determine which time of concentration controls for each pipe segment.

For ordinary conditions, storm drains should be sized on the assumption that they will flow full or practically full under the design discharge but will not flow under pressure head. The Manning's formula is recommended for capacity calculations.

6.6.2 *Design Procedures*

The design of storm drainage systems is generally divided into the following operations:

Step 1 Determine inlet location and spacing as outlined earlier in this chapter.

Step 2 Prepare the plan layout of the storm drainage system establishing the following design data:

a. location of storm drains;

b. direction of flow;

c. location of inlets and junction boxes; and

d. location of existing utilities (e.g., water, gas, underground cables and existing and proposed foundations).

Step 3 Determine drainage areas, runoff coefficients and a time of concentration to the first inlet. Using an Intensity-Duration-Frequency (IDF) curve, determine the rainfall intensity (i). Calculate the discharge by (CiA).

Step 4 Size the pipe to convey the discharge by varying the slope and pipe size as necessary. The storm drain systems are normally designed for full gravity-flow conditions using the design frequency discharges.

Step 5 Calculate travel time in the pipe to the next inlet or junction box by dividing pipe length by the velocity. This travel time is added to the time of concentration for a new time of concentration and a new rainfall intensity at the next entry point.

Step 6 Calculate the new area (A) and multiply by the runoff coefficient (C), add to the previous (CA) and multiply by the new rainfall intensity to determine the new discharge. Determine the size of pipe and slope necessary to convey the discharge.

Step 7 Continue this process to the storm drain outlet. Utilize the equations and/or nomographs to accomplish the design effort.

Step 8 Complete the design by calculating the hydraulic grade line as described in Section 6.6.4. The design procedure should include the following:

- Storm drain design computations can be made on forms as illustrated in Figure 6-13.
- All computations and design sheets should be clearly identified. The designer's initials and date of computations should be shown on every sheet. Voided or superseded sheets should be so marked. The origin of data used on one sheet but computed on another should be given.

6.6.3 *Hydraulic Capacity*

The most widely used formula for determining the hydraulic capacity of storm drains for gravity and pressure flows is the Manning's formula, expressed by the following equation:

$$V = \frac{1.486}{n} R^{2/3} \quad S^{1/2} \tag{6.10}$$

where:
V = mean velocity of flow, ft/s
n = Manning's roughness coefficient
R = hydraulic radius, ft = area of flow divided by the wetted perimeter (A/WP)
S = the slope of the energy grade line, ft/ft

In terms of discharge, the above formula becomes:

