URBAN STORM DRAINAGE

CRITERIA MANUAL

Volume 1

Urban Drainage and Flood Control District
Denver, Colorado June 2001

Revised April 2008

URBAN STORM DRAINAGE

CRITERIA MANUAL

Volume 2

Urban Drainage and Flood Control District
Denver, Colorado June 2001
Revised April 2008

Drainage Criteria Manual
(Volume 1)

June 2001

Revised April 2008

Urban Drainage and Flood Control District

Drainage Criteria Manual
(Volume 2)

June 2001

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Urban Drainage and Flood Control District

SUMMARY OF CHANGES TO VOLUME 1
of the
URBAN STORM DRAINAGE CRITERIA MANUAL
and
DISCLAIMER

2001 Edition vs. 1969 Edition

GENERAL

- All chapters edited; some totally rewritten.
- Many design aids added, including figures, nomographs, spreadsheets, etc.
- New chapters on Revegetation and Design Examples added.
- Emphasis on maintenance, public safety, aesthetics and multidisciplinary design approaches.
- Design checklists added to many chapters.
- Stronger emphasis on "designing with nature" principles such as "bioengineering."

POLICY CHAPTER

- Provides increased emphasis on staying out of the 100-year floodplain.
- Recommends reducing runoff rates, volumes and pollutants to the maximum extent practicable.
- Recommends reserving sufficient rights-of-way for lateral movement of incised floodplains.
- Clarifies the role of irrigation ditches in urban drainage.
- Revises street inundation criteria for the 100-year flood.

DRAINAGE LAW CHAPTER

- Contents totally updated.

PLANNING CHAPTER

- Also addresses the areas now being emphasized in the Policy chapter.

RAINFALL CHAPTER

- Adds a 25-year design storm and its distribution.
- Provides spreadsheets for calculations of design storms and IDF curves.
- Expands rainfall maps to include new areas of District added since 1969.

RUNOFF CHAPTER

- Clarifies the use of flows published in District's master plans and other reports.
- Also clarifies the use and applicability of statistical analysis.
- Provides spreadsheets for the Rational Method and CUHP calculations.
- Describes the use of CUHP and UDSWM software.
- Includes new procedure for calculating the runoff coefficient "*C*" in the Rational Formula.
- Clarifies which hydrologic methods to use as a function of watershed size.

STREETS/INLETS/STORM SEWERS CHAPTER

- Combines three separate chapters on design of streets, inlets and storm sewers.
- Uses protocols from the Federal Highway Administration Engineering Circular Nos. 12 and 22.
- Includes reduction factors for allowable gutter/street flow.
- Provides an inlet capacity reduction protocol that accounts for inlet clogging.
- Also provides spreadsheets for calculations and design examples.

MAJOR DRAINAGE CHAPTER

- Includes expanded and updated design guidance and criteria for each channel type.
- Provides guidance for protection of natural channels from effects of urbanization.
- Adds new section on bioengineered channel design.
- Includes new guidance on use and design of composite channels.
- Adds text on the fundamentals of open channel hydraulics and stream stability.
- Updates text on 404 permitting.

- Revises guidance for sizing trickle channels and low-flow channels.
- Includes new criteria for design of boulders and grouted boulders.
- Provides spreadsheets as design aids.

2002 through 2005 Revisions to 2001 Edition

ENTIRE VOLUME 1

2005-03: Reformat entire Volume 1 to facilitate future updates. (Significant Revision)

RUNOFF CHAPTER

2004-01: Correct typos on Page RO-35.

MAJOR DRAINAGE CHAPTER

2002-06: Correct Table MD-2.
2004-01: Revise text on Page MD-62 and MD-105 and add Figure MD-25. (Significant Revision)

STREETS/INLETS/STORM SEWERS CHAPTER

2002-06: Correct units in Eq. ST-8 and correct Eq. ST-25. (Significant Revision)
2002-06: Replace Sections 4.4.2 and 4.4.13 and UDSEWER example. (Major Revision)
2003-03: Corrects Eq. ST-17. (Significant Revision)

August 2006 Update to 2001 Edition

RUNOFF CHAPTER

- Updated description of CUHP to use of CUHP 2005 software and EPA SWMM 5.0 for routing
- Deleted use of UDSWM and described EPA SWMM 5.0 for routing CUHP 2005 hydrographs.

MAJOR DRAINAGE CHAPTER

- Cleaned up a number of figures using AutoCAD™
- Expanded on the description on use of trickle and low flow channels in grass-lined channels.
- Modified submittal checklist to include some design elements not previously listed in them.
- Clarified Froude Number and Velocity limitations for concrete and riprap lined channels.
- Clarified that concrete-lined channels are not maintenance eligible.
- Expanded the use of soil riprap to now include VL, L and M riprap sizes.
- Clarified the minimum embedment of riprap bank and channel toe lining for sandy soils.
- Clarified the need to check rock sizes for increased velocities at channel bends and transitions.
- Clarifies the use of soil-riprap lining side-slopes above the low-flow section of a channel.
- Added a figure relating grass cover type, velocity, depth and Manning's *n* in grass-lined channels.
- Added details for soil-riprap installation.
- Expanded on the need for air-venting when rectangular storm sewers are used.
- Clarified importance of pipe entrance(s) in design.
- Modified examples to reflect latest spreadsheet workbooks.

April 2008 Update to 2001 Edition

MAJOR DRAINAGE CHAPTER

- Revised Section 4.4.1.2 (page MD-62) to recommend grouted boulders on banks be covered with topsoil and vegetated.

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SUMMARY OF CHANGES TO VOLUME 2
of the
URBAN STORM DRAINAGE CRITERIA MANUAL
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DISCLAIMER

2001 Edition vs. 1969 Edition

GENERAL

- All chapters edited or rewritten. Emphasis on maintenance, public safety, and aesthetics.
- Many design aids added, including figures, nomographs, spreadsheets, etc.
- New chapters on Revegetation and Design Examples added.
- Design checklists added to many chapters.
- Stronger emphasis on “designing with nature” principles such as “bioengineering.”

HYDRAULIC STRUCTURES CHAPTER

- Revises drop structure design criteria and details.
- Provides guidance on safety considerations for boatable channels.
- Includes section on “rundowns” to convey flows into major drainageways and storage facilities.
- Also includes section on design of low tailwater basins at storm sewer discharge locations.

CULVERTS CHAPTER

- Significant changes. Emphasizes public safety with use of trash/safety racks at entrances.
- Provides minimum trash rack size guidance.
- Discourages use of grates or racks at pipe outlets.

STORAGE CHAPTER

- Totally rewritten. Emphasizes designing for maintainability, aesthetics and safety.
- Addresses protecting against catastrophic failures due to overtopping embankments.
- Gives alternative techniques for preliminary and final design sizing of facilities.
- Added guidance on sizing and use of retention facilities.
- Includes use of spreadsheets for aid in preliminary and final design.
- Provides for consistency with Volume 3 of the *Manual*.

FLOODPROOFING CHAPTER

- Contents completely rewritten, draws heavily on FEMA guidance.

REVEGETATION CHAPTER

- New chapter: provides guidance on preparation of a planting plan and use of soil amendments.
- Provides grass and wildflower seed mixes for different soil and moisture conditions.
- Lists recommended shrubs, trees and planting techniques.
- Gives details on bioengineered elements including live staking, poling and willow bundles.
- Includes a revegetation process guidance chart.

DESIGN EXAMPLES CHAPTER

- Provides a variety of design examples from around the Denver metropolitan area.

2001 through 2005 Revisions to 2001 Edition of Volume 2

ENTIRE VOLUME 2
2005-03: Reformats entire Volume 2 to facilitate future updates. (Significant Revision)

HYDRAULIC STRUCTURES CHAPTER
2002-06: Correct Tables HS-4 and HS-7a4 and Figure HS-8. (Significant Revision)

CULVERT CHAPTER
2001-07: Rewritten *Trash Rack* Section. (Major Revision)

January 2007 Update to 2001 Edition

HYDRAULIC STRUCTURES CHAPTER

- Revised Manning's *n* and Boulder sizing recommendations for grouted boulders.
- Simplifies Grouted Sloping Boulder (GSB) drop design, increases allowable maximum drop for the simplified design from 5-feet to 6-feet.
- Adds a smaller Impact Energy Dissipating Basin for use with outlets 18- to 48-inches in diameter.
- Adds details for the design of an impact basin for pipe outlets 18" and smaller in diameter.
- Modifies the guidance on pipe outlet rundowns, including details for a Grouted Boulder Rundown
- Adds details for a GSB drop for use in channels with sandy/erosive soils.
- Modifies concrete check structure details and adds design guidance for a sheet-pile checks.
- Clarifies parts of *Detailed Hydraulic Analysis* section including guidance for Manning's *n* for Concrete, Boulders and Grouted Boulders.
- Clarifies guidance on the design of low tailwater riprap basins at pipe outlets.

STORAGE CHAPTER

- Clarifies a number of topics in the *Design Storms for Sizing Storage Volumes* section, including drainage and flood control issues, spillway sizing, retention facilities, outlet works design, etc.
- Claries the uses Rational Formula-Based Modified FAA Procedure as being applicable only for the sizing of single return period control on-site detention basins.
- Adds Full Spectrum Detention procedure for the design of on-site detention facilities for tributary areas of one square mile and less.
- Adds a section: *On-Site Detention and UDFCD 100-year Floodplain Management Policy*.
- Expands on the discussion on use of vegetation in detention basins.
- Revises the submittal checklist.

June 2007 Correction of the 2001 Edition's January 2007 Update

STORAGE CHAPTER

- Corrected Equation SO-13 and equations shown on Figure SO-8 for perforation sizing.

April 2008 Correction of the 2001 Edition's January 2007 Update

HYDRAULIC STRUCTURES CHAPTER

- Revised Figure HS-9 to make consistent with narrative recommendations.

STORAGE CHAPTER

- Revised Figure SO-6 to show protection on the downstream face of spillway and/or embankment.

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URBAN STORM DRAINAGE CRITERIA MANUAL

RAINFALL

RUNOFF

STREETS/INLETS/STORM SEWERS

MAJOR DRAINAGE

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1.0 ACKNOWLEDGEMENTS

The Urban Drainage and Flood Control District (District) wishes to acknowledge and thank all individuals and organizations that contributed to development and publication of the 2001 update of the *Urban Storm Drainage Criteria Manual, Volumes 1 and 2* (*Manual*). The lists of individuals and organizations that follow are our best effort to acknowledge all of the organizations and individuals that were directly involved in the *Manual's* preparation and the members of the Stormwater Manual Advisory Committee (SMAC) who provided technical guidance and review. We apologize to anyone that may have contributed to the development of this *Manual* and has not been listed here.

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CONTRIBUTORS TO ORIGINAL (1969) VERSION OF THE *MANUAL*

All residents of the Denver metropolitan area have benefited significantly from the pioneering vision of those who were responsible for the original (1969) version of the *Manual*, including D. Earl Jones, Jr., P.E., Dr. Jack Schaeffer, Dr. Gilbert White and Kenneth R. Wright, P.E. (lead author). The vast majority of the policies, principles, and criteria in the 1969 *Manual* are found in this updated (2001) version—a true testament to the wisdom of these leaders.

2.0 PURPOSE OF THE *MANUAL*

Volumes 1 and 2 of the *Manual* provide guidance to local jurisdictions, developers, contractors, and industrial and commercial operations in selecting, designing, constructing, and maintaining stormwater drainage and flood control facilities. This *Manual* covers a variety of topics, including the following:

Chapter 1	Drainage Policy
Chapter 2	Drainage Law
Chapter 3	Planning
Chapter 4	Rainfall
Chapter 5	Runoff
Chapter 6	Streets/Inlets/Storm Sewers
Chapter 7	Major Drainage
Chapter 8	Hydraulic Structures
Chapter 9	Culverts
Chapter 10	Storage
Chapter 11	Flood Proofing
Chapter 12	Revegetation
Chapter 13	Design Examples

A reference section is provided for each chapter, and additional materials and insight on the topics presented in the *Manual* may be found by studying the papers and documents listed at the end of each chapter. Additionally, on the CD version of this *Manual*, a series of design spreadsheets and software are provided including:

Software	**Spreadsheets**
CUHP 2000	UD-Channels
HY8 (FHWA HY8 Culvert Analysis Microcomputer Program)	UD-Culvert
UD CHANNEL	UD-Detention
UD POND (Hydropond)	UD-Inlet
UD INLET	UD-Raincurve

UD SEWER	UD-Rainzone
UDSWM	UD-Rational

These software programs may also be obtained through the District's Web site (www.udfcd.org, under DOWNLOADS), which should be checked for periodic updates.

The District's Maintenance Eligibility Guidelines as of June 2001 are also provided on the CD version of this *Manual*. Again, the District's Web site should also be checked for periodic updates.

The *Manual* provides the minimum criteria and standards recommended by the District. Providing for facilities that go beyond the minimum is encouraged. In addition, there may be other requirements by local, state and federal agencies that may have to be met in addition to the minimum criteria provided herein.

3.0 LIST OF ABBREVIATIONS

Commonly Used Abbreviations

ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BCD	Baffle chute drop
BFE	Base flood elevation
BMP	Best management practice
CDOT	Colorado Department of Transportation
CDPHE	Colorado Department of Public Health and Environment
CGIA	Colorado Governmental Immunity Act
CMP	Corrugated metal pipe
CRS	Colorado Revised Statutes
CUHP	Colorado Urban Hydrograph Procedure
CWA	Clean Water Act
CWCB	Colorado Water Conservation Board
DCIA	Directly connected impervious area
DRCOG	Denver Regional Council of Governments
EGL	Energy grade line
EPA	U.S. Environmental Protection Agency
FAA	Federal Aviation Administration
FEMA	Federal Emergency Management Agency
FHAD	Flood Hazard Area Delineation
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study

FPE	Flood protection elevation
GSB	Grouted sloping boulder
HGL	Hydraulic grade line
HUD	U.S. Department of Housing and Urban Development
H:V	Horizontal to vertical ratio of a slope
ICC	Increased cost of compliance
LID	Low impact development
MDCIA	Minimized directly connected impervious area
NAVD	North American Vertical Datum
NFIA	National Flood Insurance Act
NFIP	National Flood Insurance Program
NGVD	National Geodetic Vertical Datum
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System
NRCS	Natural Resources Conservation Service
PMP	Probable maximum precipitation
RCP	Reinforced concrete pipe
SBA	Small Business Administration
SEO	Colorado State Engineer's Office
SFHA	Special Flood Hazard Area
SFIP	Standard Flood Insurance Policy
SWMM	Stormwater Management Model
TABOR	Taxpayers Bill of Rights
TWE	Tailwater elevation
UDSWM	Urban Drainage Stormwater Management Model

USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
WEF	Water Environment Federation
VHB	Vertical hard basin
WQCV	Water quality capture volume

Commonly Used Units

cfs	cubic feet per second
cfs/ft	cubic feet per second per foot
ft	foot
ft^2	square feet
ft/ft	foot per foot
ft/sec	feet per second
ft/sec^2	feet per second squared
hr	hour
in	inch
in/hr	inches per hour
in/hr/ac	inches per hour per acre
lbs	pounds
lbs/cy	pounds per cubic yard
lbs/ft^2	pounds per square foot
lbs/ft^3	pounds per cubic foot
lbs PLS/acre	pounds pure live seed per acre
min	minute
psi	pounds per square inch
psf	pounds per square foot

DRAINAGE POLICY

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1.0 STATEMENT OF POLICY

1.1 Policy

Adequate drainage for urban areas is necessary to preserve and promote the general health, welfare, and economic well being of the region. Drainage is a regional feature that affects all governmental jurisdictions and all parcels of property. This characteristic of drainage makes it necessary to formulate a program that balances both public and private involvement (Wright-McLaughlin Engineers 1969). Overall, the governmental agencies most directly involved must provide coordination and master planning, but drainage planning must also be integrated on a regional level (FEMA 1995).

When planning drainage facilities, certain underlying principles provide direction for the effort. These principles are made operational through a set of policy statements. The application of the policy is, in turn, facilitated by technical drainage criteria and data. When considered in a comprehensive manner—on a regional level with public and private involvement—drainage facilities can be provided in an urban area in a manner that will avoid uneconomic water losses and disruption, enhance the general health and welfare of the region, and assure optimum economic and social relationships (White 1945).

Photograph DP-1—Denver grass-lined channel after 35 years of service. Ann Spirn of the Massachusetts Institute of Technology refers to this channel as "urban poetry" in her publications. Spirn appreciates the soft natural lines.

The principles and policies for urban storm drainage are summarized below.

1.2 Principles

- **Drainage is a regional phenomenon that does not respect the boundaries between government jurisdictions or between properties.**
- **A storm drainage system is a subsystem of the total urban water resource system.**
- **Every urban area has an initial and a major drainage system, whether or not they are actually planned and designed.**
- **Runoff routing is primarily a space allocation problem.**
- **Planning and design of stormwater drainage systems generally should not be based on the premise that problems can be transferred from one location to another.**
- **An urban storm drainage strategy should be a multi-objective and multi-means effort.**
- **Design of the stormwater drainage system should consider the features and functions of the existing drainage system.**
- **In new developments, attempts should be made to reduce stormwater runoff rates and pollutant load increases after development to the maximum extent practicable.**
- **The stormwater management system should be designed, beginning with the outlet or point of outflow from the project, giving full consideration to downstream effects and the effects of off-site flows entering the system.**
- **The stormwater management system should receive regular maintenance.**
- **Floodplains need to be preserved whenever feasible and practicable.**
- **Reserve sufficient right-of-way to permit lateral channel movement whenever the floodplain is contained within a narrow natural channel.**

1.3 Basic Knowledge

A program for collecting and analyzing storm runoff and flood data should be maintained in order that intelligent and orderly planning may be undertaken in regard to storm drainage facilities.

A program should be maintained to delineate flood hazard areas along all waterways in the region which are urbanized or which may be in the future. This program should make full use of the information and data from the Federal Emergency Management Agency (FEMA), the U.S. Geological Survey (USGS), private consulting engineers, and the Colorado Water Conservation Board. This information should be regularly reviewed and updated to reflect changes due to urbanization, changed channel conditions, and the occurrence of extraordinary hydrologic events.

Before commencing design of any drainage project, comprehensive facts and data should be collected and examined for the particular watershed and area under consideration, and the basis for the design should then be agreed upon by the governmental entities affected.

1.4 Planning

Storm drainage is a part of the total urban environmental system. Therefore, storm drainage planning and design must be compatible with comprehensive regional plans. A master plan for storm drainage should be developed and maintained in an up-to-date fashion at all times for each urbanizing drainage watershed in the Denver region. The planning for drainage facilities should be coordinated with planning for open space and transportation. By coordinating these efforts, new opportunities may be identified that can assist in the solution of drainage problems.

Natural drainageways should be used for storm runoff waterways wherever feasible. Major consideration must be given to the floodplains and open space requirements of the area (White 1945).

Planning and design of stormwater drainage systems should not be based on the premise that problems can be transferred from one location to another.

Stormwater runoff can be stored in detention and retention reservoirs. Such storage can reduce the drainage conveyance capacity required immediately downstream. Acquisition of open space having a relationship to drainageways will provide areas where storm runoff can spread out and be stored for slower delivery downstream.

1.5 Technical Issues

Storm drainage planning and design should follow the criteria developed and presented in this *Urban Storm Drainage Criteria Manual* (*Manual*).

Every urban area has two separate and distinct drainage systems, whether or not they are actually planned and designed. One is the initial system, and the other is the major system. To provide for orderly urban growth, reduce costs to future generations and avoid loss of life and major property damage, both systems must be planned, properly engineered and maintained.

The determination of runoff magnitude should be by the Rational Formula, the Colorado Urban Hydrograph Procedure (CUHP), or statistical analyses based on an adequate record of actual measured flood occurrences as set forth in the RUNOFF chapter of this *Manual*.

Use of streets for urban drainage should fully recognize that the primary use of streets is for traffic.

Streets should not be used as floodways for initial storm runoff. Urban drainage design should have as an objective reduction of street repair and maintenance costs to the public.

Irrigation ditches should not be used as outfall points for initial or major drainage systems, unless such use is shown to be without unreasonable hazard, as substantiated by thorough hydraulic engineering analysis, and written approval of the ditch owner(s) is obtained. In addition, irrigation ditches cannot be relied on for mitigating upstream runoff.

Proper design and construction of stormwater detention and retention basins are necessary to minimize future maintenance and operating costs and to avoid public nuisances and health hazards. This is particularly important, given the many detention and retention facilities in the Denver region.

The various governmental agencies within the Denver region have adopted and need to maintain their floodplain management programs. Floodplain management must encompass comprehensive criteria designed to encourage, where necessary, the adoption of permanent measures which will lessen the exposure of life, property and facilities to flood losses, improve the long-range land management and use of flood-prone areas, and inhibit, to the maximum extent feasible, unplanned and economically unjustifiable future development in such areas.

1.6 Flood Insurance

Flood insurance is an integral part of the strategy to manage flood losses. The Denver region should encourage continued participation in the National Flood Insurance Program, set forth in the National Flood Insurance Act (NFIA) of 1968, as amended.

1.7 Implementation

This *Manual* should continue to be adopted by all governmental agencies operating within the region. Each level of government is encouraged to participate in a successful drainage program.

Problems in urban drainage administration encountered by any governmental agency can be reviewed by the Urban Drainage and Flood Control District (District) to determine if equity or public interests indicate a need for drainage policy, practice, or procedural amendments (Figure DP-1).

The financing of storm drainage improvements is fundamentally the responsibility of the affected property owners—both the persons directly affected by the water and the person from whose land the water flows.

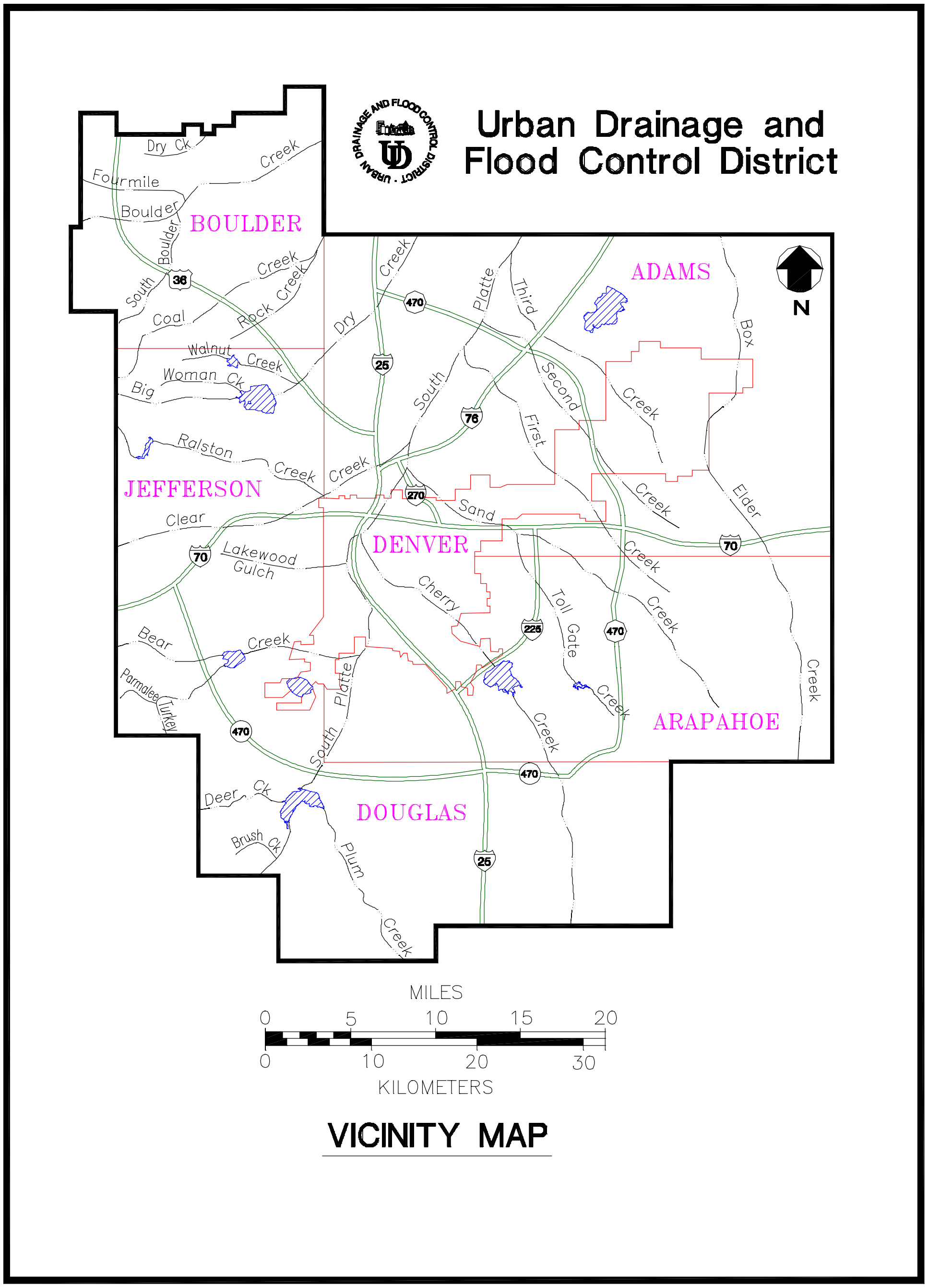

Figure DP-1—Urban Drainage and Flood Control District Boundaries

2.0 PRINCIPLES

2.1 Drainage Is a Regional Phenomenon That Does Not Respect the Boundaries Between Government Jurisdictions or Between Properties

This makes it necessary to formulate programs that include both public and private involvement. Overall, the governmental agencies most directly involved must provide coordination and master planning, but drainage planning must be integrated on a regional level if optimum results are to be achieved. The ways in which proposed drainage systems fit existing regional systems must be quantified and discussed in the master plan.

2.2 A Storm Drainage System Is a Subsystem of the Total Urban Water Resource System

Stormwater system planning and design for any site must be compatible with comprehensive regional plans and should be coordinated, particularly with planning for land use, open space and transportation. Erosion and sediment control, flood control, site grading criteria, and regional water quality all closely interrelate with urban stormwater management. Any individual master plan or specific site plan should normally address all of these considerations.

2.3 Every Urban Area Has an Initial (i.e., Minor) and a Major Drainage System, Whether or Not They Are Actually Planned and Designed

The initial drainage system, sometimes referred to as a “minor system,” is designed to provide public convenience and to accommodate moderate, frequently occurring flows. The major system carries more water and operates when the rate or volume of runoff exceeds the capacity of the minor system. Both systems should be carefully considered.

2.4 Runoff Routing Is Primarily a Space Allocation Problem

The volume of water present at a given point in time in an urban region cannot be compressed or diminished. Channels and storm sewers serve both conveyance and storage functions. If adequate provision is not made for drainage space demands, stormwater runoff will conflict with other land uses, will result in damages, and will impair or even disrupt the functioning of other urban systems.

2.5 Planning and Design of Stormwater Drainage Systems Generally Should Not Be Based on the Premise That Problems Can Be Transferred From One Location to Another

Urbanization tends to increase downstream peak flow by increasing runoff volumes and the speed of runoff. Stormwater runoff can be stored in detention facilities, which can reduce the drainage capacity required immediately downstream.

2.6 An Urban Storm Drainage Strategy Should Be a Multi-Objective and Multi-Means Effort

The many competing demands placed upon space and resources within an urban region argue for a drainage management strategy that meets a number of objectives, including water quality enhancement, groundwater recharge, recreation, wildlife habitat, wetland creation, protection of landmarks/amenities, control of erosion and sediment deposition, and creation of open spaces.

Photograph DP-2—An urban storm drainage strategy should be a multi-objective and multi-means effort.

2.7 Design of the Stormwater Drainage System Should Consider the Features and Functions of the Existing Drainage System

Every site contains natural features that may contribute to the management of stormwater without significant modifications. Existing features such as natural drainageways, depressions, wetlands, floodplains, permeable soils, and vegetation provide for infiltration, help control the velocity of runoff, extend the time of concentration, filter sediments and other pollutants, and recycle nutrients. Each development plan should carefully map and identify the existing natural system. Techniques that preserve or protect and enhance the natural features are encouraged. Good designs improve the effectiveness of natural systems rather than negate, replace or ignore them.

2.8 In New Developments, Attempts Should Be Made to Reduce Stormwater Runoff Rates and Pollutant Load Increases After Development to the Maximum Extent Practicable

1. The perviousness of the site should be maintained, to the extent feasible.

2. The rate of runoff should be slowed. Preference should be given to stormwater management systems that use practices that maximize vegetative and porous land cover. These systems will promote infiltration, filtering and slowing of the runoff. It should be noted that, due to the principle of mass conservation, it is virtually impossible to prevent increases in post-development runoff volumes when an area urbanizes. However, existing stormwater regulations can require control of peak flows to predevelopment levels to a maximum extent achievable. Increased flow volumes may present no flooding problems if the watershed has a positive outfall to a stream or river; however, these volumes may cause problems for a small, enclosed watershed draining to a lake or into streams of limited capacity.

3. Pollution control is best accomplished by implementing a series of measures, which can include source control, minimization of directly connected impervious areas, and construction of on-site and regional facilities, to control both runoff and pollution.

2.9 The Stormwater Management System Should Be Designed Beginning With the Outlet or Point of Outflow From the Project, Giving Full Consideration to Downstream Effects and the Effects of Off-Site Flows Entering the System

The downstream conveyance system should be evaluated to ensure that it has sufficient capacity to accept design discharges without adverse backwater or downstream impacts such as flooding, stream bank erosion, and sediment deposition. In addition, the design of a drainage system should take into account the runoff from upstream sites, recognizing their urban development potential.

2.10 The Stormwater Management System Should Receive Regular Maintenance

Failure to provide proper maintenance reduces both the hydraulic capacity and pollutant removal efficiency of the system. The key to effective maintenance is clear assignment of responsibilities to an established agency and a regular schedule of inspections to determine maintenance needs and to ensure that required maintenance is done. Local maintenance capabilities should be a consideration when selecting specific design criteria for a given site or project.

2.11 Floodplains Need to Be Preserved Whenever Feasible and Practicable

Nature has claimed a prescriptive easement for floods, via its floodplains, that cannot be denied without public and private cost. Floodplains often provide a natural order to the land surface with drainageways that serve as outfalls for urban drainage, bottomland for wildlife habitat, riparian corridors, and specialized vegetation. Floodplain encroachment can occur only after competent engineering and planning have been conducted to assure that flow capacity is maintained, risks of flooding are defined and risks to life and property are strictly minimized. Preservation of floodplains is a policy of the District to manage flood hazards, preserve habitat and open space, create a more livable urban environment, and protect the public health, safety, and welfare (White 1945).

Photograph DP-3—National Medal of Science winner, Dr. Gilbert White, recommends natural-like floodplains because they save people from damages and are good for the economy.

2.12 Reserve Sufficient Right-of-Way for Lateral Movement of Incised Floodplains

Whenever a floodplain is contained within a narrow (i.e., degraded) channel, its lateral movement over time can cause extensive damages to public and private structures and facilities. For this reason, whenever such a condition exists, it is recommended that, at a minimum, the channel be provided with grade control structures and a right-of-way corridor be preserved of a width equivalent to the cross section recommended for a grass-lined channel, including a maintenance access roadway.

3.0 BASIC KNOWLEDGE

3.1 Data Collection

An important step in a drainage program is to get the facts. A program for collecting and analyzing storm runoff and flood data should be maintained to promote intelligent and orderly planning (Jones 1967).

3.1.1 Storm Runoff and Flood Damage

Storm runoff and flood damage data should be collected in a systematic and uniform manner.

3.1.2 Rainfall-Runoff Relationships

A program should be maintained to collect and analyze rainfall-runoff relationships in urban areas of the Denver region.

3.1.3 Inventory of Successful Projects

Some drainage projects function better than others. It is important to determine why, so that key features may be inventoried for use on other succeeding projects.

3.1.4 Library

The District should acquire and actively maintain a library, which should be available for use by all governmental agencies, practicing planners, and engineers. The public should be encouraged to use the library as part of the District's educational and outreach programs.

3.1.5 Runoff Magnitudes

Where practical, the magnitude of computed and measured runoff peaks should be tabulated for Denver region streams and gulches so that comparisons may be readily made between watersheds and erroneous values may be more easily identified.

3.2 Floodplain Data

The program to delineate flood hazard areas along all waterways in the region should be maintained. This program should make full use such sources as the District's Flood Hazard Area Delineation studies, the FEMA Flood Insurance Studies, data from the Natural Resources Conservation Service, the USGS, and floodplain studies by private consulting engineers. This information should be regularly reviewed and updated to reflect changes due to urbanization, changed channel conditions, and the occurrence of extraordinary hydrologic events.

3.2.1 Small Waterways

Small gulches and other waterways, which are often overlooked, have a large damage potential. These waterways should receive early attention in areas subject to urbanization. Floodplain information should be shown on preliminary and final subdivision plats, including the areas inundated by major storm runoff and areas of potential erosion.

3.2.2 Data Inventory

The information collected should be stored in a central District depository available to all planners, developers, and engineers.

3.2.3 Floodplains

This effort should be aimed towards developing information on those areas that have a one percent chance of being inundated in any given year—that is, the 100-year floodplain. Local governmental agencies may choose to regulate floodplains for other frequencies of flooding; however, the 100-year floodplain based on runoff from the projected fully urbanized watershed must be defined in addition to being the minimum basis for regulation.

3.2.4 Priority for Data Acquisition

The District will establish priorities for acquisition of data because it is recognized that not all of the data can be collected at one time. When setting priorities, consideration should be given to:

a. Areas of rapid urban growth

b. Drainage problem areas

c. Local interest and capabilities in floodplain management

d. Potential for developing significant information

3.3 Data Use

Prior to the commencement of any drainage project, comprehensive facts and data should be collected and examined for the particular watershed and area under consideration.

3.3.1 Master Plan

Drainage design does not lend itself to a piece-meal approach; therefore, master plans for drainage should be prepared on a priority basis. Such plans already cover most of the developed major drainageways in the District. Additional plans will be developed for areas yet unplanned. In addition, existing master plans will be updated as needed to reflect changed conditions that take place over time.

3.3.2 Public Cost

Development of an area without the provision of adequate drainage multiplies the cost to the public because the drainage problem must be corrected later, usually at public expense.

3.3.3 Easements

Where construction occurs along a waterway not yet developed downstream or upstream, and where a master plan is not yet available, flood easements should be left which will include the future development 100-year floodplain. Where an existing master plan recommends the preservation of a defined floodplain, every effort should be made to acquire and/or preserve an easement or property right (ownership) for such a floodplain.

On any floodplain, nature possesses by prescription an easement for intermittent occupancy by runoff waters. Man can deny this easement only with difficulty. Encroachments upon or unwise land modifications within this easement can adversely affect upstream and downstream flooding occurrences during the inevitable periods of nature’s easement occupancy.

Floodplain regulation, then, must define natural easements and boundaries and must delineate floodplain occupancy that will be consistent with total public interests.

4.0 PLANNING

4.1 Total Urban System

Storm drainage is a part of the total urban environmental system. Therefore, storm drainage planning and design should be compatible with comprehensive regional plans. Master plans for storm drainage have been developed and maintained in an up-to-date fashion for most of the watersheds in the Denver region. An effort to complete the coverage of master plans for yet unplanned areas of the District should be continued until full coverage is achieved.

4.1.1 Development Plan

A development plan should be given direction by broad, general framework goals. Examples of such goals are:

1. Drainage and flood control problem alleviation
2. Economic efficiency
3. Regional development
4. Environmental preservation and enhancement
5. Social and recreational need fulfillment

These goals, or combinations of these goals, as they are pursued within an urban region, have the potential to influence greatly the type of drainage subsystem selected. Planning for drainage facilities should be related to the goals of the urban region, should be looked upon as a subsystem of the total urban system, and should not proceed independent of these considerations (Wright 1967).

4.1.2 Master Plan

Each municipality and county in the Denver region is responsible for master planning for urban storm drainage facilities within its boundaries and environs. The District can help to coordinate efforts. Cooperation between governmental agencies is needed to solve drainage problems and joint city, county and District efforts are encouraged. Carrying forward master planning is best accomplished on a priority-phased basis so that the most demanding problems, such as areas of rapid urbanization, may be addressed at an early date.

Early work includes the planning of major drainageways from the point of outfall, proceeding in an upstream direction. The major drainageways are generally well defined and often dictate the design of the initial drainage system, including storm sewers, detention facilities, and water quality systems.

The District has established a suitable format for master plan reports and drawings so that a uniform planning approach and coordination of efforts can more easily be made. Master planning should be done in enough detail and with adequate thoroughness to provide a ready drainage development guide for the future in a particular watershed. Generalized concepts based on rule-of-thumb hydrological analyses should not be used as master plans; a more rigorous analysis is necessary.

4.1.3 Planning Process Ingredients

Good urban drainage planning is a complex process. Fundamentals include:

1. Major Drainage Planning. All local and regional planning must take into consideration the major drainage system necessary to manage the runoff that is expected to occur once every 100 years. The major drainage system plans will reduce loss of life and major damage to the community and its infrastructure.

2. Initial Drainage System Planning. All local and regional planning must take into consideration the initial drainage system to transport the runoff from storms expected to occur once every 2 to 10 years. The planner of an initial system must strive to minimize future drainage complaints.

3. Environmental Design. Environmental design teams involving a range of disciplines should be convened whenever desirable to ensure that the benefits to total urban systems receive consideration in the drainage planning work. Planning should address water quality enhancements and include evaluation of the impacts of new facilities, as well as future operation and maintenance by private and public bodies.

4.1.4 Local and Regional Planning

Local and regional planning, whether performed under federal or state assistance programs or under completely local auspices, should consider and evaluate opportunities for multi-objective water resources management.

4.1.5 Site Planning

All land development proposals should receive full site planning and engineering analyses. In this regard, professional consideration must be given to the criteria outlined in the *Manual*. Where flood hazards are involved, the local planning boards should take into consideration proposed land use so that it is compatible with the flood hazard risks involved with the property, and appropriate easements need to be provided to preclude encroachment upon waterways or flood storage areas.

4.1.6 Water Quality

Protecting and enhancing the water quality of public streams is an important objective of drainage planning. Erosion control, maintaining stream channel stability, sediment and debris collection, and pollutant removal from stormwater runoff must be taken into account by using the stormwater runoff best management practices (BMPs) described in Volume 3 of this *Manual*.

Sanitary sewerage systems that overflow or bypass untreated sewage into surface streams should not be permitted in the Denver region. Existing systems that discharge sewage should be adjusted by their owners to eliminate this problem.

Full cooperation should be extended to planners and designers of sanitary sewerage works to minimize the hazards involved with the flooding of sanitary sewers by urban storm runoff. Drainage planning should include means to prevent inflow to sanitary sewers because of street flow and flooding of channels.

4.2 Multiple-Objective Considerations

Planning for drainage facilities should be coordinated with planning for open space, recreation and transportation. By coordinating these efforts, new opportunities can be identified which can assist in the solution of drainage problems (Heaney, Pitt and Field 1999).

4.2.1 Lower Drainage Costs

Planning drainage works in conjunction with other urban needs results in more orderly development and lower costs for drainage and other facilities.

4.2.2 Open Space

Open space provides significant urban social benefits. Use of stabilized, natural drainageways often is less costly than constructing artificial channels. Combining the open space needs of a community with major drainageways is a desirable combination of uses that reduces land costs and promotes riparian zone protection and establishment over time.

4.2.3 Transportation

Design and construction of new streets and highways should be fully integrated with drainage needs of the urban area for better streets and highways and better drainages and to avoid creation of flooding hazards. The location of borrow pits needed for road construction should be integrated with broad planning objectives, including storm runoff detention.

4.3 Natural Channels

Natural drainageways should be used for storm runoff waterways wherever practical. Preservation and protection of natural drainageways are encouraged; however, major consideration must be given to their stability as the area urbanizes.

4.3.1 Channelization

Natural drainageways within an urbanizing area are often deepened, straightened, lined, and sometimes put underground. A community loses a natural asset when this happens. Channelizing a natural waterway usually speeds up the flow, causing greater downstream flood peaks and higher drainage costs, and does nothing to enhance the environment.

4.3.2 Channel Storage

Drainageways having "slow-flow" characteristics, vegetated bottoms and sides, and wide water surfaces provide significant floodplain storage capacity. This storage is beneficial in that it reduces downstream runoff peaks and provides an opportunity for groundwater recharge. Wetland channels, wide natural channels, and adjacent floodplains provide urban open space.

Photograph DP-4—Drainageways having "slow-flow" characteristics, with vegetated bottoms and sides can provide many benefits.

4.3.3 Major Runoff Capacity

Drainageways and their residual floodplains should be capable of carrying the major storm runoff, which can be expected to have a one percent chance of occurring in any single year.

4.3.4 Maintenance and Maintenance Access

Waterways will require both scheduled and unscheduled maintenance for a wide array of activities such as sediment, debris and trash removal, mowing, and repair of hydraulic structures. Assured long term maintenance is essential, and it must be addressed during planning and design. The District assists with drainage facility maintenance, provided that the facilities are designed in accordance with the District's Maintenance Eligibility Guidelines. The June 2001 version of these guidelines are available on the CD version of this *Manual*, and updates to these guidelines should be obtained from the District's Web site at www.udfcd.org. Designers are strongly encouraged to adhere to the design criteria listed in the Maintenance Eligibility Guidelines. Waterways, detention structures and other facilities must have permanent access for routine and major maintenance activities.

4.4 Transfer of Problems

Planning and design of stormwater drainage systems should not be based on the premise that problems can be transferred from one location to another.

4.4.1 Intra-Watershed Transfer

Channel modifications that create unnecessary problems downstream should be avoided, both for the benefit of the public and to avoid damage to private parties. Problems to avoid include land and channel erosion and downstream sediment deposition, increase of runoff peaks, and debris transport, among others.

4.4.2 Inter-Watershed Transfer

Diversion of storm runoff from one watershed to another introduces significant legal and social problems and should be avoided unless specific and prudent reasons justify and dictate such a transfer and no measurable damages occur to the natural receiving water or urban systems or to the public.

4.4.3 Watershed Planning

Master planning must be based upon potential future upstream development, taking into consideration both upstream and downstream existing and future regional publicly owned and operated detention and retention storage facilities. Such facilities must be assured of construction, perpetual operation and maintenance. Urban development causes a major increase in the volume of runoff, even though the peak flows for certain return floods might be managed to simulate those of undeveloped historic conditions. In the absence of such detention and retention facilities, the basis of design for both the initial and major systems is fully developed upstream conditions without storage.

4.5 Detention and Retention Storage

Stormwater runoff can be stored in detention and retention reservoirs. Such storage can reduce the peak flow drainage capacity required, thereby reducing the land area and expenditures required downstream. (However, see limitation in 4.4.3 regarding taking credit for detention.) In some instances of stormwater retention and detention, there may be water rights implications, and in those instances, the State Engineer's Office should be consulted.

4.5.1 Upstream Storage

Storage of storm runoff close to the points of rainfall occurrence includes use of parking lots, ball fields, property line swales, parks, road embankments, borrow pits, and on-site basins and ponds.

Large parking lots, like those at shopping centers, create more runoff volume than before with high runoff discharge rates. The same is true for many small parking lots. Parking lots should be designed to provide for storage of runoff during infrequent events except where clearly shown that such storage is impractical. Wherever reasonably acceptable from a social standpoint, parks should be used for short-term detention of storm runoff to create drainage benefits. Such use may help justify park and greenbelt acquisition and expenditures.

The difficulty in quantifying the cumulative effects of very large numbers of small (i.e., on-site) detention/retention facilities (Malcomb 1982; Urbonas and Glidden 1983) and the virtual impossibility of assurance of their continued long-term performance or existence (Debo 1982; Prommersberger 1984) requires the District to recognize in its floodplain management only regional, publicly owned facilities. Nevertheless, upstream storage is encouraged, such as with the "Blue-Green" concept first described in *Civil Engineering* magazine (Jones 1967).

4.5.2 Minimized Directly Connected Impervious Area Development

The “minimized directly connected impervious area” (MDCIA) concept (refer to Volume 3 of this *Manual*) provides an approach to upstream stormwater management that reduces the amount of impervious surfaces in a development and their connection to the initial drainage system. In addition, it includes functional grading, wide and shallow surface flow sections, disconnection of hydrologic flow paths, and the use of porous landscape detention and porous pavement areas. Details for its use are presented in Volume 3 of this *Manual*. The technique of MDCIA is also referred to as “low impact development” (LID). Other references include Heaney, Pitt and Field (1999) and Prince George’s County, Maryland (1999).

4.5.3 Downstream Storage

The detention and retention of storm runoff is desirable in slow-flow channels, in storage reservoirs located in the channels, in off-stream reservoirs, and by using planned channel overflow ponding in park and greenbelt areas. Lengthening the time of concentration of storm runoff to a downstream point is an important goal of storm drainage and flood control strategies. This should be achieved via numerous and varied techniques.

4.5.4 Reliance on Non-Flood-Control Reservoirs

Privately owned non-flood-control reservoirs cannot be used for flood mitigation purposes in master planning because their perpetuity cannot be reasonably guaranteed. Publicly owned water storage reservoirs (city, state, water district, irrigation company, etc.) should be assumed to be full for flood planning purposes and, therefore, only the detention storage above the spillway crest can be utilized in regard to the determination of downstream flood peak flows.

4.5.5 Reliance on Embankments

The detention of floodwaters behind embankments created by railroads, highways or roadways resulting from hydraulically undersized culverts or bridges should not be utilized by the drainage engineer for flood peak mitigation when determining the downstream flood peaks for channel capacity purposes unless such detention has been covered by a binding agreement approved by the District.

5.0 TECHNICAL CRITERIA

5.1 Design Criteria

Storm drainage planning and design should adhere to the criteria developed and presented in this *Manual* maintained by the District.

5.1.1 Design Criteria

The design criteria presented herein represent current good engineering practice, and their use in the Denver region is recommended. The criteria are not intended to be an ironclad set of rules that the planner and designer must follow; they are intended to establish guidelines, standards and methods for sound planning and design.

5.1.2 Criteria Updating

The criteria contained in this *Manual* should be revised and updated as necessary to reflect advances in the field of urban drainage engineering and urban water resources management.

5.1.3 Use of Criteria

Governmental agencies and engineers should utilize this *Manual* in planning new facilities and in their reviews of proposed works by developers, private parties, and other governmental agencies, including the Colorado Department of Transportation and other elements of the state and federal governments.

5.2 Initial and Major Drainage

Every urban area has two separate and distinct drainage systems, whether or not they are actually planned and designed. One is the initial system, and the other is the major system. To provide for orderly urban growth, reduce costs to future generations, and avoid loss of life and major property damage, both systems must be planned and properly engineered.

5.2.1 Design Storm Return Periods

Storm drainage planning and design should fully recognize the need for considering two separate and distinct storm drainage systems: the initial drainage system and the major drainage system. Local governments should not be tempted to specify larger than necessary design runoff criteria for the initial drainage system because of the direct impact on the cost of urban infrastructure.

There are many developed areas within the Denver urban region that do not fully conform to the drainage standards projected in this *Manual*. The multitude of problems associated with these areas historically provided the emphasis required to proceed with development of this *Manual*. It is recognized that upgrading these developed areas to conform to all of the policies, criteria, and standards contained in this *Manual* will be difficult, if not impractical, to obtain, short of complete redevelopment or renewal. However, flood-proofing techniques can be applied to these areas.

Strict application of this *Manual* in the overall planning of new development is practical and economical; however, when planning drainage improvements and the designation of floodplains for developed areas, the use of the policies, criteria, and standards contained in this *Manual* should be adjusted to provide for economical and environmentally sound solutions consistent with other goals of the area. Where the 100-year storm is not chosen for design purposes, the impact of the 100-year storm should be investigated and made known.

5.2.2 Initial Storm Provisions

The initial storm drainage system, capable of safely handling 2- to 10-year floods depending on local criteria, is necessary to reduce the frequency of street flooding and maintenance costs, to provide protection against regularly recurring damage from storm runoff, to help create an orderly urban system, and to provide convenience to urban residents. Normally, the initial drainage system cannot economically carry major runoffs, though the major drainage system can provide for the initial runoff. A well-planned major drainage system will reduce or eliminate the need for storm sewer systems (Jones 1967). Storm sewer systems consisting of underground pipes are a part of initial storm drainage systems.

5.2.3 Major Storm Provisions

In addition to providing the storm drainage facilities for the initial storm runoff, provisions should be made to avoid major property damage and loss of life for the storm runoff expected to occur from an urbanized watershed once every 100 years on average (i.e., one percent probability of occurrence any given year). Such provisions are known as the major drainage system.

5.2.4 Critical Facilities

Drainage engineers and planners should consider that certain critical facilities may need a higher level of flood protection. For instance, hospitals, police, fire stations and emergency communication centers should be designed in a manner so that, even during a 100-year flood, their functioning will not be compromised. The use of a 500-year flood level for such facilities may be justified in many instances.

5.2.5 Major Drainage Channels

Open channels for transporting major storm runoff are more desirable than closed sewers in urban areas, and use of such channels is encouraged. Open channel planning and design objectives are often best met by using natural-type vegetated channels, which characteristically have slower velocities and large width-to-depth ratios. Additional benefits from open channels can be obtained by incorporating parks and greenbelts with the channel layout. When evaluating existing natural water courses (perennial, intermittent and ephemeral), it is desirable to minimize straightening, fill placement, and other alterations. Alterations such as these should be very carefully evaluated. Normally, however, some structural stabilization will be necessary to address the increased effects on stream stability caused by increased flows due to urbanization. For example, grade control structures and structural protection at the channel toe and on outer banks are normally required.

The filling, straightening or altering of natural water courses, perhaps wet only during and after large rainstorms, is discouraged. Such actions tend to reduce flood storage and increase the velocity to the detriment of those downstream of and adjacent to the channel work. Effort must be made to reduce flood peaks and control erosion so that the natural channel regime is preserved as much as practical. Buffer zones can be used to account for future channel meandering and bank sloughing, at least in part.

Use of open channels should receive early attention when planning a new development, along with other storm runoff features.

5.2.6 Tailwater

The depth of flow in the receiving stream must be taken into consideration for backwater computations for either the initial or major storm runoff.

5.3 Runoff Computation

The determination of runoff magnitude should be made using the techniques described in the RUNOFF chapter of this *Manual*.

5.3.1 Accuracy

The peak discharges determined by any method are approximations. Rarely will drainage works operate at the design discharge. Flow will always be more or less in actual practice as it rises and falls during a storm event. Thus, the engineer should not overemphasize the detailed accuracy of computed discharges but should emphasize the design of practical and hydraulically balanced works based on sound logic and engineering, as well as dependable hydrology. The use of more than three significant figures for estimating the flood magnitudes conveys a false sense of accuracy and should be avoided. Because of the public’s reliance on published peak flow estimates, they should only be changed when it is clear that an original error has been made and that continuing their use would not be in the public’s interest.

5.4 Streets

5.4.1 Use of Streets

Streets are significant and important in urban drainage, and full use should be made of streets for storm runoff up to reasonable limits, recognizing that the primary purpose of streets is for traffic. Reasonable limits of the use of streets for transportation of storm runoff should be governed by reasonable design criteria as summarized in Table DP-1. Urban drainage design should have as objectives reduction of street repair, maintenance costs, nuisance to the public, and disruption of traffic flow.

Table DP-1—Reasonable Use of Streets for Initial Storm Runoff in Terms of Pavement Encroachment

Street Classification	Maximum Encroachment
Local	No curb overtopping. Flow may spread to crown of street.
Collector	No curb overtopping. Flow spread must leave at least one lane free of water.
Arterial	No curb overtopping. Flow spread must leave at least one lane free of water in each direction but should not flood more than two lanes in each direction.
Freeway	No encroachment is allowed on any traffic lanes.

When maximum allowed encroachment is present, the storm sewer system design based on the initial storm should commence. Development of a major drainage system that can often drain the initial runoff from the streets is encouraged, thus making the point at which the storm sewer system should commence further downstream. Initial and major drainage planning should go hand-in-hand.

While it is the intent of this policy to have major storm runoff removed from public streets at frequent and regular intervals and routed into major drainageways, it is recognized that water will often tend to follow streets and roadways and that streets and roadways often may be aligned so they will provide a specific runoff conveyance function. Planning and design objectives for the major drainage system with regard to public streets should be based upon following the limiting criteria summarized in Table DP-2.

Table DP-2—Major Storm Runoff Recommended Maximum Street Inundation

Street Classification	Maximum Depth and Inundated Areas
Local and Collector	Residential dwellings should be no less than 12 inches above the 100-year flood at the ground line or lowest water entry of a building. The depth of water over the gutter flow line should not exceed 18 inches for local and 12 inches for collector streets.
Arterial and Freeway	Residential dwellings should be no less than 12 inches above the 100-year flood at the ground line or lowest water entry of a building. The depth of water should not exceed the street crown to allow operation of emergency vehicles. The depth of water over the gutter flow line should not exceed 12 inches.

The allowable flow across a street should be within the criteria presented in Table DP-3.

Table DP-3—Allowable Maximums for Cross-Street Flow

Street Classification	Initial Design Runoff	Major Design Runoff
Local	6 inches of depth in cross pan	18 inches of depth above gutter flow line
Collector	Where cross pans allowed, depth of flow should not exceed 6 inches	12 inches of depth above gutter flow line
Arterial/Freeway	None	No cross flow. 12 inches of maximum depth at upstream gutter or roadway edge

An arterial street crossing will generally require that a storm sewer system be commenced, unless the topography is such that day-lighted inlet culverts or other suitable means can transport the initial storm runoff under the arterial street or water can be routed to a major drainage facility. Bubblers (inverted siphons which convey flows beneath roadways) are not encouraged in the Denver region because of possible plugging with sediment and difficulty in maintaining them. Collector streets should have cross pans only at infrequent locations as specified by the governing agency and in accordance with good traffic engineering practices. The local street criteria for overtopping also apply to any private access road that serves commercial areas or more than one residence, for emergency access and safety reasons.

5.5 Irrigation Ditches

Irrigation ditches should not be used as outfall points for initial or major drainage systems, unless such use is shown to be without unreasonable hazard substantiated by adequate hydraulic engineering analysis and approval of the owner of the ditch.

5.5.1 Use of Ditches

The irrigation ditches coursing through urban areas are laid out on flat slopes and with limited carrying capacity. Based on experience and hydraulic calculations, irrigation ditches cannot, as a general rule, be used as an outfall point for the initial storm drainage system because of physical limitations. Exceptions to the rule can occur when the capacity of the irrigation ditch is adequate to carry the normal ditch flow plus the initial storm runoff with adequate freeboard to avoid creating a hazard to those below the ditch. Written approval must be obtained from the ditch owner stating that the owner understands the physical and legal (i.e., liability) consequences of accepting said runoff.

If there is a question about the use of irrigation ditches as outfalls for initial storm runoff, there is no question about their unsuitability as an outfall for the major storm runoff. Without major reworking of irrigation ditches to provide major carrying capacity without undue hazard to those downstream or below the ditch, the ditches are almost always totally inadequate for such a use and should not be used as an outfall. Moreover, because ditches are normally privately owned, one cannot assume the perpetual existence or function of a ditch. Land planners downhill from a ditch should plan for pre-ditch drainage conditions as well as continued ditch seepage.

5.5.2 Ditch Perpetuation

Irrigation ditches are sometimes abandoned in urban areas after the agricultural land is no longer farmed. Provisions must be made for a ditch's perpetuation, defined as continued operation and serviceability, prior to its being chosen and used as an outfall for urban drainage.

5.5.3 Conformance With Master Plan

Use of irrigation ditches for collection and transport of either initial or major storm runoff should be prohibited unless specifically provided in a District's master plan or approved by the District and the ditch owner.

5.6 Detention and Retention Facilities Maintenance

The significant cost of handling stormwater runoff, coupled with the social benefits to be derived from proper storm drainage facilities, points towards the use of detention and retention basins for storage of stormwater runoff in the Denver region. Maintenance provisions must be arranged. Maintenance of detention or retention facilities includes the removal of debris, excessive vegetation from the embankment, and sediment. Without maintenance, a detention/retention facility will become an unsightly social liability and eventually become ineffective.

5.6.1 Water Quality

Detention and retention facilities provide an opportunity to improve the quality of stormwater runoff before it reaches streams. Water quality BMPs will add an additional level of maintenance obligation because they are designed to remove, among other things, solid constituents from urban runoff.

Photograph DP-5—Detention basins with permanent ponding help in many ways, including flood reduction, water quality and land values.

6.0 FLOODPLAIN MANAGEMENT

6.1 Purpose

Various governmental agencies within the Denver region should initiate floodplain management programs. Floodplain management includes comprehensive criteria designed to encourage, where necessary, the adoption of permanent state or local measures which will lessen exposure of property and facilities to flood losses, improve long-range land management and use of flood-prone areas, and inhibit, to the maximum extent feasible, unplanned future development in such areas.

6.2 Goals

There are two goals in regard to floodplain management:

- *To reduce the vulnerability of Denver region residents to the danger and damage of floods.*

  The dangers of flooding include threats to life, safety, public health, and mental well being, as well as damage to properties and infrastructure and disruption of the economy. Protection from these hazards should be provided, by whatever measures are suitable, for floods having a one percent reoccurrence probability in any given year (100-year floods), at a minimum, based on projected build-out in the watershed. Protection from the effects of greater, less frequent flooding is also needed in those places where such flooding would cause unacceptable or catastrophic damages.

- *To preserve and enhance the natural values of the region's floodplains.*

  Natural floodplains serve society by providing floodwater storage, groundwater recharge, water quality enhancement, aesthetic pleasure, and habitat for plants and animals. Many floodplains also have cultural and historical significance. It is in the public's interest to avoid development that destroys these values or, in instances where the public good requires development, to assure that measures are taken to mitigate the loss through replacement or other means.

These two goals are reconcilable and achievable through appropriate management shared by the agencies involved.

6.3 National Flood Insurance Program

Flood insurance should be an integral part of a strategy to manage flood losses. The cities and counties in the Denver region are encouraged to continue to participate in the federal flood insurance program set forth in the NFIA of 1968, as amended.

6.3.1 Participation

A prerequisite for participation is the adoption of a floodplain management program by the local government that, where necessary, includes adoption of permanent state or local regulatory measures that will lessen the exposure of property and facilities to flood losses. Property owners should be encouraged to buy flood insurance, even outside the designated floodplain, to protect against local flooding where such potential exists.

6.4 Floodplain Management

The objectives of floodplain management are:

a. To adopt effective floodplain regulations.

b. To improve local land use practices, programs, and regulations in flood-prone areas.

c. To provide a balanced program of measures to reduce losses from flooding.

d. To reduce the need for reliance on local and federal disaster relief programs.

e. To minimize adverse water quality impacts.

f. To foster the creation/preservation of greenbelts, with associated wildlife and other ecological benefits, in urban areas.

Floodplain management practices must be implemented to be of value. Although hydrologic data are critical to the development of a floodplain management program, the program is largely dependent on a series of policy, planning, and design decisions. These decisions are essentially political, economic, and social in character and are developed on a geographic scale extending beyond the floodplain itself. These area-wide decisions provide the setting for floodplain usage and, when combined with hydrologic considerations and augmented by both administrative and implementing devices, constitute the floodplain management program. The program must give high priority to both flood danger and public programs, such as urban renewal, open space, etc.

6.5 Floodplain Filling

While floodplain management includes some utilization of the flood fringe (i.e., areas outside of the formal floodway), the planner and engineer should proceed cautiously when planning facilities on lands below the expected elevation of the 100-year flood. Flood peaks from urbanized watersheds are high and short-lived, which makes storage in the flood fringe important and effective. Filling the flood fringe tends to increase downstream peaks.

6.6 New Development

The decision as to whether or not a major flood control measure should be undertaken to permit intensive new urbanization or to maintain an open area within an urban floodplain or any intermediate use should be made on the basis of:

a. Relative costs of the respective alternatives (not only financial, but also non-financial economic costs such as opportunities foregone).

b. The opportunities for flood proofing and other measures in relation to the extent of flood hazard.

c. The availability of lands in non-floodplain areas for needed development.

d. The location of the high flood hazard areas, namely, defined floodways.

e. The potential adverse effect on others in or adjacent to the floodplain.

f. The fact that floods larger than the design flood will occur (i.e., exposure will still exist, even with well-designed facilities, for the one percent flood).

6.7 Strategies and Tools

The strategies and tools available to the drainage engineer for floodplain management are numerous and varied. The following menu is meant to be a list of strategies and tools available for floodplain management, but it should not be considered to be limiting (FEMA 1995).

6.7.1 Exposure to Floods

Reduce exposure to floods and disruptions by employing floodplain regulations and local regulations. The latter would include zoning, subdivision regulations, building codes, sanitary and well codes, and disclosure to property buyers.

6.7.2 Development Policies

Development policies include design and location of utility services, land acquisition, redevelopment, and permanent evacuation (purchase of properties).

6.7.3 Preparedness

Disaster preparedness is an important tool for safeguarding lives and property, and disaster assistance will reduce the impact to citizens from flooding.

6.7.4 Flood Proofing

Flood proofing of buildings is a technique that is wise and prudent where existing buildings are subject to flooding. Flood proofing can help a proposed project achieve a better benefit-cost ratio.

6.7.5 Flood Forecasting

Flood forecasting and early warning systems are important means to reduce flood losses, safeguard health, protect against loss of life and generally provide an opportunity for people to prepare for a flood event before it strikes.

6.7.6 Flood Modification

The use of methods to modify the severity of the flood is a floodplain management tool. These include regional detention, channelization, minimizing directly connected impervious area, and on-site detention.

6.7.7 Impact of Modification

Using education, flood insurance, tax adjustments, emergency measures, and a good post-flood recovery plan that can be initiated immediately can modify the impact of flooding.

7.0 IMPLEMENTATION

7.1 Adoption of Drainage Master Plans

This *Manual* and master plans should be adopted and used by all governmental agencies operating within the District.

7.1.1 Manual Potential

From a broad perspective, this *Manual* on drainage disseminated by the District will have the potential to:

a. Give direction to public agency efforts to guide private decisions.

b. Give direction to public agency efforts to regulate private decisions.

c. Provide a framework for a public agency when it seeks to guide other public agencies.

d. Provide a framework to assist in coordinating the range of public and private activities.

e. Provide direction for development of master plans and designs and for implementation of drainage facilities.

7.2 Governmental Operations

Each level of government must participate if a drainage program is to be successful.

7.3 Amendments

Problems in urban drainage administration encountered by any governmental agency should be reviewed by the District to determine if equity or public interests indicate a need for drainage policy, practice, or procedural amendments. The District should continually review the needs of the Denver region in regard to urban runoff criteria and should recommend changes as necessary to this *Manual*.

7.4 Financing

Financing storm drainage improvements is fundamentally the responsibility of the affected property owners (both the persons directly affected by the water and the person from whose land the water flows) and the local governing body.

7.4.1 Drainage Costs

Every effort should be made to keep the cost of drainage solutions reasonable. This will involve careful balancing of storage and conveyance costs and the integration of drainage with other activities such as open space and transportation efforts. Funding must be established, and budgets should be prepared to assure proper maintenance of all new drainage and storage facilities.

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DRAINAGE LAW

CONTENTS

Photographs

1.0 SUMMARY OF CURRENT GENERAL PRINCIPLES OF DRAINAGE AND FLOOD CONTROL LAW

1.1 Introduction

Drainage law not only has its basis in law made by the courts and the legislature, but also relies to a large extent on the drainage facts that exist in each case. Therefore, a party with the most reliable facts and information will have a distinct advantage in court. Similarly, drainage engineering and design revolves around drainage law as well as the natural laws of gravity.

This chapter deals with the general principles of drainage law along with local government drainage actions, financing, floodplain management, and special matters. This chapter is meant to provide an outline of the general principles of Colorado drainage law for the engineer and agency official. It is not meant to serve as a substitute for a lawyer's opinions, though this chapter may be of interest to practicing attorneys.

In using this chapter of the *Manual*, the reader should be familiar with the entire *Manual*, and should pay particular attention to the POLICY and PLANNING chapters. In the POLICY chapter, 12 principles have been stated, with which the reader of this chapter should be familiar. Similarly, the following legal principles are summarized below for ready reference.

Photograph DL-1—Using a natural floodplain, even with a wetland involved, represents sound engineering in concert with established Colorado drainage law.

1.2 Legal Principles

1. The owner of upstream property possesses a natural easement on land downstream for drainage of surface water flowing in its natural course. The upstream property owner may alter drainage

conditions so long as the water is not sent down in a manner or quantity to do more harm to the downstream land than formerly. Bittersweet Farms, Inc. v. Zimbelman, 976 P.2d 326 (Colo. App. 1998).

2. For purposes of determining liability in a negligence action, the duty of a public entity shall be determined in the same manner as if it were a private party. Leake v. Cain, 720 P.2d 152 (Colo. 1986).

3. A natural watercourse may be used as a conduit or outlet for the drainage of lands, at least where the augmented flow will not tax the stream beyond its capacity and cause flooding of adjacent lands. Ambrosio v. Pearl-Mack Construction Co., 351 P.2d 803 (Colo. 1960).

4. Ditch corporations that own ditches owe a duty to those property owners through which their ditches pass to maintain their ditches using ordinary care so as to prevent damage to adjoining real property. Oliver v. Amity Mut. Irrigation Co., 994 P.2d 495 (Colo. App. 1999).

5. Construction or enlargement of jurisdictional dams or reservoirs is subject to approval by the Colorado State Engineer, which includes consideration of requiring their spillways to be capable of passing the inflow design flood generated by 100 percent of the probable maximum precipitation. A "jurisdictional dam" is defined as a dam that impounds water above the elevation of the natural surface of the ground creating a reservoir with a capacity of more than 100 acre-feet or creating a reservoir with a surface area exceeding 20 acres at the high waterline or exceeding 10 feet in height measured vertically from the elevation of the lowest point of the natural surface of the ground where that point occurs along the longitudinal centerline of the dam up to the flow line crest of the emergency spillway of the dam. Rules 4 & 5 of the Department of Natural Resources, Division of Water Resources, Office of the State Engineer, Rules and Regulations for Dam Safety and Dam Construction.

6. The boundaries of the floodplain should be accurately determined and based on a reasonable standard. Mallett v. Mamarooneck, 125 N.E. 2d 875 (N.Y. 1955).

7. Adoption of a floodplain regulation to regulate flood-prone areas is a valid exercise of police power and is not a taking as long as the regulation does not go beyond protection of the public's health, safety, morals, and welfare. Hermanson v. Board of County Commissioners of Fremont, 595 P.2d 694 (Colo. App. 1979).

8. The adoption by a municipality of floodplain ordinances to regulate flood-prone areas is a valid exercise of police power and is not a taking. Morrison v. City of Aurora, 745 P.2d 1042 (Colo. App. 1987).

9. A zoning ordinance is not unconstitutional because it prohibits a landowner from using or developing

his land in the most profitable manner. It is not required that a landowner be permitted to make the best, maximum or most profitable use of his property. Baum v. City and County of Denver, 363 P.2d 688 (Colo. 1961) and Sundheim v. Board of County Commissioners of Douglas County, 904 P.2d 1337 (Colo. App. 1995).

10. The safest approach to avoiding liability in regard to drainage and flood control improvements is to assume that the defense of a design error will not protect a governmental entity from a lawsuit and liability for injury to property or person. Scott v. City of Greeley, 931 P.2d 525 (Colo. App. 1996) and 24-10-106 (1)(e) and (f) C.R.S.

11. A “dangerous condition” constitutes an unreasonable risk to the health or safety of the public, which is known to exist or which in the exercise of reasonable care should have been known to exist and which condition is proximately caused by the negligent act or omission of the public entity in constructing or maintaining such facility. 24-10-103 C.R.S.

12. Under the Colorado Governmental Immunity Act (CGIA), a drainage and flood control facility is considered to be a “sanitation facility” and thus not protected by the defense that the facility caused damage solely because the design of the facility was inadequate. 24-10-106 (f) and 24-10-103 C.R.S. and Burnworth v. Adams County, 826 P.2d 368 (Colo. App. 1991).

13. Under the CGIA, a governmental entity will be liable for the negligent operation and maintenance of any drainage and flood control facility. 24-10-106 (f) and 24-10-103 C.R.S. and Burnworth v. Adams County, 826 P.2d 368 (Colo. App. 1991).

14. Under the CGIA, a governmental entity will not be liable for its failure to upgrade, modernize, modify, or improve the design or construction of a drainage or flood control facility. 24-10-103 (1) C.R.S.

15. In imposing conditions upon the granting of land-use approvals, no local government shall require an owner of private property to dedicate real property to the public or pay money to a public entity in an amount that is determined on an individual and discretionary basis, unless there is an essential nexus between the dedication or payment and a legitimate local government interest and the dedication or payment is roughly proportional both in nature and extent to the impact of the proposed use or development of such property. This law does not apply to any legislatively formulated assessment, fee, or charge that is imposed on a broad class of property owners by a local government. 29-20-203 C.R.S.

16. Public entities that own dams or reservoirs are not subject to strict liability for damages caused by water escaping from their dams or reservoirs. Further, those public entities have no duty to ensure that waters released from an upstream reservoir because of a dam failure would be contained by

their facilities or would bypass those facilities without augmentation. Kane v. Town of Estes Park, 786 P.2d 412 (Colo. 1990).

17. A professional engineer is required not only to serve the interests of his or her employer/client but is also required, as his or her primary obligation, to protect the safety, health, property, and welfare of the public. Rule I 2. of The Colorado Rules of Professional Conduct of the State Board of Registration for Professional Engineers and Professional Land Surveyors.

18. Where a municipality imposes a special fee upon owners of property for purposes of providing a service and where the fee is reasonably designed to defray the cost of the service provided by the municipality, such a fee is a valid form of governmental charge within the legislative authority of the municipality. Bloom v. City of Fort Collins, 784 P.2d 304 (Colo. 1989).

2.0 GENERAL PRINCIPLES OF DRAINAGE LAW

> Very little is gained if the same act which dries up one tract of land renders the adjoining tract twice as difficult to redeem.
>
> Livingston v. McDonald, 21 Iowa 160, 170 (1866).

2.1 Private Liability

Traditionally, courts have analyzed the legal relations between parties in drainage matters in terms of such property concepts as natural easements, rights, privileges, and servitudes but have based liability for interfering with surface waters on tort principles. See Kenyon and McClure *Interferences With Surface Waters*, 24 Minn. L. Rev. 891 (1940). Drainage and flood control problems attendant with increased urbanization, the trend in tort law toward shifting the burden of a loss to the best risk-bearer, and complete or partial abolition of governmental immunity by the judiciary or the legislature will continue to change the traditional rules that have governed legal relations between parties in drainage matters. These changes are reflected in the three basic rules relating to drainage of surface waters that have been applied over a period of time in the United States: the common enemy rule, the civil law rule (later to be called a “modified civil law rule”), and the reasonable use rule.

2.1.1 Common Enemy Rule

Under the common enemy rule, which is also referred to as the common law rule, surface water is regarded as a common enemy, which each property owner may fight off or control as he or she will or is able, either by retention, diversion, repulsion, or altered transmission. Thus, there is no cause of action even if some injury occurs. All jurisdictions originally following this harsh rule have either modified the rule or adopted the civil law rule or reasonable use rule. 5 *Water and Water Rights*, §§450.6, 451.2 (R.E. Clark ed. 1972).

2.1.2 Civil Law Rule

The civil law rule, or natural flow rule, places a natural easement or servitude upon the lower land for the drainage of surface water in its natural course, and the natural flow of the water cannot be obstructed by the servient owner to the detriment of the dominant owner. 5 *Water and Water Rights*, §452.2A (R.E. Clark ed. 1972). Most states following this rule, including Colorado, have modified the rule. Under the modified rule, the owner of upper lands has an easement over lower lands for drainage of surface waters, and natural drainage conditions can be altered by an upper proprietor provided the water is not sent down in a manner or quantity to do more harm than formerly. Hankins v. Borland, 163 Colo. 575, 431 P.2d 1007 (1967); H. Gordon Howard v. Cactus Hill Ranch Company, 529 P.2d 660 (1974); Hoff v. Ehrlich, 511 P.2d 523 (1973); but see Ambrosio v. Perl-Mack Construction Company, 143 Colo. 49, 351 P.2d 803 (1960) and Bittersweet Farms, Inc. v. Zimbelman, 976 P.2d 326 (Colo. App. 1998).

2.1.3 Reasonable Use Rule

Under the reasonable use rule, each property owner can legally make reasonable use of his land, even though the flow of surface waters is altered thereby and causes some harm to others. However, liability attaches when the harmful interference with the flow of surface water is "unreasonable." Whether a landowner's use is unreasonable is determined by a nuisance-type balancing test. The analysis involves three inquiries:

1. Was there reasonable necessity for the actor to alter the drainage to make use of his or her land?
2. Was the alteration done in a reasonable manner?
3. Does the utility of the actor's conduct reasonably outweigh the gravity of harm to others?

Restatement Torts, §§822-831, 833 (1939); *Restatement (Second) Torts*, §158, Illustration 5. Alaska, Hawaii, Kentucky, Massachusetts, Minnesota, New Hampshire, New Jersey, North Carolina, North Dakota, Ohio and Utah have adopted this rule. Some states have restricted their application of the rule to urban areas (South Dakota and Texas). In Pendegast v. Aiken, 236 S.E. 2d 787 (1977), the North Carolina Supreme Court traces the common law rule to the civil law rule to adoption by that court of the reasonable use rule, starting at page 793:

> It is no longer simply a matter of balancing the interests of individual landowners; the interests of society must be considered. On the whole the rigid solutions offered by the common enemy and civil law rules no longer provide an adequate vehicle by which drainage problems may be properly resolved.

2.2 Municipal Liability

A municipality is generally treated like a private party in drainage matters. Harbison v. City of Hillsboro, 103 Ore. 257, 204 P. 613, 618 (1922); City of Golden v. Western Lumber and Pole Company, 60 Colo. 382, 154 P. 95 (1916) (a municipality undertaking a public improvement is liable like an individual for damage resulting from negligence or an omission of duty); City of Denver v. Rhodes, 9 Colo. 554, 13 P. 729 (1887). In the case of municipalities, however, the distinction between unlawful collection, diversion, or concentration of surface waters and lawful improvement is not always clear, particularly as the pace and extent of urbanization increases. City of Englewood v. Linkenheil, 146 Colo. 493, 362 P.2d 185 (1961); Aicher v. Denver, 10 Colo. App. 413, 52 P. 86 (1897).

Governmental entities are subject to the same civil law rule applied to a private party. In Metro Docheff v. City of Broomfield, 623 P.2d 69 (Colo. App. 1980), the court found that the city had accepted the streets and storm drains in a subdivision for maintenance and control and, therefore, had exclusive control over the water collected in the subdivision. The court determined that, by approving the subdivision and drainage plan and accepting control, the city interfered with the natural conditions and thereby caused surface water to be collected and discharged upon the plaintiff's land "in greater quantity or in a different manner than had previously occurred under natural conditions." The court found that the discharge of drainage water under the circumstances of the case constituted an enjoinable trespass.

2.2.1 Planning Drainage Improvements

As a general rule, municipalities are under no legal duty to construct drainage improvements unless public improvements necessitate drainage—as in those situations in which street grading and paving or construction of schools accelerates or alters storm runoff. Denver v. Mason, 88 Colo. 294, 295 P. 788 (1931); Denver v. Capelli, 4 Colo. 25, 34 Am. Rep. 62 (1877); Daniels v. City of Denver, 2 Colo. 669 (1875). This is because statutory provisions authorizing municipal drainage improvements and flood control are generally written in non-mandatory language. Thus, absent mandatory statutory language imposing a duty on municipalities or judicial imposition of an implied duty to avoid or abate injuries, municipalities are not liable for failing to provide drainage or flood control.

In Colorado, governmental immunity has been partially waived and the governmental-proprietary distinction has been abolished. 24-10-101 C.R.S. As a result, Colorado municipalities may be exposed to liability in the future for adoption or selection of defective plans or designs for drainage. Although 24-10-103 (1) C.R.S. provides that a "dangerous condition" for which a governmental entity will be liable does not include that caused by the inadequate design of a facility, it is still unclear whether that liability exemption applies to drainage and flood control facilities.

In Scott v. City of Greeley, 931 P.2d 525 (Colo. App. 1996) the court found that the city formulated a comprehensive drainage plan which called for placement of a 42-inch storm sewer line throughout the length of the street adjacent to the property of the plaintiff and down to the river. The city placed a 42-inch pipe under a section of the street. However, the sewer renovation did not extend to the river, and the 42-inch line was instead connected to the pre-existing 15-inch line at a junction near the plaintiff's property. The plaintiff's property suffered flooding several times. The city argued that the damages that the plaintiff suffered were a result of a "design flaw" and thus immunity would apply. However, the court found the plaintiff's property was damaged not as a result of any inadequacy of the plan but rather from the city having departed from it in temporally connecting the new larger pipe to the existing 15-inch pipe.

On the basis of the Scott case, two things are clear. First, once a plan is in place, it should be followed. Second, drainage improvements should be constructed from the downstream end upstream to avoid creating flows that violate the civil law rule, or special arrangements should be made to keep potential flow damage from increasing downstream of the work. One possible exception to this general rule is the construction of detention facilities, which actually reduce the potential for downstream damages.

However, the Scott case also raises the question of whether the defense that the design of a drainage and flood control improvement caused the alleged damage will convey immunity to a governmental entity.

The court in Scott raises the question but fails to answer it. The confusion arises because of two provisions of the CGIA:

> 24-10-106 (1)(e) C.R.S.: A dangerous condition of any...sanitation facility
>
> and
>
> 24-10-106 (1)(f) C.R.S.: The operation and maintenance of any public water facility, . . . sanitation facility,...by such public entity.

Section (f) does not include the defense of a design error, but Section (e) does. A Colorado court has yet to answer the question raised by the Scott case. Therefore, based upon the state of the law, the safest approach to liability in regard to drainage and flood control improvements is to assume that the defense of a design error will not protect a governmental entity from a lawsuit for injury to property or person. Thus, the legal principles of negligence will apply to the actions of a governmental entity in designing, constructing and maintaining a drainage and flood control improvement. Thus, in order to establish a case of negligence, the following must be proved: (1) the existence of a legal duty owed by the defendant to the plaintiff; (2) a breach of that duty; (3) injury to the plaintiff; and (4) a causal relationship between the breach and the injury.

2.2.2 Construction, Maintenance, and Repair of Drainage Improvements

Municipalities can be held liable for negligent construction of drainage improvements. McCord v. City of Pueblo, 5 Colo. App. 48, 36 P. 1109 (1894); Denver v. Rhodes, 9 Colo. 554, 13 P. 729 (1887); Denver v. Capelli, 4 Colo. 25, 34 Am. Rep. 62 (1877); (as well as for negligent maintenance and repair of drainage improvements) Malvernia v. City of Trinidad, 123 Colo. 394, 229 P.2d 945 (1951); Denver v. Mason, 88 Colo. 294, 295 P. 788 (1931).

In addition to negligence, other legal theories have been used to impose liability on municipalities for faulty construction and maintenance of drainage improvements. Thus, a municipality may incur liability for trespass, Barberton v. Miksch, 128 Ohio St. 169, 190 N.E. 387 (1934) (casting water upon the land of another by seepage or percolation resulting from construction and maintenance of a reservoir was a trespass by the municipality); an unconstitutional taking, Mosley v. City of Lorain, 43 Ohio St. 2d 334, 358 N.E. 2d 596 (1976) (the city had effectively appropriated the plaintiff's property by constructing a storm sewer system which channeled a greater volume of water into the creek than the creek could reasonably be expected to handle without flooding); taking, Lucas v. Carney, 167 Ohio St. 416, 149 N.E. 2d (1958) (construction of a public improvement on county property, which greatly increased the amount and force of surface water which flowed onto the plaintiff's property, overflowing and inundating it, raised a claim of pro tanto appropriation); or nuisance, Mansfield v. Bolleet, 65 Ohio St. 451, 63 N.E. 8.6 (1902) (a municipality is liable if it causes drainage to be emptied into a natural watercourse and substantially damages a downstream landowner). Even in the absence of negligence, nuisance, trespass, or taking, the evolving doctrine of inverse condemnation is being used to permit landowners to obtain compensation from a municipality where storm runoff from municipal projects is diverted across another's land on the theory that the city has taken a drainage easement. Thus, like an easement for noise emanating from the municipal airport, physical entry by the governmental entity or statutory allowance of compensatory damages is not required in order for landowners to recover damages.

In several Colorado cases, however, municipalities have not incurred liability for faulty construction where they are found to be upstream proprietors with a natural easement for drainage—even when water is sent down in a manner or quantity to do more harm than formerly. City of Englewood v. Linkenheil, 362 P.2d 186 (1961) (the city's action in channeling water by a system of drains, catch basins, intakes, and pipes, from a higher place to a place contiguous to the land of the plaintiff, which was a natural drainage area, so as to overflow onto the land of plaintiff did not constitute a taking of property without just compensation); City and County of Denver v. Stanley Aviation Corporation, 143 Colo. 182, 352 P.2d 291 (1960) (plaintiff could not recover from the city for damage caused by flood waters which backed onto lower land on its theory that the city had been negligent or failed to use due care in installing a pipe adequate to carry the waters); Aicher v. Denver, 10 Colo. App. 413, 52 P. 86 (1897) (the city was not found liable for damage where street grade was changed, trolley tracks were permitted in a street, and a culvert was built too small, but the landowner was declared to be in the unfortunate position of having built below the grade of the street).

The CGIA provides in 24-10-103 (1) C.R.S. that maintenance does not include any duty to upgrade, modernize, modify, or improve the design or construction of a facility. Therefore, a governmental entity, under this statute, would not be found to have failed to maintain a facility if it failed to perform one or more of these enumerated actions. However, if a governmental entity fails to maintain a facility other than the excluded enumerated actions above, such failure could subject that entity to a claim that such failure was negligent, and such entity would not be protected by the CGIA.

2.2.3 Summary

In general, in the absence of negligence, a municipality will not be held liable for increased runoff occasioned by the necessary and desirable construction of drains and sewers. Denver v. Rhodes, 9 Colo. 554, 13 P. 729 (1887). Nor will a municipality be held liable for damages caused by overflow of its sewers or drains occasioned by extraordinary, unforeseeable rains or floods. 18 McQuillan, *Municipal Corporations*, §53.124 (3rd ed. 1971).

Municipal liability will attach, however, where a municipality:

1. Collects surface water and casts it in a body onto private property where it did not formerly flow.

2. Diverts, by means of artificial drains, surface water from the course it would otherwise have taken and casts it in a body large enough to do substantial injury on private land, where, but for the artificial drain, it would not go.

3. Fills up, dams back, or otherwise diverts a stream of running water so that it overflows its banks and flows on the land of another. A municipality is also liable if it fails to provide a proper outlet for drainage improvements constructed to divert surface waters or if it fails to exercise ordinary care in the maintenance and repair of drainage improvements.

This latter liability attaches when it is determined that a municipality has not exercised a reasonable degree of watchfulness in ascertaining the condition of a drainage system to prevent deterioration or obstruction. 13 McQuillan, *Municipal Corporations*, §37.254 (3rd ed. 1971). See, also, Malvernia v. City of Trinidad, 123 Colo. 394, 229 P.2d 945 (1951).

Thus, the best rule to follow in planning for the construction of drainage improvements, whether following the natural watercourse or artificially draining surface water, is that a municipality is liable if it actively injures private property as a result of improvements made to handle surface water. A municipality in Colorado appears to be in a much stronger position if it can establish that the improvement followed natural drainage patterns. Drainage District v. Auckland, 83 Colo. 510, 267 P. 605 (1928); City of Englewood v. Linkenheil, 362 P.2d 186 4961); City of Boulder v. Boulder and White Rock Ditch and Reservoir Company, 73 Colo. 426, 216 P. 553 (1923). See Kenworthy, "Urban Drainage: Aspects of Public and Private Liability," July-August 1962, *DICTA*, p. 197; Shoemaker, "An Engineering-Legal Solution to Urban Drainage Problems," 45 *Denver Law Journal* 381 (1968).

2.3 Municipal Liability for Acts of Others

2.3.1 Acts or Omissions of Municipal Officers, Agents, or Employees

The general rule is that a municipality is not liable under the doctrine of respondent superior for the acts of officers, agents, or employees that are governmental in nature but is liable for negligent acts of its agents in the performance of duties relating to proprietary or private corporate purposes of the city. Denver v. Madison, 142 Colo. 1, 351 P.2d 826 (1960). The construction, maintenance and repair of drainage improvements have been regarded as proprietary or corporate functions. Denver v. Maurer, 47 Colo. 209, 106 P. 875 (1910). Although the governmental-proprietary distinction has been abolished by statute in Colorado, the distinction apparently still applies whenever the injury arises from the act, or failure to act, of a public employee who would be, "or heretofore has been personally immune from liability." 24-10-106 C.R.S. Thus, a municipality may be held liable for the acts of its officers, agents or employees for injuries resulting from negligent construction, maintenance, or dangerous conditions of a public facility. 24-10-106 (l)(e), (l)(f) C.R.S. However, it is not clear whether in Colorado liability attaches or, conversely, whether the defense of governmental immunity applies, to the adoption, selection, or approval of a defective plan or design. The governmental immunity statute provides for a waiver of governmental immunity when injuries result from the operation and maintenance or dangerous condition of a public facility. 24-10-106 (l)(e), (l)(f) C.R.S. The statute also states that “a dangerous condition shall not exist solely because the design of any facility…, is inadequate in relation to its present use.” 24-10-103 (1) C.R.S. Since the distinction between construction and design is often vague, it is difficult to predict how the Colorado courts will approach municipal liability for injuries resulting from adoption, selection, or approval of a defective plan or design by municipal officers, agents, or employees.

In three cases considered by the Colorado Court of Appeals since the enactment of the CGIA [Burnworth v. Adams County, 826 P.2d 368 (Colo. App. 1991); Scott v. City of Greeley, 931 P.2d 525 (Colo. App. 1996); and Smith v. Town of Estes Park, 944 P.2d 571 (Colo. App. 1996)], when the court was faced with the questions of whether the damage was caused by a design error or the operation and maintenance of a drainage or flood control facility, the court found that the damage was caused by the operation and maintenance of the facility. Therefore, the governmental entity had no immunity and was treated as a private citizen in regard to its negligence.

Before an individual can recover damages from a public entity for injuries caused by the public entity or one of its employees, the CGIA requires written notice to the public entity involved within 180 days after the date of discovery of the injury. Otherwise, failure to notify is a complete defense to a personal injury action against a municipality. 24-10-109 C.R.S. Kristensen v. Jones, 575 F.2d 854 (1978).

2.3.2 Municipal Liability for Acts of Developers

Unless an ordinance or statute imposes a duty on a municipality to prevent or protect land from surface water drainage, a municipality will not incur liability for wrongfully issuing building permits, failing to enforce an ordinance, or approving defective subdivision plans. Breiner v. C & P Homebuilder’s Inc., 536 F.2d 27 (3rd Cir. 1976), reversing the District Court. (In a suit by landowners in an adjacent township against a borough, its engineers, and subdivision developer for damages caused by increased flow of surface water from development where the borough approved a subdivision plan which did not provide drainage facilities and issued building permits, the borough was not liable because it owed no duty to landowners outside its boundaries. However, the developer was held liable.)

One state court, however, has held that a municipality is liable for damages where the municipality has furnished building permits to a contractor for development of an industrial complex which benefited the village financially but also diminished surface area available for drainage of water, causing flooding of neighboring servient estates. Myotte v. Village of Mayfield, 375 N.E.2d 816 (1977). In Myotte, the village’s liability was based on the following reasoning:

> To require the developer to pick up the cost of flood prevention by requiring him to acquire land along stream margins for widening or deepening to accommodate accelerated flow, would subject him to possible overreaching by riparian owners. The

> developer has no power of eminent domain. Municipalities do have powers of condemnation. Accordingly, as an advantaged party with the power to protect itself from crisis pricing, it seems reasonable and just that the municipality should either enlarge the stream to accommodate water accelerated from permitted improvements that enrich it or pay the consequences.
>
> Myotte, supra at 820. (Day, J. concurring.). See also, Armstrong v. Francis Corporation, 20 N.J. 320, 120 A.2d 4 (1956); Sheffet v. County of Los Angeles, 3 Cal. App. 3d 720 (1970); Powers, et al., County of Clark and Clark County Flood Control District, District Court, State of Nevada (No. A 125197) (1978).

There is a trend toward imposing a greater burden or responsibility on municipalities for the drainage consequences of urban development. See Wood Brothers Homes, Inc. v. City of Colorado Springs, 568 P.2d 487 (1977) (where the city abused its discretion by not granting variance and by assessing the entire cost of a major drainage channel on the developer, where the area to be served by the major drainage channel already suffered from occasional flooding and needed an expanded drainage facility whether the property was developed or not).

2.4 Personal Liability of Municipal Officers, Agents, and Employees

An injured person always has a remedy against the original tort feasor even if no recovery may be had from the municipality for acts of its officers, agents, or employees in discharge of governmental functions. Denver v. Madison, 142 Colo. 1, 351 P.2d 826 (1960). Thus, public employees generally have been personally liable for injuries caused by their negligent actions within the scope of employment, even when the defense of sovereign immunity was available to their employers. Antonpoulos v. Town of Telluride, 187 Colo. 392, 532 P.2d 346 (1975); Liber v. Flor, 143 Colo. 205, 353 P.2d 590 (1960). Since an injured person's right to sue the negligent employee of an immune entity derives from the common law, the Colorado Supreme Court will not infer legislative abrogation of that right absent clear legislative intent. Thus, the CGIA is only directed toward liability of public entities. Kristensen v. Jones, 574 P.2d 854 (1978) (a bus driver for the regional transportation district was found personally liable for injuries sustained in a collision with the district's bus, and written notice was not a condition precedent to a suit against a public employee in his or her individual capacity).

The CGIA provides both for the defense of any governmental employee who is sued individually as a result of the employee's acts during the performance of his or her duties as well as the payment of any judgment or settlement. The act provides in part that a public entity shall be liable for the payment of all judgments and settlements of claims against any of its public employees where the claim against the public employee arises out of injuries sustained from an act or omission of such employee occurring during the performance of his or her duties and within the scope of employment, except where such act or omission is willful and wanton or where sovereign immunity bars the action against the public entity (24-10-110 [b][l] C.R.S.).

Therefore, it is possible for an employee to be personally liable for a negligent act and the public entity to escape liability. Such a situation would arise when the claimant fails to give proper notice to the public entity, thus providing that entity with the defense of lack of jurisdiction against it. However, the public employee would have no such defense.

3.0 DRAINAGE IMPROVEMENTS BY A LOCAL GOVERNMENT

> In an era of increasing urbanization and suburbanization, drainage of surface water most often becomes a subordinate feature of the more general problem of proper land use—a problem acutely sensitive to social change.
>
> Pendergast v. Arkin, 236 S.E. 2d 787, 796 N. Carolina.

3.1 Constitutional Power

A municipality's inherent police powers enable it to enact ordinances that serve the public's health, safety, morals, or general welfare. Ordinances addressing drainage problems are clearly a proper exercise of a municipality's police powers. Wood Brother's Homes, Inc. v. City of Colorado Springs, 568 P.2d 487, 490 (1977). Hutchinson v. Valdosta, 227 U.S. 303, 308 (1913).

3.2 Statutory Power

3.2.1 Statutes—Municipalities

3.2.1.1 Municipal Powers—Public Property and Improvements

31-15-701, 31-15-714 C.R.S. The statute grants municipalities the power to establish, improve, and regulate such improvements as streets and sidewalks, water and water works, sewers and sewer systems, and water pollution controls. In addition, a municipality may, among other powers, "deepen, widen, cover, wall, alter or change the channel of watercourses." 31-15-711 (1) (a) C.R.S.

3.2.1.2 Public Improvements—Special Improvement Districts in Municipalities

31-25-501, 31-25-540 C.R.S. The statute authorizes municipalities to construct local improvements and assess the cost of the improvements wholly or in part upon property specially benefited by such improvements. By ordinance, a municipality may order construction of district sewers for storm drainage in districts called storm sewer districts.

3.2.1.3 Public Improvements—Improvement Districts in Municipalities

31-25-601, 31-25-630 C.R.S. The statute authorizes municipalities to establish improvement districts as taxing units for the purpose of constructing or installing public improvements. The organization of districts is initiated by a petition filed by a majority of registered electors of the municipality who own real or personal property in the district.

3.2.1.4 Sewer and Water Systems—Municipalities

31-35-401, 31-35-417 C.R.S. The statute authorizes municipalities to operate, maintain, and finance water and sewage facilities for the benefit of users within and without their territorial boundaries. Sewerage facilities are defined as "any one or more of the various devices used in the collection, treatment, or disposition of sewage or industrial wastes of a liquid nature or storm, flood, or surface drainage waters...." 31-35-491(6) C.R.S.

3.2.2 Statutes—County

3.2.2.1 Public Improvements—Sewer and Water Systems

30-20-401, 30-20-422 C.R.S. The statute authorizes county construction, maintenance, improvement and financing of water and sewerage facilities for the county's own use and for the use of the public and private consumers and users within and without the county's territorial limits.

3.2.2.2 County Public Improvement Districts

30-20-501, 30-20-531 C.R.S. The statute authorizes creation of public improvement districts within any county as taxing units for purposes of constructing, installing, or acquiring any public improvement. 30-20-513 C.R.S. lists special benefits for purposes of assessing improvements within a public improvement district, particularly with respect to storm sewer drainage and drainage improvements to carry off surface waters.

3.2.2.3 Public Improvements—Local Improvement Districts—Counties

30-20-601, 30-20-626 C.R.S. The statute authorizes a county by resolution to construct local improvements and assess costs thereof wholly or in part upon property specially benefited by such improvements.

3.2.2.4 Flood Control—Control of Stream Flow

30-30-101, 30-28-105 C.R.S. The statute authorizes the board of county commissioners of each county for flood control purposes only:

> ...to remove or cause to be removed any obstruction to the channel of any natural stream which causes a flood hazard, and for such purpose only the board of county commissioners shall have a right of access to any such natural stream, which access shall be accomplished through existing gates and lanes, if possible. Such authority includes the right to modify existing diversion or storage facilities at no expense to the diverter of a water right, but it shall in no way alter or diminish the quality or quantity of water entitled to be received under any vested water right.
>
> 30-30-102 (1) C.R.S.

3.2.2.5 Conservancy Law—Flood Control

37-1-101, 37-8-101 C.R.S. The statute authorizes the district court for any county to establish conservancy districts for any of the following purposes:

> Preventing floods; regulating stream channels by changing, widening, and deepening the same; regulating the flow of streams; diverting, controlling, or in whole or in part eliminating watercourses; protecting public and private property from inundation…

3.2.2.6 Drainage Districts

37-20-101, 37-33-109 C.R.S. The statute authorizes owners of agricultural lands susceptible to drainage by the same general system of works to petition the board of county commissioners for the organization of a drainage district.

3.2.3 Statutes—State

3.2.3.1 Colorado Land Use Act

24-65-101, 24-65-105 C.R.S. The statute establishes a nine-member Colorado land use commission. Among other powers, the commission has authority to assist counties and municipalities in developing guidelines for developing land uses and construction controls within designated floodways.

3.2.3.2 Drainage of State Lands

37-30-101, 37-30-105 C.R.S. The statute authorizes the state board of land commissioners to make contracts with any person, corporation, association, or drainage district to provide drainage of state lands.

3.2.3.3 Water Conservation Board of Colorado

37-61-101, 37-60-123 C.R.S. The statute creates a 13-member state water conservation board for purposes of water conservation and flood prevention. An important duty of this board is to "designate and approve storm or floodwater runoff channels or basins, and to make such designations available to legislative bodies of cities and incorporated towns, ...and counties of this state." 30-60-123 C.R.S.

3.2.3.4 State Canals and Reservoirs

37-88-101, 37-88-109 C.R.S. The statute authorizes the Department of Corrections to locate, acquire, and construct ditches, canals, reservoirs, and feeders for irrigating and domestic purposes for the use of the State of Colorado. The board of county commissioners have charge and control of any state reservoir in their county including the obligation to maintain and keep said reservoir in good condition at the county's expense. In addition, the county in which the state reservoir is located is liable for any damages resulting from breakage of the dams or water discharges therefrom.

3.2.3.5 Regulatory Impairment of Property Rights

29-20-201 C.R.S. This law became effective July 1, 1999. One of the legislative declarations of the act is that "The general assembly further finds and declares that an individual private property owner should not be required, under the guise of police power regulation of the use and development of property, to bear burdens for the public good that should more properly be borne by the public at large." The main thrust of the act is contained in 29-20-203 (1) C.R.S., which reads as follows:

> In imposing conditions upon the granting of land-use approvals, no local government shall require an owner of private property to dedicate real property to the public, or pay money to a public entity in an amount that is determined on an individual and discretionary basis, unless there is an essential nexus between the dedication or payment and a legitimate local government interest, and the dedication or payment is roughly proportional both in nature and extent to the impact of the proposed use or development of such property. This section shall not apply to any legislatively formulated assessment, fee, or charge that is imposed on a broad class of property owners by local government.

The act goes on to prescribe the remedies available to a private property owner who believes his or her rights have been violated under the act. However, unlike most litigation, it is the burden of the local government and not the plaintiff “to establish, based upon substantial evidence appearing in the record” that the dedication or payment required by the local government is roughly proportional to the impact of the proposed use of the subject property.

Therefore, the Colorado legislature has now established a standard that is consistent with the leading case law in this area to assist local governments with reaching a safe harbor when imposing conditions on development. The concepts are fairly simple. First, the conditions imposed have to have some causal relationship with the impact of the development and, second, those conditions must be “roughly proportional” to the impact of the development. However, it should be noted that these restrictions relate only to those instances where the local government is negotiating individually with a developer as to what conditions will be imposed by the local government. The act does provide that, if the local government is legislatively imposing conditions for development on a broad class of property owners, the “essential nexus” and “roughly proportional” requirements of the act do not apply to those legislatively imposed conditions.

3.2.3.6 Intergovernmental Relationships

29-1-201 C.R.S. In 1974, Section 2 of Article XI of the state constitution was amended to permit and encourage governments to make the most efficient and effective use of their powers and responsibilities by cooperating and contracting with other governments. 29-1-203 C.R.S. provides more detail in regard to how that cooperation is to be carried out. It reads in part as follows:

> Governments may cooperate or contract with one another to provide any function, service, or facility lawfully authorized to each of the cooperating or contracting units, including the sharing of costs, the imposition of taxes, or the incurring of debt, only if such cooperation or contracts are authorized by each party thereto with the approval of its legislative body or other authority having the power to so approve.

3.2.4 Urban Drainage and Flood Control Act

32-11-101 C.R.S., et. seq., established the Urban Drainage and Flood Control District (District), including all of the City and County of Denver and the urbanized and urbanizing portions or Adams, Arapahoe, Boulder, Douglas and Jefferson Counties. An 18-person board, comprised of 16 elected officials and 2 professional engineers, is given the power to (1) plan solutions to drainage and flood control problems (with an authorized mill levy of 0.1 mill); (2) construct drainage and flood control improvements (with an authorized mill levy of 0.4 mill); (3) maintain such improvements and other natural drainageways in the District (with an authorized mill levy of 0.4 mill); and (4) construct drainage and flood control improvements in and adjacent to the South Platte River (with an authorized mill levy of 0.1 mill). The board also has the power to adopt and enforce a floodplain regulation.

4.0 FINANCING DRAINAGE IMPROVEMENTS

> The ability of one owner to develop land, install impervious surfaces, alter drainage paths, and accelerate runoff onto other properties involves more than issues of what rights and relief should be accorded neighboring property owners. Urbanization may double or triple the peak flows of 5- and 10-year floods. Lands far downstream may be severely affected by the cumulative impact of unplanned and unregulated changes in drainage patterns due to urban clearance, grading, and development. Increasingly, the costs of uncontrolled drainage modifications and storm water management have fallen on the state and federal budgets.
>
> Westen, *Gone With the Water—Drainage Rights and Storm Water Management in Pennsylvania*, 22 Vill. L. Rev. 901, 902 (1976-77).

4.1 Capital Improvement

Resources from the current budget, usually derived from sales, property, and income taxes, can be used to finance drainage improvements. Since the cost is paid from the “general fund” or “capital improvement fund” and no specific property tax is levied, the financing is relatively simple.

4.2 Local Improvement

Financing for drainage improvements through local improvements or as part of a general bond issue requires that all property be assessed on a valuation basis. Since a majority of all taxpaying electors must approve the decision, the success of this method usually turns on how well the facts (needs) have been prepared and how well a plan has been developed.

4.3 Special Improvement

When drainage improvements are financed as special improvements, the property assessed must be specially benefited. In Colorado, benefits, for purposes of special assessments, are defined in several statutory sections. (See 30-20-513, 30-20-606, 31-25-507, and 37-23-101.5 C.R.S.). For example, 37-23-101.5 C.R.S. provides:

> Determination of special benefits—factors considered. (1) The term ‘benefit,’ for the purposes of assessing a particular property within a drainage system improvement district, includes, but is not limited to, the following: (a) any increase in the market value of the property; (b) the provision for accepting the burden from specific dominant property for discharging surface water onto servient property in a manner or quantity greater than would naturally flow because the dominant owner made some of his property impermeable; (c) any adaptability of property to a superior or more profitable use; (d) any alleviation of health and sanitation hazards accruing to particular property or accruing to public property in the improvement district, if the provision of health and sanitation is paid for wholly or partially out of funds derived from taxation of property owners of the improvement district; (e) any reduction in the maintenance costs of particular property or of public property in the improvement district, if the maintenance of the public property is paid for wholly or partially out of funds derived from taxation of property owners of the improvement district; (f) any increase in convenience or reduction in inconvenience

> accruing to particular property owners, including the facilitation of access to and travel over streets, roads, and highways; (g) recreational improvements accruing to particular property owners as a direct result of drainage improvement.

This statute was adopted by the Colorado legislature to define “benefits,” a term previously defined only by courts. See Shoemaker, “What Constitutes ‘Benefits’ for Urban Drainage Projects,” 51 *Denver L. Journal* 551 (1974).

Although a benefit to the premises assessed must at least be equal to the burden imposed, the standard of apportionment of local improvement costs to benefits is not one of absolute equality, but one of reasonable approximation. Satter v. City of Littleton, 185 Colo. 90, 522 P.2d 95 (1974). A presumption of validity inheres in a city council’s determination that benefits specifically accruing to properties equal or exceed assessments thereon. Satter, supra. Further, a determination of special benefits and assessments is left to the discretion of municipal authorities, and their determination is conclusive in the courts unless it is fraudulent or unreasonable. Orchard Court Development Co. v. City of Boulder, 182 Colo. 361, 513 P.2d 199 (1973). A determination of no benefit in an eminent domain proceeding does not preclude a subsequent special assessment providing a landowner’s property benefited from construction of the improvement. City of Englewood v. Weist, 184 Colo. 325, 520 P.2d 120 (1974). See, also, Denver v. Greenspoon, 140 Colo. 402, 344 P.2d 679 (1959); Town of Fort Lupton v. Union Pacific R.R. Co., 156 Colo. 352, 399 P.2d 248 (1965); Houch v. Little River District, 239 U.S. 254 (1915); and Miller and Lux v. Sacramento Drainage District, 256 U.S. 129 (1921).

4.4 Service Charge

The District can charge service fees for the use of its facilities or services and thereby finance its improvements. 32-11-217 (l)(e), 32-11-306 C.R.S. provides:

> Such service charges may be charged to and collected in advance or otherwise by the District at any time or from time to time from any person owning real property within the District or from any occupant of such property which directly or indirectly is, has been, or will be connected with the drainage and flood control system of the District or from which or on which originates or has originated rainfall, other surface and subsurface drainage, and storm and flood waters (or any combination thereof) which have entered or may enter such system, and such owner or occupant of any such real property shall be liable for and shall pay such service charges to the District at the time when and place where such service charges are due and payable.

Storm and flood control facilities fall within the definition of “sewerage facilities” defined in 30-35-401 (5) C.R.S; 31-35-402 (1) C.R.S. states:

> In addition to the powers which it may now have, any municipality, without any election of the taxpaying or qualified electors thereof, has power under this part for:
>
> (f) to prescribe, revise and collect in advance or otherwise, from any consumer or any owner or occupant of any real property connected therewith or receiving service

> therefrom rates, fees, tolls, and charges or any combination thereof for the services furnished by, or the direct or indirect connection with, or the use of, or any commodity from such water facilities or sewerage facilities or both,...

A service charge is neither a tax nor a special assessment but is a fee for the sole purpose of defraying the cost of establishing and maintaining a storm drainage and flood control utility. Western Heights Land Corp. v. City of Fort Collins, 146 Colo. 464, 362 P.2d 155 (1961). See, also, City of Aurora v. Bogue, 176 Colo. 198, 4-9 P.2d 1295 (1971); Brownbriar Enterprises v. City and County of Denver, 177 Colo. 198, 493 P.2d 352 (1972); and City of Boulder v. Arnold, 978 P.2d 149 (Colo. App. 1976) which upheld the City of Boulder's flood control fee. Counties in Colorado have similar powers pursuant to 30-20-402 (1) C.R.S.

4.5 Developer's Cost

1. A county planning commission or the board of adjustment of any county may condition any portion of a zoning resolution, or any amendments or exceptions thereto, upon "the preservation, improvement, or construction of any storm or floodwater runoff channel designated and approved by the Colorado Water Conservation Board." 30-28-111 (2) C.R.S.

2. Every Colorado county is required to have a planning commission to develop, adopt and enforce subdivision regulations. Among the provisions that the board of county commissioners must include in the county's regulations are those requiring developers to submit:

   a. A plat and other documentation showing the layout or plan of development, including, where applicable, the following information:

      i. Estimated construction cost and proposed method of financing of the streets and related facilities, water distribution system, sewage collection system, storm drainage facilities, and such other utilities as may be required of the developer by the county.

      ii. Maps and plans for facilities to prevent stormwater in excess of historic runoff caused by the proposed subdivision from entering, damaging, or being carried by conduits, water supply ditches and appurtenant structures, and other storm drainage facilities. 30-28-133 (3)(c) C.R.S.

In addition, subdivision regulations must include provisions governing:

> Standards and technical procedures applicable to storm drainage plans and related designs, in order to ensure proper drainage ways, which may require, in the opinion of the board of county commissioners, detention facilities which may be dedicated to the county or the public, as are deemed necessary to control, as nearly as possible, storm waters generated exclusively within a subdivision from a one-hundred year storm which

> are in excess of the historic runoff volume of storm water from the same land area in its undeveloped and unimproved condition.
>
> 30-28-133 (4)(b) C.R.S.

4.6 The Taxpayers Bill of Rights, Article X, Section 20, Colorado Constitution

On December 31, 1992 the Taxpayers Bill of Rights (TABOR) became effective. Its effect is to limit governmental spending generally so that "the maximum annual percentage change in each local district's fiscal year spending equals inflation in the prior calendar year plus annual local growth." In addition to a spending limitation, TABOR imposes a revenue limit that is similar to the spending limit. Finally, districts must have voter approval in advance for:

> ...any new tax, tax rate increase, mill levy above that for the prior year, valuation for assessment ratio increase for a property class, or extension of an expiring tax, or a tax policy change directly causing a net tax revenue gain to any district.

Prior to the passage of TABOR there were a number of cases that addressed whether a service charge was a tax. The first of note was Zelinger v. City and County of Denver, 724 P.2d 1356 (Colo. 1986) wherein a storm drainage service charge was attacked as an unconstitutional property tax and an unconstitutional denial of equal protection and due process guarantees to property owners. The storm drainage service charge applied to all owners of property in Denver and was used to pay for the operation, maintenance, improvement and replacement of the city's storm drainage facilities. The charge was based on the ratio of impervious to pervious land surface. The higher the ratio of impervious to pervious surface, the greater the charge per square foot. The Colorado Supreme Court held that such a service charge was not a tax nor was it a violation of due process or equal protection. The court concluded with the following finding:

> ...although alternative cost allocation schemes may be equally well-suited or arguably better suited to serving the governmental interest in providing storm drainage facilities than the scheme actually adopted, the equal protection clauses do not authorize the invalidation of the scheme chosen unless it is without rational foundation.

The Zelinger case has continued as good law ever since 1986 and has been cited recently as the law of Colorado in regard to these matters. Thus, a storm drainage service charge similar to that adopted by Denver is not a tax and therefore is not subject to the limitations of TABOR.