$$Q = VA = \frac{1.486}{n} AR^{2/3} \quad S^{1/2} \tag{6.11}$$

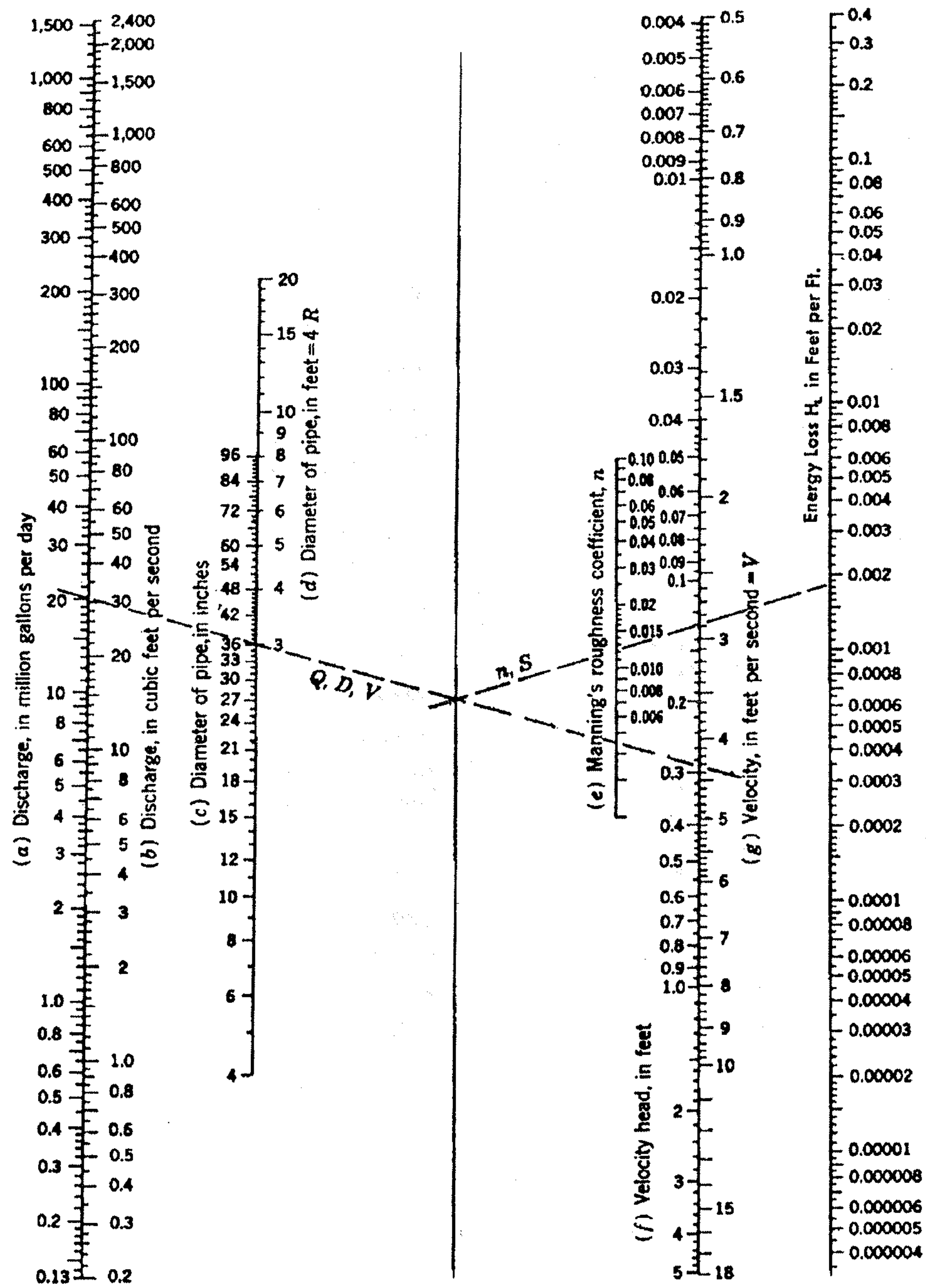
where:
Q = rate of flow, ft^3/s
A = cross sectional area of flow, ft^2

For circular storm drains flowing full, $R = D/4$ and Equations 6.10 and 6.11 become:

$$V = \frac{0.590}{n} D^{2/3} S^{1/2} \qquad Q = \frac{0.463}{n} D^{8/3} S^{1/2} \tag{6.12}$$

where:
D = diameter of pipe, ft

The nomograph solution of Manning's formula for full flow in circular storm drains is shown on Figure 6-9, Figure 6-10, and Figure 6-11. Figure 6-12 has been provided to assist in the solution of Manning's Equation for partial full flow in storm drains.



Alignment chart for energy loss in pipes, for Manning's formula.
Note: Use chart for flow computations, $H_L = S$

Solution of Manning's Equation for Flow in Storm Drains - English Units
(Taken from "Modern Sewer Design" by American Iron and Steel Intitute)

FIGURE 6-9. Manning's Formula for Full Flow in Storm Drains

Source: HEC 22 *(6)*.