In 1989 the Colorado Supreme Court revisited fees in the case of Bloom v. City of Fort Collins, 784 P.2d 304 (Colo. 1989). In that case the court considered a transportation utility fee and held that such a fee was not a property tax but rather a special fee imposed upon owners or occupants of developed lots fronting city streets and that such a fee is reasonably related to the expenses incurred by the city in carrying out its legitimate goal of maintaining an effective network of city streets. The court in reaching this conclusion considered any number of possibilities as to what this fee was and rejected the following as not applying: property tax, excise tax and special assessment. It therefore found that the fee was a special fee that was a charge imposed on persons and property and reasonably designed to meet the overall cost of the service for which the fee is imposed.

Finally, in the case of City of Littleton v. State of Colorado, 855 P.2d 448 (Colo. 1993), the Colorado Supreme Court addressed another stormwater and flood management utility fee. The fee was enacted to prevent damage to property from accumulations and uncontrolled runoff of water. The ordinance declares that as the ultimate beneficiaries and users of the contemplated system, the owners of property within the city shall be required to pay a fee for the costs of constructing, operating, maintaining and replacing the system and its facilities. The state Community Colleges Board challenged the fee as a special assessment and thus something that could not be charged against the state. The court found that, despite the fact that the service fees did not specifically benefit the property owned by the state, it did create the capacity to remove excess water from property and prevent flooding, which benefited all property owners; thus, the fee is a permissible fee.

In conclusion, drainage fees, if properly structured, are not property taxes and can be implemented without TABOR implications. However, outside of Colorado, there have been three recent cases where each have held, for various reasons, that a “stormwater service charge,” a “stormwater utility charge” and a “stormwater drainage service charge” are each a tax and not a fee. Those cases are Bolt v. City of Lansing, 561 N.W. 2d 423 (Mich. 1997); Fulton County Taxpayers Association v. City of Atlanta, Georgia, Superior Court of Fulton County, State of Georgia, Civil Action File Number: 1999 cv05897; and City of Cincinnati v. United States, United States Court of Appeals for the Federal Circuit, 98-5039.

4.7 Water Activities—Enterprise Statute 37-45.1-101 C.R.S.

This statute, which was adopted after the passage of TABOR, takes advantage of the exception in TABOR that the same does not apply to governmental enterprises by setting forth, in regard to water activities, what a governmental entity needs to do to become and remain a enterprise and thus not subject to TABOR. Numerous Front Range cities have taken advantage of this statute to adopt enterprises without a vote of the people to address drainage and flooding issues in their municipalities.

The statute provides in regard to the establishment of a water activity enterprise that:

> Any district which under applicable provisions of law has its own bonding authority may establish or may continue to maintain water activity enterprises for the purpose of pursuing or continuing water activities including...water project or facility activities, including the construction, operation, repair, and replacement of water or wastewater facilities. Any water activity enterprise established or maintained pursuant to this article is excluded from the provision of Section 20 of Article X of the state constitution.

The statute defines “water project or facility” as including a dam, storage reservoir, compensatory or replacement reservoir, canal, conduit, pipeline, tunnel, power plant, water or wastewater treatment plant, and any and all works, facilities, improvements, and property necessary or convenient for the purpose of conducting a water activity. The statute also defines water activity as including stormwater services.

Two restrictions in regard to water activity enterprises are that they cannot receive more than 10 percent of their annual revenues from grants from state and local governmental entities and that an enterprise may not tax.

5.0 FLOODPLAIN MANAGEMENT

Floodplain management involves fuller use of non-structural techniques. See 24-65.1-202 (2)(a)(I) C.R.S. Such techniques include:

1. Floodplain zoning and building code ordinances to regulate flood area construction.

2. Flood insurance programs.

3. Flood warning systems, including notification to occupants of floodplains.

See Westen, *Gone With the Water—Drainage Rights and Storm Water Management in Pennsylvania*, 22 Vill. L. Rev., 901, 972 (1976-77).

5.1 Floodplain Regulations

5.1.1 Constitutional Considerations

The general principles of zoning were established in Village of Euclid v. Amber Realty Co., 272 U.S. 365 (1926), in which the U.S. Supreme Court stated:

> While the meaning of constitutional guarantees never varies, the scope of their application must expand or contract to meet new and different conditions that are constantly coming within the field of their operation.

The court in Colorado has determined that zoning is justified as a valid exercise of police power, and that this legal basis for zoning legislation must be reconciled with the legitimate use of private property, in harmony with constitutional guarantees. Westwood Meat Market, Inc. v. McLucas, 146 Colo. 435, 361 P.2d 776 (1961); People ex rel. Grommon v. Hedgcock, 106 Colo. 300, 104 P.2d 607 (1940).

The adoption by a municipality of floodplain ordinances to regulate flood-prone areas is a valid exercise of police power and is not a taking. Morrison v. City of Aurora, 745 P.2d 1042 (Colo. App. 1987).

5.1.2 Statutory Grants of Power

Specific legislative action has given local governments authority to proceed in floodplain regulation. In Colorado, cities, counties, and the District all have plenary grants of power.

The governing body of each municipality has the following authority:

> To establish, regulate, restrict and limit such uses on or along any storm or floodwater runoff channel or basin, as such storm or floodwater runoff channel or basin has been designated and approved by the Colorado Water Conservation Board, in order to lessen or avoid the hazards to persons and damage to property resulting from the accumulation of storm or floodwaters.
>
> 31-23-301 (1) C.R.S.

Counties in Colorado are directly authorized by statute to adopt zoning plans concerned with regulating use in a floodplain area through the provisions of 30-28-111 (1) C.R.S.:

> ...the county planning commission may include in said zoning plan or plans provisions establishing, regulating, and limiting such uses upon or along any storm or water runoff channel or basin as such storm or runoff channel or basin has been designated and approved by the Colorado Water Conservation Board in order to lessen or avoid the hazards to persons and damage to property resulting from the accumulation of storm or flood waters.

Home rule counties and cities have the same powers as noted above. These powers may be expanded by charter as long as those powers do not violate the Colorado constitution dealing with home rule governmental entities.

The District is authorized to:

> ...adopt, amend, repeal, enforce, and otherwise administer under the police power such reasonable floodplain zoning resolutions, rules, regulations, and orders pertaining to properties within the district of any public body or other person (other than the federal government) reasonably affecting the collection, channeling, impounding or disposition of rainfall, other surface and subsurface drainage, and storm and flood waters (or any combination thereof), including without limitation variances in the event of any practical difficulties or unnecessary hardship and exceptions in the event of appropriate factors, as the board may from time to time deem necessary or convenient. <u>In the event of any conflict between any floodplain zoning regulation adopted under this section and any floodplain zoning regulation adopted by any other public body, the more restrictive regulation shall control</u>. (emphasis added)
>
> 32-11-218 (1) (f) (I) C.R.S.

Because of the underlined language above, the District has proceeded on the basis that if local governments within the District fail to adopt floodplain regulations, then the District would administer its regulation within that local jurisdiction. Further, since the District's regulation prohibits residential development within the floodway (the most hazardous portion of the floodplain), any local government failing to prohibit residential development within the floodway would be governed by the District's regulation inasmuch as the District's regulation would be "more restrictive" and, thus, controlling under the statute.

5.1.3 Court Review of Floodplain Regulations

The leading Colorado case is <u>Famularo v. Adams County</u>, 180 Colo. 333, 505 P.2d 958 (1973), in which the Colorado Supreme Court upheld the District Court's findings that (1) the Adams County Commissioners had authority to regulate, by resolution, the uses of land in unincorporated areas for "trade, industry, residence, recreation, or other purposes, and for flood control"; and (2) the regulation in question did not so limit the uses of plaintiff's land so as to violate the Colorado Constitution, Article II, §25 or the U.S. Constitution, Amendment XIV.

In the case of Kolwicz v. City of Boulder, 538 P.2d 482 (Colo. App. 1975) the court was asked to determine if a city resident had standing to sue the city to require the city council and its administrator to implement floodplain regulations by adopting a map that delineated the floodway and the flood storage areas within the floodplain, for which the city had adopted a map four years prior to the lawsuit. The court denied the city resident's request on the basis that nothing in the record showed that the resident herself had been aggrieved, wronged, or had any of her rights impaired or threatened as a result of the city council's failure to implement its regulations.

In the case of Hermanson v. Board of County Commissioners of Fremont, 595 P.2d 694 (Colo. App. 1979), the court addressed an assertion by the plaintiff that his property had been taken from him because of a series of regulatory obstructions to its development that had been imposed by the county. The plaintiff alleged that his property had been taken by inverse condemnation, and the court found that such an action is justified when there has been a taking of private property for public use without payment of just compensation by some public body that has the power of eminent domain. However, the court did acknowledge that it is true that the use of property may be regulated by valid exercise of the police power, if the regulation does not go beyond protection of the public health, safety, morals, and welfare. Therefore, it found that, when regulations are designed to depress value with a view to future acquisition, this may form the basis of a cause of action for compensation on the theory of inverse condemnation against the public entity initiating the regulation.

Finally, in the case of Morrison v. City of Aurora, 745 P.2d 1042 (Colo. App. 1987), a property owner alleged that the city's adoption of floodway restrictions was a taking of his property. The court found for the city, since an adoption by a municipality of floodplain ordinances to regulate flood-prone areas is a valid exercise of police power and is not a taking.

In Colorado, the legislature has taken the lead in granting local governments power to regulate flood hazard areas. Usually, courts interpret such regulation that follows on a case-by-case basis, depending on what is "reasonable" under the circumstances. Some guidelines that have emerged in anticipating "reasonableness" follow.

5.1.3.1 Restriction of Uses

The restriction of uses on property that would prevent a public harm, as opposed to the creation of a public *benefit,* removes the requirement of compensation to property owners who are restricted from the full use of their property. Dunham, *A Legal and Economic Basis for City Planning*, 58 Colum. L. Rev. 650 (1958).

The restrictions on the uses must not be so severe as to deny the owners a constitutional right to make "beneficial use" of their land because such restrictions would be confiscatory and void. Francis v. City and County of Denver, 160 Colo. 440, 418 P.2d 45 (1966). However, a zoning ordinance is not unconstitutional because it prohibits a landowner from using or developing his or her land in the most profitable manner. It is not required that a landowner be permitted to make the best, maximum or most profitable use of his or her property. Baum v. City & County of Denver, 363 P.2d 688 (Colo. 1961); and Sundheim v. Board of County Commissioners of Douglas County, 904 P.2d 1337 (Colo. App. 1995).

5.1.3.2 Health Regulations

The relationship of the zoning restrictions to the public's health, safety, morals, and general welfare must be considered. Whether the zoning provisions are reasonable and for the promotion of the public's welfare must be determined by the court from the facts, circumstances, and locality in a particular case. DiSalle v. Giggal, 128 Colo. 208, 261 P.2d 499 (1953).

A similar matter in zoning restrictions was determined by the U.S. Supreme Court in upholding the validity of the police power in a zoning ordinance that prohibited excavation below a certain water table, which in effect deprived the property of its most beneficial use, stated:

> The ordinance in question was passed as a safety measure, and the town is attempting to uphold it on that basis. To evaluate its reasonableness, we therefore need to know such things as to the nature of the menace against which it will protect, the availability and effectiveness of other less drastic protective steps, and the loss which the appellants will suffer from the imposition of the ordinance.
>
> Goldblatt v. Town of Hempstead, (N.Y.) 369 U.S. 590 (1962).

This holding appears to coincide with the Colorado cases on the requirements for the determination by the court from facts, circumstances, and locality in a particular case, as to the reasonableness of the zoning ordinances in their promotion of the general welfare, and to prove that the restrictive use would bear a substantial relation to the public's health, safety, morals, or general welfare. DiSalle v. Giggal, supra; Westwood Meat Market, Inc. v. McLucas, supra.

5.1.3.3 Determination of Boundaries

The boundaries of the floodplain should be accurately determined and based on a reasonable standard. Mallett v. Mamaroneck, 1313 N.Y. 821, 125 N.E. 2d 875 (1955).

The setting of the boundaries of the floodplain zone to determine the hydraulic reach of a potential flood should be determined accurately. The accuracy of which will be affected by terrain, river course, and other factors that will necessarily cause some variation from the initially adopted boundary.

The Federal Emergency Management Agency (FEMA), U.S. Army Corps of Engineers, Colorado Water Conservation Board (CWCB), the District, and local governments have conducted extensive stream surveys throughout Colorado. The surveys have been completed upon reasonable scientific standards and have often become an integral part of the floodplain zoning ordinances and resolutions adopted by Colorado's cities and counties.

The CWCB has actively cooperated in the past to designate and approve such areas as delineated as a storm or “floodwater runoff channel or basin.” Such approval or designation of a runoff channel or basin by the CWCB is required by statute prior to any action by a local government, including the District, to set the boundaries on proposed floodplain zoning resolutions.

5.2 Flood Insurance

The National Flood Insurance Act of 1968, as amended in 1973, provides for a federally subsidized flood insurance program conditioned on active management and regulation of flood plan development by states and local governments. 42 U.S.C., §§4001 and 4128; 24 C.F.R., §1979.1-1925.14 (1975). Communities designated as flood prone by FEMA can obtain flood insurance eligibility for structures within the community upon meeting the qualifications of the act by developing a floodplain management system. Development of a floodplain management system requires the community to promulgate a land use and building permit system that restricts development in flood hazard areas. FEMA publishes a list, updated monthly, of the status of communities. Flood insurance is provided on a subsidized basis through all licensed insurance agents.

Federally regulated lending institutions (FDIC, ESLIC, NCUA) must require flood insurance for loans made on structures in FEMA-identified flood hazard areas in communities where flood insurance is available. The lender is required to give notice to the borrower 10 days in advance that the property securing the loan is located in a flood hazard area, and written acknowledgement of the borrower’s knowledge of the flood hazard must be obtained. If flood insurance is not available in the community, the lender may still make the loan, but he or she must notify the borrower that federal disaster assistance may not be available in the event of a flood disaster. Federally insured loans (SBA, VA and FHA) have the same requirements, with the exception that they cannot be made on property located in a FEMA identified flood hazard area if flood insurance is not available in the community.

An area of great concern is whether flood hazard boundaries should be based on current development in the drainage watershed or on future development. FEMA uses current development as its criteria. The District uses future development, which results in the regulation of a larger floodplain area in most instances. Although the watershed may take time to develop in accordance with the local government’s Master Land Use Plan and land use requirements may call for on-site upstream detention, it is the District’s position that “future condition” criterion is preferable because existing floodplain users are put on notice of what the future may bring, and potential users of the floodplain are also put on notice of the potential hazard. The net result is a more restrictive regulation under 32-11-218 (l)(f) C.R.S.

5.3 Flood Warning Systems and Notification

The District has adopted a procedure to notify known occupants of identified flood hazard areas (100-year floodplains). Although larger floods can and do occur, the local governments in Colorado are directed by the legislature to identify the areas that would be affected by 100-year storms. The CWCB has been directed by the legislature to coordinate this land use program.

The District's "Flood Hazard Information Official Notice" also suggests actions that individuals can take to help themselves mitigate the hazard. This notice is mailed annually to the occupants of all residential units identified as being in the flood hazard area.

With the use of radar and a communications network, the District has put in place a system to help inform all residents of the District of potential flooding.

6.0 SPECIAL MATTERS

6.1 Irrigation Ditches

In situations in which an irrigation ditch intersects a drainage basin, the irrigation ditch does not have to take underground waters diverted by a tile drain. However, the surface drainage must be accepted if the irrigation ditch is constructed in such a way that surface water would naturally flow into it. Clark v. Beauprez, 151 Colo. 119, 377 P.2d 105 (1962) (between private parties, the owner of an irrigation ditch can prevent an upstream landowner from diverting waters from their natural course into the irrigation ditch); City of Boulder v. Boulder and White Rock Ditch & Reservoir Company, 73 Colo. 426, 216 P. 553 (1923) (where an irrigation ditch was constructed in a natural drainageway into which surface water would naturally flow, the ditch owners could not complain merely on the ground that the city, in building storm sewers, collected the surface water and accelerated its flow and precipitated or discharged it at some particular point in the line of the ditch instead of spreading it out at different places of entrance).

In urbanizing areas, the conflict between the natural flow of surface water and irrigation ditches which bisect many drainage basins continues to be a difficult condition to resolve, taking into consideration the rights and liabilities of upstream property owners and irrigation ditch owners. Innumerable natural drainageways have been blocked by irrigation ditches, although they were constructed long before the basin became urbanized. This special area of urban drainage points to the need for good land use requirements, as well as identification of potential problem areas.

7-42-108 C.R.S. provides in part that:

> Every ditch corporation organized under the provisions of law shall be required to keep its ditch in good condition so that the water shall not be allowed to escape from the same to the injury of any mining claim, road, ditch, or other property.

This provision of Colorado law was recently interpreted in the case of Oliver v. Amity Mut. Irrigation Co., 994 P.2d 495 (Colo. App. 1999). In this case, the ditch company was being sued for damages to property resulting from a break in the bank of the ditch company's ditch. The court held that the statute imposed a duty of ordinary care, such as a person of average prudence and intelligence would use, under like circumstances to protect his or her own property. The court went on to state that, in order for the ditch company to fulfill its statutory duty, it had to prevent erosion of the ditch bank, keep the ditch free of sediment and debris, and control the amount of water flowing through its ditch, among other things, keeping the spillway at the intersection of its ditch and another free of obstructions. Finally, the court concluded that, although a ditch company is not liable for damages caused solely by an act of God, the company may not escape liability if its negligence contributed to or cooperated with an act of God to cause the damage.

In conclusion, those that own ditches owe a duty to those property owners through which their ditches pass to maintain their ditches, using ordinary care so as to prevent damage to the adjoining real property.

6.2 Dams and Detention Facilities

Subdivision regulations adopted by the board of county commissioners must include provisions requiring subdivisions to submit:

> Maps and plans for facilities to prevent storm waters in excess of historic runoff, caused by the proposed subdivision, from entering, damaging, or being carried by conduits, water supply ditches and appurtenant structures, and other storm drainage facilities.
>
> 30-28-133 (3)(c)(VIII) C.R.S.

In addition, the regulations must include provisions governing:

> Standards and technical procedures applicable to storm drainage plans and related designs, in order to ensure proper drainageways, which may require, in the opinion of the board of county commissioners, detention facilities which may be dedicated to the county or the public, as are deemed necessary to control as nearly as possible, storm waters generated exclusively within a subdivision from a one-hundred year storm which are in excess of the historic runoff volume of storm water from the same land area in its undeveloped and unimproved condition.
>
> 30-28-133 (4)(b) C.R.S. See Shoptaugh v. Board of County Commissioners, 543 P.2d 524 (Colo. App. 1975).

The law in regard to liability for damages caused by failure of a dam or detention facility has recently changed. In the case of Kane v. Town of Estes Park*,* 786 P.2d 412 (Colo. 1990), the Colorado Supreme Court considered the issue of whether the Town of Estes Park was negligent for the failure of its dam and reservoir, which was the result of the failure of an upstream dam. The court held that "To impose a burden on a downstream builder to construct facilities adequate to hold or bypass the entire capacity of an upstream reservoir has the potential for foreclosing construction of beneficial downstream storage facilities because of prohibitive costs." The court then concluded as follows:

> In summary, we hold that public entities that own dams or reservoirs are not subject to strict liability for damages caused by water escaping from their dams or reservoirs. Furthermore, we hold that Estes Park had no duty to ensure that waters released from an upstream reservoir because of a dam failure of this magnitude would be contained by its facilities or would bypass those facilities without augmentation.

The Colorado legislature, in response to the 1982 flood that then resulted in the above-referenced lawsuit, amended the statute in regard to storage reservoirs to clarify the law. The applicable sections of 37-87-104 C.R.S. read as follows:

> (1) Any provision of law to the contrary notwithstanding, no entity or person who owns,

controls, or operates a water storage reservoir shall be liable for any personal injury or property damage resulting from water escaping from that reservoir by overflow or as a result of the failure or partial failure of the structure or structures forming that reservoir unless such failure or partial failure has been proximately caused by the negligence of that entity or person. No entity or person shall be required to pay punitive or exemplary damages for such negligence in excess of that provided by law. Any previous rule or law imposing absolute or strict liability on such an entity or person is hereby repealed.

(2) No such entity or person shall be liable for allowing the inflow to such reservoir to pass through it into the natural stream below such reservoir.

The law therefore is relatively clear now in regard to the ownership of dams and reservoirs and the owner's liability for them. No longer are dam owners subject to strict liability for damages caused by those dams. Meaning, that now in order to hold a dam owner responsible for damage caused by the dam, it must be established that the dam owner was negligent in maintenance or operation of the dam. However, this test of negligence is further limited by the law's permission to dam owners to pass all inflows through the dam.

The court, in the case of Barr v. Game, Fish and Parks Commission, 497 P.2d 340 (Colo. App. 1972), held that the criteria for the construction of a dam is to safely pass the probable maximum precipitation (PMP). In Barr, the Colorado Court of Appeals found that, since modern meteorological techniques provide a method of predicting the probable maximum storm and flood, liability should be imposed for injuries resulting from a failure to determine the probable maximum flood and to design and construct a dam with a spillway having the capacity to handle that storm. The court stated:

> The maximum probable storm, by definition, is both maximum and probable. It can and may occur…Thus being both predictable and foreseeable to the defendant in the design and construction of the dam, the defense of act of God is not available to them.

However, the Colorado State Engineer, pursuant to 37-87-105 (1) and (3) C.R.S. must approve plans and specifications for the alteration, modification, repair, or enlargement of a jurisdictional reservoir or dam and, pursuant to regulation, may impose less stringent requirements than those dictated by consideration of the PMP. In fact, the Colorado State Engineer has issued *Rules and Regulations for Dam Safety and Dam Construction*, 2 CCR 402-1 (September 1988) wherein at Rule 4 dams are classified based upon an evaluation of the consequences of the failure of the dam absent flooding conditions. Based upon that classification, Rule 5 sets forth the inflow design flood to be used in determining the spillway capacity of that dam.

A question arises, however, regarding the proper criteria to use in determining the size of the floodplain or channel below the dam: the 100-year flood, before the dam was constructed or after construction? This special area has not been resolved by either the legislature or the courts in Colorado. However, since some dams and reservoirs are required by law to safely pass the PMP (storms greater than the 100-year storm) it might be argued that the watercourse below the dam should be constructed to at least carry the same water as before construction of the dam. Assuming the dam safely passes a 500-year flood, for example, the 100-year floodplain would obviously be inadequate. But with no dam in place, the same floodplain would also be inadequate.

Preserving the 100-year floodplain before the dam was constructed will prevent damage below the newly constructed dam in the larger than 100-year storm, although not for the PMP.

6.3 Water Quality

> Stormwater runoff is a major non-point source of water pollution. In urbanizing areas, where land-disturbing activities are numerous, stormwater washes soil and sediment into surface waters causing increased levels of turbidity and eutrophication, threatening fish and wildlife, and blocking drainage. In developed areas, runoff carries with it the pollutants from surfaces over which it runs, including, oil, litter, chemicals, nutrients and biological wastes, together with soils eroded from downstream channels of the flow.
>
> U.S. Environmental Protection Agency, *Legal and Institutional Approaches to Water Quality Management Planning and Implementation*. VI-I (1977).

It is reasoned that water quality control should be an integral part of any drainage or stormwater management program, since stormwater management techniques are often consistent with water quality objectives. However, this special area, as related to urban drainage, has not been researched adequately enough so as to provide the facts upon which a cost-effective approach could integrate water quality objectives with plans for surface drainage improvements. See City of Boulder v. Boulder and White Rock Ditch & Reservoir Company, 73 Colo. 426, 216 P. 553, 555 (1923).

Currently, some counties and municipalities are under regulation through the U.S. Environmental Protection Agency and the State of Colorado to address water quality issues. Other portions of this *Manual* deal in detail with those requirements.

6.4 Professional Responsibility

The Colorado Rules of Professional Conduct of the State Board of Registration for Professional Engineers and Professional Land Surveyors provides in the *Basis and Purpose* section the following:

> In order to safeguard life, health and property, to promote the public welfare, and to establish and maintain a high standard of integrity and practice, the following Rules of Professional Conduct shall be binding on every person holding a certificate of registration and on all partnerships or corporations or other legal entities authorized to offer or perform engineering or land surveying services in Colorado.

These Rules were authorized by Colorado statute and in 12-25-108 (1) C.R.S.

The board has the power to deny, suspend, revoke, or refuse to renew the license and certificate of registration of, limit the scope of practice of, or place on probation, any professional engineer or engineer-intern who is found guilty of:..(e) Violating, or aiding or abetting in the violation of,...any rule or regulation adopted by the board in conformance with the provisions of this part 1,...Rule I—*Registrants shall hold paramount the safety, health and welfare of the public in the performance of their professional duties.*

2. Rule I shall include, but not be limited to, the following:

   A. Registrants shall at all times recognize that their primary obligation is to protect the safety, health, property and welfare of the public. If their professional judgment is overruled under circumstances where the safety, health, property or welfare of the public are endangered, they shall notify their employer or client and/or such other authority as may be appropriate.

Based upon the law and rule set forth above, a professional engineer is required not only to serve the interests of his or her employer/client but is also required as a primary obligation to protect the safety, health, property, and welfare of the public. Therefore, this obligation of protection is superior to the obligation to an employer/client and therefore must be considered in all professional decisions made by a professional engineer.

7.0 CONCLUSION

> The force of gravity which causes all waters flowing on the earth to seek the lowest level creates natural drainage, and provides for the distribution of all water, whether surface or otherwise. This natural drainage is necessary to render the land fit for the use of man. The streams are the great natural sewers through which the surface water escapes to the sea, and the depressions in the land are the drains leading to the streams. These natural drains are ordained by nature to be used, and so long as they are used without exceeding their natural capacity the owner of land through which they run cannot complain that the water is made to flow in them faster than it does in a state of nature.
>
> 2 Farnham, *Water and Water Rights*, p. 968.

Drainage is both simple and complicated. If the facts are ascertained and a plan is developed before initiating a proposed improvement, the likelihood of an injury to a landowner is remote, and the municipality or developer should be able to undertake such improvements relatively assured of no legal complications and be able to use several different means of financing the improvement.

A legal opinion on proposed drainage improvements should state as a minimum whether:

1. The watercourse under study has been walked.
2. There are problems involved, and what causes them (obstructions, topography, development, present or future).
3. The proposed improvements to make the situation better.
4. The proposal requires that the natural drainage be modified.
5. There is potential liability for doing something versus doing nothing.
6. Someone will benefit from the proposed improvements.
7. In general, what is proposed is “reasonable,” using the criteria set forth in paragraph 2.1.3.

PLANNING

CONTENTS

Photographs

1.0 THE DRAINAGE SUBSYSTEM

1.1 Planning

Planning of the urban storm runoff system is a very important step that requires a comprehensive understanding of city planning, drainage planning, and many of the social, technical, and environmental issues embedded in each watershed.

Urban storm runoff is a subsystem of the total urban system. It is an integral part of the urban community and should be planned as such. The drainage engineer must be included in all urban planning from the beginning. When drainage planning is done after all the other decisions are already made as to the layout of a new subdivision or commercial area or of the transportation network, drainage and urban space allocation problems often result that are costly and difficult to correct.

The city or county urban design team should think in terms of natural drainage easements and street drainage patterns and should coordinate efforts with the drainage engineers to achieve the policies and objectives presented in this *Manual*. Storm runoff will occur when rain falls or snow melts no matter how well or how poorly drainage planning is done. Drainage and flood control measures are costly when not properly planned. Good planning results in lower-cost drainage facilities for the developer and the community and a more functional community infrastructure (Jones 1967).

The drainage design team is encouraged to consider ways of creating additional benefits from drainage works such as recreation or open space.

Photograph PL-1—Bible Park with fully integrated drainage, flood control, recreation, and open space functions represents a partnership among engineers, landscape architects, planners and recreation professionals.

Consideration of multiple uses and multiple benefits in drainage planning and engineering can reduce drainage costs and increase benefits to the urban system. One way to ensure maximum consideration of these multiple uses is by preparing master plans for drainage so that the overall effort is coordinated with other predetermined objectives (ASCE and WEF 1992).

During the master planning phase, major decisions are made as to design velocities, location of structures, open space set-asides for drainage, integration with recreation, means of accommodating conflicting utilities, and potential alternate uses for open channels, detention, and water quality facilities. It is also at this time that decisions need be made on the use of downstream detention storage, either off-stream or channel ponds or reservoirs. Upstream storage and land treatment should also be evaluated.

1.2 Planning Philosophy

The planning of urban drainage should proceed on a well-organized basis with a defined set of drainage policies backed up with suitable ordinances. The policies presented in this *Manual* provide a basis upon which additional localized and specific policies can be built.

Planning of urban drainage facilities should be based upon incorporating natural waterways, artificial channels, storm sewers, and other drainage works into the development of a desirable, aesthetic, and environmentally sensitive urban community, rather than attempting to superimpose drainage works on a development after it is laid out, as is often done with water supply and sanitary sewer facilities. Surface drainage, unlike water and sanitation systems, must be integrated early into the fabric of the urban layout.

Urban drainage should be considered on the basis that two separate and distinct drainage systems exist. These are the initial drainage system and the major drainage system.

The *initial system,* as defined in the POLICY chapter, consists of grass and paved swales, streets and gutters, storm sewers, and smaller open channels. This is the system that, if properly planned and designed, will eliminate many "complaint" calls to the city or county. It provides for convenient drainage, reduces costs of streets, and directly affects the orderliness of an urban area.

A well-planned *major system* can reduce or eliminate the need for underground storm sewers, and it can protect the urban area from extensive property damage, injury, and loss of life from flooding. The major system exists in a community whether or not it has been planned and designed and whether or not development is situated wisely in respect to it. Water will obey the law of gravity and flow downhill to seek its lowest level whether or not buildings and people are in its way.

The planning process can best serve the community by making sure that nature’s prescriptive easements are maintained along major drainage routes. Here, floodplain delineation and zoning are tools that should be used freely. Small waterways and gulches lend themselves to floodplain regulations in the same manner as larger creeks.

Reshaping channel areas along small waterways is often not required, except to provide grade control, protection of certain vulnerable areas (such as the channel toe and outer banks), or unless they are in a degraded or deteriorated condition. The practice of straightening, narrowing, and filling major drainageways such as gulches, dry streams, and other natural channels is not recommended for general use in drainageway master plans.

The urban stormwater planning process should attempt to make drainage, which is often a resource out of place, a "resource in place" which can contribute to the community's general well being.

1.3 Drainage Management Measures

Urban drainage and flood control planning should consider the following management measures:

1. Appropriate measures to limit development of land that is exposed to flood damage including:
   a. Enacting floodplain management or other restrictive ordinances (i.e., building, subdivision, housing and health codes).
   b. Acquiring developed property in built-up areas.
   c. Preempting development of vacant flood fringe areas by public acquisition of land where appropriate for good drainage and community planning.
2. Appropriate measures to guide proposed development away from locations exposed to flood damage including:
   a. Developing floodplain regulations.
   b. Using warning signs.
   c. Limiting access to flood-prone areas.
   d. Using setbacks from channel banks.
   e. Withholding public financing from flood area development.
   f. Withholding utilities (electricity, water, sewers, etc.) from flood area development.
   g. Examining equivalent alternative sites.
   h. Maintaining low property assessment for tax purposes allowing flood-prone land to economically lie idle.
   i. Providing incentives for floodplain dedication to the public such as density credits.
3. Appropriate measures to assist in reducing individual losses by flooding including:
   a. Structural flood abatement devices.

b. Flood-proofing buildings.

c. Early warning systems.

d. Emergency preparedness plans (e.g., sandbagging, evacuation, etc.).

e. Ongoing maintenance of the minor and major drainage systems.

f. Disaster relief (funds and services).

g. Tax subsidies (i.e., ameliorating assessments).

Furthermore, good urban drainage planning practices and management procedures should make it possible to initiate:

1. Land use planning that recognizes flood hazards and flood damage and the value of the riparian zones that often occupy natural major drainageway routes.

2. A plan for expansion of public facilities that recognizes the implications of flood hazards for:

   a. Sewer and water extensions.

   b. Open space acquisition.

   c. Transportation.

3. Implementation measures that demonstrate an existing or proposed floodplain management program including, where appropriate:

   a. Building codes, zoning ordinances, subdivision regulations, floodplain regulations, and map regulations with flooding encroachment lines. These should be consistent with land use recommendations discussed earlier, incorporating flood-proofing requirements and reserving areas used in accordance with flood control recommendations.

   b. Participation in regional land use planning.

   c. Participation in available floodplain management services, including flood warning systems.

   d. Cooperation in flood damage data collection programs.

4. Use of major public programs that are available (e.g., urban renewal, public health, open space, code enforcement, highway programs and demonstration programs).

5. The administrative devices created to undertake and implement a floodplain management program including a commitment of personnel, financing, and other resources.

1.4 Water Quality

Drainage planning for quantity (rate and volume) should proceed hand-in-hand with planning for water quality management. Generally, in urban areas, water quantity and water quality are inseparable. There are a number of best management practices (BMPs) recommended in Volume 3 of this *Manual* for use in a newly developing area to mitigate the adverse effects of increased runoff rates and volumes and pollution, both during construction and after the occupancy permits have been issued. Another essential aspect of water quality protection is stream channel stability. Unstable channels can experience significant degradation and aggradation, both of which can damage aquatic life. Consequently, channel stability must be assured during the planning process.

Photograph PL-2—A stable channel coupled with wet detention for the outlet of a large storm sewer system provides Denver enhanced water quality in Harvard Gulch.

2.0 EARLY PLANNING ADVANTAGES

2.1 Advantages

There are many advantages to the developers, residents, and local governmental agencies when drainage planning is undertaken early. These advantages include lower-cost drainage facilities and facilities that provide integrated benefits to the community. The drainage engineer, planner, and the entire design team should work in close cooperation to achieve maximum urban benefits.

Good urban drainage planning is a complex process. Basic planning considerations that should be taken up early include planning for the major drainage system, the initial drainage system, and the environment.

2.2 New Development

When planning a new subdivision for residential purposes, various drainage concepts should be evaluated before decisions are made as to street location and block layout. It is perhaps at this point in the development process where the greatest impact can be made as to what the drainage facilities will cost and how well they will do their job. When flood hazards are involved, the planning consultant should take these hazards into consideration in land planning to avoid unnecessary complications with local planning boards and governments.

Planners, both governmental and private, are encouraged to confer and work with the drainage engineer. The earlier drainage problems are identified and planned for, the better the final resulting plan will be. Compromising on drainageways in a new development may appear to have short-term benefits, but long-term urban interests suffer as a result. Good drainage policy and practices should be uniformly and consistently applied.

2.3 Get the Facts

The importance of obtaining the facts, including technical and community-based information that affects the drainage program, cannot be overemphasized even in the early planning stages of development. With the aid of the collected facts, defining the objectives of the drainage system, as well as the problems that will be encountered in implementing the drainage plan, can be the most important step in the planning process. As the planning process progresses, the defined objectives will need to be reevaluated for affordability and practicability of implementation, sometimes requiring adjustment of the initial set of objectives.

2.4 Regulatory Considerations

One of the essential elements of early planning is to address regulatory requirements at the federal, state and local level. Drainage projects will frequently trigger the need for environmental permits related to (for example): wetlands and “Waters of the United States;” stormwater discharges; dewatering discharges;

and local water quality, wetland or other protection ordinances. A solid understanding of these and other regulatory programs is imperative, as they can significantly affect the design, construction and long-term maintenance of channels, ponds, wetlands, and other facilities.

Photograph PL-3—An engineered wetland channel can serve as a filter for low flows and yet carry the major flood event without damage.

3.0 CONSIDER DRAINAGE BENEFITS

3.1 Benefits

The planner should be cognizant of the additional benefits that can be derived from a good urban drainage plan. It is generally recognized that an urban area that has well-planned drainage facilities is usually an area that experiences orderly growth.

Some of the additional benefits that are derived from good urban drainage systems are:

1. Benefits to upstream property owners resulting from elimination of downstream constrictions and increased conveyance capacity.
2. Reduced problems to downstream property owners and receiving systems resulting from managed runoff and stable waterways.
3. Improved water quality.
4. Protection and enhancement of environmentally sensitive areas.
5. Reduced street maintenance costs.
6. Reduced street construction costs.
7. Improved traffic movement.
8. Improved public health and environment.
9. Lower-cost open space.
10. Lower-cost park areas and more recreational opportunities.
11. Development of otherwise undevelopable land.
12. Opportunities for lower building construction cost.
13. Controlled rising groundwater table after urbanization.

Professionals from other disciplines, including urban hydrologists, sociologists, economists, traffic engineers, civil engineers, public health professionals, attorneys, geographers, ecologists, landscape architects, and others can contribute to the formulation of plans for additional benefits.

4.0 MASTER PLANNING

4.1 Master Plan

A master plan is an overall plan into which the details of other specific plans are fitted, providing overall guidance for future actions and improvements for all or part of an evolving watershed. It is generally a regionally conceived plan based on examination of the total system that, with the aid of public participation, bridges a variety of perspectives and jurisdictional boundaries. It is a road map for future drainage and flood control watershed actions, irrespective of political boundaries.

A drainage master plan for an urbanizing area is helpful to both the developer and the municipality. The drainage master plan must be based on good environmental design techniques and address the goals and needs of the urban area. It should not be prepared only on the basis of drainage hydraulics and not be limited to moving stormwater runoff from one location to another.

A master plan for drainage will only be effective if it is coordinated with planning for open space, transportation, water quality, urban wildlife, and other urban considerations.

4.2 Uniformity

A uniform approach to master planning of drainage in a region brings better results than when different approaches are utilized by each planning effort, depending upon the particular planning team's past experiences and training.

Photograph PL-4—Use of uniform design standards represents a reasonable standard of care for urban flood channels.

5.0 PLANNING FOR THE FLOODPLAIN

5.1 Floodplains

Planning addresses many issues that deal with floodplains and the necessity of floodplain zoning. It is necessary to understand the nature and concept of floodplain regulation before serious floodplain management planning can proceed intelligently. The planner must also consider the national flood insurance program, set forth in the National Flood Insurance Act of 1968, as amended (NFIA 1968).

5.2 Concept of Floodplain Regulation

On any floodplain, nature possesses, by prescription, an easement for intermittent occupancy by runoff waters. Man can deny this easement only with difficulty. Encroachments upon or unwise land modifications within this easement can adversely affect upstream and downstream flooding occurrences during the inevitable periods of nature's easement occupancy.

Government has a responsibility to protect the public's health and safety. Thus, it is implicit that government may permit unwise occupancy or use of the natural easement only at the risk of incurring liability.

Urbanization typically modifies the natural hydrologic and water quality response of its drainageways. Because urbanization usually proceeds in accordance with land use rules and land development regulations prescribed by local government and with the review and approval of detailed development plans, local government in effect becomes a party to the inevitable hydrologic modifications. It follows that a community cannot disclaim liability from consequences of such development, either upon the developed area itself or downstream there from.

Floodplain regulation is the government's response to limit its liability along natural drainageways and is an exercise of its health and safety protective function. The concept of the existence of a natural easement for the storage and passage of floodwaters is fundamental to the assumption of regulatory powers in a definable flood zone. Floodplain regulation, then, must define the natural easement's boundaries and must delineate easement occupancy that will be consistent with total public interests.

5.3 Tools

Key components of floodplain planning include reduction of the exposure to floods, use of development policies, disaster preparedness, flood proofing (see the FLOOD PROOFING chapter), flood forecasting, flood modification, and modification of the impact of flooding.

6.0 PLANNING FOR MAJOR DRAINAGE

6.1 Major Drainage

The major drainage system planning is the key to good urban drainage in newly developing areas. The general lack of good, open-surface major drainage in older urban areas often requires expensive storm sewer retrofit projects.

A major conduit or channel has an impact upon an urban area, and much depends upon its proper functioning. It is usually a box culvert, a large pipe, or an open channel. As an open channel, it may be a stabilized natural waterway, a modified natural channel, or an artificial channel with grass or other lining. The character of the major drainageway often changes from reach to reach to account for neighborhood needs and general environmental requirements.

The planner and designer have great opportunities when working on major drainageways to help provide a better urban environment for all citizens. The challenges and opportunities are particularly great for those having the opportunity to plan and design works in core areas of cities.

The conceptual design of a major drainageway channel or conduit is that portion of the engineer's job that is most important and that has the greatest effect on the performance and cost of the works. Imagination and general hydraulic experience of the engineer are the most important tools in the preliminary planning and design stage.

6.2 Initial Route Considerations

A preliminary estimate of the design rate of flow is necessary to approximate the channel's or conduit's capacity and size. This estimate can be made by comparisons with other similar basins where unit rates of discharge have been computed or by computing preliminary hydrographs.

Routing of the major drainageway is usually a straightforward matter of following the natural valley thalweg (i.e., the lowest point in the drainageway, sometimes also called channel invert) and defining it on a map. In many urbanized areas, however, there is no thalweg, or the thalweg has been filled and built upon. For these cases, it is necessary to determine many factors before the route is chosen. A meeting should be held with the owner and with the appropriate government officials to explain the routes studied, the conclusions, and the choice. At the same time, the types of channels or conduits being considered should be presented and suggestions or concurrence should be obtained. A dialogue with citizen groups is encouraged where various alternates can be explained.

6.3 The Master Plan

The major drainage master plan must be true to its name to be effective in urban drainage. It must be a team consensus with thorough attention to engineering concepts and details. The completed plan must be suitable for day-to-day use by local and regional governmental administrators.

The master plan portion of the planning phase is where major decisions are made as to design velocities, location of structures, means of accommodating conflicting utilities, approaches to minimize adverse environmental impacts and the potential alternate uses in the case of an open channel, among others.

The master plan is also where decisions need be made on the use of downstream detention storage, either off-stream or channel ponds or reservoirs. Upstream storage should also be evaluated along with BMPs for both quantity and quality.

6.4 Open Channels

Open channels for use in the major drainage system have significant advantages in regard to cost, capacity, multiple uses for recreational and aesthetic purposes, environmental protection/enhancement, and potential for detention storage. Disadvantages include right-of-way needs and the need for more frequent maintenance. Careful planning and design are needed to minimize the disadvantages and to increase the benefits.

Channel instability is a well-recognized problem in urbanizing areas because of the significant increase in low flows, storm runoff flow rates and volumes, and erosion along the waterways that cause increased sediment concentrations. The volume of storm runoff, peak discharge rate, and frequency of bankfull discharges from an urban area are usually significantly larger than under historic conditions (Leopold 1994; Urbonas 1980; ASCE and WEF 1992; and WEF and ASCE 1998). A natural channel must be studied to determine what measures are needed to avoid future bottom scour and bank cutting. Structural measures can be implemented that will preserve the natural appearance, minimize cost, and assure proper channel function during large events. These include features such as grade control structures, drop structures, and bank stabilization.

In cases of a meandering channel, it may be necessary to provide a buffer zone outside of the floodway or floodplain to account for future channel movement. Likewise, where a deep, incised channel exists, a buffer zone allowance should be provided for bank sloughing and future channel modification by creating a setback line computed at a bank slope of 4(H) to 1(V) measured from the channel bank’s bottom.

The ideal channel is one shaped by nature over a long period of time. Unfortunately, urbanization changes the hydrology that has shaped the channel, which, in turn, destabilizes it. Providing for features to keep a natural channel from rapid degradation is an important part of any master plan. The benefits of a stabilized natural channel can include:

1. Lower flow velocities, resulting in longer concentration times and lower downstream peak flows.

2. Channel and adjacent floodplain storage that tends to decrease peak flows.

3. Lower maintenance needs.

4. Protection of riparian and aquatic habitat.

5. A desirable greenbelt and recreational area that adds significant social benefits.

While recognizing the need for at least some stabilization measures to address the hydrologic changes caused by urbanization, the closer an artificial channel character can be made to that of a natural channel, the greater the public acceptance.

In many areas about to be urbanized, the runoff has been so minimal that well-defined natural channels do not exist. However, subtle low areas nearly always exist that provide an excellent basis for location and construction of channels. Good land planning should reflect even these minimal drainageways to reduce development costs and minimize drainage problems. In many cases, wise utilization of natural water routes in the development of a major drainage system will eliminate the need for an underground storm sewer system.

A wide variety of channel types are available to the design team, depending on good hydraulic practice, environmental design, sociological impact, basic project requirements and other factors. However, from a practical standpoint, the basic choice to be made initially is whether or not the channel is to be a lined one for higher velocities, a wetland bottom channel, a grass-lined channel, a stabilized existing natural channel, or a bioengineered channel, all of which are discussed in the MAJOR DRAINAGE chapter.

The actual choice must be based upon a variety of multidisciplinary factors and complex considerations that include, among others:

1. Hydraulic Factors

    - Slope of thalweg
    - Right-of-way
    - Capacity needed
    - Basin sediment yield
    - Topography
    - Ability to drain adjacent lands
    - Permitting requirements

2. Structural Factors

    - Costs
    - Availability of material
    - Areas for wasting fill

3. Environmental Factors

    - Water quality
    - Neighborhood character
    - Neighborhood aesthetic requirements
    - Needs for new green and riparian areas
    - Street and traffic patterns
    - Municipal or county policies

4. Sociological Factors

    - Neighborhood social patterns
    - Neighborhood children population
    - Pedestrian traffic
    - Recreational needs

5. Regulatory Factors

    - Federal government permits, such as a Section 404 permit
    - State government permits
    - Local government permits

Prior to choosing the channel type, the designer should be sure to consult with experts in related fields in order that the channel chosen will create the greatest overall benefits. When practical, the channel should have slow flow characteristics, be wide and shallow, and be natural in its appearance and functioning.

Grass-lined channels, wetland bottom channels, and bioengineered channels with adequate structural enhancement may be the most desirable artificial channels. The channel storage, lower velocities, environmental benefits, and sociological benefits obtainable create significant advantages over other types. The design must give full consideration to aesthetics, sediment deposition, water quality, maintenance, scour, and hydraulics.

Many open waterways in the western and southern parts of the Denver region have experienced the effects of urbanization and are often steep-banked gulches that have erodible banks and bottoms. On the other hand, a number of natural waterways exist in the northern and eastern parts of the District that have milder slopes, are somewhat stable, and are not in an obvious state of degradation. However, for either type of channel, when it begins to carry storm runoff from an urbanized area, the changed runoff regime will result in new and highly active erosional tendencies. Careful hydraulic analysis of natural channels must be made to foresee and counteract these tendencies. In nearly all cases, some modification of the channel will be required to create a more stabilized condition so it can handle changes to surface runoff created by urbanization.

With most Denver area natural waterways, it is necessary to construct grade controls or drop structures at regular intervals to decrease the thalweg (channel invert) slope and control erosion. When site conditions are conducive, channels should be left in as near a natural condition as feasible, subject to the requirement of demonstrated stability during the major event. Extensive channel modifications should not be undertaken unless they are found to be necessary to avoid excessive erosion with subsequent sediment deposition downstream and water quality deterioration.

Because of the decided advantages that are available to a community by utilizing natural waterways for urban storm drainage purposes, the designer should consult with experts in related fields for the method of development. It is important to convene a design team to develop the best means for using a natural waterway. Sometimes it will be concluded that park and greenbelt areas should be incorporated into the channel works. In these cases the usual constraints of freeboard depth, curvature, and other rules applicable to artificial channels may be different or may not apply. For instance, there are significant advantages that may accrue if the designer incorporates relatively frequent (e.g., every five years) overtopping of the formal channel, thus creating localized flooding of adjacent areas that are laid out and developed for the purpose of being inundated during the major runoff peak.

Photograph PL-5—A wide-open waterway carries floodwater at modest depths while maintaining low velocities to inhibit erosion.

7.0 PLANNING FOR INITIAL DRAINAGE

7.1 Initial Drainage

Planning and design for urban storm runoff must be considered from the viewpoint of the regularly expected storm occurrence, which includes the initial storm and the major storm. The initial storm has been defined for the area served by the District to have a return frequency ranging from once in 2 years to once in 10 years. The major storm has been defined to have a return period of 100 years. The objective of major storm runoff planning and design is to reduce the potential for major damage and loss of life. The initial drainage system is necessary to reduce inconvenience, frequently recurring damages, and high street maintenance and to help create an orderly urban system with significant sociological benefits.

The initial system is sometimes termed the “convenience system,” “minor system,” “local system,” “collector system,” or “storm sewer system.”

The initial drainage system is that part of the storm drainage system frequently used for collecting, transporting, and disposing of snowmelt, miscellaneous minor flows, and storm runoff up to the capacity of the system. The capacity should be equal to the maximum rate of runoff to be expected from the initial design storm.

The initial system may include a variety of features such as swales, curbs and gutters, storm sewer pipes, open drainageways, on-site detention, “minimized directly connected impervious area” features, and water quality BMPs.

7.2 Streets

Streets serve an important and necessary drainage service, even though their primary function is for the movement of traffic. Traffic and drainage uses are compatible up to a point, beyond which drainage is, and must be, subservient to traffic needs.

Gutter flow in streets or flow in adjacent swales is necessary to transport runoff water to storm inlets and to major drainage channels. Good planning of streets can substantially help in reducing the size of, and sometimes eliminate the need for, a storm sewer system in newly urbanized areas.

Design criteria for collecting and moving runoff water on or adjacent to public streets are based on a reasonable frequency of traffic interference. That is, depending on the character of the street and as discussed in the POLICY chapter of this *Manual*, certain traffic lanes can be fully inundated during the initial design storm return period, usually once each two years. However, during this design period, lesser storms occur that will produce runoff, which will inundate traffic lanes to some smaller degree.

Drainage practices as related to streets are dependent on the type of street use and construction.

Classification of streets is based upon traffic volume, parking practices, design and construction, relationship to cross streets, and other criteria. The classification adopted for use herein includes:

- Local/residential.
- Collector.
- Arterial.
- Freeway.

Streets should be classified with respect to pedestrian traffic as well as vehicular traffic. As an example, streets that are classified as local for vehicles and located adjacent to a school are arterials for pedestrian traffic. The allowable width of gutter or swale flow and ponding should reflect this fact.

Inverted crown or “dished” streets shall not be utilized. The dished street design violates the basic function of a street: that of a safe vehicular traffic carrier.

Photograph PL-6—District drainage criteria are aimed at respecting the needs of safe, unimpeded traffic movement. This intersection represents a long-standing drainage problem needing a solution.

8.0 PLANNING FOR STORAGE

8.1 Upstream Storage

The drainage designer usually controls upstream storage utilization (sometimes called on-site detention) and land-shaping BMPs in the early stages of laying out a development. The architect, engineer, homebuilder, land developer, and governmental officials, however, all have a responsibility to work towards more upstream storage and effective land shaping. Upstream storage and land treatment, such as use of grass buffers and swales described in Volume 3 of this *Manual,* have the greatest potential for making good urban drainage less costly to the urban resident.

Many new urban area plans contain parks, both the neighborhood type and the large central type. Parks and recreational fields create little runoff of their own; however, they provide excellent detention potential for storage of runoff from adjacent areas. The use of parks for temporary detention of stormwater runoff can measurably increase benefits to the public, and the use of parks for such purposes is encouraged.

8.2 Downstream Storage

Downstream storage is defined as retention or detention storage situated in the downstream portions of the basin. Typically these are larger facilities that can include channel reservoirs, channel storage, and off-stream storage. The use of downstream storage to reduce storm runoff, and hence drainage costs, should be considered as supplementary to upstream storage. Benefits to be derived from downstream storage are significant and should be taken advantage of wherever possible.

The construction of pond embankments in the channel, generally where topography is favorable to the storage of stormwater runoff, can provide significant benefits in regard to reducing peak flows and settling sediment and debris, the latter helping to improve the quality of water downstream. Multiple benefits, including water quality, can be obtained by the use of on-stream storage ponds by planning and designing for a small permanent pool.

While upstream storage is usually the responsibility of upstream land developers, downstream storage is usually the responsibility of the local governmental unit because the water stored there is derived from a larger area representing many upstream tributary sources.

8.3 Channel Storage

The use of wide, slow-flow swales and natural-type channels also provides storage without constructing special embankments.

8.4 Other Benefits

Both upstream storage and downstream storage have significant multipurpose use potentials generally centered around recreational, water quality, aesthetic and, possibly, wildlife benefits. In regard to such multiple uses, it is necessary for the designer to work closely with the city planner and the recreational department of the local government.

Photograph PL-7—Urban stormwater detention basins can create neighborhood amenities that at the same time serve their flood control function.

9.0 PLANNING FOR STORM SEWERS

9.1 Storm Sewers

The term storm sewer system refers to the system of inlets, conduits, manholes, and other appurtenances that are designed to collect and convey storm runoff from the initial storm to a point of discharge into a major drainage outfall. Storm sewers are a portion of the initial drainage system that includes street gutters, roadside drainage ditches and swales, culverts, storm sewers, small open channels, and any other feature designed to handle runoff from the initial storm. Alternate terms for the storm sewer system are convenience or minor drainage system. These names are derived from the function of the storm sewers, which is to prevent inconvenience and frequently recurring damage caused by the more frequently occurring smaller storm events.

The initial drainage system, including storm sewers, is that portion of the total drainage system that often receives the most attention from engineers. It is what the average citizen considers to be the urban drainage system. It is what directly contributes to the orderly growth of a community by handling the storm runoff expected to occur once every two to ten years.

The initial system exists even without storm sewers. Storm sewers are needed only when the other parts of the initial system no longer have capacity for additional runoff. A good major system of drainage coupled with wise layout of streets can often significantly reduce the need for storm sewers. The more inadequate the major system is, the more costly the storm sewers are.

9.2 Function of Storm Sewers

Storm sewers belong to the initial drainage system, as do curbs and gutters, roadside swales and roadside ditches. The more distant the point of outfall for the storm sewer, the more extensive the system must be. It is for this reason that the major drainage system takes on importance in regard to the storm sewer system. Generally, the better the major system is, the shorter the storm sewers.

In older built-up urban areas, the storm sewer system may be the only existing planned drainage works. When the capacity of the storm sewers is exceeded, the excess water flows in an unplanned manner overland, often causing damage and loss. The intent of planning and designing for major drainage is to control and manage the large runoff, which exceeds the capacity of the initial system.

9.3 Layout Planning

The preliminary layout of a storm sewer system should consider urban drainage objectives, urban hydrology, and hydraulics. The preliminary layout of the system has more effect on the success and cost of the storm sewers than the final hydraulic design, preparation of the specifications, and choice of materials.

The ideal time to undertake early work on the layout of the storm sewers is prior to finalizing the street layout in a new development. Once the street layout is set, the options open to the drainage engineer to provide a more cost-effective system are greatly reduced. Various layout concepts should be developed and reviewed, and critical analyses should be done to arrive at the best layouts. For example, the longer street flow can be kept from concentrating in one street, the further the distance from the divide the storm sewer system can begin. In storm sewer design remember that small-diameter laterals represent a large part of the construction cost. Planning a storm sewer system should have as its objective the design of a balanced system in which all portions will be used to their full capacity without adversely affecting the drainage of any area.

9.4 System Sizing

The runoff or rainfall return period to be utilized for designing a storm sewer system is a choice local governments must make. Whenever the system crosses jurisdictional boundaries, differences in sizing policies for the initial system must be worked out between these jurisdictions so that a consistent design is achieved for the entire system serving two or more communities.

The suggested design return periods to be used by local jurisdictions in the Denver region for storm sewer design for all land uses is 2- to 10-years. This is a departure from the policy of recommending different return periods for different land uses. Experience has shown that it is not practical to vary storm sewer design by land use because a single system often serves multiple land uses. Instead, greater attention is necessary to ensure that the major system is adequate to protect the public and property within all areas, regardless of land use.

Once the overall design return period has been set, the system should be reviewed for points where deviation is justified or necessary. For example, it may be necessary to plan a storm sewer to receive more than the initial runoff from a sump area that has no other method of drainage. The sewer might be planned to receive only necessary initial runoff both upstream and downstream of this particular area.

An area must be reviewed on the basis of both the initial and the major storm occurrence. When an analysis implies that increasing the storm sewer capacity is necessary to help convey the major storm, the basic system layout of the major drainage should be analyzed and changed, as necessary.

9.5 Inlets

A stormwater inlet is an opening into a storm sewer system for entrance of surface storm runoff. There are four typical categories of inlets:

1. Curb opening inlets

2. Grated inlets

3. Combination inlets

4. Multiple inlets

In addition, inlets may be further classified as being on a continuous grade or in a sump. It is recommended that curb opening and combination inlets generally be utilized in the design of storm sewer systems, particularly when a sump condition exists. Although these inlets will not guarantee against plugging, they are the most dependable.

9.6 Alternate Selection

The best alternate is chosen on the basis of numerous considerations, one of which is cost. Cost, however, should not be overemphasized. The choice should be based, in part, upon the total benefit-cost ratio, taking into consideration other community benefits and needs.

Photograph PL-8—Planning for storm sewers is aimed at maintaining an orderly urban area where stormwater street flow is limited to predetermined levels.

10.0 PLANNING FOR OPEN SPACE

10.1 Greenbelts

Waterways can make excellent greenbelts and riparian zones because the needs for drainage and the needs for greenbelts and riparian zones are often compatible.

The land along natural streams and gulches has already been chosen by Mother Nature as a storm runoff easement for intermittent occupancy. Only humans, based on cost and difficulty, can deny this easement. Nature will always extract some price for use of its floodplains.

Zoning land for floodplains and limiting the potential use of such land provide ideally situated open space, greenbelts and potential riparian zones. Acquisition cost of the land for greenbelts and riparian zones should be lower because of the limited potential of the land for development without costly works and major federal regulatory constraints. In appraisal work, adjustments are made to comparable sales to make them equal to the subject property. One adjustment is typically the risk factor for flooding and whether or not the subject property is in a floodplain or a floodway.

The design team should develop the park and greenbelt needs in conjunction with the master planning of the major drainage channels and floodplain zoning. To wait means that a good opportunity may be lost.

Photograph PL-9—Open space, stable channels and recreation go hand-in-hand towards creating urban amenities.

11.0 PLANNING FOR TRANSPORTATION

11.1 Coordination Needed

The planning, design, and construction of transportation facilities including local, state, and federal highways, railroads, utilities involving conduits, and airports often involve crossing or paralleling major channels and streams. Many of the flood problems presently existing are created by inadequate waterway openings (bottlenecks) under transportation facilities. These inadequate openings have been a result of various deficiencies, including lack of appropriate basic criteria, lack of good planning, lack of proper hydraulic engineering, and lack of coordination between the various agencies involved with drainageways.

Many storm drainage problems can be avoided by special cooperation and coordination between the various governmental, state, county, local, and publicly owned agencies in the very early stages of planning for storm drainage works. This is absolutely essential if proper drainage is to be provided at the lowest reasonable cost. Proper coordination will make it possible to solve many of the inherent initial design and monetary problems connected with storm drainage.

Transportation agencies often get involved in constructing drainage works that are necessary for draining their own facilities. Planning such drainage facilities should be integrated with the total urban system and the drainage subsystem of the adjacent urban area in question. At times this will indicate that the drainage facilities constructed for a transportation facility, for instance, should intercept and convey storm runoff from a significant urban drainage basin. In design and construction of sound barriers along freeways, which in essence can act as dams across drainageways, it is possible for the highway designers to neglect the major drainage needs of the uphill land, sometimes creating flooding problems upstream of the sound barrier. A similar situation develops when a roadway embankment or a median barrier is constructed across a drainageway. These can create community costs that should be avoided. It is in these cases that cooperation with the local governmental entity is particularly advantageous so that joint planning, design, and construction can result in a better urban environment.

12.0 CLEAN WATER ACT SECTION 404 PERMITTING PROCESS

12.1 Purpose of the 404 Permit

The stated purpose of the U.S. Army Corps of Engineers (USACE) Section 404 program is to insure that the physical, biological, and chemical quality of our nation's water is protected from irresponsible and unregulated discharges of dredged or fill material that could permanently alter or destroy these valuable resources.

12.2 Activities Requiring Permit

Section 404 of the Clean Water Act requires approval from the USACE prior to discharging dredged or fill material into the waters of the United States. Typical activities within the waters of the United States (which include adjacent wetlands) requiring Section 404 permits are:

- Site development fill for residential, commercial, or recreational construction
- Construction of revetments, groins, breakwaters, levees, dams, dikes, and weirs
- Placement of riprap
- Construction of roads
- Construction of dams
- Any grading work affecting waters of the United States

12.3 Who Should Obtain a Permit

Any person, firm, or agency (including federal, state, and local government agencies) planning to work, dump, or place dredged or fill material in waters of the United States, must first obtain a permit from the USACE. Other permits, licenses, or authorizations may also be required by other federal, state, and local agencies, and the issuance of a 404 permit does not relieve the proponent from obtaining such permits, approvals, licenses, etc.

12.4 Definition of Waters of the United States

Waters of the United States include essentially all surface waters such as all navigable waters and their tributaries, all interstate waters and their tributaries, all wetlands adjacent to these waters, and all impoundments of these waters.

"Wetlands" are areas characterized by growth of wetland vegetation (e.g., bulrush, cattails, rushes, sedges, willows, pickleweed, andiodine bush) where the soil is saturated during a portion of the growing season or the surface is flooded during some part of most years. Wetlands generally include swamps, marshes, bogs, and similar areas.

12.5 Pre-Application Meetings

Pre-application meetings with the USACE and other regulatory agencies are encouraged by the USACE to facilitate the review of potentially complex or controversial projects, or projects that could have significant impacts on the human environment. Pre-application meetings can help streamline the permitting process by alerting the applicant to potentially time-consuming concerns that are likely to arise during the evaluation of their project.

13.0 REFERENCES

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RAINFALL

CONTENTS

1.0 OVERVIEW

The purpose of this chapter is to present the analytical methods used to develop the rainfall information needed in order to carry out the hydrological analyses described in the RUNOFF chapter of this *Manual*. Specifically, this chapter describes: (a) the development of point precipitation values for locations within the Urban Drainage and Flood Control District (District) (Section 2), (b) the temporal distribution of point rainfall to develop the hyetograph necessary for the Colorado Urban Hydrograph Procedure (CUHP) hydrological modeling (Section 3), and (c) preparation of intensity-duration-frequency graphs used in Rational Method hydrologic computations (Section 4). This chapter includes analysis of the 2-, 5-, 10-, 25-, 50-, and 100-year return storm events. If information is needed regarding other storm return periods or areas in Colorado but outside the District, the reader is directed to the *Precipitation-Frequency Atlas of the Western United States, Volume III-Colorado* (NOAA Atlas) published by the National Oceanic and Atmospheric Administration (NOAA) in 1973, which contains a more complete description of rainfall analysis in the State of Colorado.

The *Urban Storm Drainage Criteria Manual* that was originally published in 1969 contained rainfall depth-duration-frequency maps for the 2-, 10-, and 100-year recurrence frequencies. A detailed set of guidelines was given on how to use the depth-duration-frequency maps to develop design rainstorms and time-intensity-frequency curves for any location within the District. The NOAA Atlas published in 1973 was based on a longer period of record and a large number of gages within Colorado (NOAA 1973). Unfortunately the maps in the *Manual* and the NOAA Atlas did not agree with each other.

Since 1977 the District has studied the rainfall and runoff relationships in the Denver metropolitan area. As part of this effort, the rainfall depth-frequency distribution was investigated for a 73-year period at the Denver rain gage. Inconsistencies between the rainfall frequency distribution obtained using a long-term data record and the rainfall depth-frequency-duration maps in the *Manual* were discovered and reported (Urbonas 1978). Further investigations indicated that the NOAA Atlas maps, although not perfect, were more in line with the rainfall frequency distribution of the long-term record.

As the 1982 version of CUHP was being developed, it became apparent that the information in the NOAA Atlas could be converted to a family of design rainstorms by distributing these design storms in a manner that yielded reasonable peak runoff recurrence frequency distributions. For the above-stated reasons and to use rainfall information consistent with the information being used by the State of Colorado, it was concluded that the NOAA Atlas rainfall information should also be used within the District.

2.0 RAINFALL DEPTH-DURATION-FREQUENCY

In order to use CUHP or the Rational Method, it is necessary to find the 1-hour point rainfall for the area of interest. In order to use CUHP method for watersheds larger than 10 square miles in size, the 3-hour and 6-hour point rainfall depths are also required.

2.1 Rainfall Depth-Duration-Frequency Maps

Using the information contained in the NOAA Atlas, rainfall depth-duration-frequency maps were prepared for the Denver Region. Maps are presented for the 1-hour and 6-hour durations for the 2-, 5-, 10-, 25-, 50-, and 100-year recurrence frequencies as Figures RA-1 through RA-12.

2.2 Rainfall Depths For Durations Between 1- and 6-Hours

The 2-hour point rainfall depth can be determined using the nomograph presented in Figure RA-13 or the equation:

$$P_2 = P_1 + (P_6 - P_1)0.342 \qquad \text{(RA-1)}$$

Rainfall depths for the 3-hour storm can be determined using Figure RA-13 or the equation:

$$P_3 = P_1 + (P_6 - P_1)0.597 \qquad \text{(RA-2)}$$

in which:

P_1 = 1-hour point rainfall (inches)

P_2 = 2-hour point rainfall (inches)

P_3 = 3-hour point rainfall (inches)

P_6 = 6-hour point rainfall (inches)

In order to use Figure RA-13, the 1-hour and 6-hour point precipitation depths for any particular storm return period are determined using the maps in Figures RA-1 through RA-12. The two values are plotted on the vertical lines of Figure RA-13 labeled 1- and 6-hour duration. A straight line is drawn between the two points. The intersection with the 2-hour or the 3-hour line yields the point rainfall depth for that duration. See Section 7.1 for an example of the calculation of point rainfall depths.

3.0 DESIGN STORM DISTRIBUTION FOR CUHP

The 1-hour point precipitation value described in Section 2 is distributed into 5-minute increments for use with CUHP model (i.e., temporal distribution). This is described in Section 3.1 and summarized in Table RA-2. The rainfall duration used with CUHP varies with the size of the watershed being analyzed. Also, for watersheds 10 square miles or greater, there is an adjustment made to the incremental precipitation depths to take into account the greater watershed size (i.e., area adjustment). This is described in Section 3.2 and summarized in Table RA-3. A summary of the storm duration and whether area adjustments for different watershed sizes are needed is provided in Table RA-1.

Table RA-1—Storm Duration and Area Adjustment for CUHP Modeling

Watershed Area (square miles)	Suggested Minimum Storm Duration	Area Adjustment Required?
Less than 10.0	2 hours	No
10.0 to 20.0	3 hours	Yes
20.0 and larger	6 hours	Yes

3.1 Temporal Distribution

The current version of CUHP (see RUNOFF) was designed to be used with the NOAA 1-hour rainfall depths described in Section 2.1. The 1-hour rainfall depths for areas within the District are provided in Figures RA-1 through RA-6. To obtain a temporal distribution for a design storm for use in the Denver region, the 1-hour depth is transferred into a 2-hour design storm by multiplying the 1-hour depth(s) by the percentages for each time increment given in Table RA-2. The resultant design storm(s) may then be used with CUHP.

The total of all the incremental depths for the first hour of the design storm does not agree with the 1-hour depth used to develop the design storm. Do not be alarmed. The temporal distribution presented in Table RA-2 represents a design storm for use with a distributed rainfall-runoff routing model. The distribution is the result of a calibration process performed by the District to provide, in conjunction with the use of CUHP, runoff flow peak rates and volumes of the same return period as the design storm. The NOAA Atlas values are “embedded” in the 2-hour and other duration design storms.

In order to develop the temporal distribution for the 3-hour design storm (for watersheds between 10.0 and 20.0 square miles in size), first prepare the first two hours of the storm using the 1-hour storm point precipitation and the temporal percentage distribution shown in Table RA-2. The difference between the 3-hour point precipitation from Equation RA-2 and the 2-hour point precipitation (Table RA-2) is then distributed evenly over the period of 125 minutes to 180 minutes. In order to develop the temporal distribution for the 6-hour design storm (watersheds greater than 20.0 square miles), first prepare the first three hours of the storm as described above. The difference between the 6-hour point precipitation from

Figures RA-1 through RA-12 and the 3-hour point precipitation is distributed evenly over the period of 185 minutes to 360 minutes.

Table RA-2—Design Storm Distributions of 1-Hour NOAA Atlas Depths

Time	Percent of 1-Hour NOAA Rainfall Atlas Depth				
Minutes	2-Year	5-Year	10-Year	25- and 50-Year	100- and 500-Year
5	2.0	2.0	2.0	1.3	1.0
10	4.0	3.7	3.7	3.5	3.0
15	8.4	8.7	8.2	5.0	4.6
20	16.0	15.3	15.0	8.0	8.0
25	25.0	25.0	25.0	15.0	14.0
30	14.0	13.0	12.0	25.0	25.0
35	6.3	5.8	5.6	12.0	14.0
40	5.0	4.4	4.3	8.0	8.0
45	3.0	3.6	3.8	5.0	6.2
50	3.0	3.6	3.2	5.0	5.0
55	3.0	3.0	3.2	3.2	4.0
60	3.0	3.0	3.2	3.2	4.0
65	3.0	3.0	3.2	3.2	4.0
70	2.0	3.0	3.2	2.4	2.0
75	2.0	2.5	3.2	2.4	2.0
80	2.0	2.2	2.5	1.8	1.2
85	2.0	2.2	1.9	1.8	1.2
90	2.0	2.2	1.9	1.4	1.2
95	2.0	2.2	1.9	1.4	1.2
100	2.0	1.5	1.9	1.4	1.2
105	2.0	1.5	1.9	1.4	1.2
110	2.0	1.5	1.9	1.4	1.2
115	1.0	1.5	1.7	1.4	1.2
120	1.0	1.3	1.3	1.4	1.2
Totals	115.7	115.7	115.7	115.6	115.6

3.2 Adjustment to Rainfall Distribution for Watershed Size

The NOAA Atlas provides guidelines for adjusting the rainfall depths with increasing catchment area. Area-depth adjustments are given in the Atlas for durations of ½-, 1-, 3-, 6- and 24-hours. Figure RA-14 was based on a similar figure in the NOAA Atlas. The 15-minute curve was extrapolated by the District from the information shown for other storm durations on Figure RA-14. The fast response times of urbanized watersheds and sharp rainstorm distribution gradients in the Denver area require adjustments of rainfall depths for storm durations that are less than ½-hour.

The area adjustment procedure can be tedious and time consuming; therefore, Table RA-3 is provided to assist the engineer with the area-depth adjustment calculations. To adjust the design storm distribution to account for the averaging effects of larger watersheds, follow these three steps:

Step 1—Begin with the unadjusted design rainstorm for the needed storm duration (see Table RA-1) developed using the procedure described in Section 3.1.

Step 2—On the basis of total watershed size, select the appropriate column(s) of adjustment factors in Table RA-3.

Step 3—Multiply each incremental design storm depth by its respective adjustment factor for that time increment.

Table RA-3—Area Adjustment Factors for Design Rainfall Distributions

	2-, 5-, and 10-Year Design Rainfall Area—Square Miles				25-, 50-, 100-, and 500-Year Design Rainfall Area—Square Miles			
Time Minutes	10-20	20-30	30-50	50-75	10-20	20-30	30-50	50-75
5	1.00	1.00	1.10	1.10	1.00	1.00	1.05	1.10
10	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.10
15	1.00	1.00	1.05	1.00	1.00	1.00	1.05	1.10
20	0.90	0.81	0.74	0.62	1.00	1.00	1.05	1.00
25	0.90	0.81	0.74	0.62	0.90	0.81	0.74	0.60
30	0.90	0.81	0.74	0.62	0.90	0.81	0.74	0.60
35	1.00	1.00	1.05	1.00	0.90	0.81	0.74	0.70
40	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.00
45	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.10
50	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.10
55	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.10
60	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.10
65 - 120	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.10
125 - 180	1.00	1.15	1.20	1.40	1.00	1.15	1.20	1.40
185 - 360	N/A	1.15	1.20	1.20	N/A	1.15	1.20	1.20

See Section 7.2 for an example of the preparation of a design rainfall for use with CUHP.

4.0 INTENSITY-DURATION CURVES FOR RATIONAL METHOD

To develop depth-duration curves or intensity-duration curves for the Rational Method of runoff analysis take the 1-hour depth(s) obtained from Figures RA-1 through RA-6 and multiply by the factors in Table RA-4 to determine rainfall depth and rainfall intensity at each duration. The intensity can then be plotted as illustrated in Figure RA-15.

TABLE RA-4—Factors for Preparation of Intensity-Duration Curves

Duration (minutes)	5	10	15	30	60
Rainfall Depth at Duration (inches)	$0.29P_1$	$0.45P_1$	$0.57P_1$	$0.79P_1$	$1.0P_1$
Intensity (inches per hour)	$3.48P_1$	$2.70P_1$	$2.28P_1$	$1.58P_1$	$1.0P_1$

Alternatively, the rainfall intensity for the area within the District can be approximated by the equation:

$$I = \frac{28.5\,P_1}{(10 + T_c)^{0.786}} \qquad \text{(RA-3)}$$

in which:

I = rainfall intensity (inches per hour)

P_1 = 1-hour point rainfall depth (inches)

T_c = time of concentration (minutes)

5.0 BASIS FOR DESIGN STORM DISTRIBUTION

The orographic effects of the Rocky Mountains and the high plains near the mountains as well as the semi-arid climate influence rainfall patterns in the Denver area. Rainstorms can often have an “upslope” character where the easterly flow of moisture settles against the mountains. These types of rainstorms have durations that can exceed six hours and, although they may produce large amounts of total precipitation, they are rarely intense. Although upslope storms may cause local drainage problems or affect the flood levels of large watersheds, they are not the cause of 2- through 100-year type of flooding of small urban catchments in the Denver area.

Very intense rainfall in the Denver area results from convective storms or frontal stimulated convective storms. These types of storms often have their most intense periods that are less than one or two hours in duration. They can produce brief periods of high rainfall intensities. It is these short-duration, intense rainstorms that appear to cause most of the flooding problems in the great majority of urban catchments.

Analysis of a 73-year record of rainfall at the Denver rain gage revealed that an overwhelming majority of the intense rainstorms produced their greatest intensities in the first hour of the storm. In fact, of the 73 most intense storms analyzed, 68 had the most intense period begin and end within the first hour of the storm, and 52 had the most intense period begin and end within the first half hour of the storm. The data clearly show that the leading intensity storms predominate among the “non-upslope” type storms in the Denver region.

The recommended design storm distribution takes into account the observed “leading intensity” nature of the convective storms. In addition, the temporal distributions for the recommended design storms were designed to be used with the 1982 and later version of CUHP, the published NOAA 1-hour precipitation values (NOAA 1973) and Horton’s infiltration loss equation. They were developed to approximate the recurrence frequency of peak flows and volumes (i.e., 2- through 100-years) that were found to exist for the watersheds for which rainfall-runoff data were collected. The procedure for the development of these design storm distributions and the preliminary results were reported in literature and in District publications (Urbonas 1978; Urbonas 1979). The recommendations contained in this *Manual* are the result of refinements to the work originally reported in 1979.

6.0 SPREADSHEET DESIGN AIDS

Two spreadsheet design aids have been developed in order to facilitate computation of design rainfall.

The UD-Raincurve Spreadsheet computes the temporal distribution and area-adjusted design of rainfall for use with CUHP model. Input to the spreadsheet includes the 1-hour and 6-hour point rainfall amounts determined from Figures RA-1 through RA-12. The rainfall amount(s) should be entered into the page of the spreadsheet with the desired return period. The output is the rainfall distribution in 5-minute increments (including any required area adjustment) that may be used for CUHP modeling.

7.0 EXAMPLES

7.1 Example Computation of Point Rainfall

Find the 2-year and 100-year design storm point rainfall for Section 1, Township 1 South, Range 68 West.

Determine 1-hour and 6-hour point rainfall values from Figures RA-1, RA-6, RA-7, and RA-12.

Storm Event	Point Precipitation (Inches)	Map Reference
2-year, 1-hour	0.95	RA-1
2-year, 6-hour	1.46	RA-7
100-year, 1-hour	2.67	RA-6
100-year, 6-hour	3.67	RA-12

Determine 2-hour point precipitation values from Equation RA-1:

$$P_2 \text{ (2-year)} = \text{2-year, 2-hour} = 0.95 + (1.46 - 0.95)\ 0.342 = 1.12 \text{ inches}$$

$$P_2 \text{ (100-year)} = \text{100-year, 2-hour} = 2.67 + (3.67 - 2.67)\ 0.342 = 3.01 \text{ inches}$$

Determine 3-hour point precipitation values from Equation RA-2:

$$P_3 \text{ (2-year)} = \text{2-year, 3-hour} = 0.95 + (1.46 - 0.95)\ 0.597 = 1.25 \text{ inches}$$

$$P_3 \text{ (100-year)} = \text{100-year, 3-hour} = 2.67 + (3.67 - 2.67)\ 0.597 = 3.27 \text{ inches}$$

7.2 Example Distribution of Point Rainfall

Prepare a 100-year rainfall distribution to be used in CUHP computer model for a 15-square-mile catchment centered about Section 7, Township 4 South, Range 67 West.

As per Table RA-1, a 15.0-square-mile watershed requires a 3-hour storm with area adjustment.

Using Figures RA-6 and RA-12, the 100-year, 1-hour, and 6-hour point precipitation values are 2.60 inches and 3.50 inches respectively. The 3-hour point precipitation is calculated using Equation RA-2.

$$P_3 = 2.60 + (3.5 - 2.6)\ 0.597 = 3.14 \text{ inches}$$

Use the design storm distribution from Table RA-2 for 0 to 120 minutes. The period 125 to 180 minutes is calculated as the difference of P_3 from Equation RA-2 and P_2 from Table RA-2 evenly distributed over that time period. Area adjustment factors from Table RA-3 are applied. The results of the calculations are shown in Table RA-5.

Table RA-5—CUHP Rainfall Distribution for Example 7.2

Time (minutes)	Percentage of 1-Hour Rainfall[1]	Rainfall Without Area Adjustment (inches)[2]	Area Adjustment Factor[3]	Rainfall With Area Adjustment (inches)[4]
5	1.0%	0.026	1.0	0.026
10	3.0%	0.078	1.0	0.078
15	4.6%	0.120	1.0	0.120
20	8.0%	0.208	1.0	0.208
25	14.0%	0.364	0.9	0.328
30	25.0%	0.650	0.9	0.585
35	14.0%	0.364	0.9	0.328
40	8.0%	0.208	1.0	0.208
45	6.2%	0.161	1.0	0.161
50	5.0%	0.130	1.0	0.130
55	4.0%	0.104	1.0	0.104
60	4.0%	0.104	1.0	0.104
65	4.0%	0.104	1.0	0.104
70	2.0%	0.052	1.0	0.052
75	2.0%	0.052	1.0	0.052
80	1.2%	0.031	1.0	0.031
85	1.2%	0.031	1.0	0.031
90	1.2%	0.031	1.0	0.031
95	1.2%	0.031	1.0	0.031
100	1.2%	0.031	1.0	0.031
105	1.2%	0.031	1.0	0.031
110	1.2%	0.031	1.0	0.031
115	1.2%	0.031	1.0	0.031
120	1.2%	0.031	1.0	0.031
125-180		0.011[5]	1.0	0.011

Notes:

[1] From Table RA-2.

[2] Precipitation = 2.6 inches x Column 2.

[3] From Table RA-3.

[4] Column 3 x Column 4.

[5] (3.14 – (2.6 · 1.156))/12.

Alternatively, the 1-hour and 6-hour point precipitation values can be inserted into the spreadsheet to obtain CUHP rainfall distribution.

7.3 Example Preparation of Intensity-Duration-Frequency Curve

Prepare a rainfall intensity-duration curve for a 2.6-inch, 1-hour point precipitation.

Calculations are prepared using both Table RA-4 and Equation RA-3. They are summarized below in Table RA-6.

Table RA-6—Rainfall Intensity-Duration Values for a 2.6-inch, 1-Hour Point Precipitation

Duration (minutes)	Rainfall Intensity (inches/hour)	
	Table RA-4	**Equation RA-3**
5	$3.48 \cdot 2.6 = 9.05$	$28.5 \cdot 2.6/(10 + 5)^{0.786} = 8.82$
10	$2.70 \cdot 2.6 = 7.02$	$28.5 \cdot 2.6/(10 + 10)^{0.786} = 7.03$
15	$2.28 \cdot 2.6 = 5.93$	$28.5 \cdot 2.6/(10 + 15)^{0.786} = 5.90$
30	$1.58 \cdot 2.6 = 4.11$	$28.5 \cdot 2.6/(10 + 30)^{0.786} = 4.08$
60	$1.0 \cdot 2.6 = 2.60$	$28.5 \cdot 2.6/(10 + 60)^{0.786} = 2.63$

Using the two different methods (Table RA-4 and Equation RA-3) yields similar results. The values from Equation RA-3 are plotted in Figure RA-15.

8.0 REFERENCES

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Urbonas, B. 1979. Reliability of Design Storms In Modeling. In *Proceedings International Symposium on Urban Storm Runoff*, 27-36. Lexington, KY: University of Kentucky.

Urbonas, B. 1988. Some Findings in the Rainfall-Runoff Data Collected in the Denver Area. *Flood Hazard News* 18(1):10.