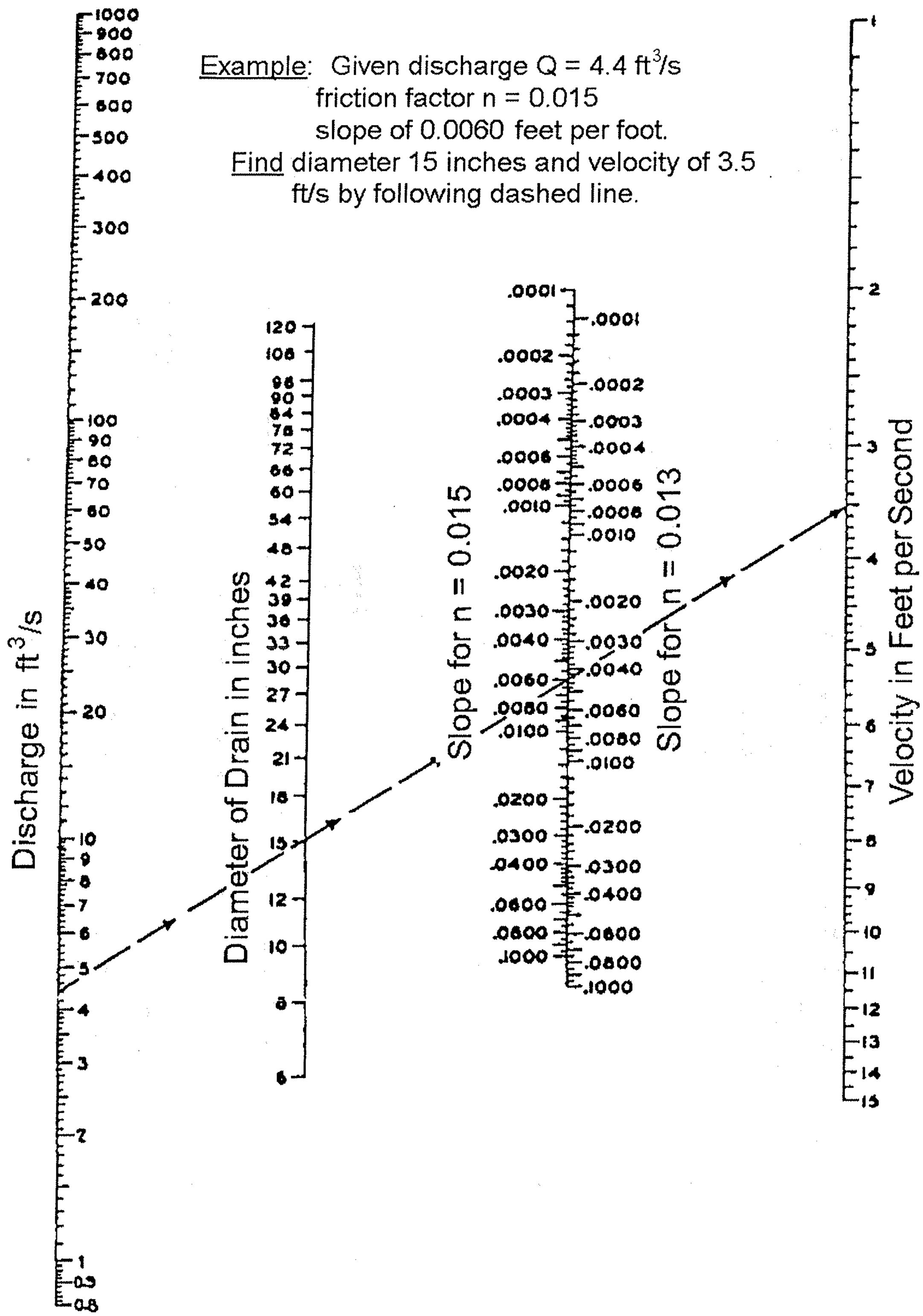


FIGURE 6-10. Nomograph for Computing Required Size of Circular Drain for Full Flow (n = 0.013 or 0.015)