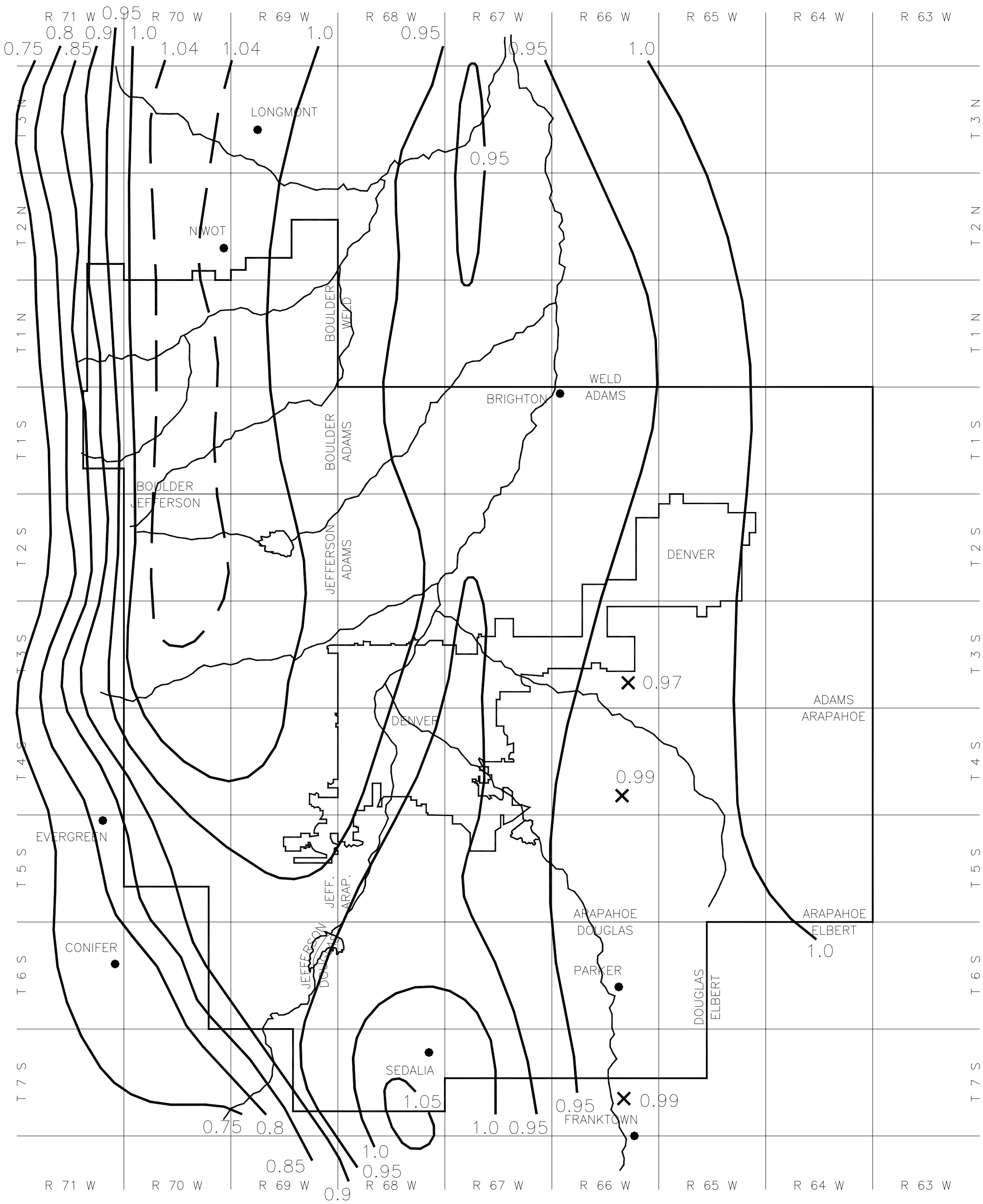


Figure RA-1—Rainfall Depth-Duration-Frequency: 2-Year, 1-Hour Rainfall

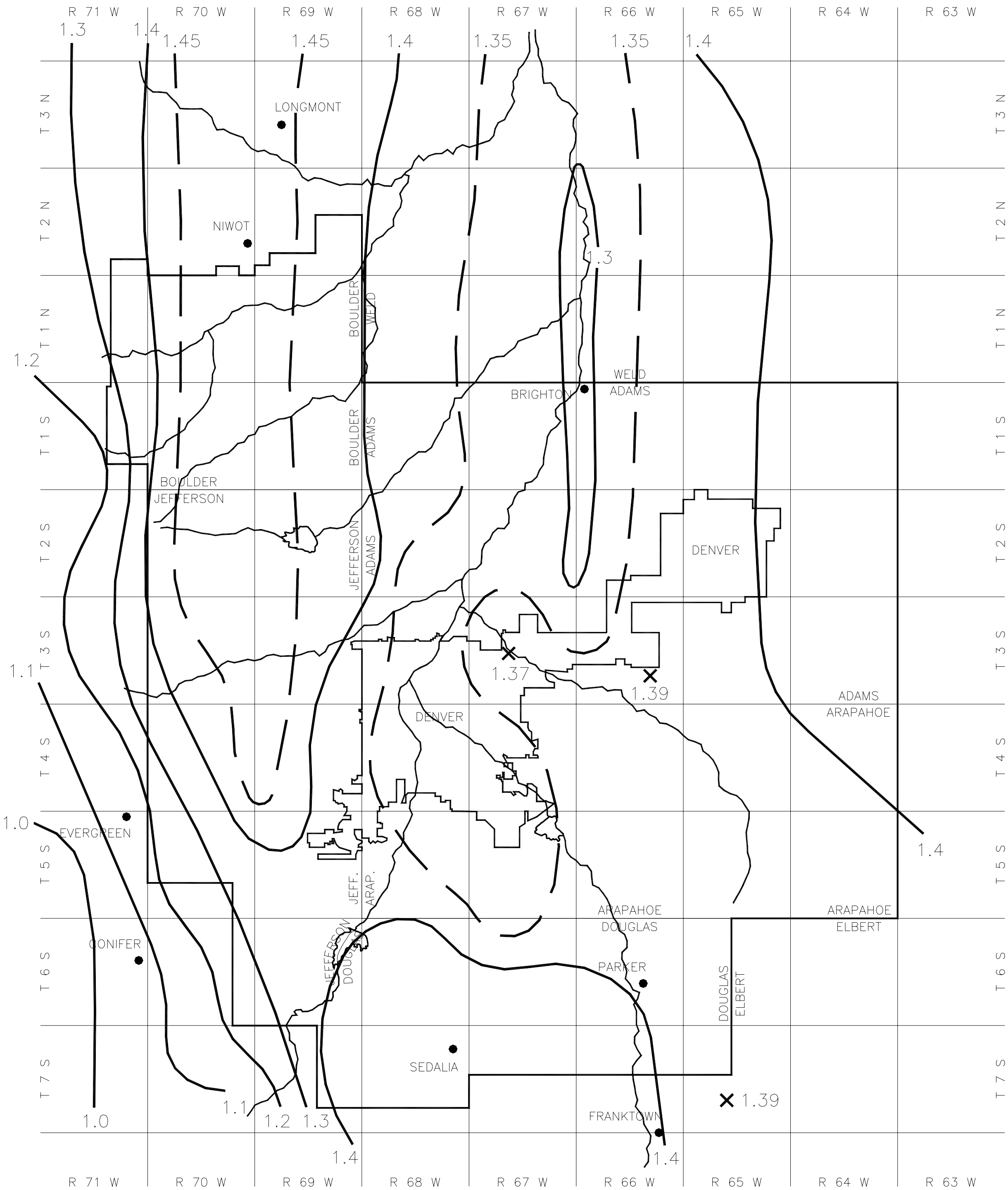


Figure RA-2—Rainfall Depth-Duration-Frequency: 5-Year, 1-Hour Rainfall

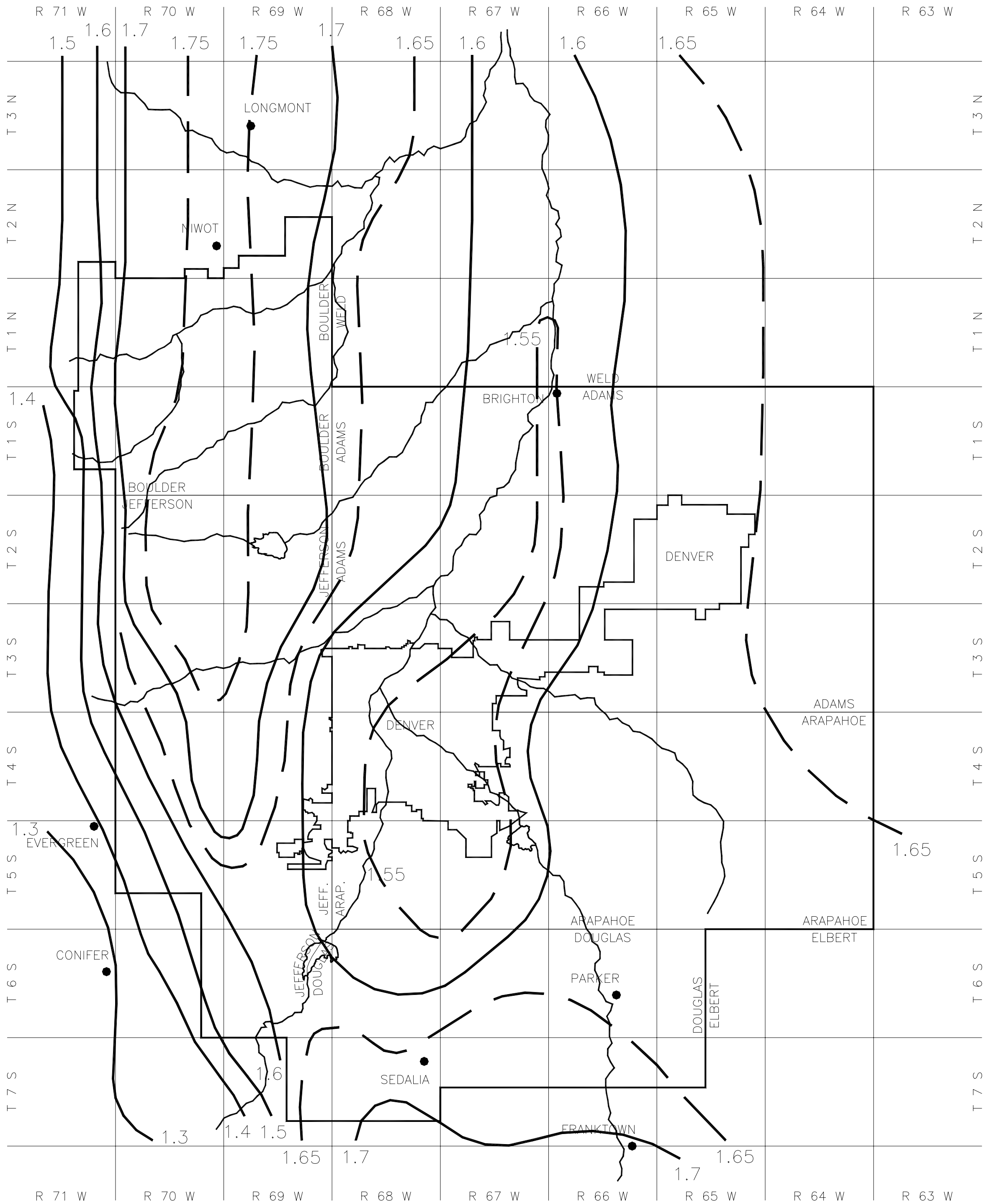


Figure RA-3—Rainfall Depth-Duration-Frequency: 10-Year, 1-Hour Rainfall

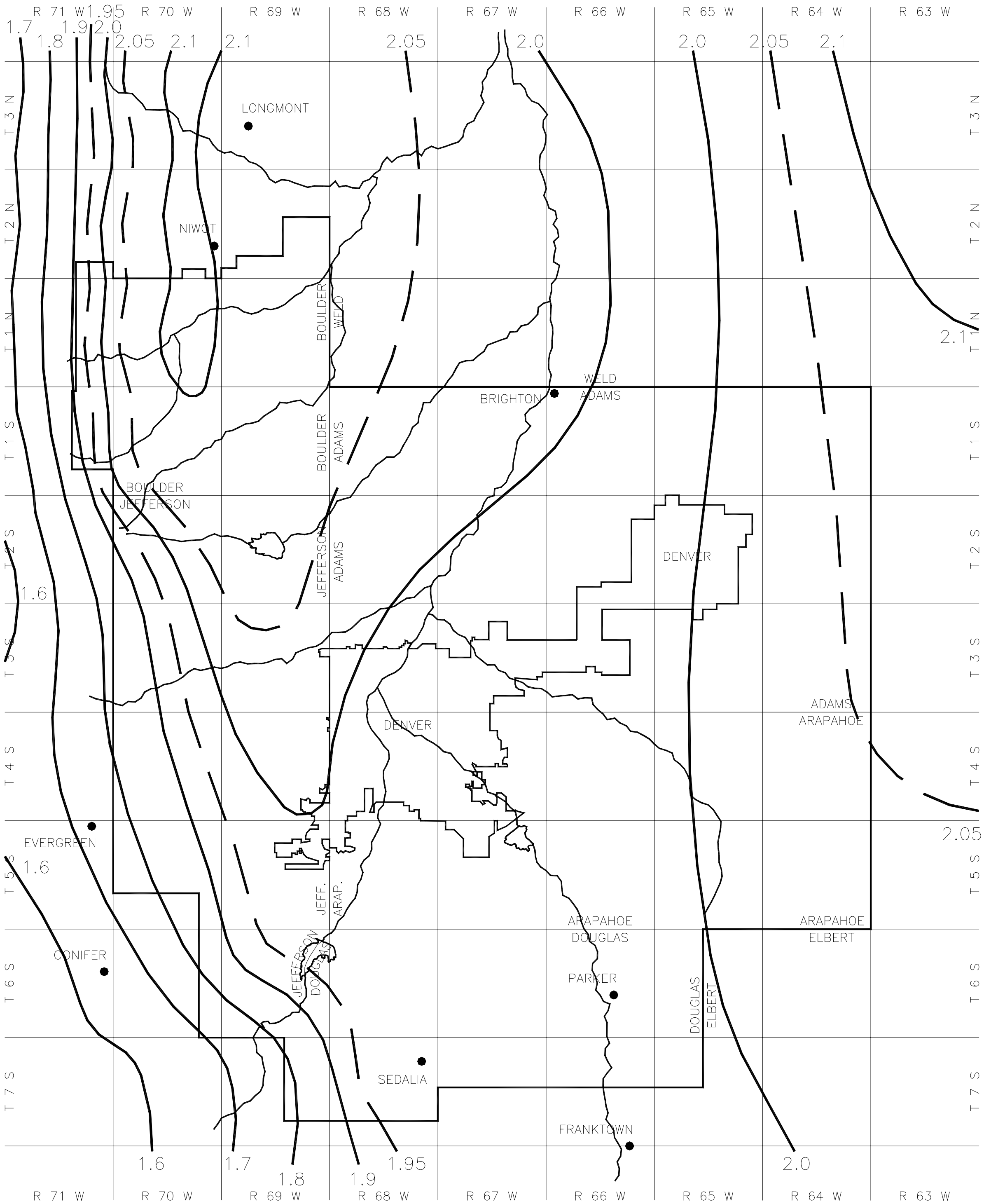


Figure RA-4—Rainfall Depth-Duration-Frequency: 25-Year, 1-Hour Rainfall

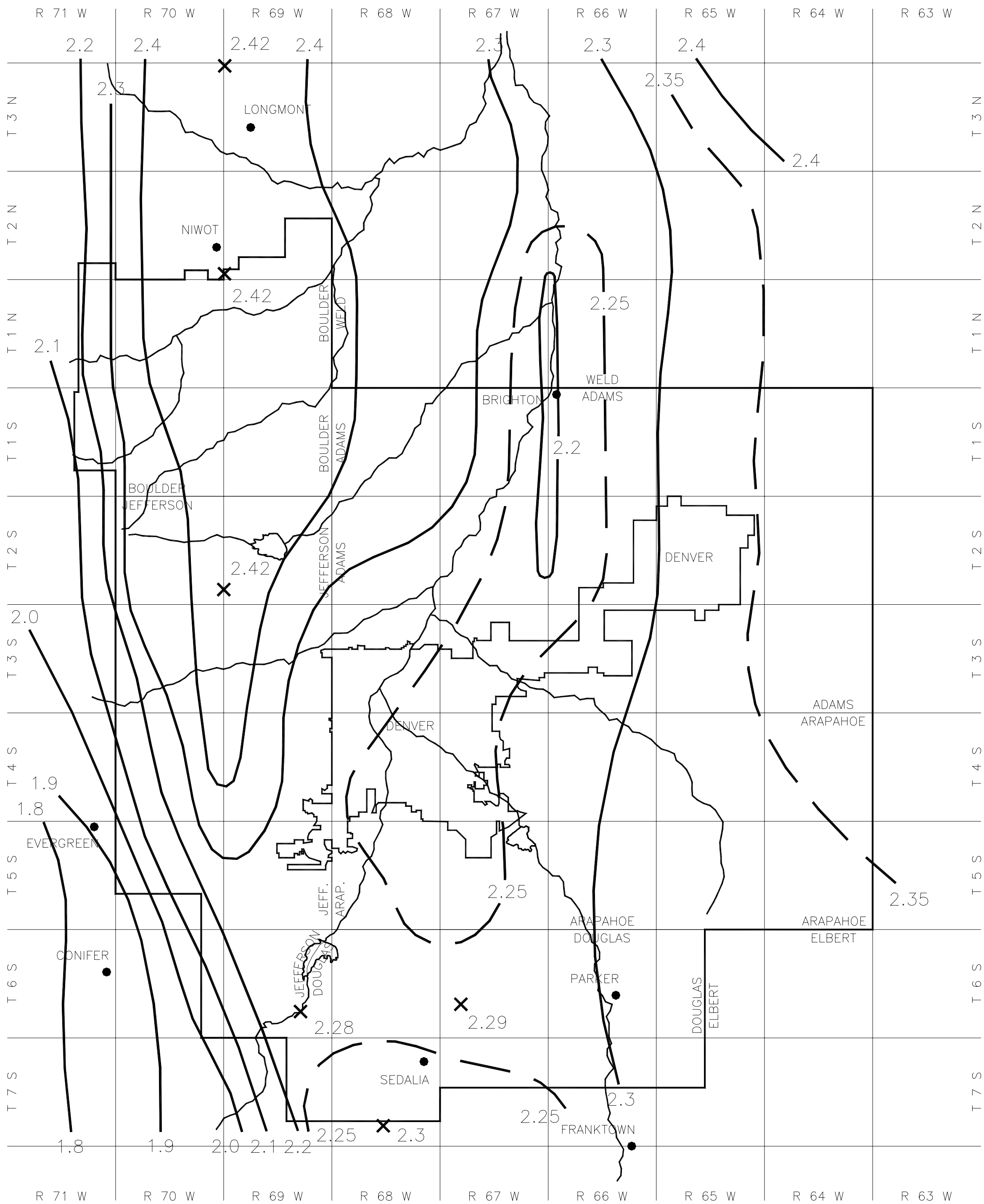


Figure RA-5—Rainfall Depth-Duration-Frequency: 50-Year, 1-Hour Rainfall

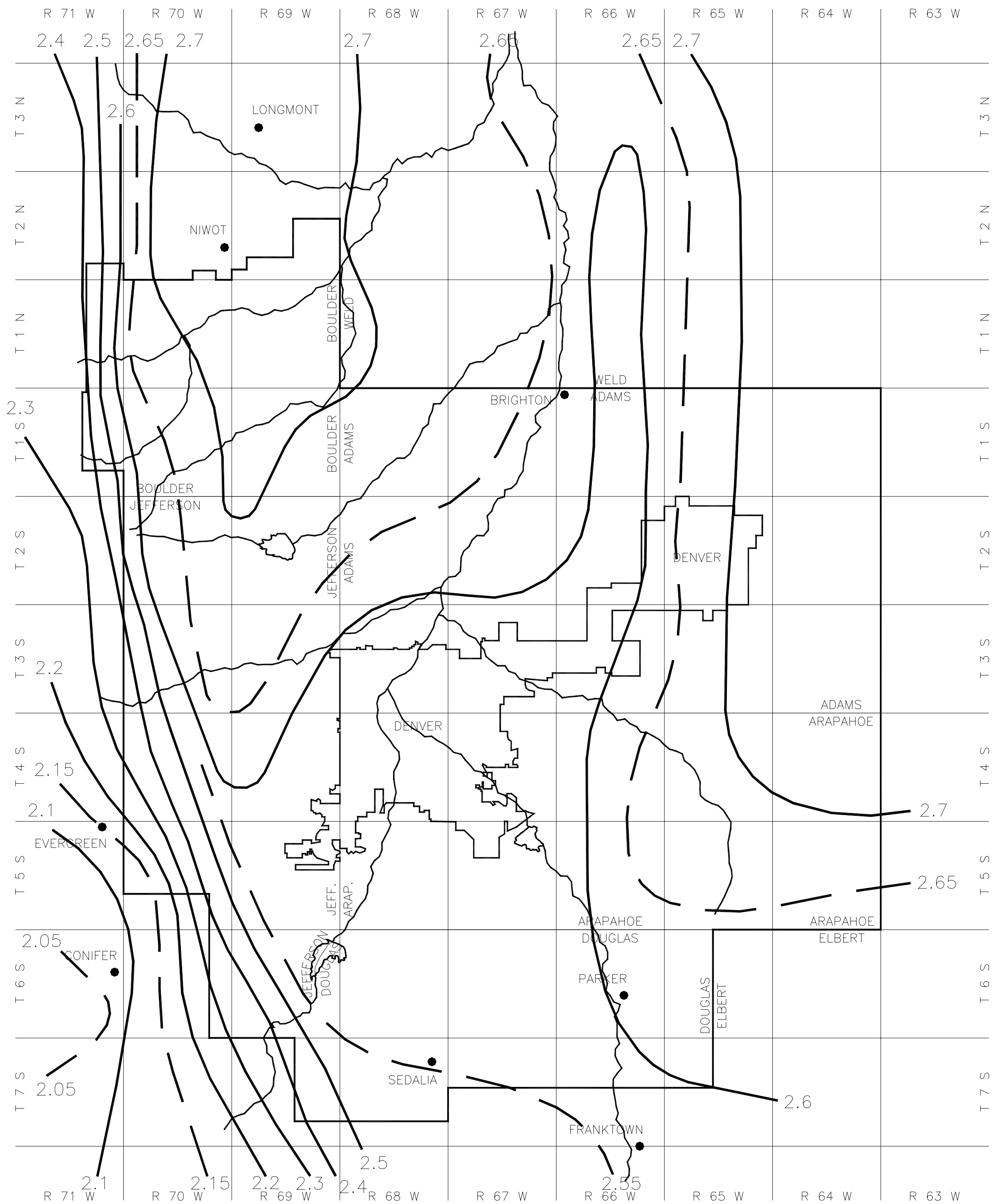


Figure RA-6—Rainfall Depth-Duration-Frequency: 100-Year, 1-Hour Rainfall

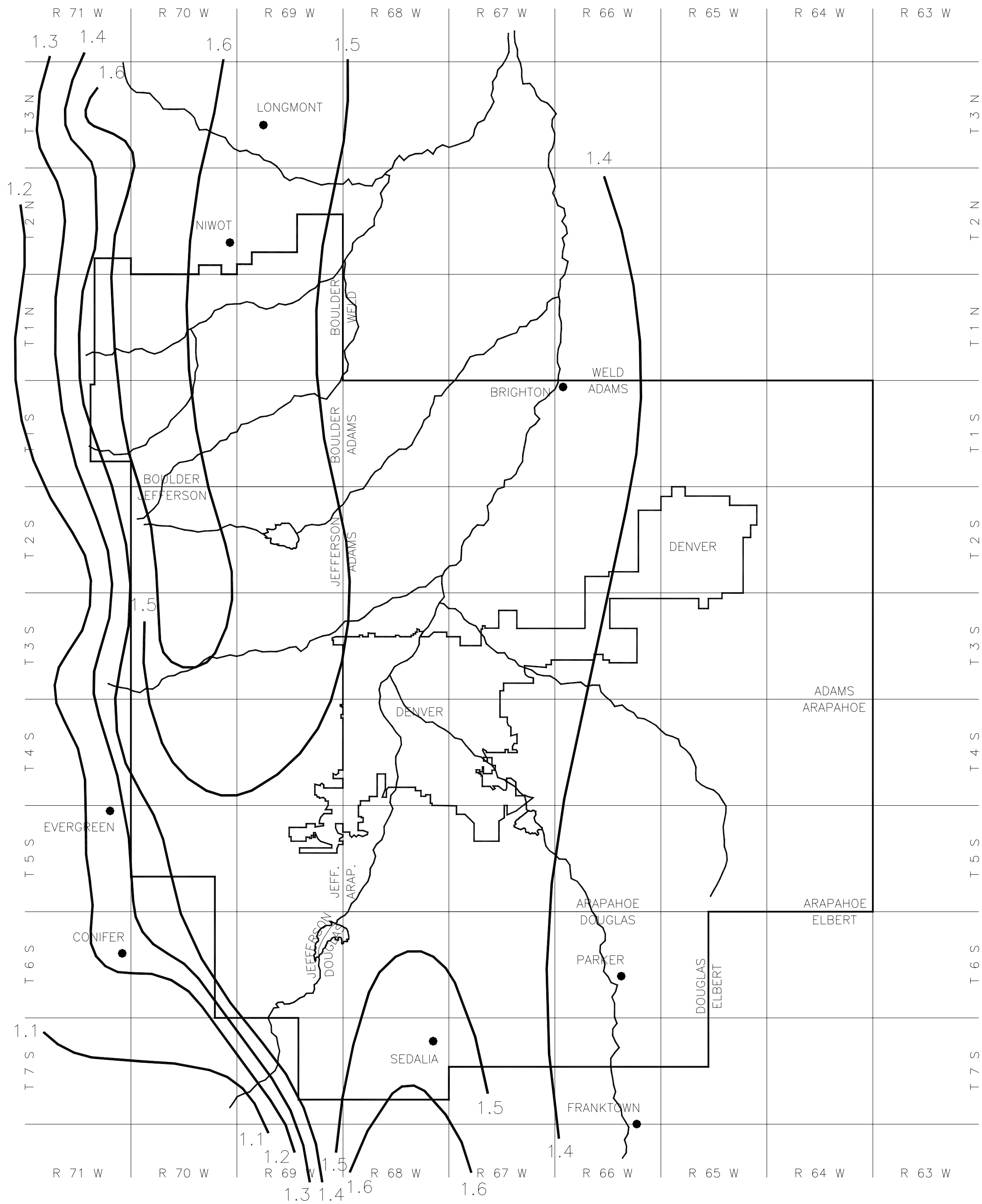


Figure RA-7—Rainfall Depth-Duration-Frequency: 2-Year, 6-Hour Rainfall

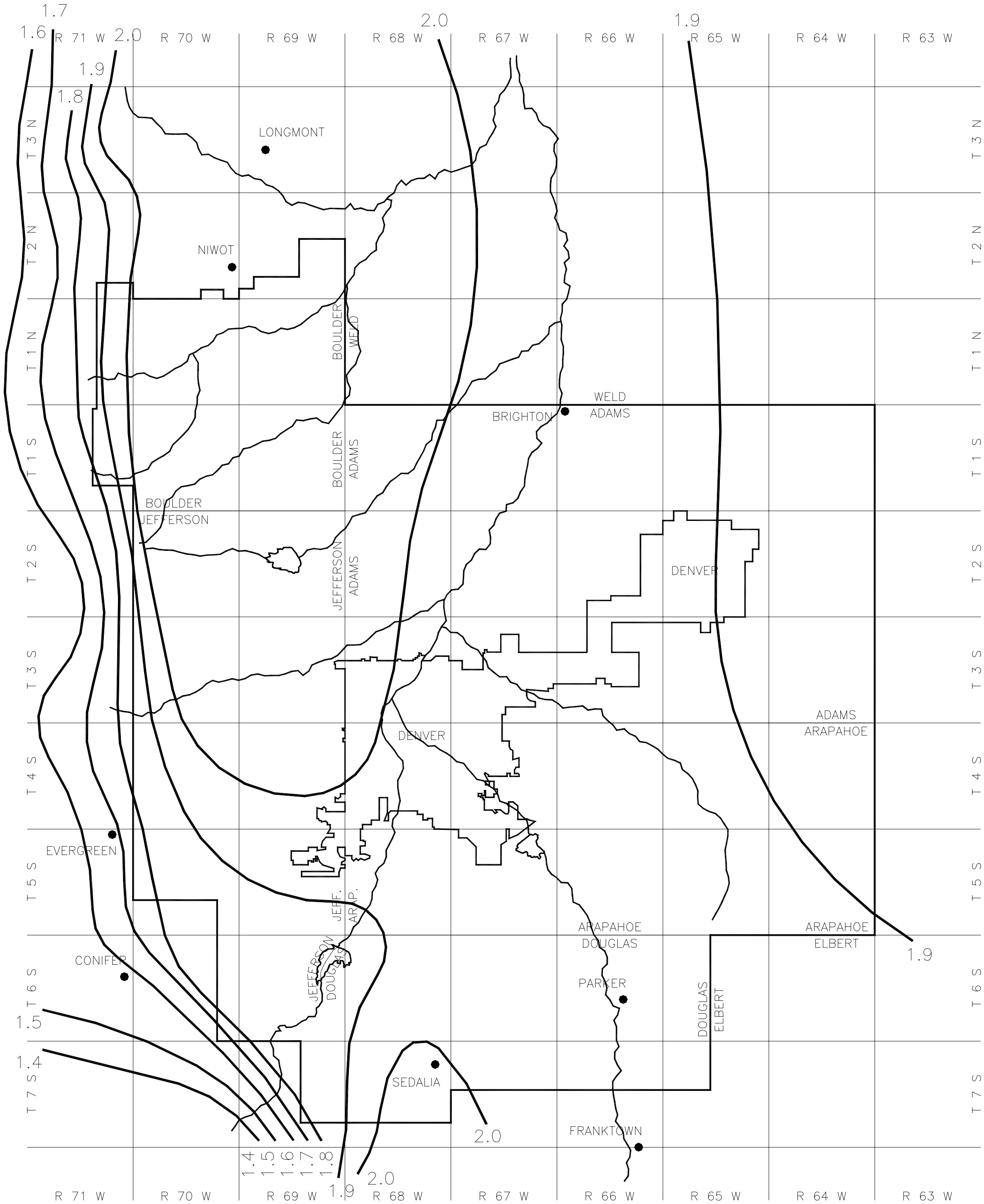


Figure RA-8—Rainfall Depth-Duration-Frequency: 5-Year, 6-Hour Rainfall

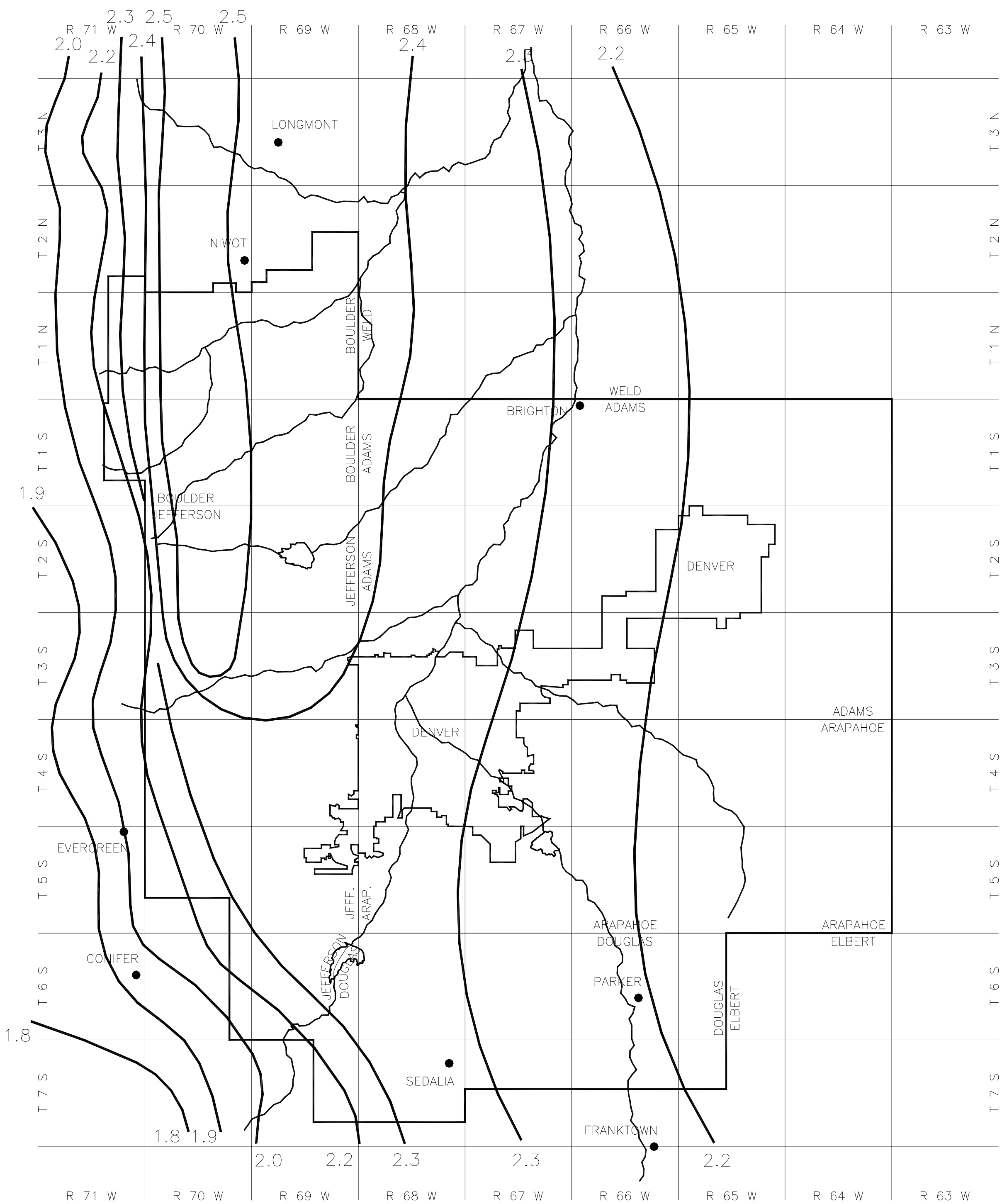


Figure RA-9—Rainfall Depth-Duration-Frequency: 10-Year, 6-Hour Rainfall

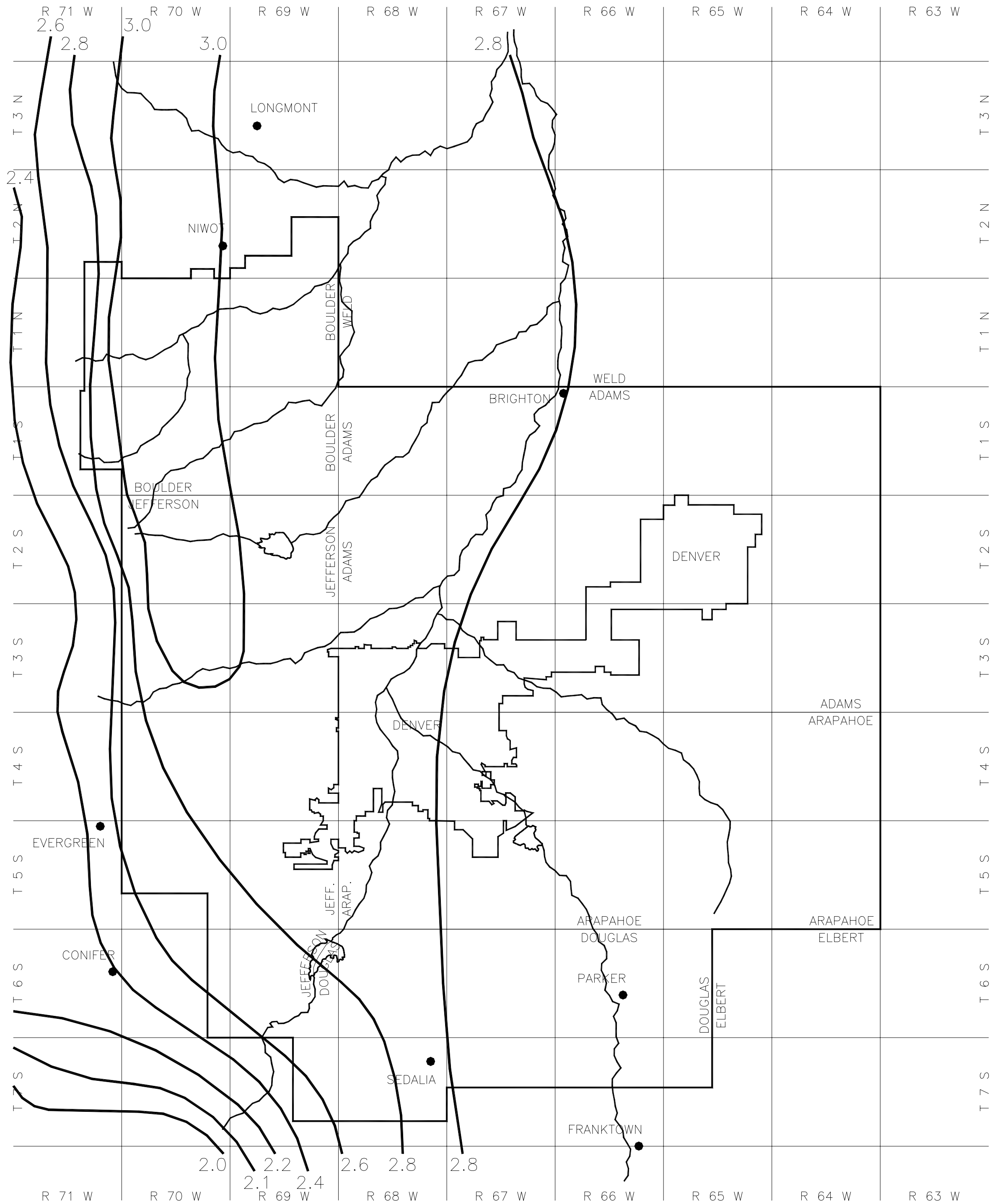


Figure RA-10—Rainfall Depth-Duration-Frequency: 25-Year, 6-Hour Rainfall

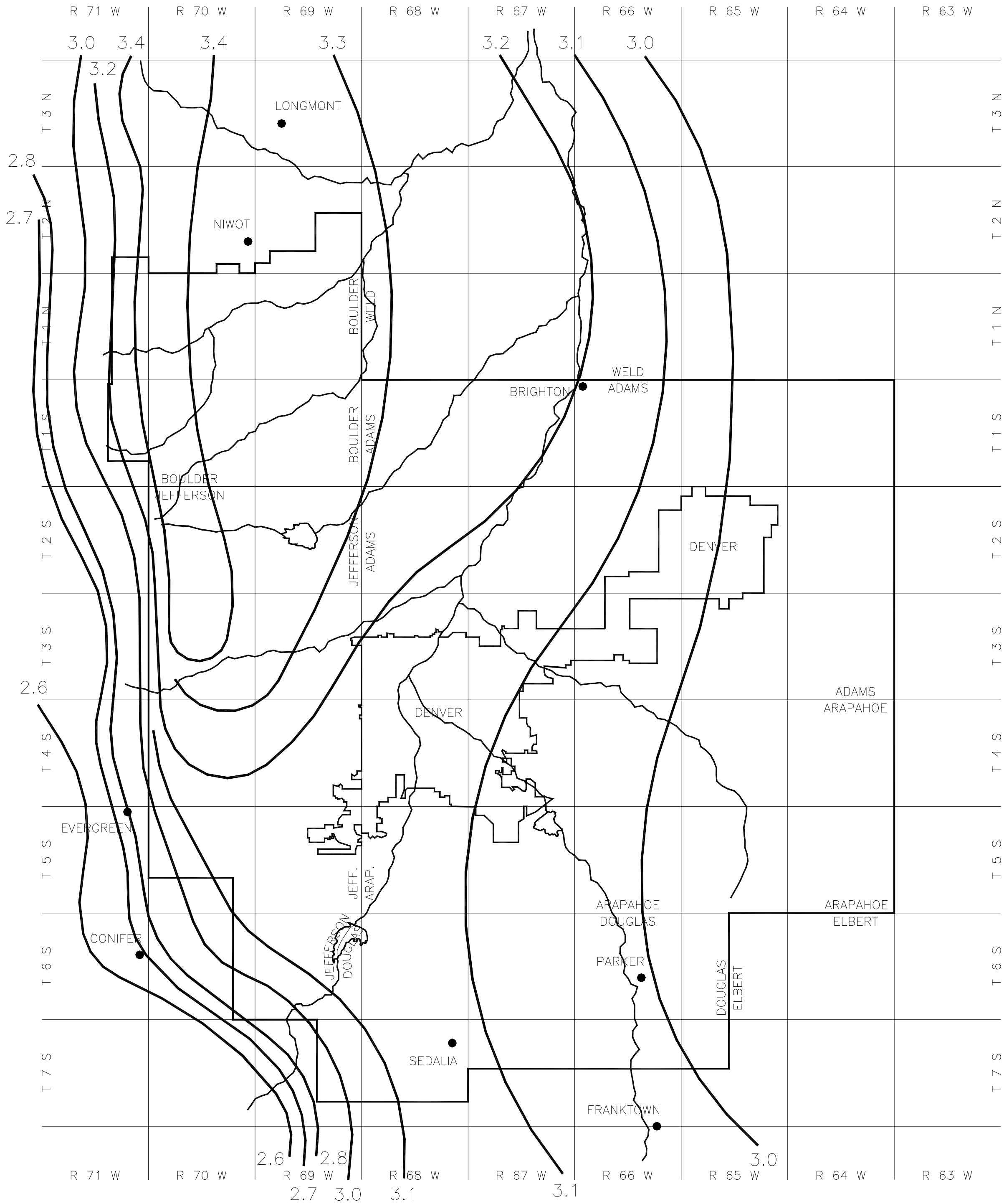


Figure RA-11—Rainfall Depth-Duration-Frequency: 50-Year, 6-Hour Rainfall

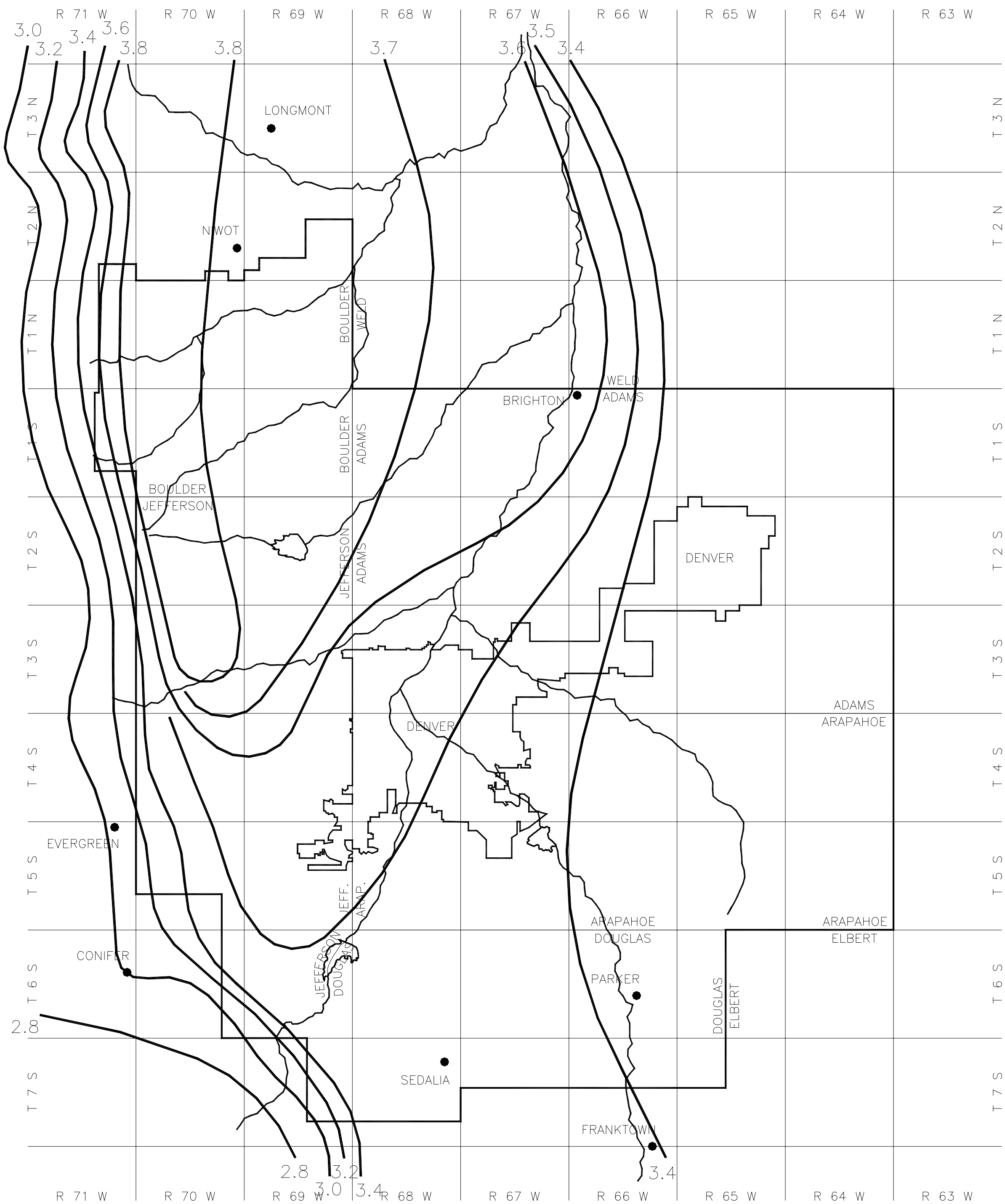
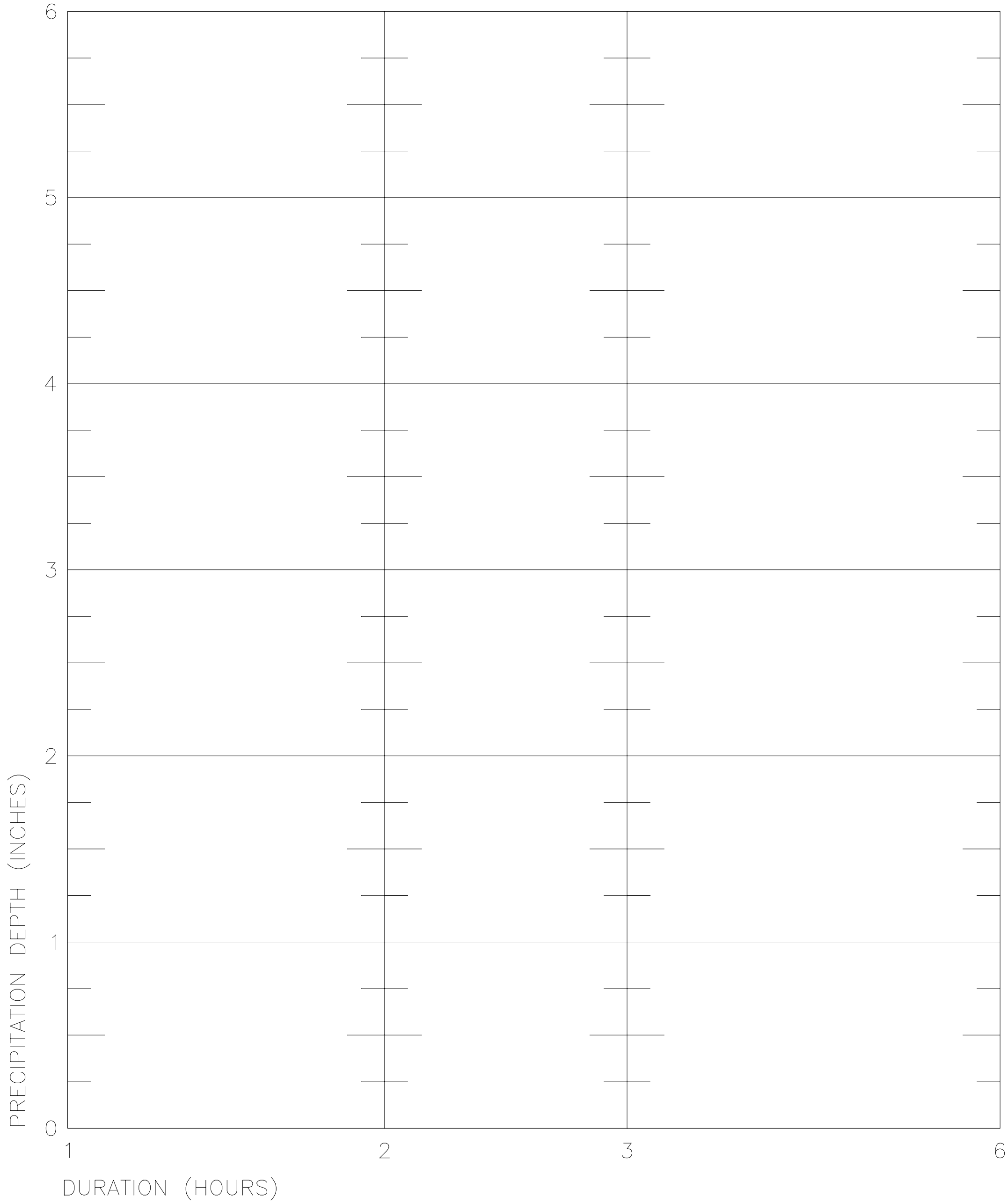


Figure RA-12—Rainfall Depth-Duration-Frequency: 100-Year, 6-Hour Rainfall



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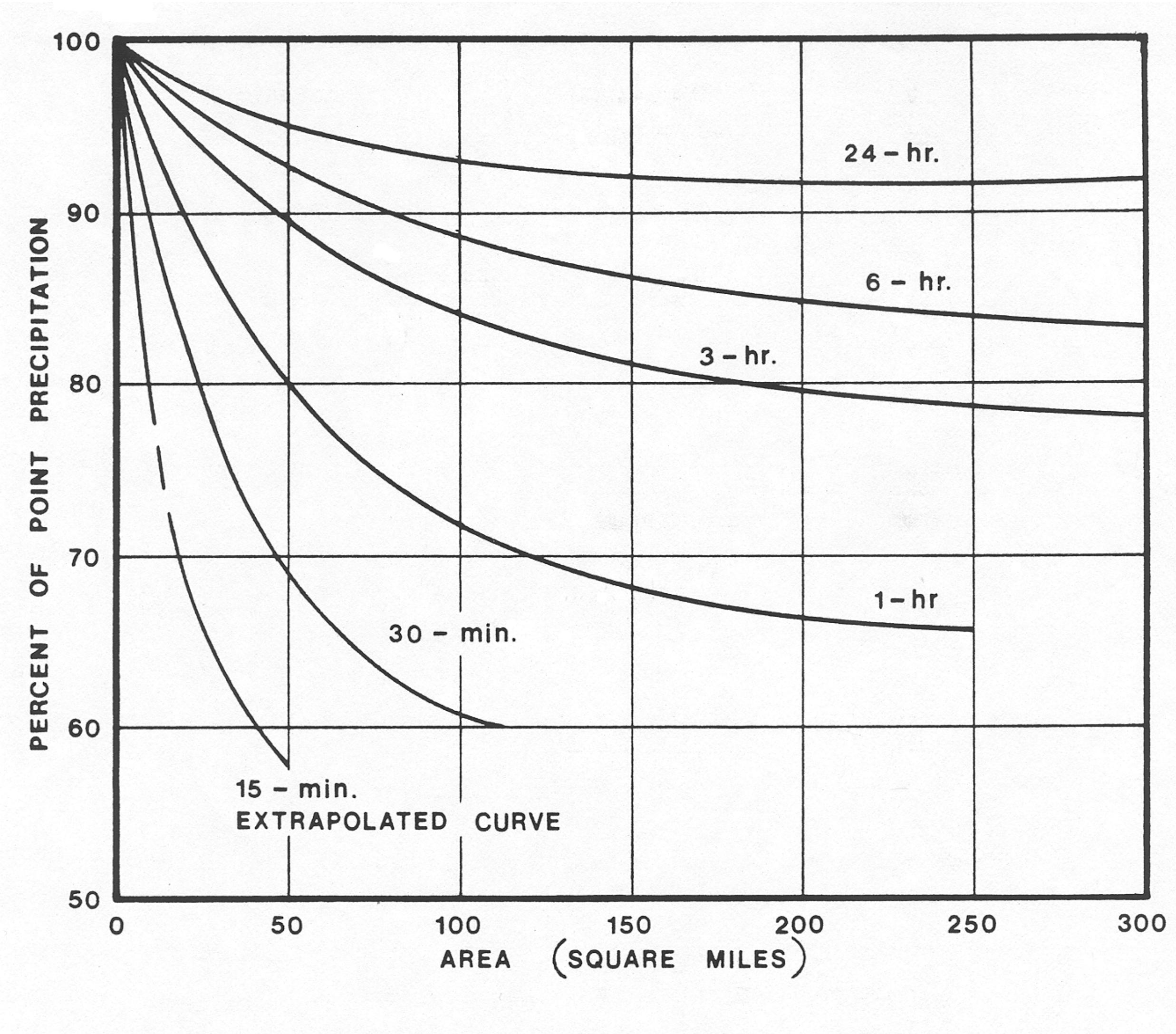

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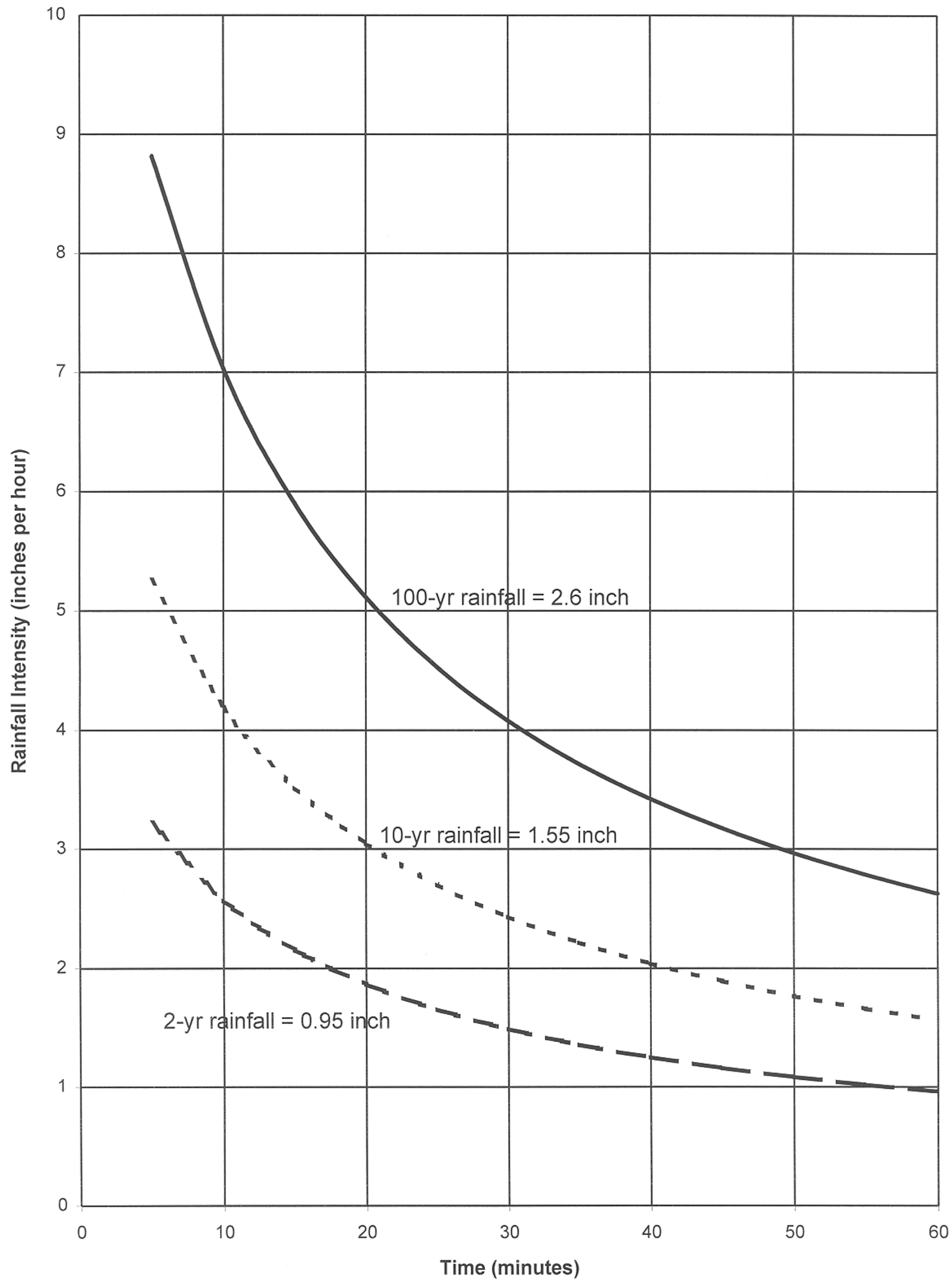

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RUNOFF

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1.0 OVERVIEW

The importance of accurate runoff quantification cannot be overstated. Estimates of peak rate of runoff, runoff volume, and the time distribution of flow provide the basis for all planning, design, and construction of drainage facilities. Erroneous hydrology results in works being planned and built that are either undersized, oversized, or out of hydraulic balance. On the other hand, it must be kept in mind that the result of the runoff analysis is an approximation. Thus, the intent of this chapter of the *Manual* is to provide a reasonably dependable and consistent method of approximating the characteristics of urban runoff for areas of Colorado and the United States having similar meteorology and hydrology to what is found within the Denver region.

Photograph RO-1—Devastating flooding from the South Platte River in 1965 emphasizes the importance of accurate flood flow projections.

Five methods of hydrologic analysis are described in this *Manual:* (1) the Rational Method; (2) the Colorado Urban Hydrograph Procedure (CUHP) for generating hydrographs from watersheds, (3) the EPA's Storm Water Management Model (SWMM), mostly for combining and routing the hydrographs generated using CUHP; (4) use of published runoff information; and (5) statistical analyses. CUHP has been calibrated for the Denver area using data that were collected for a variety of watershed conditions and has been used extensively since 1969. The vast majority of major drainage facilities within the District have been designed based upon the hydrology calculated using the CUHP and a previously used routing model used by the District, namely the Urban Drainage Stormwater Model (UDSWM). In 2005 the District has began using the EPA's SWMM and has also upgraded the CUHP software to be compatible with the EPA model.

There have been hydrologic studies carried out for a majority of the major drainageways within the

District. Often the use of published flow data (available from the District) may make the need for additional hydrologic analysis along major drainageways for a particular study unnecessary.

Statistical analyses may be used in certain situations. The use of this approach requires the availability of acceptable, appropriate, and adequate data.

Calculations for the Rational Method can be carried out by hand or using the UD-Rational Spreadsheet that may be downloaded from the District's Web site (www.udfcd.org). CUHP-SWMM calculations are extensive and are best carried out using the computer models provided by the District as an attachment to the CD version of this *Manual* or downloaded from the District's Web site.

Most of this chapter focuses on the Rational Method and on the CUHP method in combination with SWMM routing. The Rational Method is generally used for smaller catchments when only the peak flow rate or the total volume of runoff is needed (e.g., storm sewer sizing or simple detention basin sizing). CUHP-SWMM is used for larger catchments and when a hydrograph of the storm event is needed (e.g., sizing large detention facilities). A summary of applicability of both the methods is provided in Table RO-1.

Table RO-1—Applicability of Hydrologic Methods

Watershed Size (acres)	Is the Rational Method Applicable?	Is CUHP Applicable?
0 to 5	Yes	Yes (1)
5 to 90	Yes	Yes (1)
90 to 160	Yes	Yes
160 to 3,000	No	Yes (2)
Greater than 3,000	No	Yes (if subdivided into smaller catchments) (2)

(1) If one-minute unit hydrograph is used.

(2) Subdividing into smaller sub-catchments and routing the resultant hydrographs using SWMM may be needed to accurately model a catchment with areas of different soil types or percentages of imperviousness.

2.0 RATIONAL METHOD

For urban catchments that are not complex and are generally 160 acres or less in size, it is acceptable that the design storm runoff be analyzed by the Rational Method. This method was introduced in 1889 and is still being used in most engineering offices in the United States. Even though this method has frequently come under academic criticism for its simplicity, no other practical drainage design method has evolved to such a level of general acceptance by the practicing engineer. The Rational Method properly understood and applied can produce satisfactory results for urban storm sewer and small on-site detention design.

2.1 Rational Formula

The Rational Method is based on the Rational Formula:

$$Q = CIA \tag{RO-1}$$

in which:

Q = the maximum rate of runoff (cfs)

C = a runoff coefficient that is the ratio between the runoff volume from an area and the average rate of rainfall depth over a given duration for that area

I = average intensity of rainfall in inches per hour for a duration equal to the time of concentration, t_c

A = area (acres)

Actually, Q has units of inches per hour per acre (in/hr/ac); however, since this rate of in/hr/ac differs from cubic feet per second (cfs) by less than one percent, the more common units of cfs are used. The time of concentration is typically defined as the time required for water to flow from the most remote point of the area to the point being investigated. The time of concentration should be based upon a flow length and path that results in a time of concentration for only a portion of the area if that portion of the catchment produces a higher rate of runoff.

The general procedure for Rational Method calculations for a single catchment is as follows:

1. Delineate the catchment boundary. Measure its area.
2. Define the flow path from the upper-most portion of the catchment to the design point. This flow path should be divided into reaches of similar flow type (e.g., overland flow, shallow swale flow, gutter flow, etc.). The length and slope of each reach should be measured.
3. Determine the time of concentration, t_c, for the catchment.

4. Find the rainfall intensity, I, for the design storm using the calculated t_c and the rainfall intensity-duration-frequency curve. (See Section 4.0 of the RAINFALL chapter.)

5. Determine the runoff coefficient, C.

6. Calculate the peak flow rate from the watershed using Equation RO-1.

2.2 Assumptions

The basic assumptions that are often made when the Rational Method is applied are:

1. The computed maximum rate of runoff to the design point is a function of the average rainfall rate during the time of concentration to that point.

2. The depth of rainfall used is one that occurs from the start of the storm to the time of concentration, and the design rainfall depth during that time period is converted to the average rainfall intensity for that period.

3. The maximum runoff rate occurs when the entire area is contributing flow. However, this assumption has to be modified when a more intensely developed portion of the catchment with a shorter time of concentration produces a higher rate of maximum runoff than the entire catchment with a longer time of concentration.

2.3 Limitations

The Rational Method is an adequate method for approximating the peak rate and total volume of runoff from a design rainstorm in a given catchment. The greatest drawback to the Rational Method is that it normally provides only one point on the runoff hydrograph. When the areas become complex and where sub-catchments come together, the Rational Method will tend to overestimate the actual flow, which results in oversizing of drainage facilities. The Rational Method provides no direct information needed to route hydrographs through the drainage facilities. One reason the Rational Method is limited to small areas is that good design practice requires the routing of hydrographs for larger catchments to achieve an economic design.

Another disadvantage of the Rational Method is that with typical design procedures one normally assumes that all of the design flow is collected at the design point and that there is no water running overland to the next design point. However, this is not the fault of the Rational Method but of the design procedure. The Rational Method must be modified, or another type of analysis must be used, when analyzing an existing system that is under-designed or when analyzing the effects of a major storm on a system designed for the minor storm.

2.4 Time of Concentration

One of the basic assumptions underlying the Rational Method is that runoff is a function of the average rainfall rate during the time required for water to flow from the most remote part of the drainage area under consideration to the design point. However, in practice, the time of concentration can be an empirical value that results in reasonable and acceptable peak flow calculations. The time of concentration relationships recommended in this *Manual* are based in part on the rainfall-runoff data collected in the Denver metropolitan area and are designed to work with the runoff coefficients also recommended in this *Manual*. As a result, these recommendations need to be used with a great deal of caution whenever working in areas that may differ significantly from the climate or topography found in the Denver region.

For urban areas, the time of concentration, t_c, consists of an initial time or overland flow time, t_i, plus the travel time, t_t, in the storm sewer, paved gutter, roadside drainage ditch, or drainage channel. For non-urban areas, the time of concentration consists of an overland flow time, t_i, plus the time of travel in a defined form, such as a swale, channel, or drainageway. The travel portion, t_t, of the time of concentration can be estimated from the hydraulic properties of the storm sewer, gutter, swale, ditch, or drainageway. Initial time, on the other hand, will vary with surface slope, depression storage, surface cover, antecedent rainfall, and infiltration capacity of the soil, as well as distance of surface flow. The time of concentration is represented by Equation RO-2 for both urban and non-urban areas:

$$t_c = t_i + t_t \tag{RO-2}$$

in which:

t_c = time of concentration (minutes)

t_i = initial or overland flow time (minutes)

t_t = travel time in the ditch, channel, gutter, storm sewer, etc. (minutes)

2.4.1 Initial Flow Time

The initial or overland flow time, t_i, may be calculated using equation RO-3:

$$t_i = \frac{0.395(1.1 - C_5)\sqrt{L}}{S^{0.33}} \tag{RO-3}$$

in which:

t_i = initial or overland flow time (minutes)

C_5 = runoff coefficient for 5-year frequency (from Table RO-5)

L = length of overland flow (500 ft maximum for non-urban land uses, 300 ft maximum for urban land uses)

S = average basin slope (ft/ft)

Equation RO-3 is adequate for distances up to 500 feet. Note that, in some urban watersheds, the overland flow time may be very small because flows quickly channelize.

2.4.2 Overland Travel Time

For catchments with overland and channelized flow, the time of concentration needs to be considered in combination with the overland travel time, t_t, which is calculated using the hydraulic properties of the swale, ditch, or channel. For preliminary work, the overland travel time, t_t, can be estimated with the help of Figure RO-1 or the following equation (Guo 1999):

$$V = C_v S_w^{0.5} \qquad \text{(RO-4)}$$

in which:

V = velocity (ft/sec)

C_v = conveyance coefficient (from Table RO-2)

S_w = watercourse slope (ft/ft)

Table RO-2—Conveyance Coefficient, C_v

Type of Land Surface	Conveyance Coefficient, C_v
Heavy meadow	2.5
Tillage/field	5
Short pasture and lawns	7
Nearly bare ground	10
Grassed waterway	15
Paved areas and shallow paved swales	20

The time of concentration, t_c, is then the sum of the initial flow time, t_i, and the travel time, t_t, as per Equation RO-2.

2.4.3 First Design Point Time of Concentration in Urban Catchments

Using this procedure, the time of concentration at the first design point (i.e., initial flow time, t_i) in an urbanized catchment should not exceed the time of concentration calculated using Equation RO-5.

$$t_c = \frac{L}{180} + 10 \qquad \text{(RO-5)}$$

in which:

t_c = maximum time of concentration at the first design point in an urban watershed (minutes)

L = waterway length (ft)

Equation RO-5 was developed using the rainfall-runoff data collected in the Denver region and, in essence, represents regional "calibration" of the Rational Method.

The first design point is the point where runoff first enters the storm sewer system. An example of definition of first design point is provided in Figure RO-2.

Normally, Equation RO-5 will result in a lesser time of concentration at the first design point and will govern in an urbanized watershed. For subsequent design points, the time of concentration is calculated by accumulating the travel times in downstream drainageway reaches.

2.4.4 Minimum Time of Concentration

Should the calculations result in a t_c of less than 10 minutes, it is recommended that a minimum value of 10 minutes be used for non-urban watersheds. The minimum t_c recommended for urbanized areas should not be less than 5 minutes and if calculations indicate a lesser value, use 5 minutes instead.

2.4.5 Common Errors in Calculating Time of Concentration

A common mistake in urbanized areas is to assume travel velocities that are too slow. Another common error is to not check the runoff peak resulting from only part of the catchment. Sometimes a lower portion of the catchment or a highly impervious area produces a larger peak than that computed for the whole catchment. This error is most often encountered when the catchment is long or the upper portion contains grassy parkland and the lower portion is developed urban land.

2.5 Intensity

The rainfall intensity, I, is the average rainfall rate in inches per hour for the period of maximum rainfall of a given recurrence frequency having a duration equal to the time of concentration.

After the design storm's recurrence frequency has been selected, a graph should be made showing rainfall intensity versus time. The procedure for obtaining the local data and drawing such a graph is explained and illustrated in Section 4 of the RAINFALL chapter of this *Manual*. The intensity for a design point is taken from the graph or through the use of Equation RA-3 using the calculated t_c.

2.6 Watershed Imperviousness

All parts of a watershed can be considered either pervious or impervious. The pervious part is that area where water can readily infiltrate into the ground. The impervious part is the area that does not readily allow water to infiltrate into the ground, such as areas that are paved or covered with buildings and sidewalks or compacted unvegetated soils. In urban hydrology, the percentage of pervious and impervious land is important. The percentage of impervious area increases when urbanization occurs and the rainfall-runoff relationships change significantly. The total amount of runoff volume normally increases, the time to the runoff peak rate decreases, and the peak runoff rates increase.

Photograph RO-2—Urbanization (impervious area) increases runoff volumes, peak discharges, frequency of runoff, and receiving stream degradation.

When analyzing a watershed for design purposes, the probable future percent of impervious area must be estimated. A complete tabulation of recommended values of the total percent of imperviousness is provided in Table RO-3 and Figures RO-3 through RO-5, the latter developed by the District after the evolution of residential growth patterns since 1990.

2.7 Runoff Coefficient

The runoff coefficient, C, represents the integrated effects of infiltration, evaporation, retention, and interception, all of which affect the volume of runoff. The determination of C requires judgment and understanding on the part of the engineer.

Based in part on the data collected by the District since 1969, an empirical set of relationships between C and the percentage imperviousness for the 2-year and smaller storms was developed and are expressed in Equations RO-6 and RO-7 for Type A and C/D Soil groups (Urbonas, Guo and Tucker 1990). For Type B soil group the impervious value is found by taking the arithmetic average of the values found using these two equations for Type A and Type C/D soil groups. For larger storms (i.e., 5-, 10, 25-, 50- and 100-year) correction factors listed in Table RO-4 are applied to the values calculated using these two equations.

Table RO-3—Recommended Percentage Imperviousness Values

Land Use or Surface Characteristics	Percentage Imperviousness
Business:	
Commercial areas	95
Neighborhood areas	85
Residential:	
Single-family	*
Multi-unit (detached)	60
Multi-unit (attached)	75
Half-acre lot or larger	*
Apartments	80
Industrial:	
Light areas	80
Heavy areas	90
Parks, cemeteries	5
Playgrounds	10
Schools	50
Railroad yard areas	15
Undeveloped Areas:	
Historic flow analysis	2
Greenbelts, agricultural	2
Off-site flow analysis (when land use not defined)	45
Streets:	
Paved	100
Gravel (packed)	40
Drive and walks	90
Roofs	90
Lawns, sandy soil	0
Lawns, clayey soil	0

* See Figures RO-3 through RO-5 for percentage imperviousness.

$$C_A = K_A + \left(1.31i^3 - 1.44i^2 + 1.135i - 0.12\right) \text{ for } C_A \geq 0 \text{, otherwise } C_A = 0 \qquad \text{(RO-6)}$$

$$C_{CD} = K_{CD} + \left(0.858i^3 - 0.786i^2 + 0.774i + 0.04\right) \qquad \text{(RO-7)}$$

$$C_B = \left(C_A + C_{CD}\right)/2$$

in which:

i = % imperviousness/100 expressed as a decimal (see Table RO-3)

C_A = Runoff coefficient for Natural Resources Conservation Service (NRCS) Type A soils

C_B = Runoff coefficient for NRCS Type B soils

C_{CD} = Runoff coefficient for NRCS Type C and D soils

K_A = Correction factor for Type A soils defined in Table RO-4

K_{CD} = Correction factor for Type C and D soils defined in Table RO-4

Table RO-4—Correction Factors K_A and K_{CD} for Use with Equations RO-6 and RO-7

NRCS Soil Type	Storm Return Period					
	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
C and D	0	$-0.10i + 0.11$	$-0.18i + 0.21$	$-0.28i + 0.33$	$-0.33i + 0.40$	$-0.39i + 0.46$
A	0	$-0.08i + 0.09$	$-0.14i + 0.17$	$-0.19i + 0.24$	$-0.22i + 0.28$	$-0.25i + 0.32$

The values for various catchment imperviousnesses and storm return periods are presented graphically in Figures RO-6 through RO-8, and are tabulated in Table RO-5. These coefficients were developed for the Denver region to work in conjunction with the time of concentration recommendations in Section 2.4. Use of these coefficients and this procedure outside of the semi-arid climate found in the Denver region may not be valid. The *UD-Rational* spreadsheet performs all the needed calculations to find the runoff coefficient given the soil type and imperviousness and the reader may want to take advantage of this macro-enabled Excel workbook that is available for download from the District's web site www.udfcd.org under "Download" – "Technical Downloads."

See Examples 7.1 and 7.2 that illustrate the Rational method. The use of the Rational method in storm sewer design is illustrated in Example 6.13 of the STREETS/INLETS/STORM SEWERS chapter.

Table RO-5— Runoff Coefficients, *C*

Percentage Imperviousness	Type C and D NRCS Hydrologic Soil Groups					
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
0%	0.04	0.15	0.25	0.37	0.44	0.50
5%	0.08	0.18	0.28	0.39	0.46	0.52
10%	0.11	0.21	0.30	0.41	0.47	0.53
15%	0.14	0.24	0.32	0.43	0.49	0.54
20%	0.17	0.26	0.34	0.44	0.50	0.55
25%	0.20	0.28	0.36	0.46	0.51	0.56
30%	0.22	0.30	0.38	0.47	0.52	0.57
35%	0.25	0.33	0.40	0.48	0.53	0.57
40%	0.28	0.35	0.42	0.50	0.54	0.58
45%	0.31	0.37	0.44	0.51	0.55	0.59
50%	0.34	0.40	0.46	0.53	0.57	0.60
55%	0.37	0.43	0.48	0.55	0.58	0.62
60%	0.41	0.46	0.51	0.57	0.60	0.63
65%	0.45	0.49	0.54	0.59	0.62	0.65
70%	0.49	0.53	0.57	0.62	0.65	0.68
75%	0.54	0.58	0.62	0.66	0.68	0.71
80%	0.60	0.63	0.66	0.70	0.72	0.74
85%	0.66	0.68	0.71	0.75	0.77	0.79
90%	0.73	0.75	0.77	0.80	0.82	0.83
95%	0.80	0.82	0.84	0.87	0.88	0.89
100%	0.89	0.90	0.92	0.94	0.95	0.96
	TYPE B NRCS HYDROLOGIC SOILS GROUP					
0%	0.02	0.08	0.15	0.25	0.30	0.35
5%	0.04	0.10	0.19	0.28	0.33	0.38
10%	0.06	0.14	0.22	0.31	0.36	0.40
15%	0.08	0.17	0.25	0.33	0.38	0.42
20%	0.12	0.20	0.27	0.35	0.40	0.44
25%	0.15	0.22	0.30	0.37	0.41	0.46
30%	0.18	0.25	0.32	0.39	0.43	0.47
35%	0.20	0.27	0.34	0.41	0.44	0.48
40%	0.23	0.30	0.36	0.42	0.46	0.50
45%	0.26	0.32	0.38	0.44	0.48	0.51
50%	0.29	0.35	0.40	0.46	0.49	0.52
55%	0.33	0.38	0.43	0.48	0.51	0.54
60%	0.37	0.41	0.46	0.51	0.54	0.56
65%	0.41	0.45	0.49	0.54	0.57	0.59
70%	0.45	0.49	0.53	0.58	0.60	0.62
75%	0.51	0.54	0.58	0.62	0.64	0.66
80%	0.57	0.59	0.63	0.66	0.68	0.70
85%	0.63	0.66	0.69	0.72	0.73	0.75
90%	0.71	0.73	0.75	0.78	0.80	0.81
95%	0.79	0.81	0.83	0.85	0.87	0.88
100%	0.89	0.90	0.92	0.94	0.95	0.96

TABLE RO-5 (Continued)—Runoff Coefficients, *C*

Percentage Imperviousness	Type A NRCS Hydrologic Soils Group					
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
0%	0.00	0.00	0.05	0.12	0.16	0.20
5%	0.00	0.02	0.10	0.16	0.20	0.24
10%	0.00	0.06	0.14	0.20	0.24	0.28
15%	0.02	0.10	0.17	0.23	0.27	0.30
20%	0.06	0.13	0.20	0.26	0.30	0.33
25%	0.09	0.16	0.23	0.29	0.32	0.35
30%	0.13	0.19	0.25	0.31	0.34	0.37
35%	0.16	0.22	0.28	0.33	0.36	0.39
40%	0.19	0.25	0.30	0.35	0.38	0.41
45%	0.22	0.27	0.33	0.37	0.40	0.43
50%	0.25	0.30	0.35	0.40	0.42	0.45
55%	0.29	0.33	0.38	0.42	0.45	0.47
60%	0.33	0.37	0.41	0.45	0.47	0.50
65%	0.37	0.41	0.45	0.49	0.51	0.53
70%	0.42	0.45	0.49	0.53	0.54	0.56
75%	0.47	0.50	0.54	0.57	0.59	0.61
80%	0.54	0.56	0.60	0.63	0.64	0.66
85%	0.61	0.63	0.66	0.69	0.70	0.72
90%	0.69	0.71	0.73	0.76	0.77	0.79
95%	0.78	0.80	0.82	0.84	0.85	0.86
100%	0.89	0.90	0.92	0.94	0.95	0.96

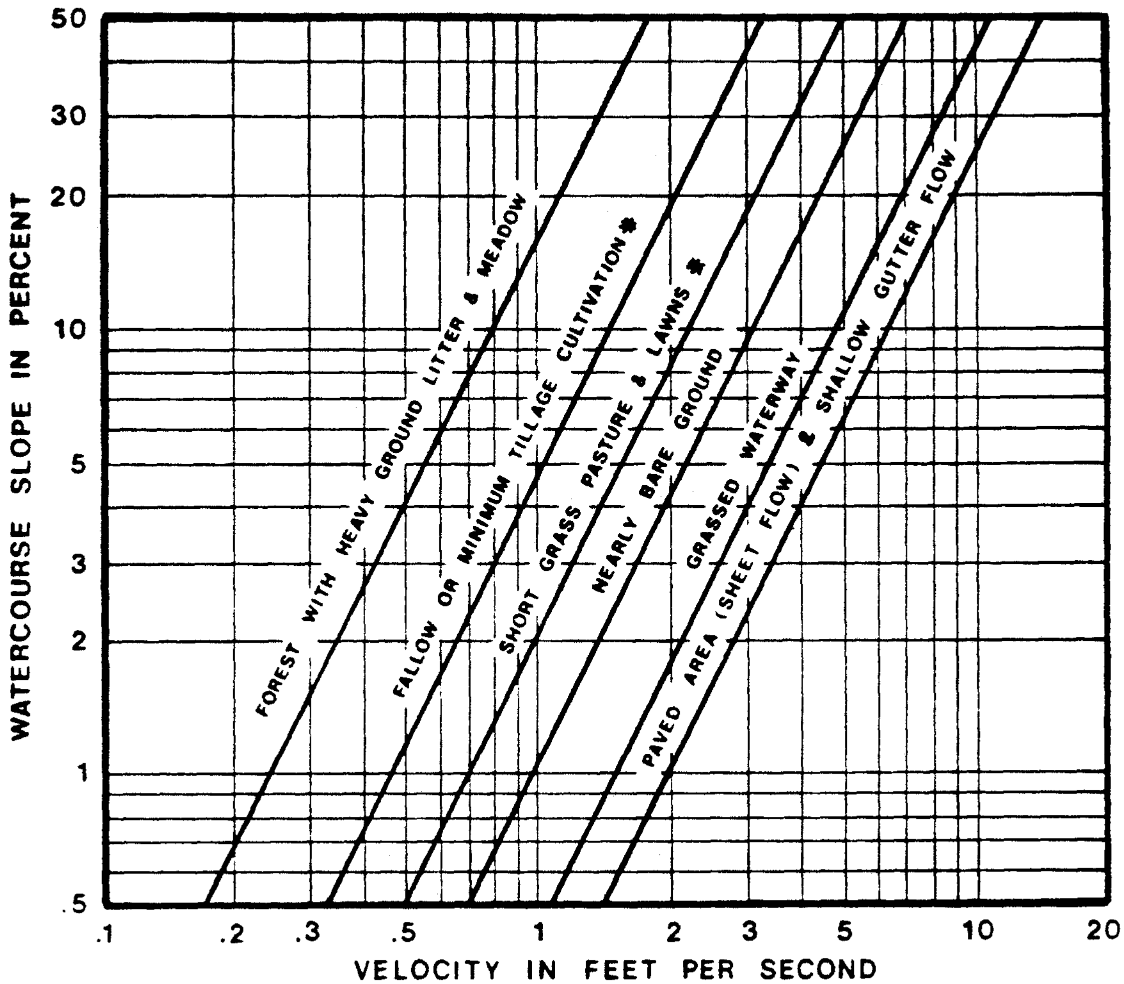
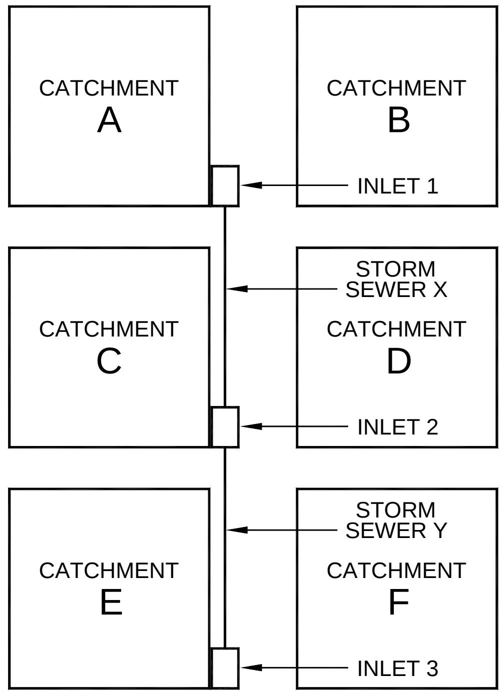


Figure RO-1—Estimate of Average Overland Flow Velocity for Use With the Rational Formula



NOTE:
INLETS 1, 2, 3 AND STORM SEWER X ARE EACH THE "FIRST DESIGN POINT" AND THE REGIONAL Tc SHOULD BE CHECKED. STORM SEWER Y IS NOT THE FIRST DESIGN POINT.

Figure RO-2—Diagram of First Design Point

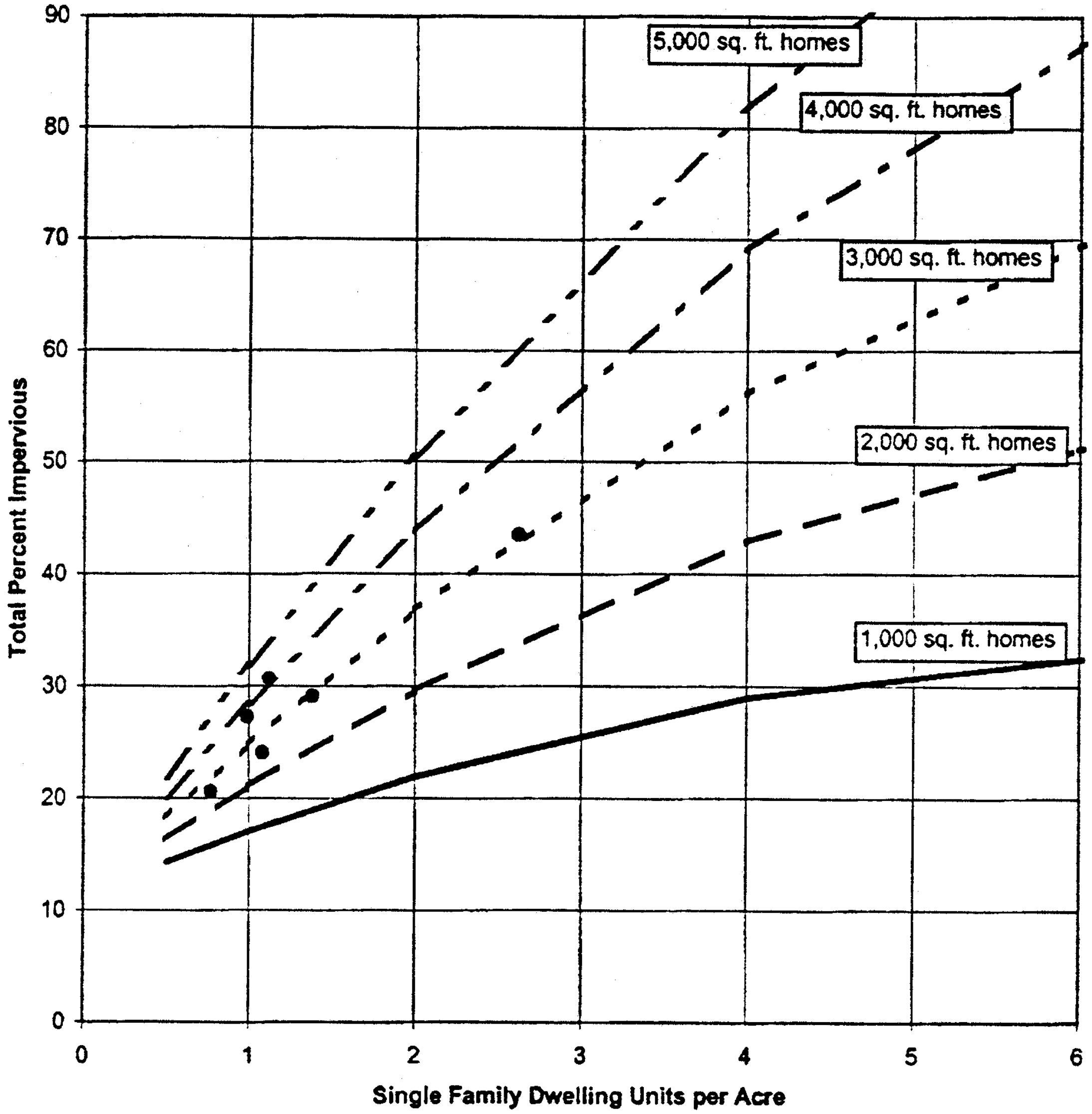


Figure RO-3— Watershed Imperviousness, Single-Family Residential Ranch Style Houses

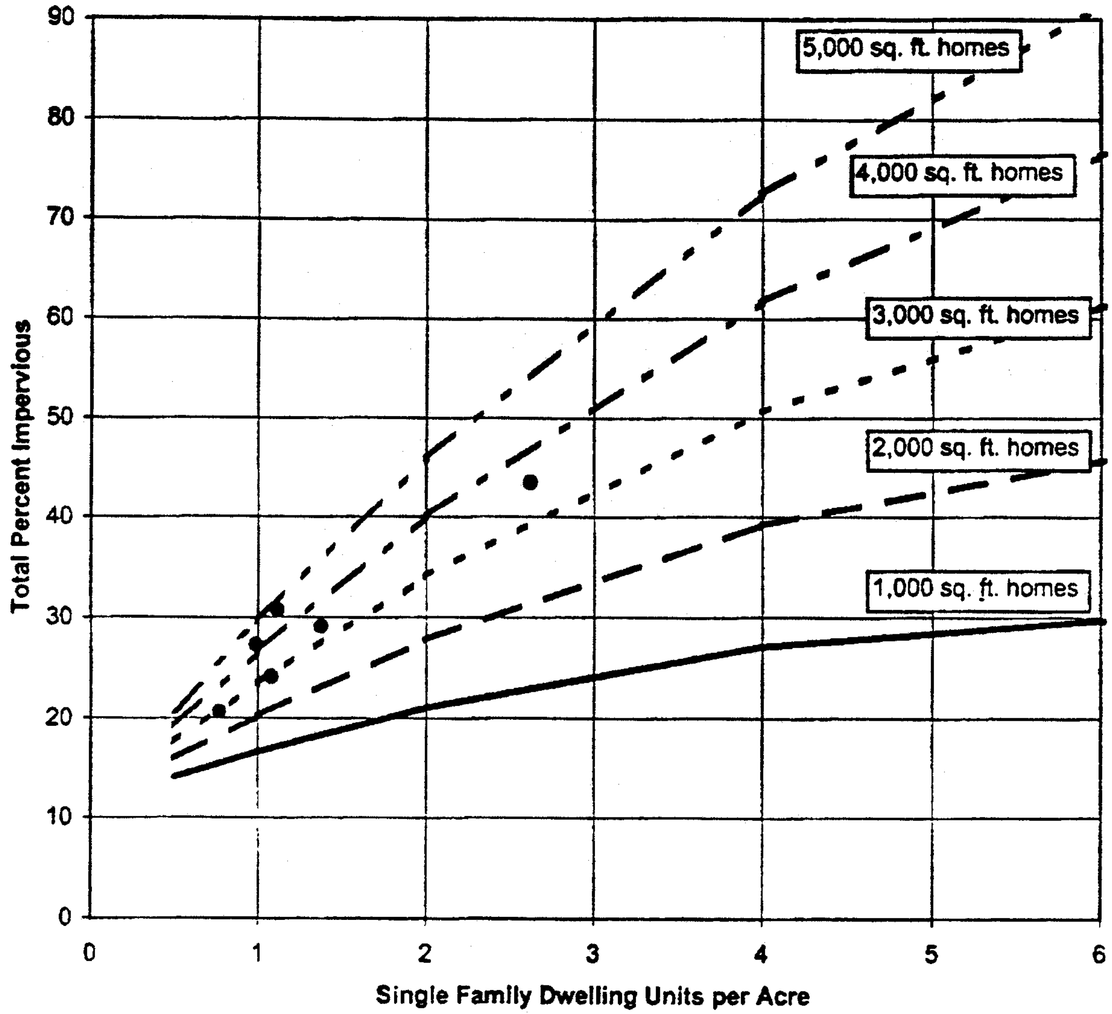


Figure RO-4—Watershed Imperviousness, Single-Family Residential Split-Level Houses

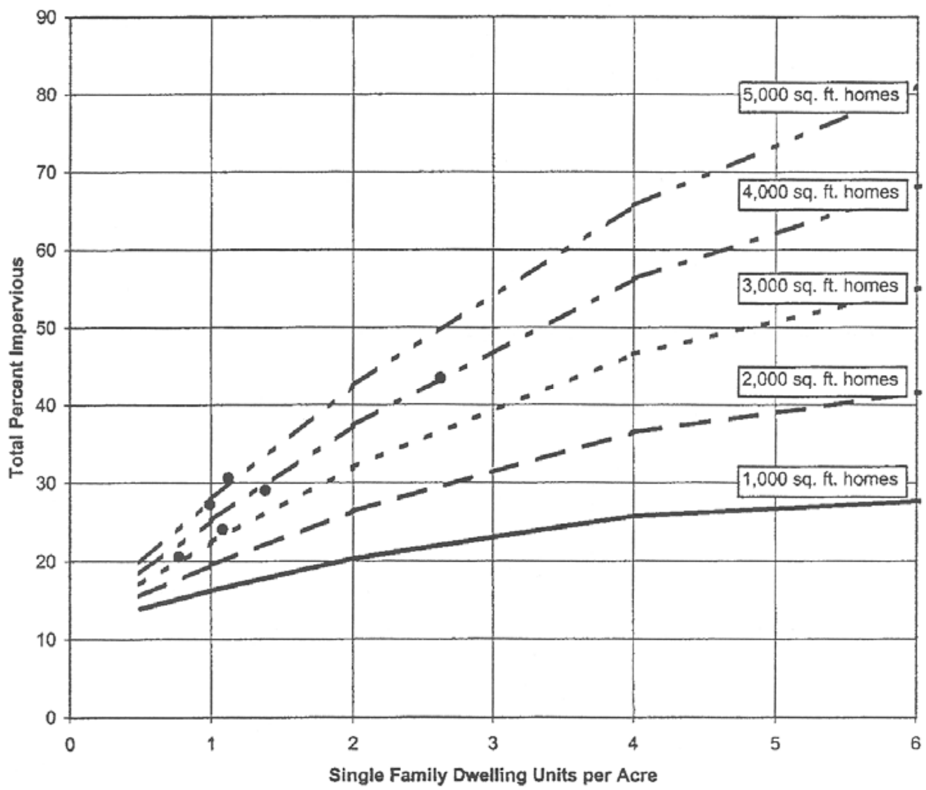


Figure RO-5—Watershed Imperviousness, Single-Family Residential Two-Story Houses

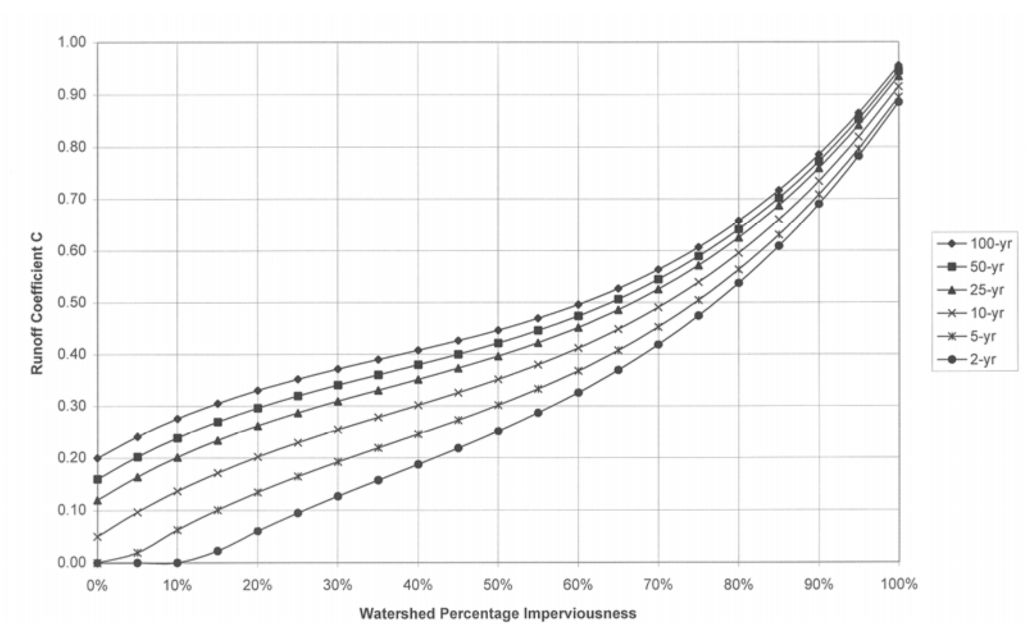


Figure RO-6—Runoff Coefficient, *C*, vs. Watershed Percentage Imperviousness NRCS Hydrologic Soil Group A

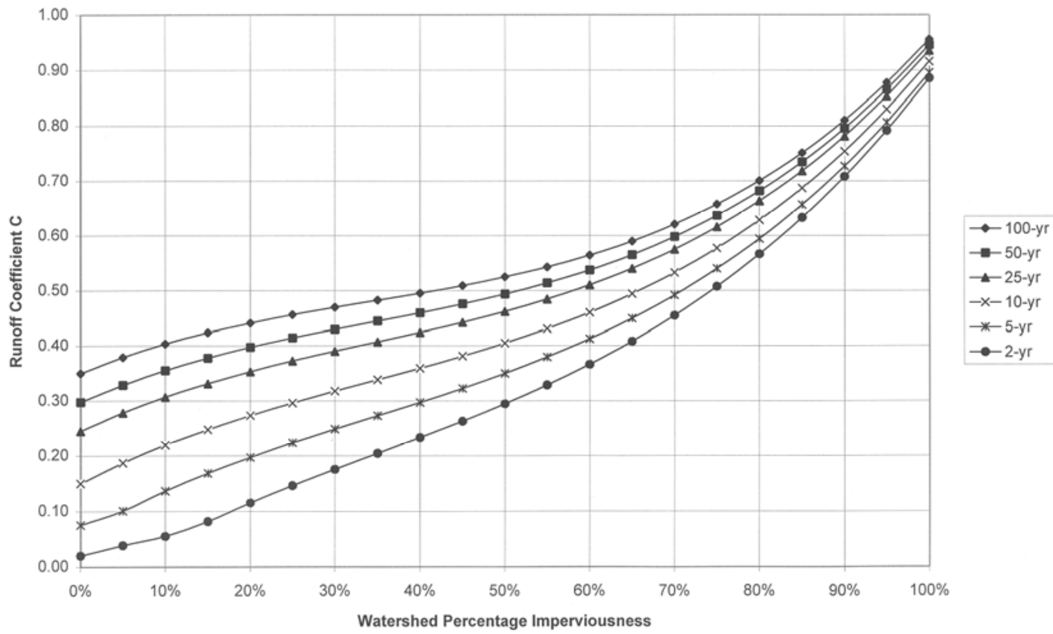


Figure RO-7—Runoff Coefficient, *C*, vs. Watershed Percentage Imperviousness NRCS Hydrologic Soil Group B

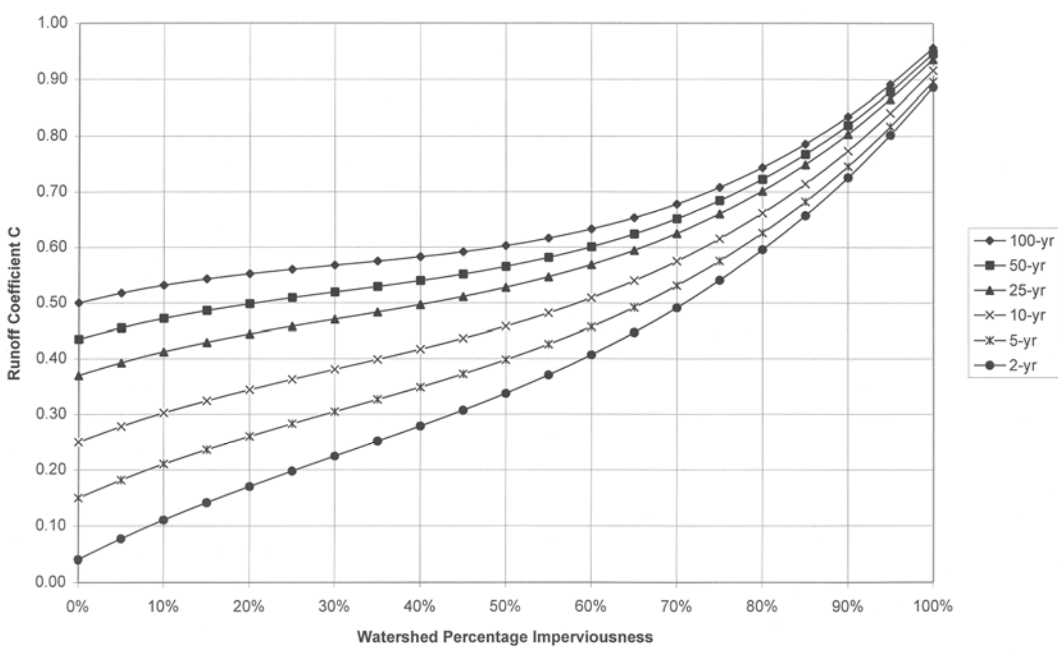


Figure RO-8—Runoff Coefficient, *C*, vs. Watershed Percentage Imperviousness NRCS Hydrologic Soil Groups C and D

3.0 COLORADO URBAN HYDROGRAPH PROCEDURE

3.1 Background

The Colorado Urban Hydrograph Procedure (CUHP) is a method of hydrologic analysis based upon the unit hydrograph principle. It has been developed and calibrated using rainfall-runoff data collected in Colorado (mostly in the Denver/Boulder metropolitan area). This section provides a general background in the use of the computer version of CUHP to carry out stormwater runoff calculations. A detailed description of the CUHP procedure and the assumptions and equations used, including a hand calculation example, are provided in Appendix A to this chapter. For more detailed information regarding the latest CUHP computer model including data requirements, data format, and model execution, the reader is directed to the program's users' manual. The latest version of CUHP macro-enabled software is ***CUHP 2005*** and users' manual are available for downloading from the District's Web site www.udfcd.org under "Downloads".

3.2 Effective Rainfall for CUHP

Effective rainfall is that portion of precipitation during a storm event that runs off the land to drainageways. Those portions of precipitation that do not reach drainageways are called abstractions and include interception by vegetation, evaporation, infiltration, storage in all surface depressions, and long-time surface retention. The total design rainfall depth for use with CUHP should be obtained from the RAINFALL chapter of this *Manual*. This RUNOFF chapter illustrates a method for estimating the amount of rainfall that actually becomes surface runoff whenever a design rainstorm is used.

3.2.1 Pervious-Impervious Areas

As was described in Section 2.6, the urban landscape is comprised of pervious and impervious surfaces. The degree of imperviousness is the primary variable that affects the volumes and rates of runoff calculated using CUHP. When analyzing a watershed for design purposes, the probable future percent of impervious area must first be estimated. A complete tabulation of recommended values of total percentage imperviousness is provided in Table RO-3 and Figures RO-3 through RO-5. References to impervious area and all calculations in this chapter are based on the input of total impervious areas. The pervious-impervious area relationship can be further refined for use in CUHP as follows:

1. *DCIA*—Impervious area portion directly connected to the drainage system.
2. *UIA*—Impervious area portion that drains onto or across impervious surfaces.
3. *RPA*—The portion of pervious area receiving runoff from impervious portions.
4. *SPA*—The separate pervious area portion not receiving runoff from impervious surfaces.

This further refinement is explained in some detail in the CUHP users' manual and shown schematically in Figure RO-A6 in Appendix A at the end of this chapter.

3.2.2 Depression Losses

Rainwater that is collected and held in small depressions and does not become part of the general surface runoff is called depression loss. Most of this water eventually infiltrates or is evaporated. Depression losses also include water intercepted by trees, bushes, other vegetation, and all other surfaces. The CUHP method requires numerical values of depression loss as inputs to calculate the effective rainfall. Table RO-6 can be used as a guide in estimating the amount of depression (retention) losses to be used with CUHP.

Table RO-6—Typical Depression Losses for Various Land Covers

(All Values in Inches. For use with the CUHP Method)

Land Cover	Range in Depression (Retention) Losses	Recommended
Impervious:		
Large paved areas	0.05 - 0.15	0.1
Roofs-flat	0.1 - 0.3	0.1
Roofs-sloped	0.05 - 0.1	0.05
Pervious:		
Lawn grass	0.2 - 0.5	0.35
Wooded areas and open fields	0.2 - 0.6	0.4

When an area is analyzed for depression losses, the pervious and impervious loss values for all parts of the watershed must be considered and accumulated in proportion to the percent of aerial coverage for each type of surface.

3.2.3 Infiltration

The flow of water into the soil surface is called infiltration. In urban hydrology much of the infiltration occurs on areas covered with grass. Urbanization can increase or decrease the total amount of infiltration.

Soil type is the most important factor in determining the infiltration rate. When the soil has a large percentage of well-graded fines, the infiltration rate is low. In some cases of extremely tight soil, there may be, from a practical standpoint, essentially no infiltration. If the soil has several layers or horizons, the least permeable layer near the surface will control the maximum infiltration rate. The soil cover also plays an important role in determining the infiltration rate. Vegetation, lawn grass in particular, tends to increase infiltration by loosening the soil near the surface. Other factors affecting infiltration rates include slope of land, temperature, quality of water, age of lawn and soil compaction.

As rainfall continues, the infiltration rate decreases. When rainfall occurs on an area that has little antecedent moisture and the ground is dry, the infiltration rate is much higher than it is with high antecedent moisture resulting from previous storms or land irrigation such as lawn watering. Although antecedent precipitation is very important when calculating runoff from smaller storms in non-urbanized areas, the runoff data from urbanized basins indicates that antecedent precipitation has a limited effect on runoff peaks and volumes in the urbanized portions of the District.

There are many infiltration models in use by hydrologists. These models vary significantly in complexity. Because of the climatic condition in the semi-arid region and because runoff from urban watersheds is not very sensitive to infiltration refinements, the infiltration model proposed by Horton was found to provide a good balance between simplicity and reasonable physical description of the infiltration process for use in CUHP. Horton's infiltration model is described by Equation RO-8 and is illustrated graphically in Figure RO-9.

$$f = f_o + \left(f_i - f_o\right) e^{-at} \tag{RO-8}$$

in which:

f = infiltration rate at any given time t from start of rainfall (in/hr)

f_o = final infiltration rate (in/hr)

f_i = initial infiltration rate (in/hr)

e = natural logarithm base

a = decay coefficient (1/second)

t = time (seconds)

In developing Equation RO-8, Horton observed that infiltration is high early in the storm and eventually decays to a steady state constant value as the pores in the soil become saturated. The coefficients and initial and final infiltration values are site specific and depend on the soils and vegetative cover complex. It is possible to develop these values for each site if sufficient rainfall-runoff observations are made. However, such an approach is rarely practical.

Since 1977, the District has analyzed a considerable amount of rainfall-runoff data. On the basis of this analysis, the values in Table RO-7 are recommended for use within the District with CUHP. The NRCS Hydrologic Soil Groups C and D occur most frequently within the District; however, areas of NRCS Group A and B soils are also fairly common. Consult NRCS soil surveys for appropriate soil classifications.

Table RO-7—Recommended Horton's Equation Parameters

NRCS Hydrologic Soil Group	Infiltration (inches per hour)		Decay Coefficient—a
	Initial—f_i	Final—f_o	
A	5.0	1.0	0.0007
B	4.5	0.6	0.0018
C	3.0	0.5	0.0018
D	3.0	0.5	0.0018

To calculate the maximum infiltration depths that may occur at each time increment, it is necessary to integrate Equation RO-8 and calculate the values for each time increment. Very little accuracy is lost if, instead of integrating Equation RO-8, the infiltration rate is calculated at the center of each time increment. This "central" value can then be multiplied by the unit time increment to estimate the infiltration depth. This was done for the four NRCS hydrologic soil groups, and the results are presented in Table RO-8. Although Tables RO-7 and RO-8 provide recommended values for various Horton equation parameters, these recommendations are being made specifically for the urbanized or urbanizing watersheds in the Denver metropolitan area and may not be valid in different meteorologic and climatic regions.

Table RO-8—Incremental Infiltration Depths in Inches*

Time in Minutes**	NRCS Hydrologic Soil Group		
	A	B	C and D
5	0.384	0.298	0.201
10	0.329	0.195	0.134
15	0.284	0.134	0.096
20	0.248	0.099	0.073
25	0.218	0.079	0.060
30	0.194	0.067	0.052
35	0.175	0.060	0.048
40	0.159	0.056	0.045
45	0.146	0.053	0.044
50	0.136	0.052	0.043
55	0.127	0.051	0.042
60	0.121	0.051	0.042
65	0.115	0.050	0.042
70	0.111	0.050	0.042
75	0.107	0.050	0.042
80	0.104	0.050	0.042
85	0.102	0.050	0.042
90	0.100	0.050	0.042
95	0.098	0.050	0.042
100	0.097	0.050	0.042
105	0.096	0.050	0.042
110	0.095	0.050	0.042
115	0.095	0.050	0.042
120	0.094	0.050	0.042

* Based on central value of each time increment in Horton's equation.
** Time at end of the time increment.

3.3 CUHP Parameter Selection

3.3.1 Rainfall

The ***CUHP 2005*** Excel-based computer program requires the input of a design storm, either as a detailed hyetograph or as a 1-hour rainfall depth. A detailed hyetograph distribution is generated by the program for the latter using the standard 2-hour storm distribution recommended in the RAINFALL chapter of this *Manual*. In addition, this software will also distribute the one-hour values for longer storm durations with area corrections accounted for cases where larger watersheds are studies.

3.3.2 Catchment Description

The following catchment parameters are required for the program to generate a unit and storm hydrograph.

1. Area—Catchment area in square miles. See Table RO-1 for catchment size limits.
2. Catchment Length—The length in miles from the downstream design point of the catchment or sub-catchment along the main drainageway path to the furthest point on its respective catchment or sub-catchment. When a catchment is subdivided into a series of sub-catchments, the sub-catchment length used shall include the distance required for runoff to reach the major drainageway from the farthest point in the sub-catchment.
3. Centroid Distance—Distance in miles from the design point of the catchment or sub-catchment along the main drainageway path to its respective catchment or sub-catchment centroid.
4. Percent Impervious—The portion of the catchment's total surface area that is impervious, expressed as a percent value between 0 and 100. (See 3.2.1 for more details.)
5. Catchment Slope—The length-weighted, corrected average slope of the catchment in feet per foot.

   There are natural processes at work that limit the time to peak of a unit hydrograph as a natural drainageway becomes steeper. To account for this phenomenon, it is recommended that the slope used in CUHP for natural drainageways and existing manmade grass-lined channels be adjusted using Figure RO-10.

   When a *riprap channel* is evaluated, use the measured (i.e., uncorrected) average channel invert slope.

   In *concrete-lined channels* and *buried conduits,* the velocities can be very high. For this reason, it is recommended that the average ground slope (i.e., not flow-line slope) be used where concrete-lined channels and/or storm sewers dominate the basin drainageways. There is no correction factor or upper limit recommended to the slope for concrete-lined channels and buried conduits.

Where the flow-line slope varies along the channel, calculate a weighted basin slope for use with CUHP. Do this by first segmenting the major drainageway into reaches having similar longitudinal slopes. Then calculate the weighted slope using the Equation RO-9.

$$S = \left[\frac{L_1 S_1^{\;0.24} + L_2 S_2^{\;0.24} + ... + L_n S_n^{\;0.24}}{L_1 + L_2 + L_3 .... L_n} \right]^{4.17} \qquad \text{(RO-9)}$$

in which:

S = weighted basin waterway slopes in ft/ft

$S_1, S_2, \ldots S_n$ = slopes of individual reaches in ft/ft (after adjustments using Figure RO-10)

$L_1, L_2, \ldots L_n$ = lengths of corresponding reaches

6. Unit Hydrograph Time Increment—Typically a 5-minute unit hydrograph is used. For catchments smaller than 90 acres, using a 1-minute unit hydrograph may be needed if significant differences are found between the “excess precipitation” and “runoff hydrograph” volumes listed in the summary output. For very small catchments (i.e. smaller than 10 acres), especially those with high imperviousness the 1-minute unit hydrograph will be needed to preserve runoff volume integrity.

7. Pervious Retention—Maximum depression storage on pervious surfaces in inches. (See Section 3.2.2 for more details.)

8. Impervious Retention—Maximum depression storage on impervious surfaces in inches. (See Section 3.2.2 for more details.)

9. Infiltration Rate—Initial infiltration rate for pervious surfaces in the catchment in inches per hour. If this entry is used by itself, it will be used as a constant infiltration rate throughout the storm. (See Section 4.2.3 for more details.)

10. Decay—Exponential decay coefficient in Horton's equation in "per second" units.

11. Final Infiltration—Final infiltration rate in Horton's equation in inches per hour.

The program computes the coefficients C_t and C_p; however, values for these parameters can be specified by the user as an option. The unit hydrograph is developed by the computer using the algorithm described in *CUHP 2005 User Manual*.

The shaping of the unit hydrograph also relies on proportioning the widths at 50% and 75% of the unit hydrograph peak. The proportioning is based on 0.35 of the width at 50% of peak being ahead of the “time to peak” and 0.45 of the width at 75% of peak being ahead of the “time to peak.” These proportioning factors were selected after observing a number of unit hydrographs derived from the rainfall-runoff data collected by the USGS for the District. It is possible for the user to override the unit hydrograph widths and the proportioning of these widths built into the program. For drainage and flood studies within the District, the program values shall be used. If the user has derived unit hydrographs from reliable rainfall-runoff data for a study catchment and can develop a “calibrated” unit hydrograph for this catchment, this option permits reshaping the unit hydrograph accordingly.

The following catchment parameters are also optional inputs and are available to the user to account for the effects of directly connected/disconnected impervious areas:

1. DCIA—Specifies the directly connected impervious area (DCIA) level of practice as defined in the STRUCTURAL BMPs chapter in Volume 3 of this *Manual*. The user may specify 1 or 2 for the level of DCIA to model.

2. D—Defines the fraction of the total impervious area directly connected to the drainage system. Values range from 0.01 to 1.0.

3. R—Defines the fraction of total pervious area receiving runoff from the “disconnected” impervious areas. Values range from 0.01 to 1.0.

A sample calculation for effective rainfall is presented in Example 7.3.

3.3.3 Catchment Delineation Criteria

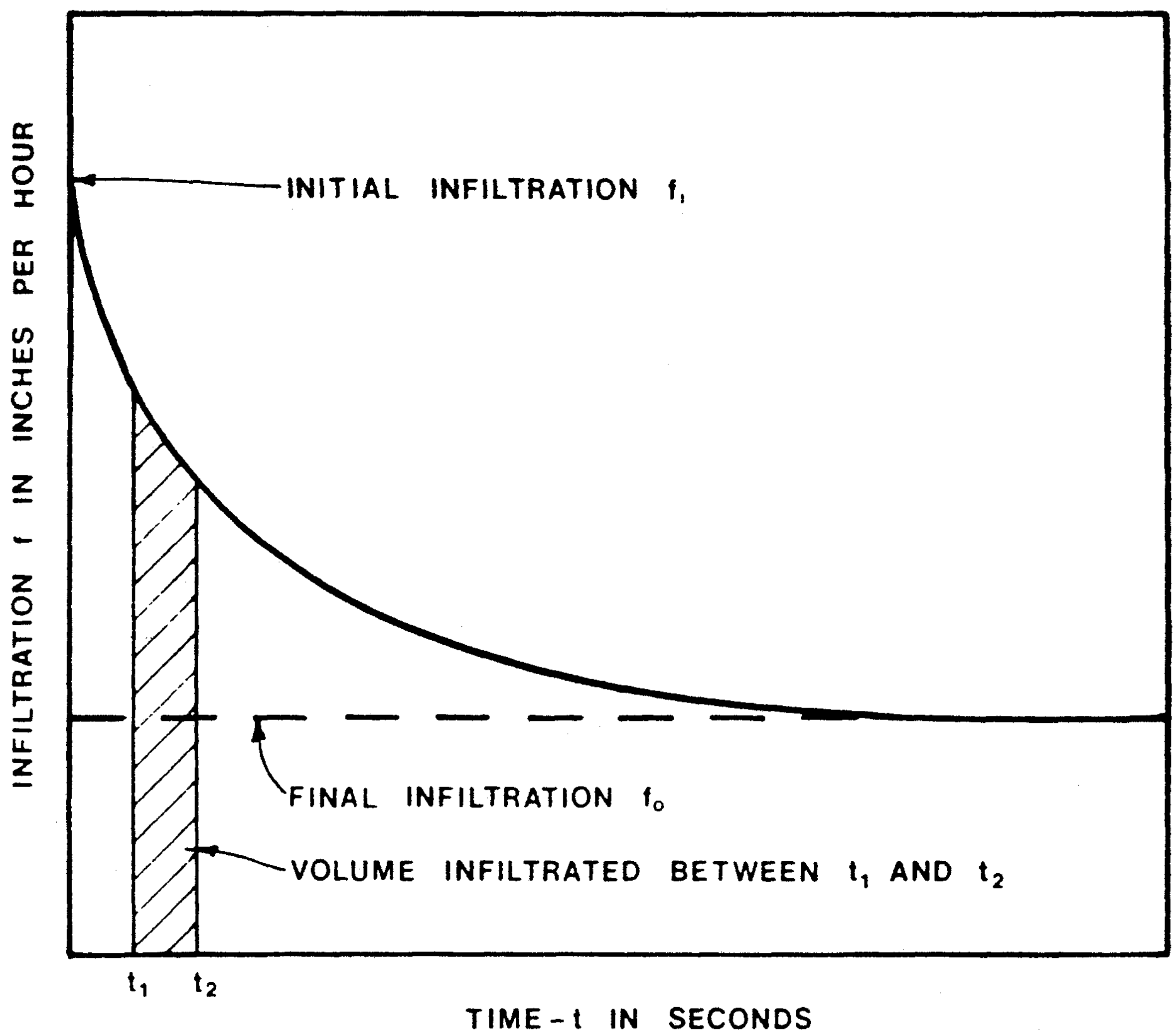
The maximum size of a catchment to be analyzed with a single unit hydrograph is limited to 5 square miles. Whenever a larger catchment is studied, it should be subdivided into sub-catchments of 5 square miles or less and individual sub-catchment storm hydrographs should be routed downstream using appropriate channel routing procedures such as the EPA’s SWMM 5 model. The routed hydrographs are then added to develop a single composite storm hydrograph. See Table RO-1 for a description of catchment size limitations for CUHP.

The catchment shape can have a profound effect on the final results and, in some instances, can result in underestimates of peak flows. Experience with the 1982 version of CUHP has shown that, whenever catchment length is increased faster than its area, the storm hydrograph peak will tend to decrease. Although hydrologic routing is an integral part of runoff analysis, the data used to develop CUHP are insufficient to say that the observed CUHP response with disproportionately increasing basin length is valid. For this reason, it is recommended to subdivide irregularly shaped or very long catchments (i.e., catchment length to width ratio of four or more) into more regularly shaped sub-catchments. A composite catchment storm hydrograph can be developed using appropriate routing and by adding the individual sub-catchment storm hydrographs.

3.3.3 Combining and Routing Sub-Catchment CUHP Hydrographs

When analyzing large and complex systems, it is necessary to combine and route the runoff hydrographs from a number of sub-catchments to determine the flows and volumes throughout the system. The ***CUHP 2005*** software provides input parameters that identify to which junction in EPS' SWMM each sub-Catchment's hydrograph is to be linked and to then generate an output file that SWMM recognizes as external flow file. All of these and other features are covered in the ***CUHP 2005*** User's Manual.

Figure RO-9—Representation of Horton's Equation



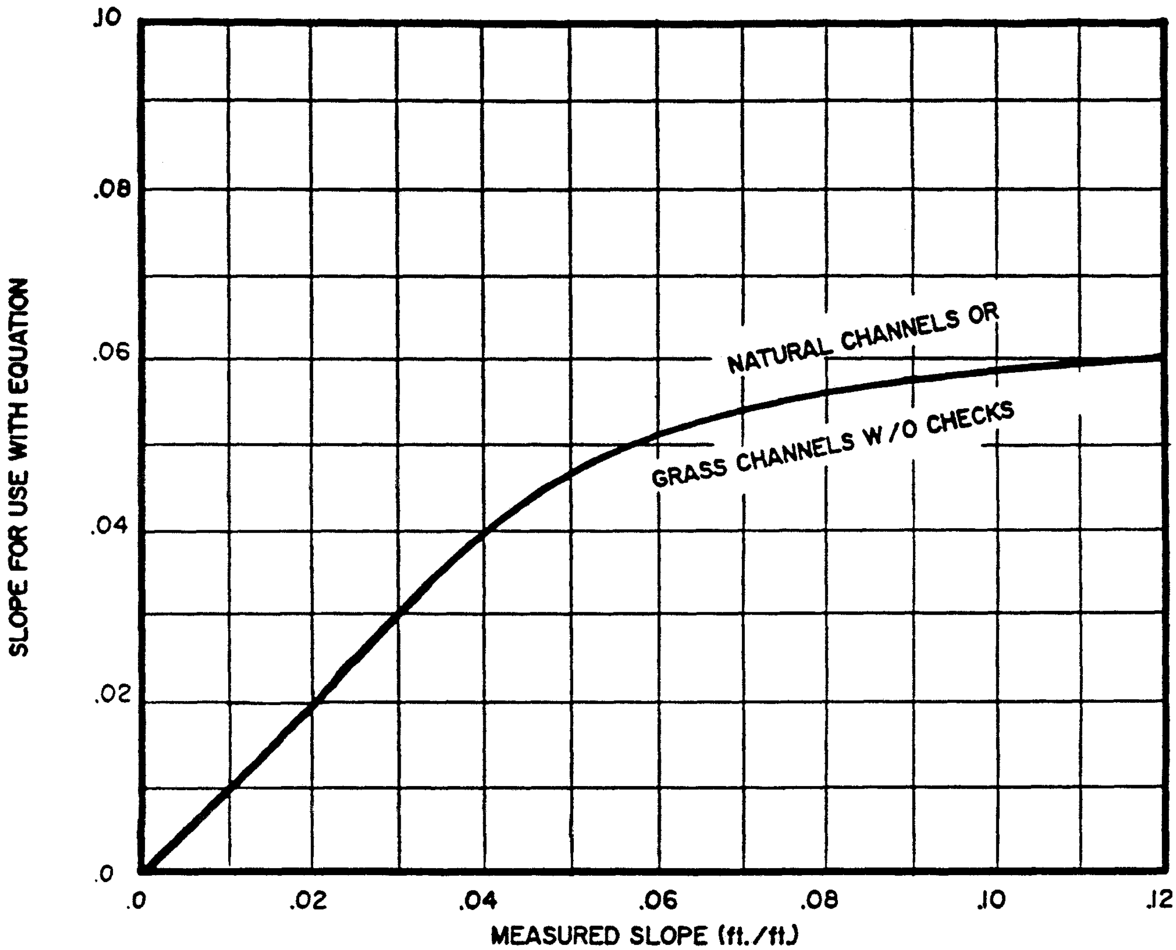


Figure RO-10—Slope Correction for Natural and Grass-Lined Channels

4.0 EPA SWMM AND HYDROGRAPH ROUTING

EPA's SWMM 5 is a computer model that is used to generate surface runoff hydrographs from sub-catchments and then route and combine these hydrographs. The procedure described here is limited to the routing of hydrographs generated using CUHP software. Originally this was done using UDSWM, a modified version of the Runoff Block of the Environmental Protection Agency's (EPA's) SWMM (Storm Water Management Model). It has been modified by the District so that it may be used conjunctively with CUHP. In 2005 the District adopted the use of ***EPA's SWMM 5.0*** model and recommends its use for all future hydrology studies.

The purpose of the discussion of SWMM in this chapter is to provide general background on the use of the model with ***CUHP 2005*** software to perform more complex stormwater runoff calculations using SWMM. Complete details about this model's use, specifics of data format and program execution is provided in the users' manual for SWMM 5.0. Software, users manual and other information about EPA's SWMM 5.0 may be downloaded from http://www.epa.gov/ednnrmrl/models/swmm/index.htm.

4.1 Software Description

SWMM represents a watershed by an aggregate of idealized runoff planes, channels, gutters, pipes and specialized units such as storage nodes, outlets, pumps, etc. The program can accept rainfall hyetographs and make a step-by-step accounting of rainfall infiltration losses in pervious areas, surface retention, overland flow, and gutter flow leading to the calculation of hydrographs. However, this portion of the model is normally not used by the District because the calculation of hydrographs for each sub-catchment is typically carried out using the CUHP software. If, however, the user wants to use SWMM to calculate runoff, the model must be calibrated against the CUHP calculations for the same watershed being studied.

After the CUHP 2005 software is used to calculate hydrographs from a number of sub-watersheds, the resulting hydrographs from these sub-watersheds can be combined and routed through a series of links (i.e., channels, gutters, pipes, dummy links, etc.) and nodes (i.e., junctures, storage, diversion, etc.) to compute the resultant hydrographs at any number of design points within the watershed.

4.1.1 Surface Flows and Flow Routing Features

Stormwater runoff hydrographs generated using CUHP 2005 can be routed through a system of stormwater conveyance, diversion, storage, etc. elements of a complex urban watershed. In setting up the SWMM model, it is critical that overflow links for storm sewers and diversion junctions are provided in the model. The combination of these allows the user to model flows accurately when pipes and/or smaller channels that do not have the capacity to convey higher flows, at which time the excess flows are diverted to the overflow channels and a “choking” of the flow is avoided and errors in the calculated peak flow values downstream are prevented.

There are several types of conveyance elements that one can select from a menu in SWMM. One element that is now available, that was not available in older versions, is a user-defined irregular channel cross-section, similar to the way cross-sections are defined in HEC-RAS. This makes the model very flexible in modeling natural waterways and composite man-made channels. For a complete description of the routing elements and junction types available for modeling see the SWMM User Manual published by EPA and available from their web site mentioned earlier.

4.1.2 Flow Routing Method of Choice

The District recommends the use of *kinematic wave routing* as the "routing" option in SWMM for planning purposes. *Dynamic wave routing* for most projects is not necessary, does not improve the accuracy of the runoff estimates and can be much more difficult to implement because it requires much information to describe, in minute detail, the entire flow routing system. In addition, it has tendencies to go unstable when modeling some of the more complex elements and/or junctions. When planning for growth, much of the required detail may not even be available (e.g., location of all drop structures and their crest and toe elevations for which a node has to be defined in the model). In addition, with dynamic routing setting up of overflow links and related nodes is much more complicated and exacting.

The use of dynamic wave routing is appropriate when evaluating complex exiting elements of a larger system. It is an option that can also offer some advantages in final design and its evaluation, as it provides hydraulic grade lines and accounts for backwater effects.

4.2 Data Preparation for the SWMM Software

Use of SWMM requires three basic steps:

Step 1—Identify or define the geometries watershed, sub-watersheds and routing/storage elements.
Step 2—Estimates of roughness coefficients and functional/tabular relationships for storage and other special elements.
Step 3—Prepare input data for the model.

4.2.1 Step 1—Method of Discretization

Discretization is a procedure for the mathematical abstraction of the watershed and of the physical drainage system. Discretization begins with the identification of drainage area boundaries, the location of storm sewers, streets, and channels, and the selection of those routing elements to be included in the system. For the computation of hydrographs, the watershed may be conceptually represented by a network of hydraulic elements (i.e., sub-catchments, gutters, pipes, etc.) Hydraulic properties of each element are then characterized by various parameters such as size, slope, and roughness coefficient.

4.2.2 Step 2—Estimate Coefficients and Functional/Tabular Characteristic of Storage and Outlets

For hydrologic routing through conveyance elements such as pipes, gutters, and channels, the resistance (Manning's n) coefficients should not necessarily be the same as those used in performing hydraulic design calculations. As a general rule, it was found that increasing the "typical" values of Manning's n by approximately 25 percent was appropriate when using UDSWM in the past and should be appropriate for use in SWMM as well. Thus, if a pipe is estimated to have n = 0.013 for hydraulic calculations, it is appropriate to use n = 0.016 in SWMM.

When modeling the hydrologic routing of natural streams, grass-lined channels, or riprap-lined channels in Colorado, it is recommended that Manning's n be estimated for SWMM using Equation RO-10 (Jarrett 1984 and 1985).

$$n = 0.393\, S^{0.38} R^{-0.16} \qquad \text{(RO-10)}$$

in which:

n = Manning's roughness coefficient

S = friction slope (ft/ft)

R = hydraulic radius (ft)

To estimate the hydraulic radius of a natural, grass-lined, or riprap-lined channel for Equation RO-10, it is suggested that one half of the estimated hydrograph peak flow be used to account for the variable depth of flow during a storm event.

SWMM does not have built-in shapes that define geometries of gutters or streets. The user can use the irregular shape option to define the shape of the gutter and street. For storage junctions, the user can define relationships such as stage vs. storage-surface area using mathematical functions or tables. For storage outlets or downstream outfalls, the user can use tables or functions to define their stage-discharge characteristics. As and alternative, the user can define geometries and characteristic for weirs and orifices and let the program calculate the functional relationships. Use of the weirs can sometimes be particularly troublesome when the dynamic wave routing option is used.

4.2.3 Step 3—Preparation of Data for Computer Input

The major preparation effort is forming a tree structure of all the runoff and conveyance elements and dividing the watershed into sub-watersheds. The conveyance elements network is developed using a watershed map, subdivision plans, and "as-built" drawings of the drainage system. Pipes with little or no backwater effects, channels, reservoirs, or flow dividers are usually designated as conveyance elements for computation by SWMM. Once the conveyance element system is set and labeled, CUHP 2005 is used to generate an output file that contains runoff hydrograph for all sub-watersheds. This file is called in by SWMM as an external inflow file and the hydrograph data is then routed by SWMM. The reader needs to study the SWMM users' manual for complete details about data input preparation.

5.0 OTHER HYDROLOGIC METHODS

5.1 Published Hydrologic Information

The District has prepared hydrologic studies for the majority of the major drainageways within District boundaries. These studies contain information regarding peak flow and runoff volume from the 2-year through 100-year storm events for numerous design points within the watershed. They also contain information regarding watershed and sub-watershed boundaries, soil types, percentage imperviousness, and rainfall. The studies are available at the District library. When published flow values are available from the District for any waterway of interest, these values should be used for design unless there are compelling reasons to modify the published values.

5.2 Statistical Methods

Statistical analysis of measured streamflow data is also an acceptable means of hydrologic analysis in certain situations. Statistical analysis should be limited to streams with a long period of flow data (30 years as a recommended minimum) where there have been no significant changes in land use in the tributary watershed during the period of the flow record. It should be recognized that there is no good way to extrapolate calculated flow from a statistical analysis to estimate the flow for expected future watershed development conditions.

6.0 SPREADSHEETS AND OTHER SOFTWARE

District provides following freeware to help with the calculations and protocols in this *Manual*. All of these can be found on the District's Web site (www.udfcd.org) under Downloads, Technical or Software.

The Colorado Urban Hydrograph Procedure has been computerized and is loaded using macro-driven spreadsheet. The software package is titled **CUHP 2005 Version x.x.x,** and includes a Converter to converts older version CUHP files and UDSWM files into CUHP 2005 and EPA's SWMM 5.0 formats.

A spreadsheet has been prepared to facilitate runoff calculations using the Rational Method, namely, **UD-Rational** (Guo 1995). Inputs needed include catchment area, runoff coefficient, 1-hour point rainfall depth, and flow reach characteristics (length, slope, and type of ground surface). The spreadsheet then calculates the peak runoff flow rate in cfs.

Storm sewers may be designed using the Rational Method with the aid of GUI-based software **Neo UD-Sewe**r. This software will pre-size storm sewers using the same input mentioned for UD-Rational, except that it permits definition of existing sewer link and that it also checks to insure that the most critical portions of the catchment are being accounted for in sizing the sewers. After the sewers are sized, or if you have an existing system, it can be used to analyze the hydraulic and energy grade lines of the system. A recent update includes a feature to generate a profile plot of the sewer, ground line, hydraulic grade line and energy grade line.

UD-RainZone is a spreadsheet that help the user find the Intensity-Duration-Frequency curve for any region in Colorado based on site elevation.

UD-Raincurve is a spreadsheet that helps the user develop design storm distributions for use with CUHP or other models based on the protocols described in this *Manual*. It will generate design storm hyetographs for small catchments (i.e., < 5 sq. mi.) all the way up to ones that are 75 sq. mi. in size, using area correction factors for the latter.

Latest release of the EPA SWMM 5.0 software is available for downloading from EPA's web site at (http://www.epa.gov/ednnrmrl/models/swmm/index.htm)

It is recommended that the users of these software check for updates on regular basis. Corrections of discovered bugs and enhancements are constantly under development and are posted as they are completed.

7.0 EXAMPLES

7.1 Rational Method Example 1

Find the 100-year peak flow rate for a 60-acre catchment in an undeveloped grassland area located in Section 13, R65W, T1S. The upper 400 feet of the catchment is sloped at 2%, the lower 1,500 feet is grassed waterway that is sloped at 1%. The area has type C soils.

From Figure RA-6, the 1-hour point precipitation value is 2.7 inches. From Table RO-3, in the category “Undeveloped Areas, historic flow analysis,” a percent impervious value of 2% (or 0.02) is selected.

Determine C_5 from Equation RO-7:

$$C_5 = \left(-0.10\,(0.02) + 0.11\right) + 0.858(0.02)^3 - 0.786\,(0.02)^2 + 0.774\,(0.02) + 0.04$$

$$= 0.16$$

Determine t_i from Equation RO-3:

$$t_i = \frac{0.395(1.1 - 0.16)\sqrt{400}}{(0.02)^{0.33}}$$

$= 27.0$ minutes

Find t_t:

$$t_t = \frac{L}{60V}$$

From Table RO-2, for a grassed waterway, $C_V = 15$

From Equation RO-4:

$$V = 15\,(0.01)^{0.5}$$

$= 1.5$ ft/sec

Find t_t:

$$t_t = \frac{1{,}500}{1.5 \cdot 60}$$

$= 16.67$ minutes

From Equation RO-2:

$t_c = 27.0 + 16.67$

$= 43.67$ minutes

Use 44 minutes

Determine C_{100} from Equation RO-7:

$$C_{100} = (-0.39(0.02) + 0.46) + 0.858(0.02)^3 - 0.786(0.02)^2 + 0.774(0.02) + 0.04$$

$= 0.51$

Determine rainfall intensity, I, from Equation RA-3:

$$I = 28.5 \cdot 2.7/(10 + 44)^{0.786}$$

$= 3.35$ in/hr

Determine Q from Equation RO-1:

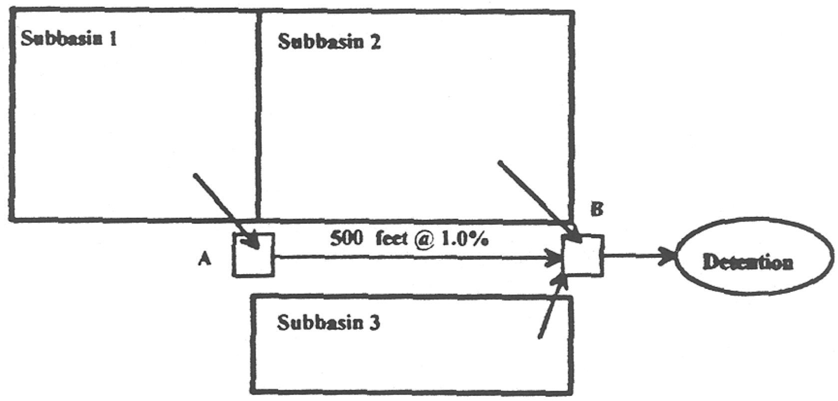
$$Q = 0.51 \cdot 3.35 \cdot 60$$

$= 102$ cfs

Alternately, use the runoff spreadsheet to calculate the peak flow rate as shown.

7.2 Rational Method Example 2

A watershed is divided into three subbasins in the City of Denver. The drainage system is designed to collect Subbasin 1 at Point A, and Subbasins 2 and 3 at Point B, and then drains into a detention system. Determine the 10-year peak discharge at Point B using the watershed parameters summarized in the table.



Subbasin	Drainage Area A (acres)	Runoff Coefficient C	Time of Concentration T_c (minutes)
1.00	2.00	0.55	15.00
2.00	5.00	0.65	22.00
3.00	1.50	0.81	12.00

As shown in the figure, there are three flow paths to reach Point B. Their flow times are:

1. From Subbasin 2: T_2 = 22 minutes
2. From Subbasin 3: T_3 = 12 minutes
3. From Subbasin 1: The flow time includes the time of concentration of Subbasin 1, and the flow time from Point A to Point B through the street. According to the SCS upland method, the conveyance parameter for the paved gutter flow is 20.0. The flow time from Subbasin 1 to Point B is the sum of the time of concentration of Subbasin 1 and the flow time through the 500-foot gutter as:

$$T_i = 15 + \frac{500}{60 \cdot 20 \cdot \sqrt{0.01}} = 19.17 \text{ minutes}$$

At Point B, the design rainfall duration T_d = max (T_1, T_2, T_3) = 22 minutes.