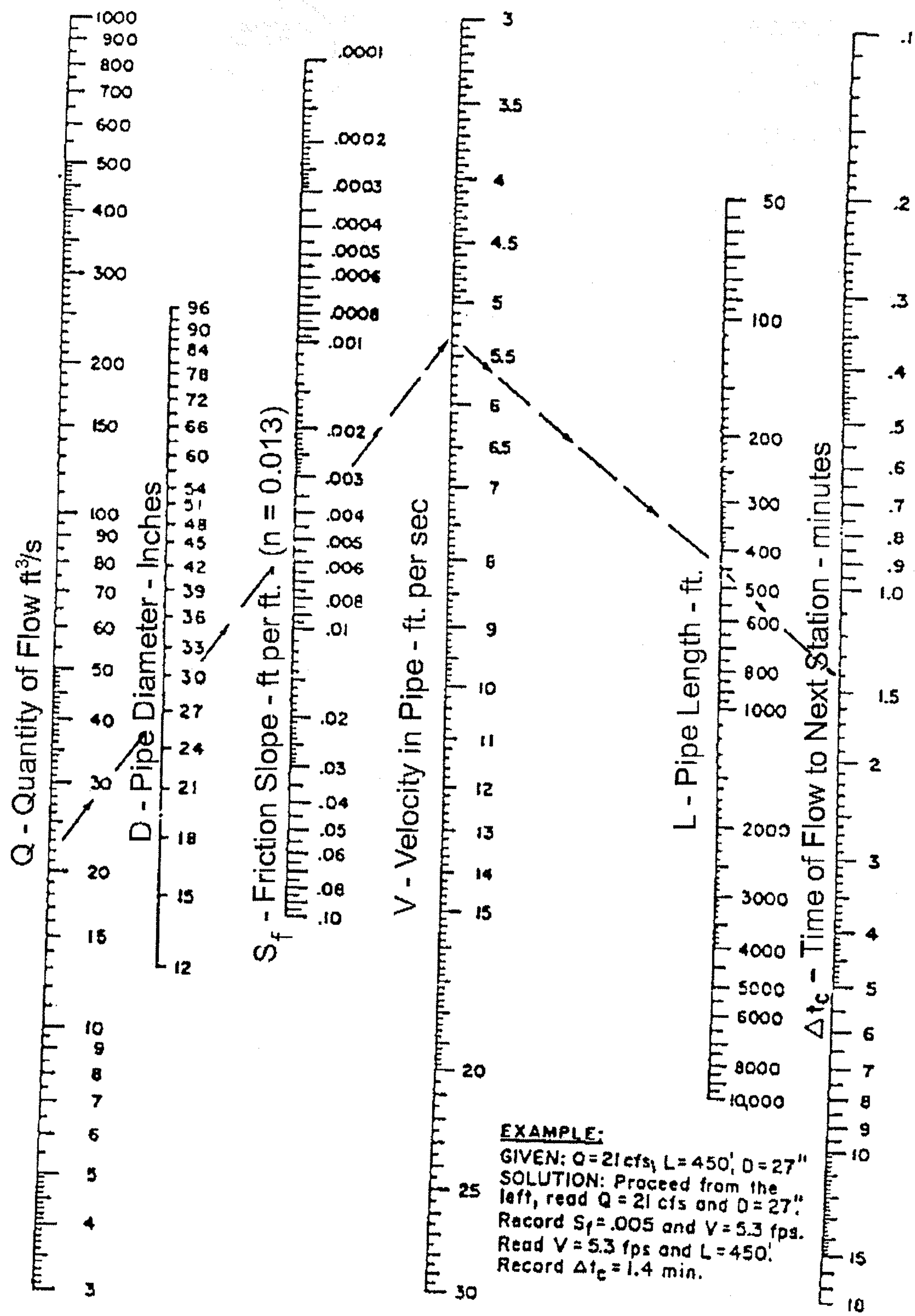
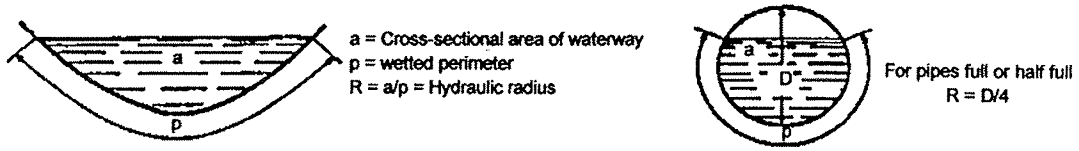