The 10-year design rainfall intensity for Denver is:

$$I = \frac{28.5 \cdot 1.61}{(10 + 22)^{0.786}} = 3.01 \text{ in/hr}$$

The total effective area at Point B is:

$$A_e = 0.81 \cdot 1.50 + 0.55 \cdot 2.0 + 0.65 \cdot 5.0 = 5.565 \text{ acres}$$

The 10-year peak discharge is:

$$Q = IA_e = 16.75 \text{ cfs}$$

7.3 Effective Rainfall Example

Calculate the effective rainfall from a 1.6-inch storm for a catchment that is 40% impervious. Sixty percent of the impervious area flows into pervious areas. Half of the pervious area receives flow from the impervious area. The depression losses are 0.1 inches for impervious areas and 0.3 inches for pervious areas.

Calculations are included in Table RO-9.

Table RO-9—Effective Rainfall Calculations

One Hour Precipitation	2.65	Inches
Return Period	100	years

NRCS Hydrologic Soil Type	C or D	
Final Infiltration Rate	0.5	in/hr
Initial Infiltration Rate	3.0	in/hr
Decay Coefficent	0.00180	1/sec

Impervious Fraction- IA	40%	
Impervious Area Depression Loss	0.1	Inches
Directly Connected % of Total Impervious Area (% CIA)	80%	
Unconnected % of Total Impervious Area (% UIA)	20%	

Pervious Fraction- PA	60%	
Pervious Area Depression Loss	0.3	Inches
Separate Pervious % of Total Perervious Area (% SPA)	40%	
Receiving Pervious % of Total Pervious Area (% RPA)	60%	

		Impervious Area						Pervious Area									
								Horton's Infiltration		SPA- Separate Pervious Area			RPA- Receiving Pervious Area				
Time (minutes) (1)	Incremental Precipitation (2)	Depression Storage (3)	Five Percent Loss (4)	Effective Precipitation (5)	Percent Effective Precipitation (6)	% DCI Effective Precipitation (7)	% UIA Effective Precipitation (8)	Value at Midpoint (9)	Incremental Infiltration Actual Depth (10)	Depression Storage (11)	Effective Precipitation (12)	% Effective Precipitation (13)	Combined Precepitaion Depth (14)	Depression Storage (15)	Effective Precipitation (16)	% Effective Precipitation (17)	Total Effective Precipitation (18)
0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
5	0.027	0.027	0.000	0.000	0.000	0.000	0.000	2.408	0.201	0.000	0.000	0.000	0.027	0.000	0.000	0.000	0.000
10	0.080	0.074	0.000	0.006	0.002	0.002	0.000	1.612	0.134	0.000	0.000	0.000	0.080	0.000	0.000	0.000	0.002
15	0.122	0.000	0.006	0.116	0.046	0.037	0.009	1.148	0.096	0.026	0.000	0.000	0.131	0.035	0.000	0.000	0.037
20	0.212	0.000	0.011	0.201	0.081	0.064	0.016	0.878	0.073	0.139	0.000	0.000	0.228	0.155	0.000	0.000	0.064
25	0.371	0.000	0.019	0.352	0.141	0.113	0.028	0.720	0.060	0.135	0.176	0.042	0.399	0.110	0.230	0.083	0.238
30	0.663	0.000	0.033	0.629	0.252	0.201	0.050	0.628	0.052	0.000	0.610	0.146	0.713	0.000	0.660	0.238	0.586
35	0.371	0.000	0.019	0.352	0.141	0.113	0.028	0.575	0.048	0.000	0.323	0.078	0.399	0.000	0.351	0.126	0.317
40	0.212	0.000	0.011	0.201	0.081	0.064	0.016	0.544	0.045	0.000	0.167	0.040	0.228	0.000	0.183	0.066	0.170
45	0.164	0.000	0.008	0.156	0.062	0.050	0.012	0.525	0.044	0.000	0.121	0.029	0.177	0.000	0.133	0.048	0.127
50	0.133	0.000	0.007	0.126	0.050	0.040	0.010	0.515	0.043	0.000	0.090	0.022	0.143	0.000	0.100	0.036	0.098
55	0.106	0.000	0.005	0.101	0.040	0.032	0.008	0.509	0.042	0.000	0.064	0.015	0.114	0.000	0.072	0.026	0.073
60	0.106	0.000	0.005	0.101	0.040	0.032	0.008	0.505	0.042	0.000	0.064	0.015	0.114	0.000	0.072	0.026	0.073
65	0.106	0.000	0.005	0.101	0.040	0.032	0.008	0.503	0.042	0.000	0.064	0.015	0.114	0.000	0.072	0.026	0.074
70	0.053	0.000	0.003	0.050	0.020	0.016	0.004	0.502	0.042	0.000	0.011	0.003	0.057	0.000	0.015	0.005	0.024
75	0.053	0.000	0.003	0.050	0.020	0.016	0.004	0.501	0.042	0.000	0.011	0.003	0.057	0.000	0.015	0.006	0.024
80	0.032	0.000	0.002	0.030	0.012	0.010	0.002	0.501	0.042	0.000	0.000	0.000	0.034	0.000	0.000	0.000	0.010
85	0.032	0.000	0.002	0.030	0.012	0.010	0.002	0.500	0.042	0.000	0.000	0.000	0.034	0.000	0.000	0.000	0.010
90	0.032	0.000	0.002	0.030	0.012	0.010	0.002	0.500	0.042	0.000	0.000	0.000	0.034	0.000	0.000	0.000	0.010
95	0.032	0.000	0.002	0.030	0.012	0.010	0.002	0.500	0.042	0.000	0.000	0.000	0.034	0.000	0.000	0.000	0.010
100	0.032	0.000	0.002	0.030	0.012	0.010	0.002	0.500	0.042	0.000	0.000	0.000	0.034	0.000	0.000	0.000	0.010
105	0.032	0.000	0.002	0.030	0.012	0.010	0.002	0.500	0.042	0.000	0.000	0.000	0.034	0.000	0.000	0.000	0.010
110	0.032	0.000	0.002	0.030	0.012	0.010	0.002	0.500	0.042	0.000	0.000	0.000	0.034	0.000	0.000	0.000	0.010
115	0.032	0.000	0.002	0.030	0.012	0.010	0.002	0.500	0.042	0.000	0.000	0.000	0.034	0.000	0.000	0.000	0.010
120	0.032	0.000	0.002	0.030	0.012	0.010	0.002	0.500	0.042	0.000	0.000	0.000	0.034	0.000	0.000	0.000	0.010
Totals:	3.063	0.100	0.148	2.815	1.126	0.901	0.225	16.574	1.381	0.300	1.700	0.408	3.289	0.300	1.903	0.685	1.994

(0) All values in inches of rainfall unless otherwise specified
(1) Enter time T in Minutes
(2) Enter Rainfall Hyetograph
(3) = (2) Until cumulative sum of column (3) equals the user entered impervious depression losses
(4) = 0.05 * ((2) -(3))
(5) = (2) - (3) - (4)
(6) = (5) * IA
(7) = (6) * CIA
(8) = (6) * UIA
(9) Horton's infiltration value using equation RO-8 at the midpoint of the last interval, as described on page RO-22
(10) Actual infiltraiton over the perceeding time increment, but not exceeding precipitation depth.
(11) = MAX (0.0, (2)-(10)) Until the cumulative sum of (10) equals the pervious depression loss
(12) = MAX (0.0, (2)-(10)-(11))
(13) = (12) * SPA * PA
(14) = (2)+(8)
(15) = MAX(0.0, (14) - (10) - (15)) Until the cumulative sum of (15) equals the pervious depression loss
(16) = MAX (0.0, (14)-(10)-(15))
(17) = (16) * RPA * PA
(18) = (7)+(13)+(17)

8.0 REFERENCES

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Urbonas, B., J.C.Y. Guo, and L.S. Tucker. 1990. Optimization Stormwater Quality Capture Volume. In *Urban Stormwater Quality Enhancement—Source Control, Retrofitting and Combined Sewer Technology*, ed. H.C. Torno, 94-110. New York: ASCE.

APPENDIX A - DETAILS OF THE COLORADO URBAN HYDROGRAPH PROCEDURE (CUHP)

For watersheds that are larger than 90 acres, the District recommends that the design storm runoff be analyzed by deriving synthetic unit hydrographs. Sherman originally developed the unit hydrograph principle in 1932. Snyder developed the synthetic unit hydrograph, which is used for analysis when there are no rainfall-runoff data for the basin under study, as is often the case in the Denver region, in 1938. The presentation given in this chapter is termed CUHP because coefficients and the form of the equation are based upon data collected in the Denver region of Colorado and on studies conducted or financed by the District. The U.S. Geological Survey (USGS) collected the data for use in the development of the 1982 version of CUHP between 1969 and 1981 under a cooperative agreement with the District. Data collection activities are continuing under a similar cooperative agreement between the District and USGS; however, the number of stations has been reduced. The goal of the currently ongoing data collection effort is to develop a long-term database for further refinements to the hydrologic techniques in the Denver region.

A.1 Definition

A unit hydrograph is defined as the hydrograph of one inch of direct runoff from the tributary area resulting from a unit storm. The unit hydrograph thus represents the integrated effects of factors such as tributary area, shape, street pattern, channel capacities, and street and land slopes.

The basic premise of the unit hydrograph is that individual hydrographs resulting from the successive increments of rainfall excess that occur throughout a storm period will be proportional in discharge throughout their runoff period. Thus, the hydrograph of total storm discharge is obtained by summing the ordinates of the individual sub-hydrographs.

A.2 Basic Assumptions

The derivation and application of the unit hydrograph are based on the following assumptions:

1. The rainfall intensity is constant during the storm that produces the unit hydrograph.
2. The rainfall is uniformly distributed throughout the whole area of the watershed.
3. The time duration of the unit hydrograph resulting from an effective rainfall of unit duration is constant.
4. The ordinates of the design runoff with a common unit time are directly proportional to the total amount of direct runoff represented by each sub-hydrograph.
5. The effects of all physical characteristics of a given watershed, including shape, slope, detention,

infiltration, drainage pattern, channel storage, etc., are reflected in the shape of the unit hydrograph for that watershed.

A.3 Equations

There are four basic equations used in defining the limits of the synthetic unit hydrograph. The first equation defines the lag time of the basin in terms of time to peak, t_p, which, for the CUHP method, is defined as the time from the center of the unit duration storm to the peak of the unit hydrograph as shown in Figure RO-A1.

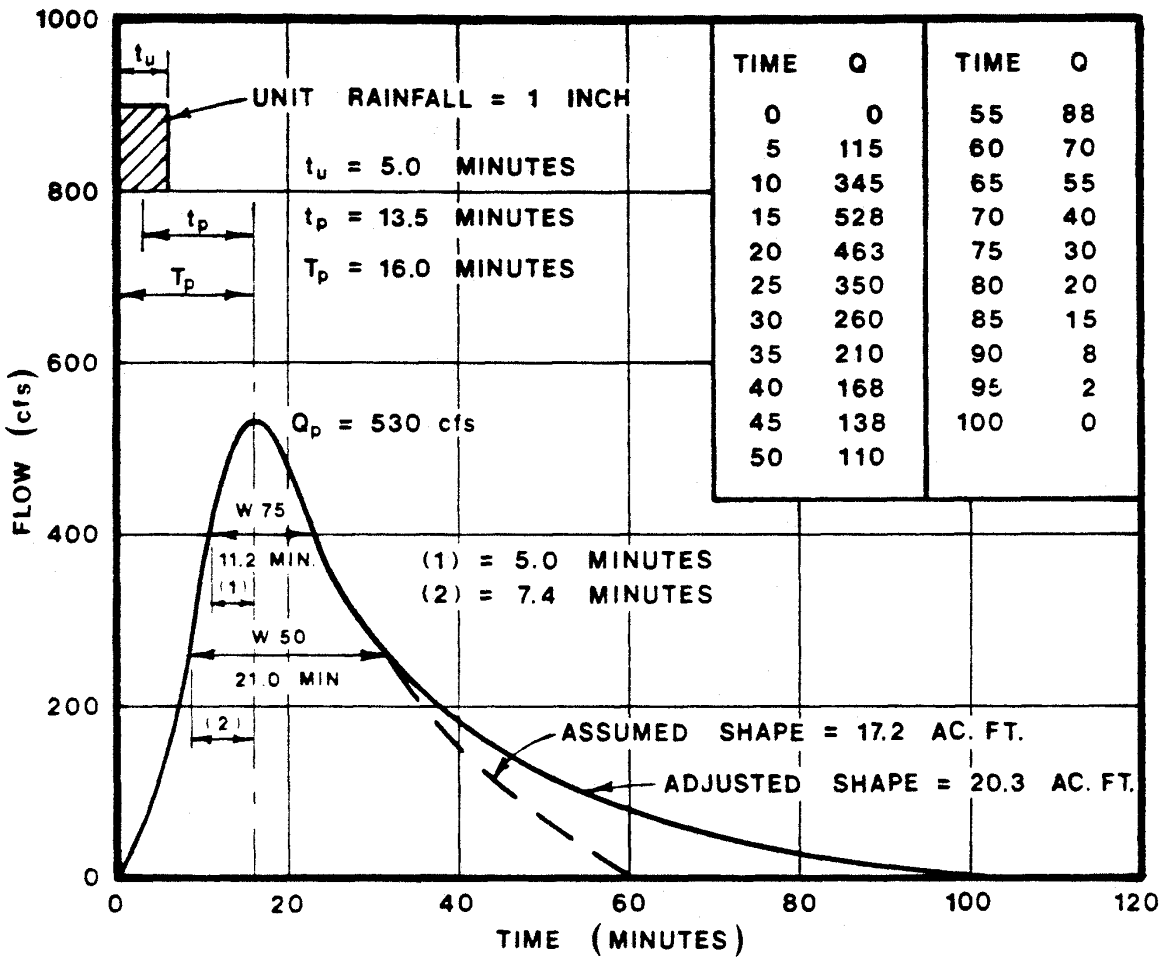


Figure RO-A1—Example of Unit Hydrograph Shaping

$$t_p = C_t \left(\frac{L L_{ca}}{\sqrt{S}} \right)^{0.48} \qquad \text{(RO-A1)}$$

in which:

t_p = time to peak of the unit hydrograph from midpoint of unit rainfall in hours

L = length along stream from study point to upstream limits of the basin in miles

L_{ca} = length along stream from study point to a point along stream adjacent to the centroid of the basin in miles

S = weighted average slope of basin along the stream to upstream limits of the basin in feet per foot

C_t = coefficient reflecting time to peak

The time from the beginning of unit rainfall to the peak of the unit hydrograph is determined by:

$$T_p = 60t_p + 0.5t_u \qquad \text{(RO-A2)}$$

in which:

T_p = time from beginning of unit rainfall to peak of hydrograph in minutes

t_u = time of unit rainfall duration in minutes

The unit peak of the unit hydrograph is defined by:

$$q_p = \frac{640C_p}{t_p} \qquad \text{(RO-A3)}$$

in which:

q_p = peak rate of runoff in cfs per square mile

C_p = coefficient related to peak rate of runoff

Once q_p is determined, the peak of the unit hydrograph for the basin is computed by:

$$Q_p = q_p A \qquad \text{(RO-A4)}$$

in which:

Q_p = peak of the unit hydrograph in cfs

A = area of basin in square miles

A.4 Unit Storm Duration

For most urban studies, the unit storm duration, t_u, should be 5 minutes. However, the unit duration may be increased for larger watersheds. It is convenient to have the unit duration incremented in multiples of 5 minutes (i.e., 10 or 15 minutes) with the maximum unit duration recommended at 15 minutes.

An acceptable unit storm duration, whenever it is larger than 5 minutes, should not exceed one-third of t_p. As an example, if the watershed has a t_p = 35 minutes, then an appropriate unit storm duration would be 5 minutes or 10 minutes (i.e., less than or equal to 1/3 t_p).

A.5 Watershed Size Limits

The rainfall-runoff data used in the development of the current version of CUHP were obtained primarily from small watersheds that ranged from 0.15 square miles to 3.08 square miles. Although some extrapolation is justified, unlimited extrapolation of how the watershed responds to rainfall is not. It is recommended that the maximum size of a watershed to be analyzed with a single unit hydrograph be limited to 5 square miles. Whenever a larger watershed needs to be studied, it is suggested it be subdivided into sub-watersheds of 5 square miles or less and individual sub-watershed storm hydrographs be routed downstream using appropriate channel routing procedures such as SWMM. The routed hydrographs then need to be added to develop a single composite storm hydrograph.

Because of the way a unit hydrograph responds, it is also suggested that the minimum watershed size be 90 acres. The 5-minute unit hydrograph procedure may be used for a smaller watershed provided t_p is greater than 10 minutes.

A.6 Watershed Shape Limits

The watershed shape can have a profound effect on the final results. It affects and suggested limitations in the coding of individual watersheds is discussed in Pragraph 3.3.3 in the main body of this Runoff Chapter.

A.7 Watershed Slope Limits and Considerations

The current version of CUHP was developed using data from watersheds having a range of major drainageway slopes between 0.005 ft/ft and 0.037 ft/ft. Caution must be used when extrapolating beyond this range.

A.7.1 Natural and Grass-Lined Waterways

In natural and grass-lined drainageways, channels become unstable when a Froude Number of 1.0 is approached. There are natural processes at work that limit the time to peak of a unit hydrograph as the drainageway becomes steeper. To account for this phenomenon, it is recommended that the slope used in Equation RO-9 for natural drainageways and existing manmade grass-lined channels be adjusted using Figure RO-10.

A.7.2 Grass-Lined Channels

Grass-lined channels designed and built using District criteria have a slope that limits maximum flow velocities. A typical range in longitudinal slopes for such channels is 0.003 ft/ft to 0.006 ft/ft. It is recommended that for preliminary estimating purposes a longitudinal slope of 0.005 ft/ft be used for grass-lined channels that are to be designed using District criteria.

A.7.3 Riprap-Lined Channels

The District's criteria also limit the Froude Number to less than 0.8 for riprap-lined channels. For this reason it is suggested that, for preliminary estimating purposes where riprap channels are contemplated, a longitudinal slope of 0.01 be used with Figure RO-10. When a riprap channel is in existence, use the measured average channel profile slope.

A.7.4 Concrete Channels and Storm Sewers

In concrete-lined channels and buried conduits, the velocities can be very high. For this reason, it is recommended that the average ground slope (i.e., not flow-line slope) be used where concrete-lined channels and/or storm sewers dominate the watershed drainageways. There is no upper limit recommended to the slope for such watersheds.

A.7.5 Weighted Watershed Slope

Where the flow-line slope varies along the channel, calculate a weighted basin slope for use with Equation RO-9. Do this by first segmenting the major drainageway path into reaches having similar longitudinal slopes. Calculate the weighted slope using Equation RO-9.

A.8 Watershed Land Use Consideration

A lumped parameter model such as CUHP relies on data from watersheds having relatively uniform land use. It is recommended that watersheds having zones of differing land use be subdivided into sub-watersheds having relatively uniform land use. As an example, if the lower half of a watershed has been urbanized and the upper half is to remain as open space, it is best to develop two distinct hydrographs. The upper sub-watershed hydrograph will be based on the coefficients for undeveloped land, and the lower sub-watershed hydrograph will be the result of coefficients for the developed area.

A.9 Determination of C_t and C_p Coefficients

The value of C_t in Equation RO-A1 may be determined using Figure RO-A2. Note that the curve in Figure RO-A2 can be represented using parabolic equations having the percent imperviousness, I_a, as an independent variable.

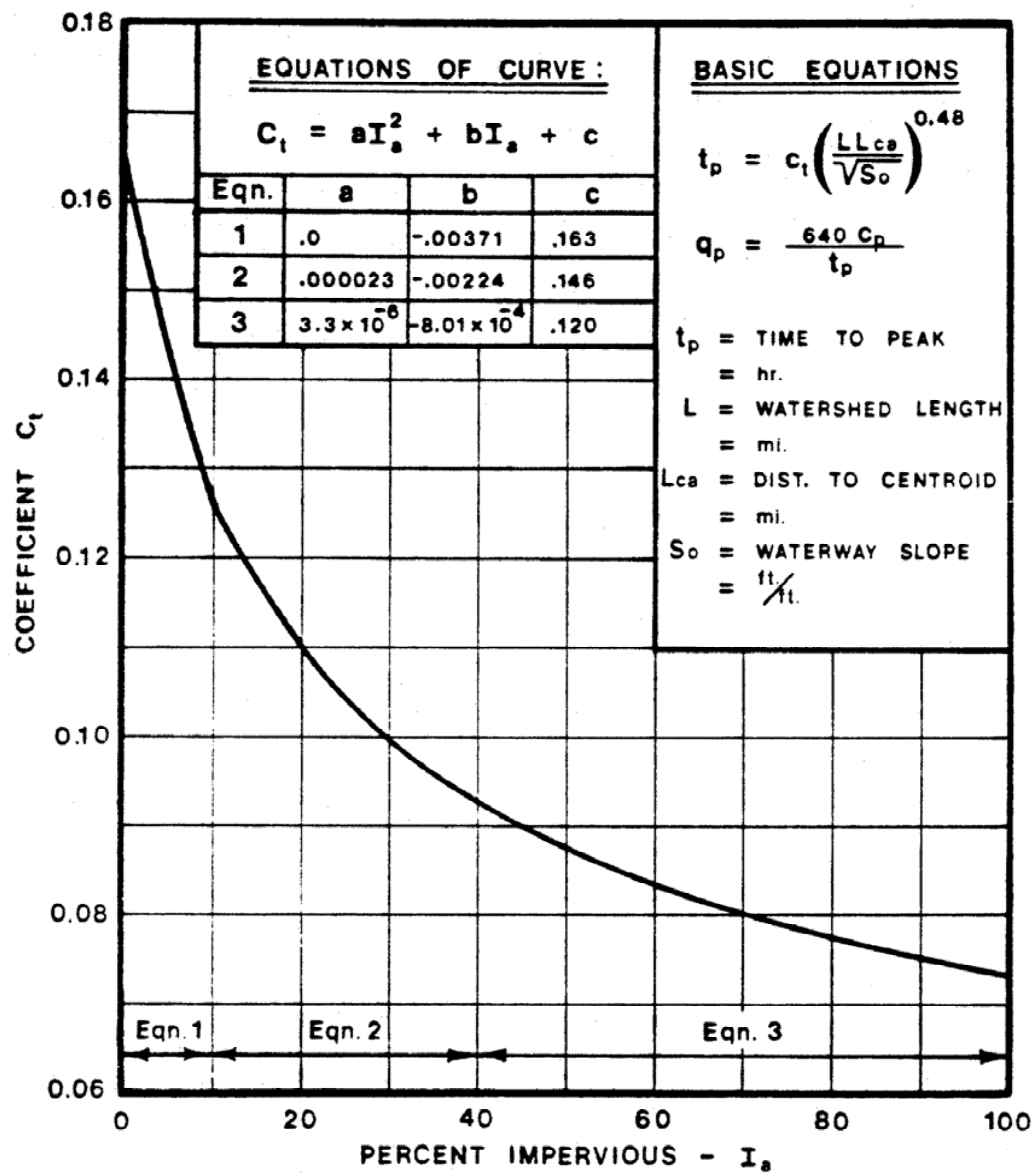


Figure RO-A2—Relationship Between C_t and Imperviousness

The value of C_p to be used in Equation RO-A3 may be determined using Figure RO-A3. The curve in Figure RO-A3 is also represented with a parabolic equation. To determine C_p, first obtain the value of the Peaking Parameter, P, from Figure RO-A3. Then calculate C_p using Equation RO-A5.

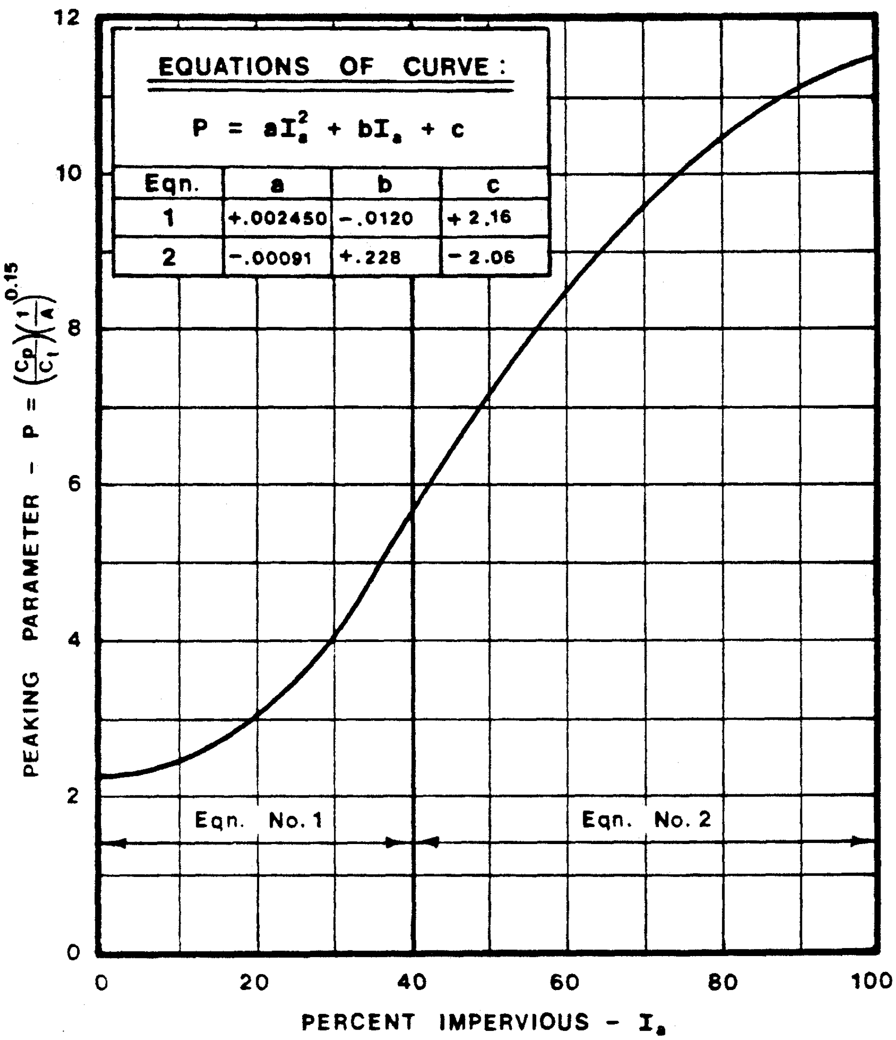


Figure RO-A3—Relationship Between Peaking Parameter and Imperviousness

$$C_p = PC_t A^{0.15} \tag{RO-A5}$$

in which:

P = peaking parameter from Figure RO-A3

C_t = coefficient from Figure RO-A2

A = basin area in square miles

A.10 Unit Hydrograph Shape

The shape of the unit hydrograph is a function of the physical characteristics of the watershed. It incorporates the effects of watershed size, shape, degree of development, slope, type, and size of drainage system, soils, and many other watershed factors. The shape of the unit hydrograph is also dependent on the temporal and spatial distribution of rainstorms and will vary with each storm event. As a result, a unit hydrograph based on rainfall-runoff data is an approximation that provides the engineer or hydrologist with a reasonable unit hydrograph shape for a given hydrologic region and land development practices.

Equations RO-A1 through RO-A5 are used to define the peak discharge and its location for the unit hydrograph. The widths of the unit hydrograph at 50% and 75% of the peak can be estimated using Figure RO-A4. Note that the unit hydrograph widths at 50% and 75% of the peak are given in hours. The two equations shown on Figure RO-A4 mathematically describe the two lines on the figure.

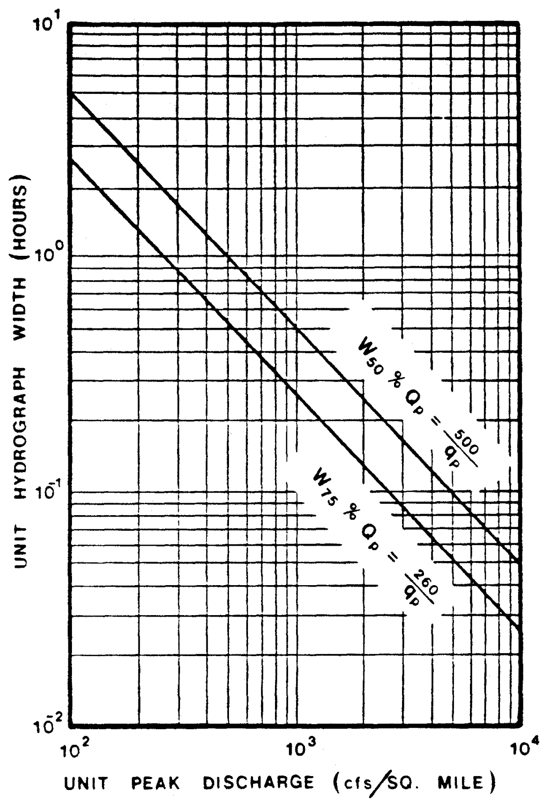
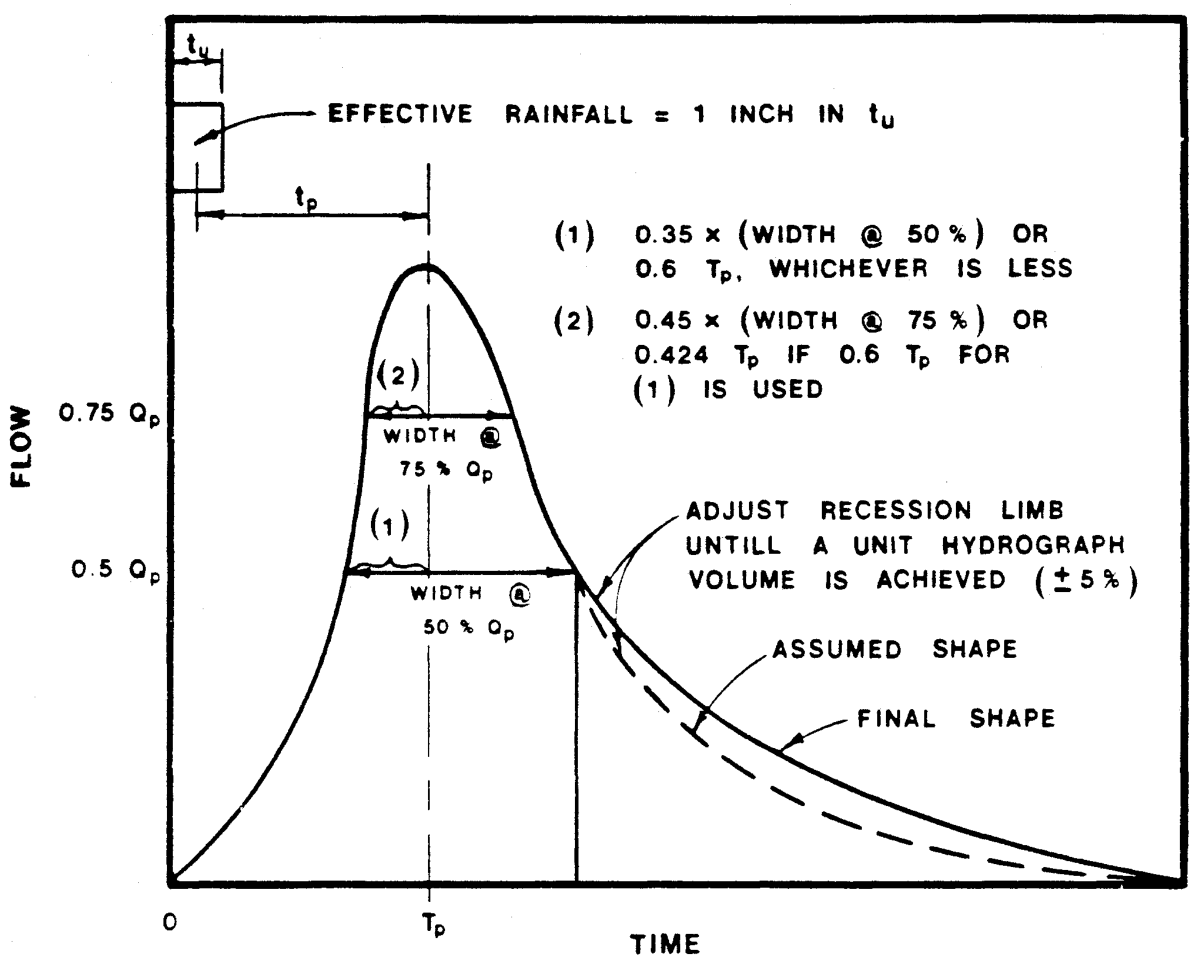


Figure RO-A4—Unit Hydrograph Widths

In addition to knowing the location of the unit hydrograph peak and its width at two points on its ordinate, it also helps to know how to distribute the two widths around the peak. A study of many unit hydrographs generated using recorded rainfall and runoff events indicates that, as a general rule, 0.35 of the width at 50% of peak is to the left of the peak and 0.65 of the width is to the right of the peak. At 75% of the peak, 0.45 of the width is left of the peak and 0.55 of the width is to the right of the peak. However, on some hydrographs this rule needs to be modified. Whenever the above rule results in the hydrograph at 50% of peak being to the left of the peak by more than $0.6T_p$ (T_p = the distance from zero to the peak of the unit hydrograph); the x coordinate at 50% of peak should be placed at $0.6T_p$, and at 75% of the peak it should be placed at $0.424T_p$. Figure RO-A5 shows how a typical unit hydrograph may be shaped to best approximate the trends found in the rainfall-runoff data.

Figure RO-A5—Unit Hydrograph

A.11 Conceptual Relationships for Directly Connected Imperviousness Modeling



In 1995, the CUHP computer model was modified to recognize the effects of directly connected impervious areas on excess precipitation and its response in calculating runoff volumes and peaks.

It is possible to conceptualize any urban catchment as having four separate surface runoff components:

1. Impervious area directly connected to the drainage system (DCIA).

2. Impervious area that drains onto or across impervious surfaces (UIA).

3. The pervious area receiving runoff from impervious portions (RPA).

4. The separate pervious area (SPA) not receiving runoff from impervious surfaces.

This concept is illustrated in Figure RO-A6. To model the excess precipitation process and the losses to it that occur within an urban catchment, the following variables were defined and used in the CUHPF/PC version of the program:

IA = total impervious fraction

PA = total pervious fraction

PIA = effective precipitation from impervious fraction*

PPA = effective precipitation from impervious fraction*

CIA = directly (hydraulically) connected impervious fraction

ICIA = indirectly connected impervious area

UIA = unconnected impervious fraction

RPA = receiving pervious fraction

SPA = separate pervious fraction

D = CIA/IA, fraction of impervious area directly connected to drainage system

R = RPA/PA, fraction of pervious area receiving disconnected impervious runoff

K = UIA/RPA, the ratio of unconnected impervious area to pervious receiving area

* Effective precipitation before adjustment

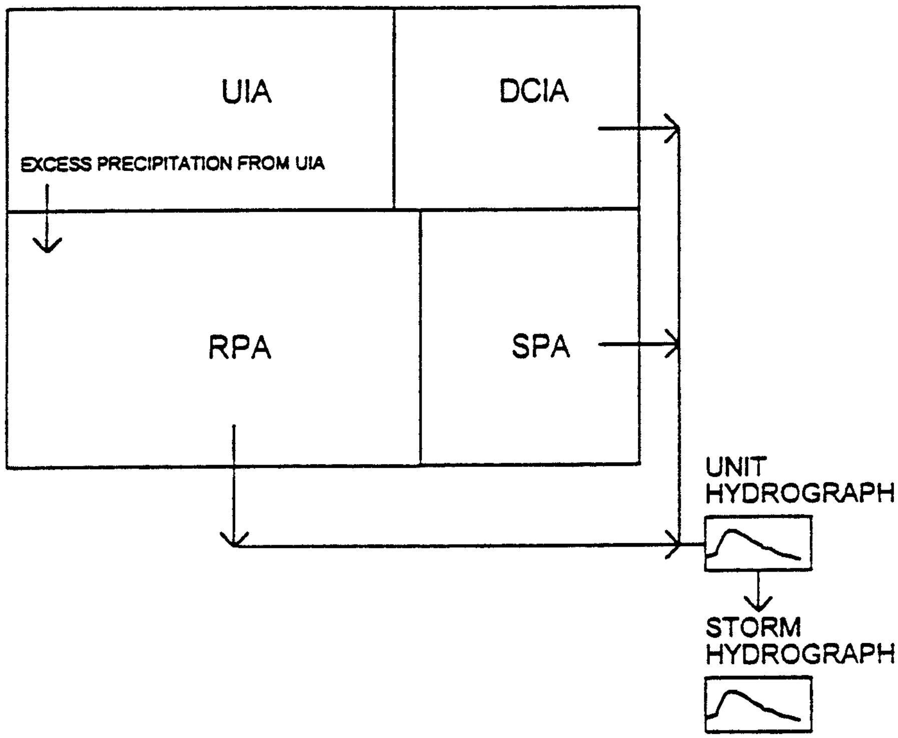


Figure RO-A6—Runoff Flow Diagram for the CUHPF/PC Model

The CUHP model was modified in 1995 to account for the effects on runoff from impervious areas that first travel over pervious areas before entering a drainageway. This was done to estimate the effects on runoff rates and volumes of intentionally routing flow onto pervious areas. Based on field observations, experience, and a lot of assumptions, a set of values was developed defining how much of the total impervious area is likely to drain onto the catchment's impervious area. Likewise, default estimates of how much of the pervious area is likely to receive the "disconnected" impervious drainage were developed. These were then incorporated as default values into the CUHPF/PC model. The flow chart shown in Figure RO-A7 illustrates the concept in more detail.

The default relationships for D, the ratio of directly connected impervious area to the total impervious area, and R, the ratio of pervious area receiving runoff from impervious areas to the total pervious area of the catchment, as a function of the total impervious fraction used as default values in CUHPF are given in Figure RO-A8 and Figure RO-A9. Level 1 of directly connected imperviousness (DCIA) assumes that all roof gutters are disconnected form driveways, gutters and stormwater conveyance elements. All roof drains are drained onto lawns. Level 2 of DCIA is for developments that already use Level 1 and do not have any curbs and gutters, including concrete swale gutters. All runoff from streets and parking areas is directed as sheet flow across grass surfaces. Intermittent curbs with frequent opening to the grass surface qualifies as Level 2 DCIA.

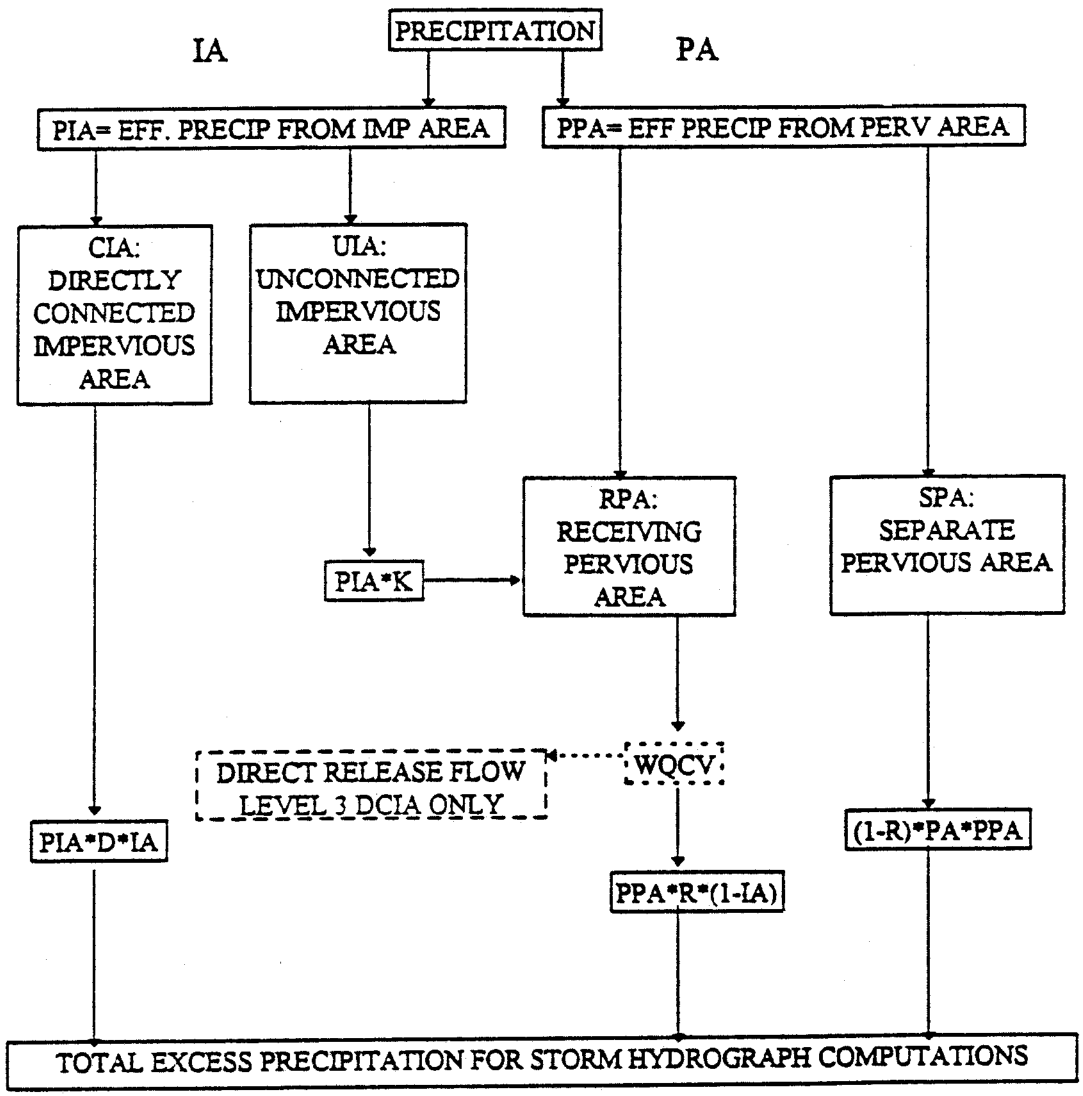


Figure RO-A7—Rainfall and Runoff Schematic for CUHPF/PC

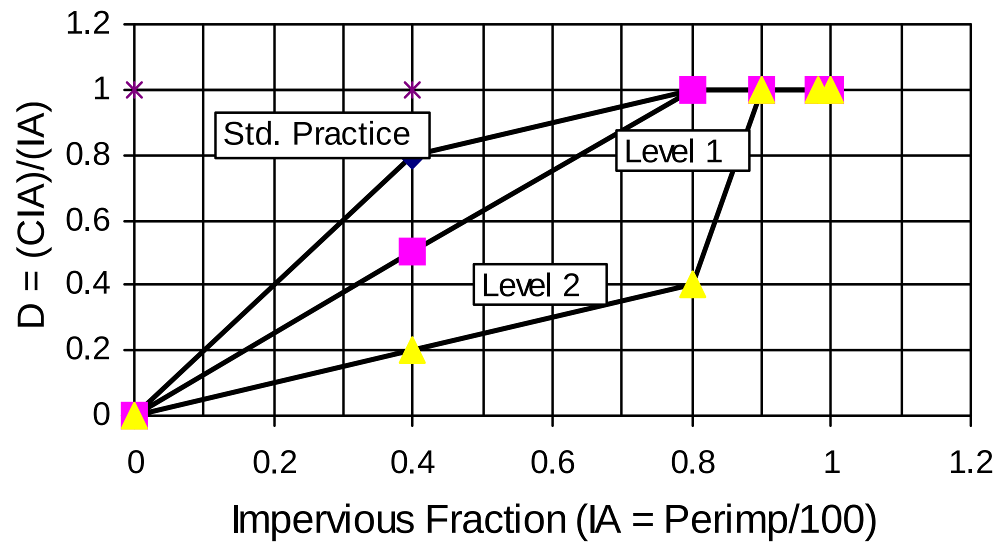


Figure RO-A8—Default Values for Directly Connected Impervious Fraction (D)

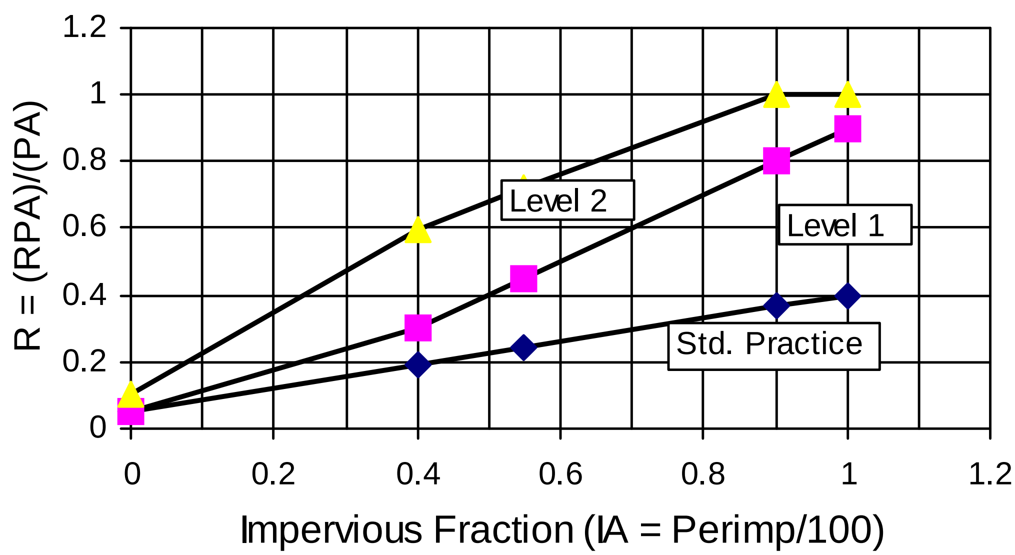


Figure RO-A9—Default Values for Receiving Pervious Area Fraction (R)

The primary change from the earlier version of this software was in the setup and execution of the Effective Rainfall worksheet. The calculations are done for each time increment, same as before, only with the additional losses experienced within the receiving portion of the pervious area taken into account. Whereas the old method was to simply multiply the effective precipitation, if any, by the percentage of the impervious area (IA) and pervious area (PA) as appropriate, each of the four elements illustrated in Figure RO-A7 are now taken into account. Pervious area calculations are segregated using (1-R) to remove the RPA portion. The IA is multiplied by the effective precipitation for impervious area, PIA, and the directly connected impervious area fraction, D, to find the excess precipitation from the hydraulically connected fraction (CIA). The PIA is multiplied by (1-D)*IA to find the excess precipitation from the unconnected impervious area, UIA, which is added to the incremental pervious area precipitation, PPA, and R*PA to calculate the net water contributed to the receiving pervious area, RPA, during each time step. The excess precipitation form the separate pervious area, SPA, is found by multiplying the remaining fraction of pervious area, (1-R) by PA*PPA. The same values for retention/detention losses are used for each pervious fraction, but they will obviously be filled at different rates. Finally, the sum of the excess precipitations from SPA, CIA, and RPA become the total excess precipitation, sometimes referred to as “effective rainfall.” All these calculations are illustrated in Table RO-9 in Section 7.3 *Effective Rainfall Example*.

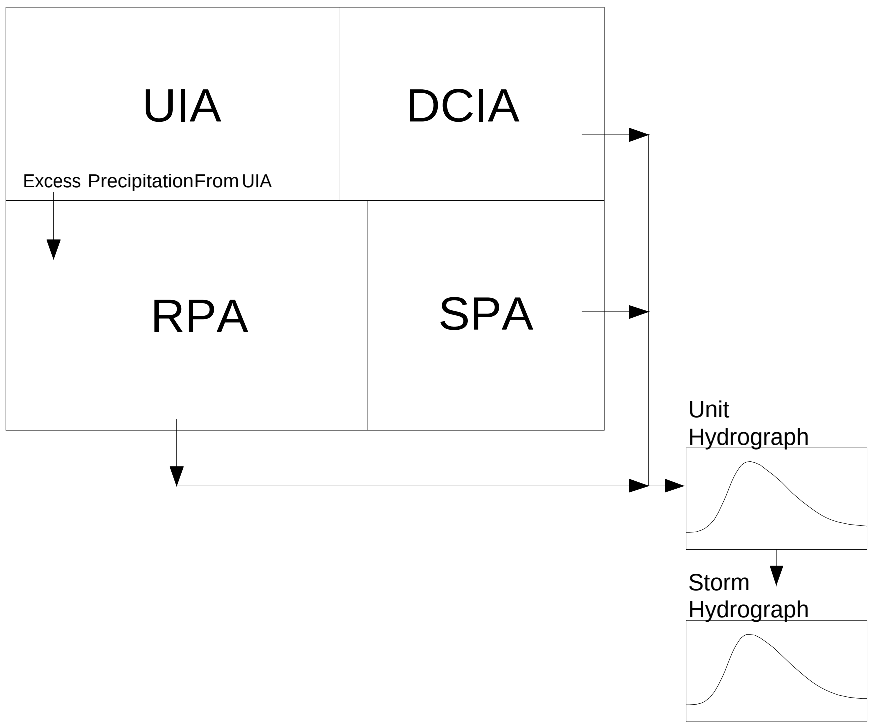


Figure RO-A10—Area and Runoff Diagram With Level 3 DCIA

The “effective” impervious area is determined using Figure ND-1 from the NEW DEVELOPMENT PLANNING chapter of Volume 3 of this *Manual*. However, only 50% of the effects of the effective imperviousness is used for Level 2 DCIA in adjusting lag time.

The user has the option of entering values to override the defaults. Any user-input values for D and R must be accompanied by a specified level of DCIA (i.e., Level 1 or 2) for them to be considered. Note: Since there are default values of D and R for “established practice,” the results of CUHP may be somewhat different than from old versions of CUHP for the same catchment.

For complete instruction and definitions of input parameters, study and follow the latest version of the *USER MANUAL -COLORADO URBAN HYDROGRAPH PROCEDURE, EXCEL-BASED COMPUTER PROGRAM (CUHP2005)*, which may be downloaded from the District's web site.

Calculating the Final Storm Hydrograph: The text in this chapter and in this appendix described how a unit hydrograph is shaped, its ordinates at unit time steps taken off, and how excess precipitation is calculated for each step of the design storm hyetograph. These two sets of calculation need to be now combined to find the storm hydrograph for the catchment given these calculated values. Thus, once the unit hydrograph and the excess precipitation hyetographs are known, the storm hydrograph is calculated by cross-multiplying these two row/column matrixes. Table RO-A1 illustrates these sets of calculations to find the final storm hydrograph for a given catchment and a design storm.

Table RO-A1—Example for Determination a Storm Hydrograph

Time min (1)	Unit Graph cfs (2)	Excess Rainfall Depth in inch																						Hydro graph cfs (25)
		0.04 (3)	0.09 (4)	0.31 (5)	0.15 (6)	0.06 (7)	0.05 (8)	0.03 (9)	0.03 (10)	0.03 (11)	0.03 (12)	0.03 (13)	0.03 (14)	0.03 (15)	0.02 (16)	0.01 (17)	0.01 (18)	0.01 (19)	0.01 (20)	0.01 (21)	0.01 (22)	0.01 (23)	0.01 (24)	
5	115	4.6																						4.6
10	345	13.8	10.4																					24.2
15	520	20.8	31.1	35.7																				87.5
20	463	18.5	46.8	107.0	17.3																			189.5
25	350	14.0	41.7	161.2	51.8	6.9																		275.5
30	260	10.4	31.5	143.5	78.0	20.7	5.8																	289.9
35	210	8.4	23.4	108.5	69.5	31.2	17.3	3.5																261.7
40	168	6.7	18.9	80.6	52.5	27.8	26.0	10.4	3.5															226.3
45	138	5.5	15.1	65.1	39.0	21.0	23.2	15.6	10.4	3.5														198.3
50	110	4.4	12.4	52.1	31.5	15.6	17.5	13.9	15.6	10.4	3.5													176.8
55	88	3.5	9.9	42.8	25.2	12.6	13.0	10.5	13.9	15.6	10.4	3.5												160.8
60	70	2.8	7.9	34.1	20.7	10.1	10.5	7.8	10.5	13.9	15.6	10.4	3.5											147.7
65	55	2.2	6.3	27.3	16.5	8.3	8.4	6.3	7.8	10.5	13.9	15.6	10.4	3.5										136.9
70	40	1.6	5.0	21.7	13.2	6.6	6.9	5.0	6.3	7.8	10.5	13.9	15.6	10.4	2.3									126.7
75	30	1.2	3.6	17.1	10.5	5.3	5.5	4.1	5.0	6.3	7.8	10.5	13.9	15.6	6.9	1.2								114.5
80	20	0.8	2.7	12.4	8.3	4.2	4.4	3.3	4.1	5.0	6.3	7.8	10.5	13.9	10.4	3.5	1.2							98.7
85	15	0.6	1.8	9.3	6.0	3.3	3.5	2.6	3.3	4.1	5.0	6.3	7.8	10.5	9.3	5.2	3.5	1.2						83.3
90	8	0.3	1.4	6.2	4.5	2.4	2.8	2.1	2.6	3.3	4.1	5.0	6.3	7.8	7.0	4.6	5.2	3.5	1.2					70.3
95	2	0.1	0.7	4.7	3.0	1.8	2.0	1.7	2.1	2.6	3.3	4.1	5.0	6.3	5.2	3.5	4.6	5.2	3.5	1.2				60.6
100	0	0.0	0.2	2.5	2.3	1.2	1.5	1.2	1.7	2.1	2.6	3.3	4.1	5.0	4.2	2.6	3.5	4.6	5.2	3.5	1.2			52.4
105			0.0	0.6	1.2	0.9	1.0	0.9	1.2	1.7	2.1	2.6	3.3	4.1	3.4	2.1	2.6	3.5	4.6	5.2	3.5	1.2		45.6
110				0.0	0.3	0.5	0.8	0.6	0.9	1.2	1.7	2.1	2.6	3.3	2.8	1.7	2.1	2.6	3.5	4.6	5.2	3.5	1.2	41.0
115					0.0	0.1	0.4	0.5	0.6	0.9	1.2	1.7	2.1	2.6	2.2	1.4	1.7	2.1	2.6	3.5	4.6	5.2	3.5	36.8
120						0.0	0.1	0.2	0.5	0.6	0.9	1.2	1.7	2.1	1.8	1.1	1.4	1.7	2.1	2.6	3.5	4.6	5.2	31.2
125							0.0	0.1	0.2	0.5	0.6	0.9	1.2	1.7	1.4	0.9	1.1	1.4	1.7	2.1	2.6	3.5	4.6	24.4
130								0.0	0.1	0.2	0.5	0.6	0.9	1.2	1.1	0.7	0.9	1.1	1.4	1.7	2.1	2.6	3.5	18.5
135									0.0	0.1	0.2	0.5	0.6	0.9	0.8	0.6	0.7	0.9	1.1	1.4	1.7	2.1	2.6	14.0
140										0.0	0.1	0.2	0.5	0.6	0.6	0.4	0.6	0.7	0.9	1.1	1.4	1.7	2.1	10.7
145											0.0	0.1	0.2	0.5	0.4	0.3	0.4	0.6	0.7	0.9	1.1	1.4	1.7	8.1
150												0.0	0.1	0.2	0.3	0.2	0.3	0.4	0.6	0.7	0.9	1.1	1.4	6.1
155													0.0	0.1	0.2	0.2	0.2	0.3	0.4	0.6	0.7	0.9	1.1	4.5
160														0.0	0.0	0.1	0.2	0.2	0.3	0.4	0.6	0.7	0.9	3.3
165															0.0	0.0	0.1	0.2	0.2	0.3	0.4	0.6	0.7	2.4
170																0.0	0.0	0.1	0.2	0.2	0.3	0.4	0.6	1.7
175																	0.0	0.0	0.1	0.2	0.2	0.3	0.4	1.2
180																		0.0	0.0	0.1	0.2	0.2	0.3	0.8
185																			0.0	0.0	0.1	0.2	0.2	0.5
190																				0.0	0.0	0.1	0.2	0.3
195																					0.0	0.0	0.1	0.1
200																						0.0	0.0	0.0
205																							0.0	0.0

A.15 Basis for the 1982 Version of the Colorado Urban Hydrograph Procedure

Rainfall and runoff data were collected by U.S. Geological Survey in the Denver metropolitan area since 1969 under a cooperative agreement with the Urban Drainage and Flood Control District. Analysis of this data by the District staff began in earnest in 1977. Of the original thirty gaging stations, data from only seven sites (nine different basin conditions) were used by the District to develop the 1982 version of the Colorado Urban Hydrograph Procedure (CUHP). Data from other sites were also evaluated but were determined not suitable for use due to various gaging problems and watershed definition problems. Because the metropolitan area database lacked an undeveloped watershed, data from a small watershed (Kiowa Creek Tributary at Elbert) recoded by USGS for the Colorado Highway Department was used.