FIGURE 6-11. Concrete Pipe Flow Nomograph

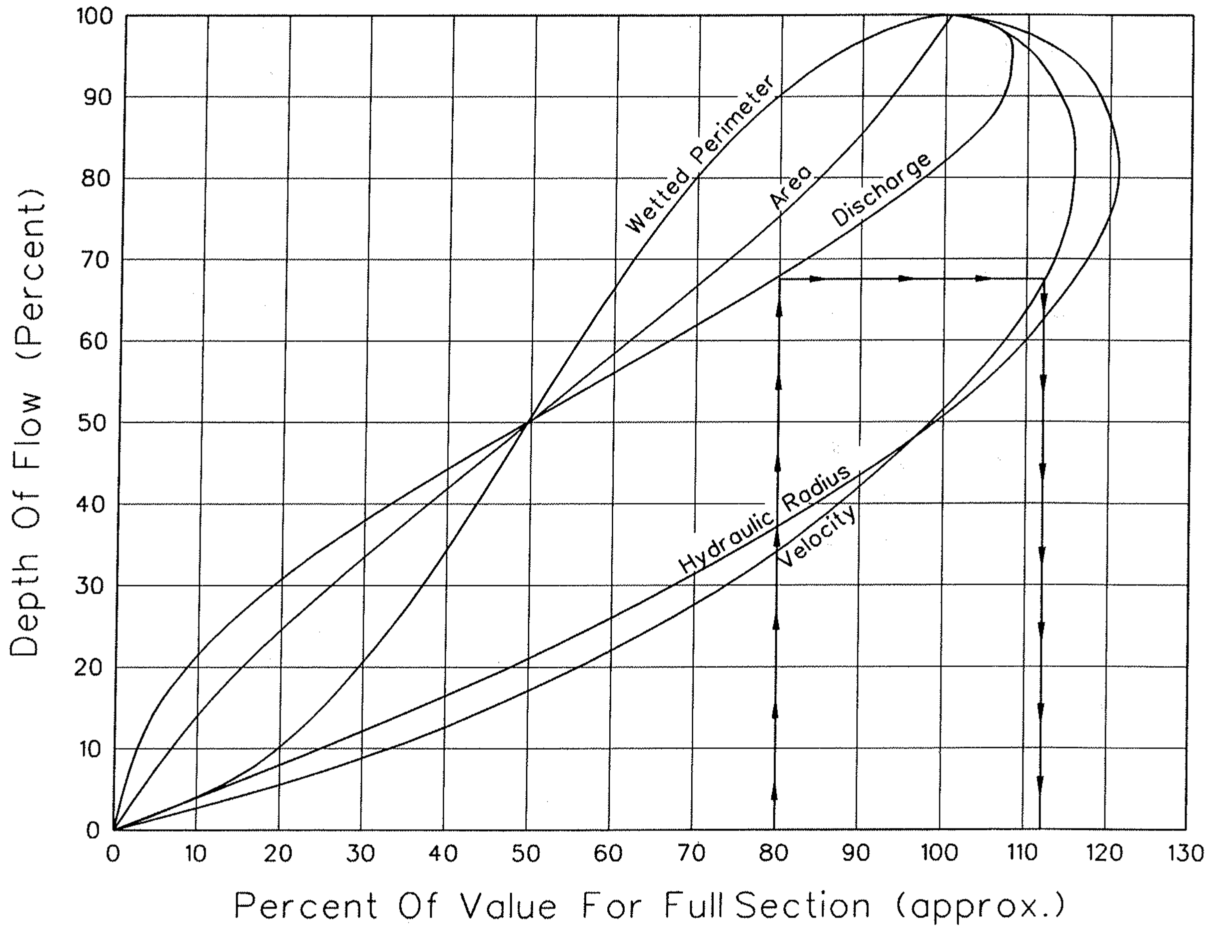


Section of Any Channel

Section of Circular Pipe

V = Average or mean velocity in ft/s
Q = a V = Discharge of pipe or channel in ft^3/s
n = Coefficient of roughness of pipe or channel surface
S = Slope of hydraulic gradient (water surface in open channels or pipes not under pressure, same as slope of channel or pipe invert only when flow is uniform in constant section)

Hydraulic Elements of Channel Sections



Source: Reference *(2)*.

FIGURE 6-12. Values of Hydraulic Elements of Circular Section for Various Depths of Flow

Storm Drain Computation Sheet

Computed ________ Date ________

Checked ________ Date ________

Route ______________

Section ______________

County ______________

Station		Length (ft)	Drainage Area A (ac)		Runoff Coefficient C	A x C		Flow Time (min)		Rainfall Intensity I (in/h)	Total Runoff 0.0278 C I A = Q (ft³/s)	Diameter Pipe (in)	Capacity Full (ft³/s)	Velocity (ft/s)		Inert Elev.		Manhole Invert Drop	Slope of Drain (ft/ft)
From	To		Increment	Total		Increment	Total	To Upper End	In Section					Flowing Full	Design Flow	Upper End	Lower End		

FIGURE 6-13. Storm Drain Computation Sheet

6.6.4 *Hydraulic Grade Line*

The hydraulic grade line (HGL) is the last important feature to be established for the hydraulic design of storm drains. This grade line aids the designer in determining the acceptability of the proposed system by establishing the elevations along the system to which the water will rise when the system is operating from a flood of design frequency.

All head losses in a storm drainage system are considered in computing the hydraulic grade line to determine the water surface elevations, under design conditions in the various inlets, junction boxes, etc. Hydraulic control is a set water surface elevation from which the hydraulic calculation are begun. All hydraulic controls along the alignment are established. If the control is at a mainline upstream inlet, the hydraulic grade line is the water surface elevation minus the entrance loss minus the differences in velocity head. If the control is at the outlet, the water surface is the outlet pipe hydraulic grade line.

6.6.4.1 Design Procedure

The head losses are calculated beginning from the control point to the first junction and the procedure is repeated for the next junction. The computation for an outlet control may be tabulated on Figure 6-14 using the following procedure:

Step 1 Enter in Col. 1 the station for the junction immediately upstream of the outflow pipe. Hydraulic grade line computations begin at the outfall and are worked upstream taking each junction into consideration.

Step 2 Enter in Col. 2 the outlet water surface elevation if the outlet will be submerged during the design storm or 0.8 diameter plus invert out elevation of the outflow pipe, whichever is greater.

Step 3 Enter in Col. 3 the diameter (D_o) of the outflow pipe.

Step 4 Enter in Col. 4 the design discharge (Q_o) for the outflow pipe.

Step 5 Enter in Col. 5 the length (L_o) of the outflow pipe.

Step 6 Enter in Col. 6 the friction slope (SF_o) in ft/ft of the outflow pipe. This can be determined by using the following formula:

$$SF = [(Qn)/(1.49AR^{2/3})]^2 \qquad (6.13)$$

where:
SF = friction slope, ft/ft

Step 7 Multiply the friction slope (SF_o) in Col. 6 by the length (L_o) in Col. 5 and enter the friction loss (H_f) in Col. 7.

HYDRAULIC GRADE LINE

PROJECT

STATION	Outlet Water Surface Elev.	D_o	Q_o	L_o	S_{f_o} %	H_f	JUNCTION LOSS											Final H	Inlet Water Surface Elev.	Rim Elev.
							V_o	H_o	Q_i	V_i	Q_iV_i	$\frac{V_i^2}{2g}$	H_i	Angle	H_Δ	H_t	1.3 H_t	0.5 H_t			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)		(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)

$H_i = 0.35\frac{V_i^2}{2g}$ $H_o = 0.25\frac{V_o^2}{2g}$ $H_\Delta = K\frac{V_i^2}{2g}$ FINAL $H = H_f + H_t$

$H_t = H_o + H_i + H_\Delta$

Figure 6-14. Hydraulic Grade Line Computation Form

Step 8 Enter in Col. 8 the outlet pipe velocity of the flow (V_o).

Step 9 Enter in Col. 9 the contraction loss (H_o) by using the formula $H_o = [0.25(V_o^2)]/2g$, where $g = 32.2$ ft/s^2.

Step 10 Enter in Col. 10 the design discharge (Q_i) for each pipe flowing into the junction, except lateral pipes with inflows of ten percent or less of the mainline outflow. Inflow must be adjusted to the mainline outflow duration time before a comparison is made.

Step 11 Enter in Col. 11 the velocity of flow (V_i) for each pipe flowing into the junction (for exception see Step 10).

Step 12 Enter in Col. 12 the product of Q_i x V_i for each inflowing pipe. When several pipes inflow into a junction, the line producing the greatest Q_i x V_i product is the line which will produce the greatest expansion loss (H_i). (For exception, see Step 10).

Step 13 Enter in Col. 13 the controlling expansion loss (H_i) using the formula $H_i = [0.35(V_i^2)]/2g$.

Step 14 Enter in Col. 14 the angle of skew of each inflowing pipe to the outflow pipe (for exception, see Step 10).

Step 15 Enter in Col. 15 the greatest bend loss (H_Δ) calculated by using the formula $H_\Delta = [K(V_i^2)]/2g$, where K = the bend loss coefficient corresponding to the various angles of skew of the inflowing pipes (see Figure 6-15).

Step 16 Enter in Col. 16 the total head loss (H_t) by summing the values in Col. 9 (H_o), Col. 13 (H_i), and Col. 15 (H_Δ).

Step 17 If the junction loss incorporates adjusted surface inflow of ten percent or more of the mainline outflow, i.e., drop inlet, increase H_t by 30 percent and enter the adjusted H_t in Col. 17.

Step 18 If the junction incorporates partial diameter inlet shaping, reduce the value of H_t by 50 percent and enter the adjusted value in Col. 18.

Step 19 Enter in Col. 19 the FINAL H, the sum of H_f and H_t, where H_t is the final adjusted value of H_t.

Step 20 Enter in Col. 20 the sum of the elevation in Col. 2 and the final H in Col. 19. This elevation is the potential water surface elevation for the junction under design conditions.

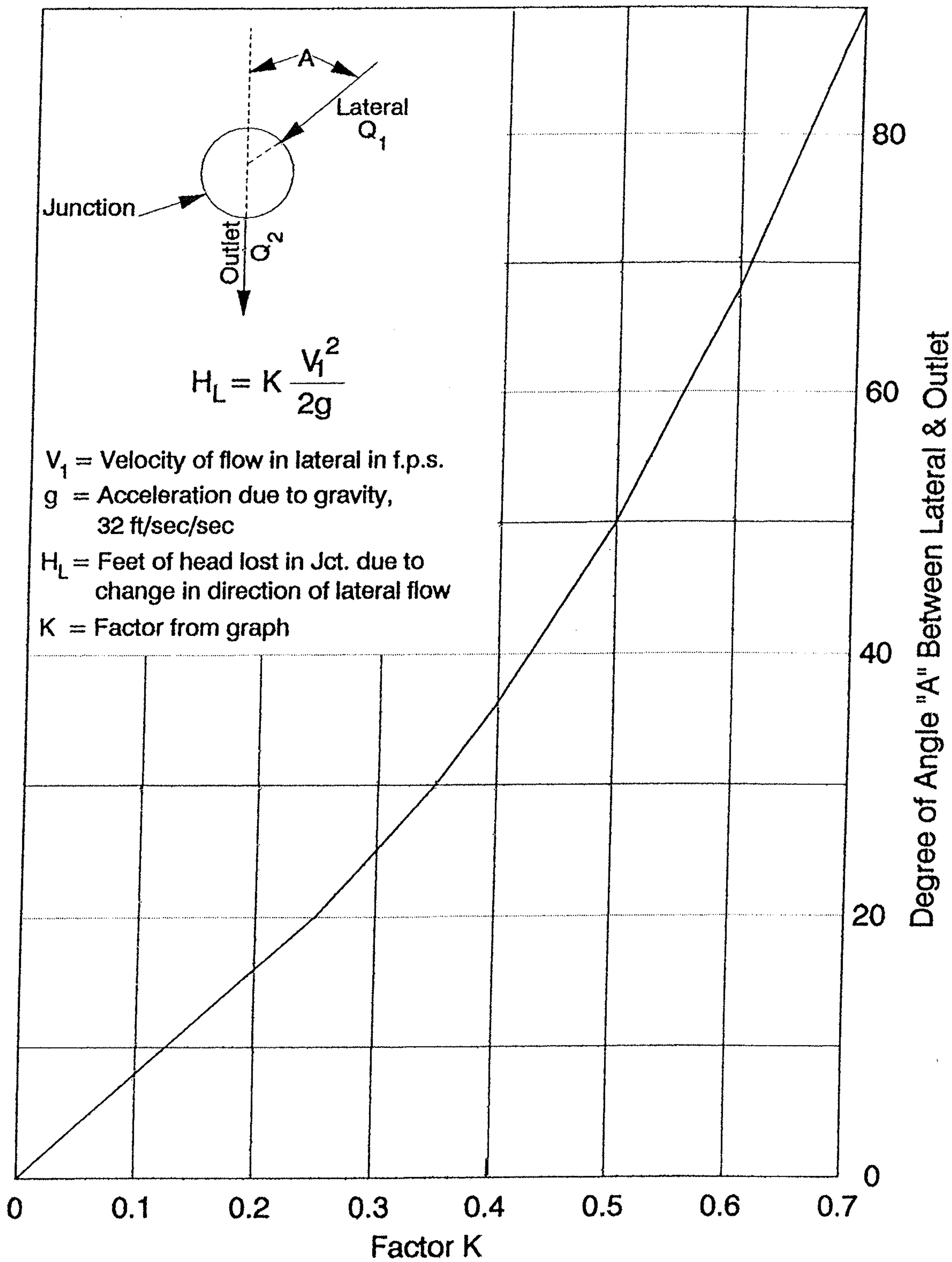


Figure 6-15. Loss In Junction Due To Change In Direction Of Flow In Lateral

Step 21 Enter in Col. 21 the rim elevation or the gutter flow line, whichever is lowest, of the junction under consideration in Col. 20. If the potential water surface elevation exceeds the 9-inch freeboard elevation (9 inches below the rim elevation or gutter flow line), adjustments are needed in the system to reduce the elevation of the hydraulic grade line.

Step 22 Repeat the procedure starting with Step 1 for the next junction upstream.

6.6.4.2 Computer Methods

Suitable computer programs may be used to determine the hydraulic grade line. All programs used must have approval of the Engineering Department.

6.7 REFERENCES

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(4) FHWA. *HYDRAIN, Drainage Design Computer System*, Version 6.1. FHWA-IF-99-008. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 1999.

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