Peak flows from each recorded hydrograph at all test sites were compared with the calculated peak flows using the 1982 version of CUHP. These comparisons are plotted in Figure RO-A12 and substantiate the validity of the CUHP procedure.

Those wishing to compare the older version (i.e., pre 1982) of the CUHP with the new version will find that the new unit hydrograph have a significantly shorter time to peak. This is particularly true for smaller urbanized catchments. However, the new version will often produce peak flow results comparable to those obtained using the old version over a wide range of watershed conditions that are typically used in drainage studies in the Denver Metropolitan Area.

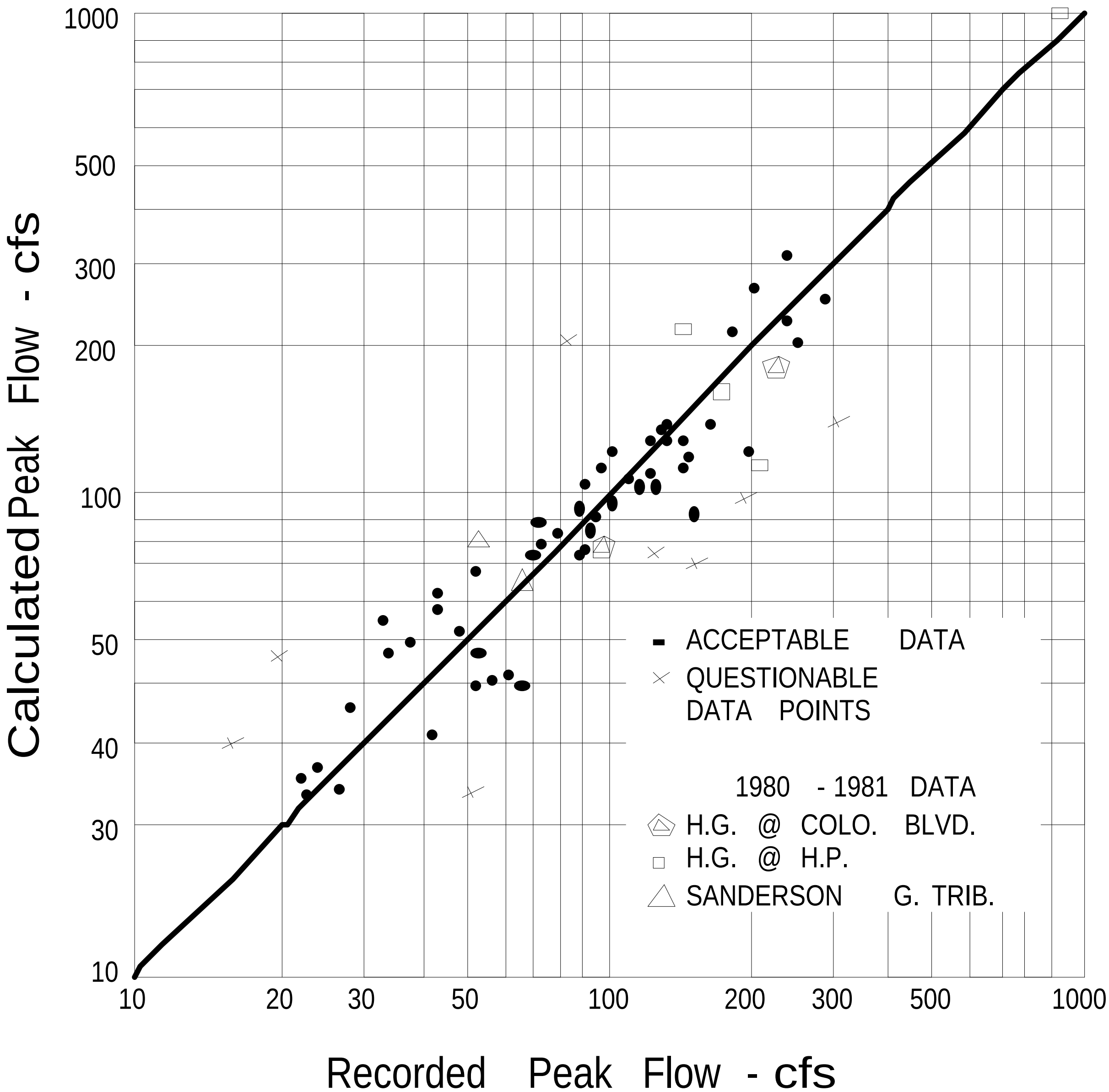


Figure RO-A11—Comparison of Measured Peak Flow Rated Against Peak Flow Rates Calculated Using the Post 1982 Colorado Urban Hydrograph Procedure.

STREETS/INLETS/STORM SEWERS

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Tables

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Photographs

1.0 INTRODUCTION

1.1 Purpose

The purpose of this chapter is to give concise, practical guidelines for the design of urban stormwater collection and conveyance systems. Procedures and equations are presented for the hydraulic design of street drainage, locating inlets and determining capture capacity, and sizing storm sewers. In addition, examples are provided to illustrate the hydraulic design process. Spreadsheet solutions accompany the hand calculations for most example problems.

The design procedures presented in this chapter are based upon fundamental hydrologic and hydraulic design concepts. The design equations provided are well accepted and widely used. They are presented without derivations or detailed explanation, but are properly referenced if the reader wishes to study their background. Therefore, it is assumed that the reader has a fundamental understanding of basic hydrology and hydraulics. A working knowledge of the Rational equation (RUNOFF chapter) and open channel hydraulics (MAJOR DRAINAGE chapter) is particularly helpful.

1.2 Urban Stormwater Collection and Conveyance Systems

Urban stormwater collection and conveyance systems are critical components of the urban infrastructure. Proper design of these systems is essential to minimize flood damage and disruptions in urban areas during storm events while protecting the urban water resources environment. Their primary function is to collect excess stormwater from street gutters, convey the excess stormwater through storm sewers and along the street right-of-way, and discharge it into a detention basin, water quality best management practice (BMP) or the nearest receiving water body (FHWA 1996).

Urban stormwater collection and conveyance systems must fulfill many objectives. Properly functioning urban drainage systems:

- Minimize disruption to the natural drainage system.
- Promote safe passage of vehicular traffic during minor storm events.
- Maintain public safety and manage flooding during major storm events.
- Preserve and protect the urban stream environment.
- Minimize capital and maintenance costs of the system.

All of these objectives are important, but the public is the most vocal about disruptions to traffic and street flooding when storm drainage systems are not designed properly.

Photograph ST-1—The critical role that streets play in urban inlet and storm sewer drainage is often not properly taken into account.

Photograph ST-2—The capital costs of storm sewer construction are large, emphasizing the importance of sound design.

1.3 Components of Urban Stormwater Collection and Conveyance Systems

Urban stormwater collection and conveyance systems within the District are comprised of three primary components: (1) street gutters and roadside swales, (2) stormwater inlets, and (3) storm sewers (and appurtenances like manholes, junctions, etc.). Street gutters and roadside swales collect runoff from the street (and adjacent areas) and convey the runoff to a stormwater inlet while maintaining the street's level-of-service.

Inlets collect stormwater from streets and other land surfaces, transition the flow into storm sewers, and often provide maintenance access to the storm sewer system. Storm sewers convey stormwater in excess of a street's or a swale's capacity along the right-of-way and discharge it into a stormwater management facility or a nearby receiving water body. In rare instances, stormwater pump stations (the design of which is not covered in this *Manual*) are needed to lift and convey stormwater away from low-lying areas where gravity drainage is not possible. All of these components must be designed properly to achieve the stormwater collection and conveyance system's objectives.

1.4 Minor and Major Storms

Rainfall events vary greatly in magnitude and frequency of occurrence. Major storms produce large flow rates but rarely occur. Minor storms produce smaller flow rates but occur more frequently. For economic reasons, stormwater collection and conveyance systems are not normally designed to pass the peak discharge during major storm events.

Stormwater collection and conveyance systems are designed to pass the peak discharge of the minor storm event (and smaller events) with minimal disruption to street traffic. To accomplish this, the spread of water on the street is limited to some maximum, mandated value during the minor storm event. Inlets must be strategically placed to pick up the excess gutter or swale flow once the limiting spread of water is reached. The inlets direct the water into storm sewers, which are typically sized to pass the peak flow rate from the minor storm without any surcharge. The magnitude of the minor storm is established by local ordinances or criteria, and the 2-, 5-, or 10-year storms are most commonly specified.

On occasion, storms will occur that surpass the magnitude of the minor storm event. When this happens, the spread of water on the street exceeds the allowable spread and the capacity of the storm sewers designed for the minor storm event. Street flooding occurs and traffic is disrupted. However, proper design requires that public safety be maintained and the flooding be managed to minimize flood damage. Thus, local ordinances also often establish the return period for the major storm event, generally the 100-year storm. For this event, the street becomes an open channel and must be analyzed to determine that the consequences of the flood are acceptable with respect to flood damage and public safety.

2.0 STREET DRAINAGE

2.1 Street Function and Classification

The primary function of a street or roadway is to provide for the safe passage of vehicular traffic at a specified level of service. If stormwater collection and conveyance systems are not designed properly, this primary function can be impaired. To make sure this does not happen, streets are classified for drainage purposes based on their traffic volume, parking practices, and other criteria (Wright-McLaughlin Engineers 1969). The four street classifications are:

- Local (low-speed traffic for residential or industrial area access).
- Collector (low/moderate-speed traffic providing service between local streets and arterials).
- Arterial (moderate/high-speed traffic moving through urban areas and accessing freeways).
- Freeway (high-speed travel, generally over long distances).

Table ST-1 provides additional information on the classification of streets for drainage purposes.

Table ST-1—Street Classification for Drainage Purposes

Street Classification	**Function**	Speed/Number of Lanes	**Signalization at Intersections**	**Street Parking**
Local	Provide access to residential and industrial areas	Low speed with 2 moving lanes	Stop signs	One or both sides of the street
Collector	Collect and convey traffic between local and arterial streets	Low to moderate speed with 2 or 4 moving lanes	Stop signs or traffic signals	One or both sides of the street
Arterial	Function as primary through-traffic conduits in urban areas	Moderate to high speeds with 4 to 6 lanes	Traffic signals (controlled access)	Usually prohibited
Freeway	Provide rapid and efficient transport over long distances	High-speed travel with 4 lanes or more	Cloverleafs, access ramps (limited access)	Always prohibited

Streets serve another important function other than traffic flow. They contain the first component in the urban stormwater collection and conveyance system. That component is the street gutter or adjacent swale, which collects excess stormwater from the street and adjacent areas and conveys it to a stormwater inlet. Proper street drainage is essential to:

- Maintain the street's level-of-service.

- Reduce skid potential.
- Minimize the potential for cars to hydroplane.
- Maintain good visibility for drivers (by reducing splash and spray).
- Minimize inconvenience/danger to pedestrians during storm events (FHWA 1984).

2.2 Design Considerations

Certain design considerations must be taken into account in order to meet street drainage objectives. The primary design objective is to keep the spread (encroachment) of stormwater on the street below an acceptable value for a given return period of flooding. As mentioned previously, when stormwater collects on the street and flows down the gutter, the top width (or spread) of the water widens as more stormwater is collected. If left unchecked, the spread of water would eventually hinder traffic flow and possibly become hazardous (i.e., reduced skid resistance, hydroplaning, splash, etc.). Based on these considerations, the District has established encroachment (spread) standards for the minor storm event. These standards were given in the POLICY chapter and are repeated in Table ST-2 for convenience.

Table ST-2—Pavement Encroachment Standards for the Minor Storm

Street Classification	Maximum Encroachment
Local	No curb overtopping. Flow may spread to crown of street.
Collector	No curb overtopping. Flow spread must leave at least one lane free of water.
Arterial	No curb overtopping. Flow spread must leave at least one lane free of water in each direction, but should not flood more than two lanes in each direction.
Freeway	No encroachment is allowed on any traffic lanes.

Standards for the major storm and street cross flows are also required. The major storm needs to be assessed to determine the potential for flooding and public safety. Cross flows also need to be regulated for traffic flow and public safety reasons. The District has established street inundation standards during the major storm event and allowable cross-street flow standards. These standards were given in the POLICY chapter and are repeated in Table ST-3 and Table ST-4 for convenience.

Table ST-3—Street Inundation Standards for the Major (i.e., 100-Year) Storm

Street Classification	Maximum Depth and Inundated Area
Local and Collector	Residential dwellings and public, commercial, and industrial buildings should be no less than 12 inches above the 100-year flood at the ground line or lowest water entry of the building. The depth of water over the gutter flow line should not exceed 18 inches.
Arterial and Freeway	Residential dwellings and public, commercial, and industrial buildings should be no less than 12 inches above the 100-year flood at the ground line or lowest water entry of the building. The depth of water should not exceed the street crown to allow operation of emergency vehicles. The depth of water over the gutter flow line should not exceed 12 inches.

Table ST-4—Allowable Cross-Street Flow

Street Classification	Initial Storm Flow	Major (100-Year) Storm Flow
Local	6 inches of depth in cross pan.	18 inches of depth above gutter flow line.
Collector	Where cross pans allowed, depth of flow should not exceed 6 inches.	12 inches of depth above gutter flow line.
Arterial/Freeway	None.	No cross flow. Maximum depth at upstream gutter on road edge of 12 inches.

Once an allowable spread (pavement encroachment) has been established for the minor storm, the placement of inlets can be determined. The inlets will remove some or all of the excess stormwater and thus reduce the spread. The placement of inlets is covered in Section 3.0. It should be noted that proper drainage design utilizes the full allowable capacity of the street gutter in order to limit the cost of inlets and storm sewers.

Another important design consideration is the frequency of occurrence of the minor storm. In other words, how often will the spread of stormwater reach or exceed the maximum encroachment limit. This is addressed by assigning a frequency (or recurrence interval) to the minor storm. The selection of a design frequency is based on many factors including street function, traffic load, vehicle speed, etc. The minor storm is generally between the 2-year and 10-year storm. The major storm is normally defined as the 100-year storm. The minor and major storm return periods are mandated by local governments.

Two additional design considerations of importance in street drainage are gutter (channel) shape and street slope. Most urban streets contain curb and gutter sections. Various types exist which include spill shapes, catch shapes, curb heads, and roll gutters. The shape is chosen for functional, cost, or aesthetic reasons and does not dramatically affect the hydraulic capacity. Swales are common along some urban and semi-urban streets, and roadside ditches are common along rural streets. Their shapes are important in determining hydraulic capacity and are covered in the next section.

2.3 Hydraulic Evaluation

Hydraulic computations are performed to determine the capacity of roadside swales and street gutters and the encroachment of stormwater onto the street. The design discharge is usually determined using the Rational method (covered in the next two sections). Stormwater runoff ends up in swales, roadside ditches and street gutters where the flow is unsteady and non-uniform. However, uniform, steady flow is usually assumed for the short period of time during peak flow conditions.

2.3.1 Curb and Gutter

Street slope can be divided into two components: longitudinal slope and cross slope. The longitudinal slope of the gutter essentially mimics the street slope. The hydraulic capacity of a gutter increases as the longitudinal slope increases. The District prescribes a minimum grade of 0.4% (Wright-McLaughlin 1969). The allowable flow capacity of the gutter on steep slopes is limited to provide for public safety. The cross (transverse) slope represents the slope from the street crown to the gutter section. A compromise is struck between large cross slopes that facilitate pavement drainage and small cross slopes for driver safety and comfort. The District prescribes a minimum cross slope of 1% for pavement drainage. Composite sections are often used with gutter cross slopes being steeper than street cross slopes to increase the gutter capacity.

The hydraulic evaluation of street capacity includes the following steps:

1. Calculate the theoretical street gutter flow capacity to convey the minor storm based upon the allowable **spread** defined in Table ST-2.

2. Calculate the theoretical street gutter flow capacity to convey the minor storm based upon the allowable **depth** defined Table ST-2.

3. Calculate the allowable street gutter flow capacity by multiplying the theoretical capacity (calculated in number 2) by a reduction factor. This reduction factor is used for safety considerations. The lesser of the capacities calculated in step 1 and this step is the allowable street gutter capacity.

4. Calculate the theoretical major storm conveyance capacity based upon the road inundation criteria in Table ST-3. Reduce the major storm capacity by a reduction factor to determine the allowable storm conveyance capacity.

2.3.1.1 Gutters With Uniform Cross Slopes (i.e., Where Gutter Cross Slope = Street Cross Slope)

Since gutter flow is assumed to be uniform for design purposes, Manning's equation is appropriate with a slight modification to account for the effects of a small hydraulic radius. For a triangular cross section

(Figure ST-1a), the Manning formula for gutter flow is written as:

$$Q = \frac{0.56}{n} S_x^{5/3} S_L^{1/2} T^{8/3} \tag{ST-1}$$

in which:

Q = calculated flow rate for the street (cfs)

n = Manning's roughness coefficient, (typically = 0.016)

S_x = street cross slope for the street (ft/ft)

S_L = longitudinal slope (ft/ft)

T = top width of flow spread (ft)

The flow depth, y, at the curb can be found using:

$$y = TS_x \tag{ST-2}$$

Note that the flow depth must be less than the curb height during the minor storm based on Table ST-2. Manning's equation can be written in terms of the flow depth, as:

$$Q = \frac{0.56}{nS_x} S_L^{1/2} y^{8/3} \tag{ST-3}$$

The cross-sectional flow area, A, can be expressed as:

$$A = (1/2)\, S_x T^2 \tag{ST-4}$$

The gutter velocity at peak capacity may be found from the continuity equation ($V = Q/A$). Triangular gutter cross-section calculations are illustrated in Example 6.1.

2.3.1.2 Gutters With Composite Cross Slopes (i.e., Where Gutter Cross Slope ≠ Street Cross Slope)

Gutters with composite cross slopes (Figure ST-1b) are often used to increase the gutter capacity. For a composite gutter section:

$$Q = Q_w + Q_s \tag{ST-5}$$

in which:

Q_w = flow rate in the depressed section of the gutter (cfs)

Q_s = discharge in the section that is above the depressed section (cfs)

The Federal Highway Administration (FHWA 1996) provides the following equations for obtaining the flow rate in gutters with composite cross slopes. The theoretical flow rate, Q, is:

$$Q = \frac{Q_s}{1 - E_o} \tag{ST-6}$$

in which:

$$E_o = \frac{1}{1 + \dfrac{S_w / S_x}{\left[1 + \dfrac{S_w / S_x}{(T/W) - 1}\right]^{8/3} - 1}} \tag{ST-7}$$

in which S_w is the gutter cross slope (ft/ft), and,

$$S_w = S_x + \frac{a}{W} \tag{ST-8}$$

in which a is the gutter depression (feet) and W is width of the gutter (ft).

Figure ST-1b depicts all geometric variables. From the geometry, it can be shown that:

$$y = a + TS_x \tag{ST-9}$$

and,

$$A = \frac{1}{2} S_x T^2 + \frac{1}{2} aW \tag{ST-10}$$

in which y is the flow depth (at the curb) and A is the flow area. Composite cross-section gutter flow calculations are illustrated in Examples 6.2 and 6.3.

2.3.1.3 Allowable Gutter Hydraulic Capacity

Stormwater flows along streets exert momentum forces on cars, pavement, and pedestrians. To limit the hazardous nature of heavy street flows, it is necessary to set limits on flow velocities and depths. As a result, the allowable gutter hydraulic capacity is determined as the lesser of:

$$Q_A = Q_T \tag{ST-11}$$

or

$$Q_A = R\, Q_F \tag{ST-12}$$

in which Q_A = allowable street hydraulic capacity, Q_T = street hydraulic capacity limited by the maximum water spread, R = reduction factor, and Q_F = gutter capacity when flow depth equals allowable depth.

There are two sets of reduction factors developed for Denver metropolitan areas (Guo 2000b). One is for the minor event, and another is for the major event. Figure ST-2 shows that the reduction factor remains unity (1.0) for a street slope <1.5%, and then decreases as the street slope increases.

It is important for street drainage designs that the allowable street hydraulic capacity be used instead of the calculated gutter-full capacity. Thus, wherever the accumulated stormwater amount on the street is close to the allowable capacity, a street inlet shall be installed.

2.3.2 Swale Sections (V-Shaped With the Same or Different Side Slopes)

Swales are often used to convey runoff from pavement where curb and gutter sections are not used. It is very important that swale depths and side slopes be as shallow as possible for safety and maintenance reasons. Street-side swales are not the same as roadside ditches that can be considered part of a major drainageway system. Street-side swales serve as collectors of initial runoff and transport it to the nearest inlet or major drainageway. To be effective, they need to be limited to the velocity, depth, and cross-slope geometries considered acceptable. The following limitations shall apply to street-side swales:

- Maximum 2-year flow velocity = 3 ft/sec
- Maximum flow depth = 1.0 ft
- Maximum side slope of each side = 5H:1V.*

  * Use of flatter side slopes is strongly recommended.

Swales generally have V-sections (Figure ST-3). Equation ST-1 can be used to calculate the flow rate in a V-section (if the section has a constant Manning's *n* value) with an adjusted slope found using:

$$S_x = \frac{S_{x1} S_{x2}}{S_{x1} + S_{x2}} \qquad \text{(ST-13)}$$

in which:

S_x = adjusted side slope (ft/ft)

S_{x1} = right side slope (ft/ft)

S_{x2} = left side slope (ft/ft)

Figure ST-3 shows the geometric variables.

Examples 6.4 and 6.5 show V-shaped swale calculations.

Under no circumstances shall a street-side swale have a longitudinal slope steeper than 2%. Use grade control checks to control the grade if the adjacent street is steeper.

Note that the slope of roadside ditches and swales is often different than the adjacent street. The hydraulic characteristics of the swale can therefore change from one location to another on a given swale. The flow depth and spread limitations of Tables ST-2 and ST-4 are also valid for swales and roadside ditches. There is no capacity reduction for safety considerations for roadside swales.

The designer is cautioned when using swales. If not properly designed and maintained, they can become a nuisance to the local residents.

Manning's equation can be used to calculate flow characteristics.

$$Q = \frac{1.49}{n} A R^{2/3} S_L^{\ 1/2} \qquad \text{(ST-14)}$$

in which:

Q = flow rate (cfs)

n = Manning's roughness coefficient

A = flow area (ft^2)

R = A/P (ft)

P = wetted perimeter (ft)

S_L = longitudinal slope (ft/ft)

2.4 Major Storm Hydraulics

2.4.1 Purpose and Objectives

As previously mentioned, the primary objective of street drainage design is not to exceed the spread (encroachment) criteria during the minor storm event. Since larger storms do occur, it is prudent to determine the consequences of the major storm event. Table ST-3 lists the street inundation standards recommended by this *Manual* for the major storm event. Proper street design requires that the major storm be assessed in the interest of public safety and to minimize the potential for flood damages.

2.4.2 Street Hydraulic Capacity

During major storms, streets typically become wide, open channels that convey stormwater flow in excess of the storm sewer capacity. Manning's equation (Equation ST-14) is generally appropriate to determine flow depths and street capacities assuming uniform flow.

The general form of Manning's equation is the most appropriate solution method for this situation since many different flow situations and channel shapes may be encountered. The allowable street capacity for a major storm is also subject to safety considerations using the reduction factor taken from Figure ST-2.

Major storm street hydraulic capacity calculations are shown in Example 6.6.

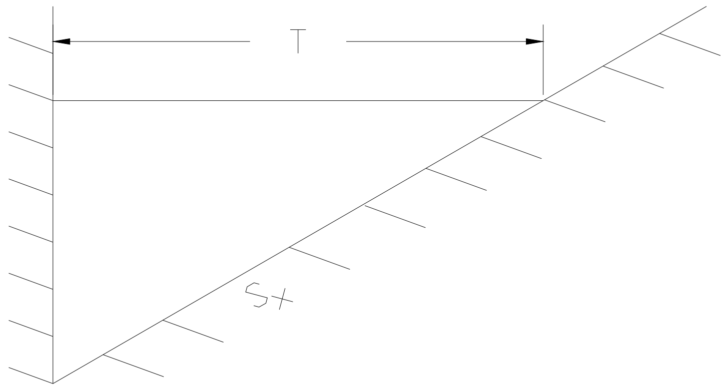


Figure ST-1a—Typical Gutter Sections—Constant Cross Slope

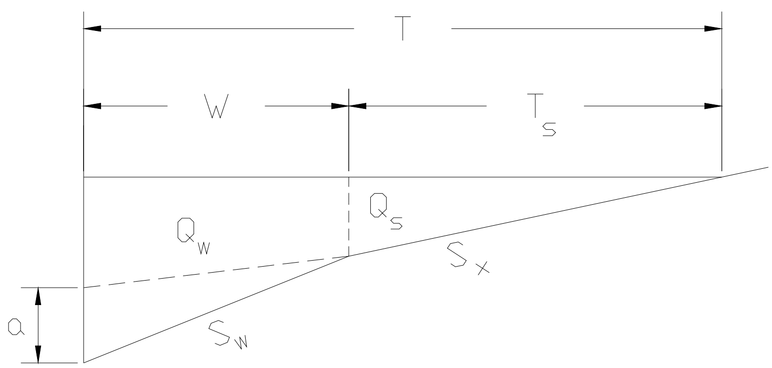


Figure ST-1b—Typical Gutter Sections—Composite Cross Slope

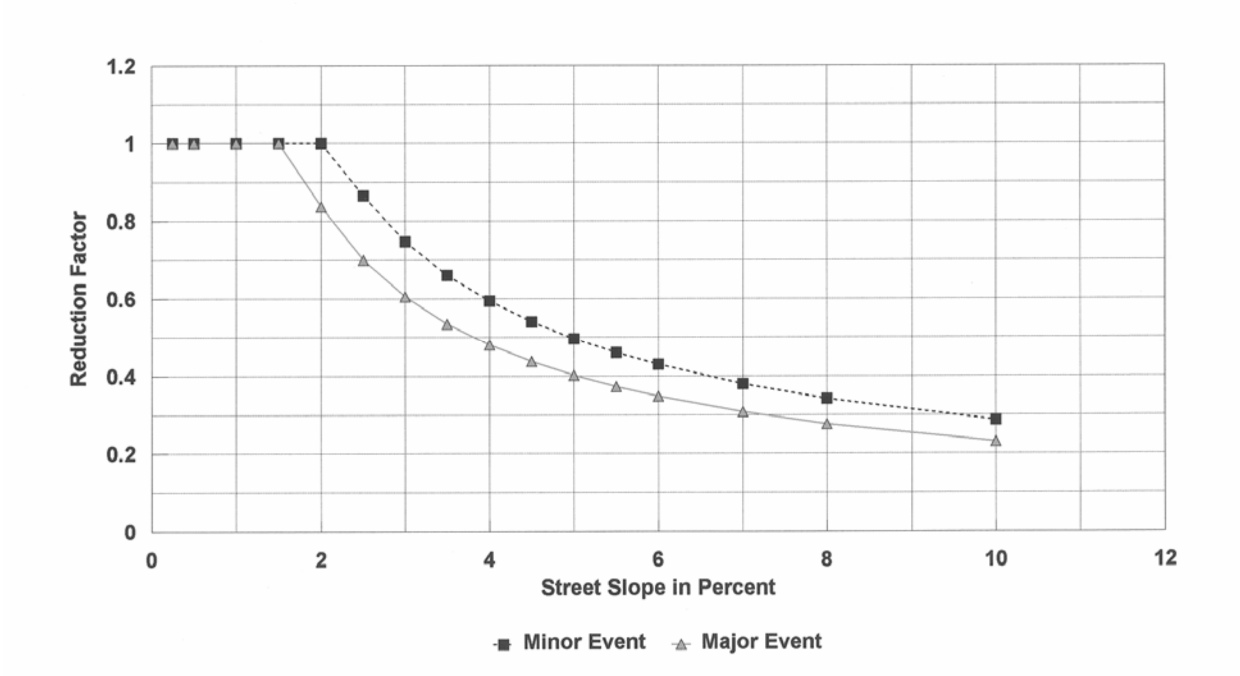
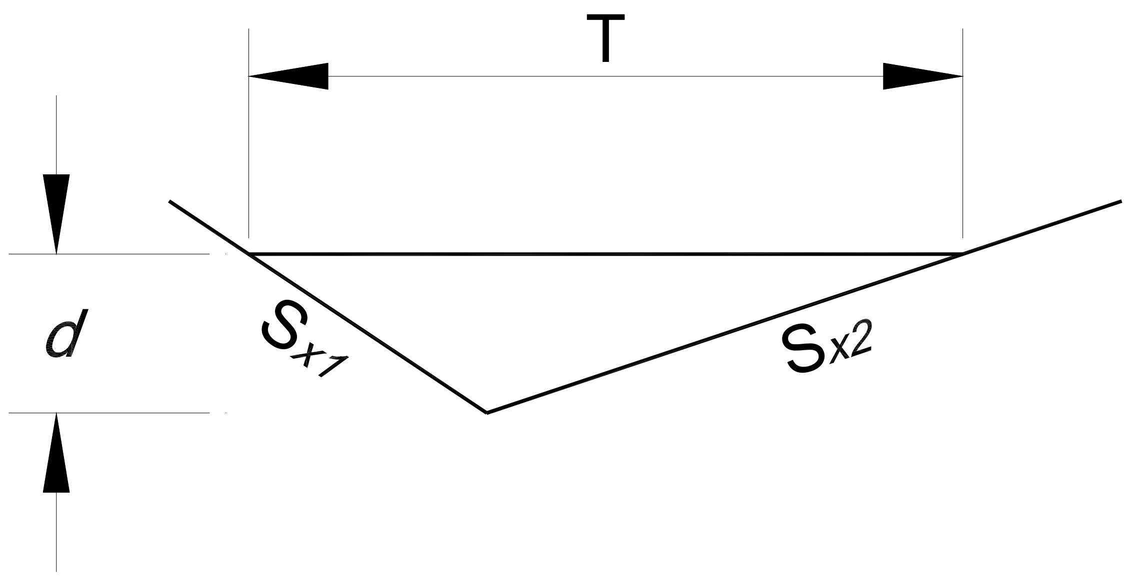


Figure ST-2—Reduction Factor for Gutter Flow



Notes:

1. S_{x1} and S_{x2} ≤ 5H:1V.
2. d ≤ 1.0 feet.
3. Normal flow velocity in a grass-lined swale shall be less than 3 ft/sec during a 2-year storm.
4. Longitudinal grade of a grass-lined swale shall be less than 2%. Use grade control checks if adjacent street is steeper to limit the swale's flow.

Figure ST-3—Typical Street-Side Swale Sections—V-Shaped

3.0 INLETS

3.1 Inlet Functions, Types and Appropriate Applications

Stormwater inlets are a vital component of the urban stormwater collection and conveyance system. Inlets collect excess stormwater from the street, transition the flow into storm sewers, and can provide maintenance access to the storm sewer system. They can be made of cast-iron, steel, concrete, and/or pre-cast concrete and are installed on the edge of the street adjacent to the street gutter or in the bottom of a swale.

Roadway geometrical features often dictate the location of pavement drainage inlets. In general, inlets are placed at all low points (sumps or sags) in the gutter grade, median breaks, intersections, and crosswalks. The spacing of inlets placed between those required by geometric controls is governed by the design flow spread (i.e., allowable encroachment). In other words, the drainage inlets are spaced so that the spread under the design (minor) storm conditions will not exceed the allowable flow spread (Akan and Houghtalen 2002).

There are four major types of inlets: grate, curb opening, combination, and slotted. Figure ST-4 depicts the four major types of inlets along with some associated geometric variables. Table ST-5 provides information on the appropriate application of the different inlet types along with advantages and disadvantages of each.

Table ST-5—Applicable Settings for Various Inlet Types

Inlet Type	Applicable Setting	Advantages	Disadvantages
Grate	Sumps and continuous grades (should be made bicycle safe)	Perform well over wide range of grades	Can become clogged Lose some capacity with increasing grade
Curb-opening	Sumps and continuous grades (but not steep grades)	Do not clog easily Bicycle safe	Lose capacity with increasing grade
Combination	Sumps and continuous grades (should be made bicycle safe)	High capacity Do not clog easily	More expensive than grate or curb-opening acting alone
Slotted	Locations where sheet flow must be intercepted.	Intercept flow over wide section	Susceptible to clogging

3.2 Design Considerations

Stormwater inlet design takes two forms: inlet placement location and inlet hydraulic capacity. As previously mentioned, inlets must be placed in sumps to prevent ponding of excess stormwater. On streets with continuous grades, inlets are required periodically to keep the gutter flow from exceeding the encroachment limitations. In both cases, the size and type of inlets need to be designed based upon their hydraulic capacity.

Inlets placed on continuous grades rarely intercept all of the gutter flow during the minor (design) storm. The effectiveness of the inlet is expressed as an efficiency, *E*, which is defined as:

$$E = Q_i / Q \qquad \text{(ST-15)}$$

in which:

E = inlet efficiency

Q_i = intercepted flow rate (cfs)

Q = total gutter flow rate (cfs)

Bypass (or carryover) flow is not intercepted by the inlet. By definition,

$$Q_b = Q - Q_i \qquad \text{(ST-16)}$$

in which:

Q_b = bypass (or carryover) flow rate (cfs)

The ability of an inlet to intercept flow (i.e., hydraulic capacity) on a continuous grade generally increases with increasing gutter flow, but the capture efficiency decreases. In other words, even though more stormwater is captured, a smaller percentage of the gutter flow is captured. In general, the inlet capacity depends upon:

- The inlet type and geometry.
- The flow rate (depth and spread of water).
- The cross (transverse) slope.
- The longitudinal slope.

The hydraulic capacity of an inlet varies with the type of inlet. For grate inlets, the capacity is largely dependent on the amount of water flowing over the grate, the grate configuration and spacing, and the velocity of flow. For curb opening inlets, the capacity is largely dependent on the length of the opening, the flow velocity, street and gutter cross slope, and the flow depth at the curb. Local gutter depression along the curb opening helps boost the capacity. On the other hand, top slab supports can decrease the capacity. Combination inlets do not intercept much more than their grates alone if they are placed side by side and are of nearly equal lengths but are much less likely to clog. Slotted inlets function in a manner similar to curb opening inlets (FHWA 1996).

Inlets in sumps operate as weirs for shallow pond depths, but eventually will operate as orifices as the depth increases. A transition region exists between weir flow and orifice flow, much like a culvert. Grate inlets and slotted inlets tend to clog with debris, so calculations should take that into account. Curb opening inlets tend to be more dependable for this reason.

3.3 Hydraulic Evaluation

The hydraulic capacity of an inlet is dependent on the type of inlet (grate, curb opening, combination, or slotted) and the location (on a continuous grade or in a sump). The methodology for determination of hydraulic capacity of the various inlet types is described in the following sections: (a) grate inlets on a continuous grade (Section 3.3.1), (b) curb opening inlets on a continuous grade (Section 3.3.2), (c) combination inlets on a continuous grade (Section 3.3.3), (d) slotted inlets on a continuous grade (Section 3.3.4), and (e) inlets located in sumps (Section 3.3.5).

3.3.1 Grate Inlets (On a Continuous Grade)

The capture efficiency of a grate inlet is highly dependent on the width and length of the grate and the velocity of gutter flow. If the gutter velocity is low and the spread of water does not exceed the grate width, all of the flow will be captured by the grate inlet. This is not normally the case during the minor (design) storm. The spread of water often exceeds the grate width and the flow velocity can be high. Thus, some water gets by the inlet. Water going over the grate may be capable of “splashing over” the grate, and usually little of the water outside the grate width is captured.

In order to determine the efficiency of a grate inlet, gutter flow is divided into two parts: frontal flow and side flow. Frontal flow is defined as that portion of the flow within the width of the grate. The portion of the flow outside the grate width is called side flow. By using Equation ST-1, the frontal flow can be evaluated and is expressed as:

$$Q_w = Q\left[1 - \left(1 - \left(W/T\right)\right)\right]^{2.67} \tag{ST-17}$$

in which:

Q_w = frontal discharge (flow within width W) (cfs)

Q = total gutter flow (cfs) found using Equation ST-1

W = width of grate (ft)

T = total spread of water in the gutter (ft)

It should be noted that the grate width is generally equal to the depressed section in a composite gutter section. Now by definition:

$$Q_s = Q - Q_w \tag{ST-18}$$

in which:

Q_s = side discharge (i.e., flow outside the depressed gutter or grate) (cfs)

The ratio of the frontal flow intercepted by the inlet to total frontal flow, R_f, is expressed as:

$$R_f = Q_{wi}/Q_w = 1.0 - 0.09(V - V_o) \text{ for } V \geq V_o \text{, otherwise } R_f = 1.0 \tag{ST-19}$$

in which:

Q_{wi} = frontal flow intercepted by the inlet (cfs)

V = velocity of flow in the gutter (ft/sec)

V_o = splash-over velocity (ft/sec)

The splash-over velocity is defined as the minimum velocity causing some water to shoot over the grate. This velocity is a function of the grate length and type. The splash-over velocity can be determined using the empirical formula (Guo 1999):

$$V_o = \alpha + \beta L_e - \gamma L_e^{\ 2} + \eta L_e^{\ 3} \tag{ST-20}$$

in which:

V_o = splash-over velocity (ft/sec)

L_e = effective unit length of grate inlet (ft)

$\alpha, \beta, \gamma, \eta$ = constants from Table ST-6

Table ST-6—Splash Velocity Constants for Various Types of Inlet Grates

Type of Grate	α	β	γ	η
Bar P-1-7/8	2.22	4.03	0.65	0.06
Bar P-1-1/8	1.76	3.12	0.45	0.03
Vane Grate	0.30	4.85	1.31	0.15
45-Degree Bar	0.99	2.64	0.36	0.03
Bar P-1-7/8-4	0.74	2.44	0.27	0.02
30-Degree Bar	0.51	2.34	0.20	0.01
Reticuline	0.28	2.28	0.18	0.01

The ratio of the side flow intercepted by the inlet to total side flow, R_s, is expressed as:

$$R_s = \frac{1}{1 + \frac{0.15V^{1.8}}{S_x L^{2.3}}} \quad \text{(ST-21)}$$

in which:

V = velocity of flow in the gutter (ft/sec)

L = length of grate (ft)

The capture efficiency, E, of the grate inlet may now be determined using:

$$E = R_f\left(Q_w / Q\right) + R_s\left(Q_s / Q\right) \quad \text{(ST-22)}$$

Example 6.9 shows grate inlet capacity calculations.

3.3.2 Curb-Opening Inlets (On a Continuous Grade)

The capture efficiency of a curb-opening inlet is dependent on the length of the opening, the depth of flow at the curb, street cross slope and the longitudinal gutter slope (see Photograph ST-3). If the curb opening is long, the flow rate is low, and the longitudinal gutter slope is small, all of the flow will be captured by the inlet. This is not normally the case during the minor (design) storm. In fact, it is generally uneconomical to install a curb opening long enough to capture all of the flow. Thus, some water gets by the inlet, and the inlet efficiency needs to be determined.

Photograph ST-3—Gutter/street slope is a major design factor for both street and inlet capacity.

The hydraulics of curb opening inlets are less complicated than grate inlets. The efficiency, E, of a curb-opening inlet is calculated as:

$$E = 1 - \left[1 - \left(L/L_T\right)\right]^{1.8} \text{ for } L < L_T\text{, otherwise } E = 1.0 \quad \text{(ST-23)}$$

in which:

L = installed (or designed) curb-opening length (ft)

L_T = curb-opening length required to capture 100% of gutter flow (ft)

and, for a curb-opening inlet that is not depressed,

$$L_T = 0.6\, Q^{0.42} S_L^{\ 0.3} \left(\frac{1}{nS_x}\right)^{0.6} \quad \text{(ST-24)}$$

in which:

Q = gutter flow (cfs)

S_L = longitudinal street slope (ft/ft)

S_x = steel cross slope (ft/ft)

n = Manning's roughness coefficient

For a depressed curb-opening inlet,

$$L_T = 0.6\, Q^{0.42} S_L^{\ 0.3} \left(\frac{1}{nS_e}\right)^{0.6} \quad \text{(ST-25)}$$

The equivalent cross slope, S_e, can be determined from

$$S_e = S_x + \frac{a}{W} E_o \quad \text{(ST-26)}$$

in which a = gutter depression and W = depressed gutter section as shown in Figure ST-1b. The ratio of the flow in the depressed section to total gutter flow, E_o, can be calculated from Equation ST-7. See Examples 6.8 and 6.9 for curb-opening inlet calculations.

3.3.3 Combination Inlets (On a Continuous Grade)

Combination inlets take advantage of the debris removal capabilities of a curb-opening inlet and the capture efficiency of a grate inlet. If the grate and the curb opening are side-by-side and of approximately equal length, the interception capacity is found by assuming the grate acts alone. If all or part of the curb-opening inlet lies upstream from the grate (a desirable configuration), the inlet capacity is enhanced by the upstream curb-opening capacity. The appropriate equations have already been presented, but Example 6.10 illustrates the procedure.

3.3.4 Slotted Inlets (On a Continuous Grade)

Slotted inlets can generally be used to intercept sheet flow that is crossing the pavement in an undesirable location. Unlike grate inlets, they have the advantage of intercepting flow over a wide section. They do not interfere with traffic operations and can be used on both curbed and uncurbed sections. Like grate inlets, they are susceptible to clogging.

Slotted inlets function like a side-flow weir, much like curb-opening inlets. The FHWA (1996) suggests the hydraulic capacity of slotted inlets closely corresponds to curb-opening inlets if the slot openings exceed 1.75 inches. Therefore, the equations developed for curb-opening inlets (Equations ST-23 through ST-26) are appropriate for slotted inlets.

3.3.5 Inlets Located in Sumps

All of the stormwater excess that enters a sump (i.e., a depression or low point in grade) must pass through an inlet to enter the stormwater conveyance system. If the stormwater is laden with debris, the inlet is susceptible to clogging. The ponding of water is a nuisance and could be hazardous. Therefore, the capacity of inlets in sumps must account for this clogging potential. Grate inlets acting alone are not recommended for this reason. Curb-opening inlets are more appropriate, as are combination inlets. Photograph ST-4 shows a curb opening inlet in a sump condition.

Photograph ST-4—Inlets that are located in street sags and sumped can be highly efficient.

As previously mentioned, inlets in sumps function like weirs for shallow depths, but as the depth of stormwater increases, they begin to function like an orifice. Orifice and weir flows have been exhaustively studied. Equations are readily available to compute requisite flow rates. However, the transition from weir flow to orifice flow takes place over a relatively small range of depth that is not well defined. The FHWA provides guidance on the transition region based on significant testing.

The hydraulic capacity of grate, curb-opening, and slotted inlets operating as weirs is expressed as:

$$Q_i = C_w L_w \, d^{1.5} \tag{ST-27}$$

in which:

Q_i = inlet capacity (cfs)

C_w = weir discharge coefficient

L_w = weir length (ft)

d = flow depth (ft)

Values for C_w and L_w are presented in Table ST-7 for various inlet types. Note that the expressions given for curb-opening inlets without depression should be used for depressed curb-opening inlets if L > 12 feet.

The hydraulic capacity of grate, curb-opening, and slotted inlets operating as orifices is expressed as:

$$Q_i = C_o A_o \left(2gd\right)^{0.5} \tag{ST-28}$$

in which:

Q_i = inlet capacity (cfs)

C_o = orifice discharge coefficient

A_o = orifice area (ft^2)

d = characteristic depth (ft) as defined in Table ST-7

g = 32.2 ft/sec^2

Values for C_o and A_o are presented in Table ST-7 for different types of inlets.

Combination inlets are commonly used in sumps. The hydraulic capacity of combination inlets in sumps depends on the type of flow and the relative lengths of the curb opening and grate. For weir flow, the capacity of a combination inlet (grate length equal to the curb opening length) is equal to the capacity of the grate portion only. This is because the curb opening does not add any length to the weir equation (Equation ST-27). If the curb opening is longer than the grate, the capacity of the additional curb length should be added to the grate capacity. For orifice flow, the capacity of the curb opening should be added to the capacity of the grate.

Table ST-7—Sag Inlet Discharge Variables and Coefficients

(Modified From Akan and Houghtalen 2002)

Inlet Type	C_w	L_w[1]	**Weir Equation Valid For**	**Definitions of Terms**
Grate Inlet	3.00	$L + 2W$	$d < 1.79(A_o/L_w)$	L = Length of grate W = Width of grate d = Depth of water over grate A_0= Clear opening area[2]
Curb Opening Inlet	3.00	L	$d < h$	L = Length of curb opening h = Height of curb opening $d = d_i - (h/2)$ d_i = Depth of water at curb opening
Depressed Curb Opening Inlet[3]	2.30	$L + 1.8W$	$d < (h + a)$	W = Lateral width of depression a = Depth of curb depression
Slotted Inlets	2.48	L	$d < 0.2$ ft	L = Length of slot d = Depth at curb
[1] The weir length should be reduced where clogging is expected. [2] Ratio of clear opening area to total area is 0.8 for P-1-7/8-4 and reticuline grates, 0.9 for P-1-7/8 and 0.6 for P-1-1/8 grates. Curved vane and tilt bar grates are not recommended at sag locations. [3] If L > 12 ft, use the expressions for curb opening inlets without depression.				
	C_o	A_0[4]	**Orifice Equation Valid for**	**Definition of Terms**
Grate Inlet	0.67	Clear opening area[5]	$d > 1.79(A_o/L_w)$	d = Depth of water over grate
Curb Opening Inlet (depressed or undepressed, horizontal orifice throat[6])	0.67	$(h)(L)$	$d_i > 1.4h$	$d = d_i - (h/2)$ d_i = Depth of water at curb opening h = Height of curb opening
Slotted Inlet	0.80	$(L)(W)$	$d > 0.40$ ft	L = Length of slot W = Width of slot d = Depth of water over slot
[4] The orifice area should be reduced where clogging is expected. [5] The ratio of clear opening area to total area is 0.8 for P-1-7/8-4 and reticuline grates, 0.9 for P-1-7/8 and 0.6 for P-1-1/8 grates. Curved vane and tilt bar grates are not recommended at sag locations. [6] See Figure ST-5 for other types of throats.				

3.3.6 Inlet Clogging

Inlets are subject to clogging effects (see Photographs ST-5 and ST-6). Selection of a clogging factor reflects the condition of debris and trash on the street. During a storm event, street inlets are usually loaded with debris by the first-flush runoff volume. As a common practice for street drainage, 50% clogging is considered for the design of a single grate inlet and 10% clogging is considered for a single curb-opening inlet. Often, it takes multiple units to collect the stormwater on the street. Since the amount of debris is largely associated with the first-flush volume in a storm event, the clogging factor applied to a multiple-unit street inlet should be decreased with respect to the length of the inlet. Linearly applying a single-unit clogging factor to a multiple-unit inlet leads to an excessive increase in length. For instance, a six-unit inlet under a 50% clogging factor will function as a three-unit inlet. In fact, continuously applying a 50% reduction to the discharge on the street will always leave 50% of the residual flow on the street. This means that the inlet will never reach a 100% capture and leads to unnecessarily long inlets.

Photograph ST-5—Clogging is an important consideration when designing inlets.

Photograph ST-6—Field inlets frequently need maintenance.

With the concept of first-flush volume, the decay of clogging factor to curb opening length is described as (Guo 2000a):

$$C = \frac{1}{N}(C_o + eC_o + e^2C_o + e^3C_o + ... + e^{N-1}C_o) = \frac{C_o}{N}\sum_{i=1}^{i=N} e^{i-1} = \frac{KC_o}{N} \qquad \text{(ST-29)}$$

in which:

C = multiple-unit clogging factor for an inlet with multiple units

C_o = single-unit clogging factor

e = decay ratio less than unity, 0.5 for grate inlet, 0.25 for curb-opening inlet

N = number of units

K = clogging coefficient from Table ST-8

Table ST-8—Clogging Coefficients to Convert Clogging Factor From Single to Multiple Units[1]

N =	1	2	3	4	5	6	7	8	>8
Grate Inlet (*K*)	1	1.5	1.75	1.88	1.94	1.97	1.98	1.99	2
Curb Opening (*K*)	1	1.25	1.31	1.33	1.33	1.33	1.33	1.33	1.33

[1] This table is generated by Equation ST-29 with e = 0.5 and e = 0.25.

When N becomes large, Equation ST-29 converges to:

$$C = \frac{C_o}{N(1-e)} \qquad \text{(ST-30)}$$

For instance, when e = 0.5 and C_o = 50%, C = 1.0/N for a large number of units, N. In other words, only the first unit out of N units will be clogged. Equation ST-30 complies with the recommended clogging factor for a single-unit inlet and decays on the clogging effect for a multiple-unit inlet.

The interception of an inlet on a grade is proportional to the inlet length, and in a sump is proportional to the inlet opening area. Therefore, a clogging factor shall be applied to the length of the inlet on a grade as:

$$L_e = (1 - C)L \qquad \text{(ST-31)}$$

in which L_e = effective (unclogged) length. Similarly, a clogging factor shall be applied to the opening area of an inlet in a sump as:

$$A_e = (1 - C)A \qquad \text{(ST-32)}$$

in which:

A_e = effective opening area

A = opening area

3.4 Inlet Location and Spacing on Continuous Grades

3.4.1 Introduction

Locating (or positioning) stormwater inlets rarely requires design computations. They are simply required in certain locations based upon street design considerations, topography (sumps), and local ordinances. The one exception is the location and spacing of inlets on continuous grades. On a long, continuous grade, stormwater flow increases as it moves down the gutter and picks up more drainage area. As the flow increases, so does the spread. Since the spread (encroachment) is not allowed to exceed some specified maximum, inlets must be strategically placed to remove some of the stormwater from the street. Locating these inlets requires design computations by the engineer.

3.4.2 Design Considerations

The primary design consideration for the location and spacing of inlets on continuous grades is the spread limitation. This was addressed in Section 2.2. Table ST-2 lists pavement encroachment standards for minor storms in the Denver metropolitan area.

Proper design of stormwater collection and conveyance systems makes optimum use of the conveyance capabilities of street gutters. In other words, an inlet is not needed until the spread reaches its allowable limit during the design (minor) storm. To place an inlet prior to that point on the street is not economically efficient. To place an inlet after that point would violate the encroachment standards. Therefore, the primary design objective is to position inlets along a continuous grade at the locations where the allowable spread is about to be exceeded for the design storm.

3.4.3 Design Procedure

Based on the encroachment standard and street geometry, the allowable street hydraulic capacity can be determined using Equation ST-11 or Equation ST-12. This flow rate is then equated to some hydrologic technique (equation) that contains drainage area. In this way, the inlet is positioned on the street so that it will service the requisite drainage area. The process of locating the inlet is accomplished by trial-and-error. If the inlet is moved downstream (or down gutter), the drainage area increases. If the inlet is moved upstream, the drainage area decreases.

The hydrologic technique most often used in urban drainage design is the Rational method. The Rational method was discussed in the RUNOFF chapter. The Rational equation, repeated here for convenience, is:

$$Q = CIA \tag{ST-33}$$

in which:

Q = peak discharge (cfs)

C = runoff coefficient described in the RUNOFF chapter

I = design storm rainfall intensity (in/hr) described in the RAINFALL chapter

A = drainage area (acres)

As previously mentioned, the peak discharge is found using the allowable spread and street geometry. The runoff coefficient is dependent on the land use as discussed in the RUNOFF chapter. The rainfall intensity is discussed in the RAINFALL chapter. The drainage area is the unknown variable to be solved.

Once the first inlet is positioned along a continuous grade, an inlet type and size can be specified. The first inlet's hydraulic capacity is then assessed. Generally, the inlet will not capture all of the gutter flow. In fact, it is uneconomical to size an inlet (on continuous grades) large enough to capture all of the gutter flow. Instead, some carryover flow is expected. This practice reduces the amount of new flow that can be picked up at the next inlet. However, each inlet should be positioned at the location where the allowable spread is about to reach its allowable limit.

The gutter discharge for inlets, other than the first inlet, consists of the carryover from the upstream inlet plus the stormwater runoff generated from the intervening local drainage area. The carryover flow from the upstream inlet is added to the peak flow rate obtained from the Rational method for the intervening local drainage area. The resulting peak flow is approximate since the carryover flow peak and the local runoff peak do not necessarily coincide.

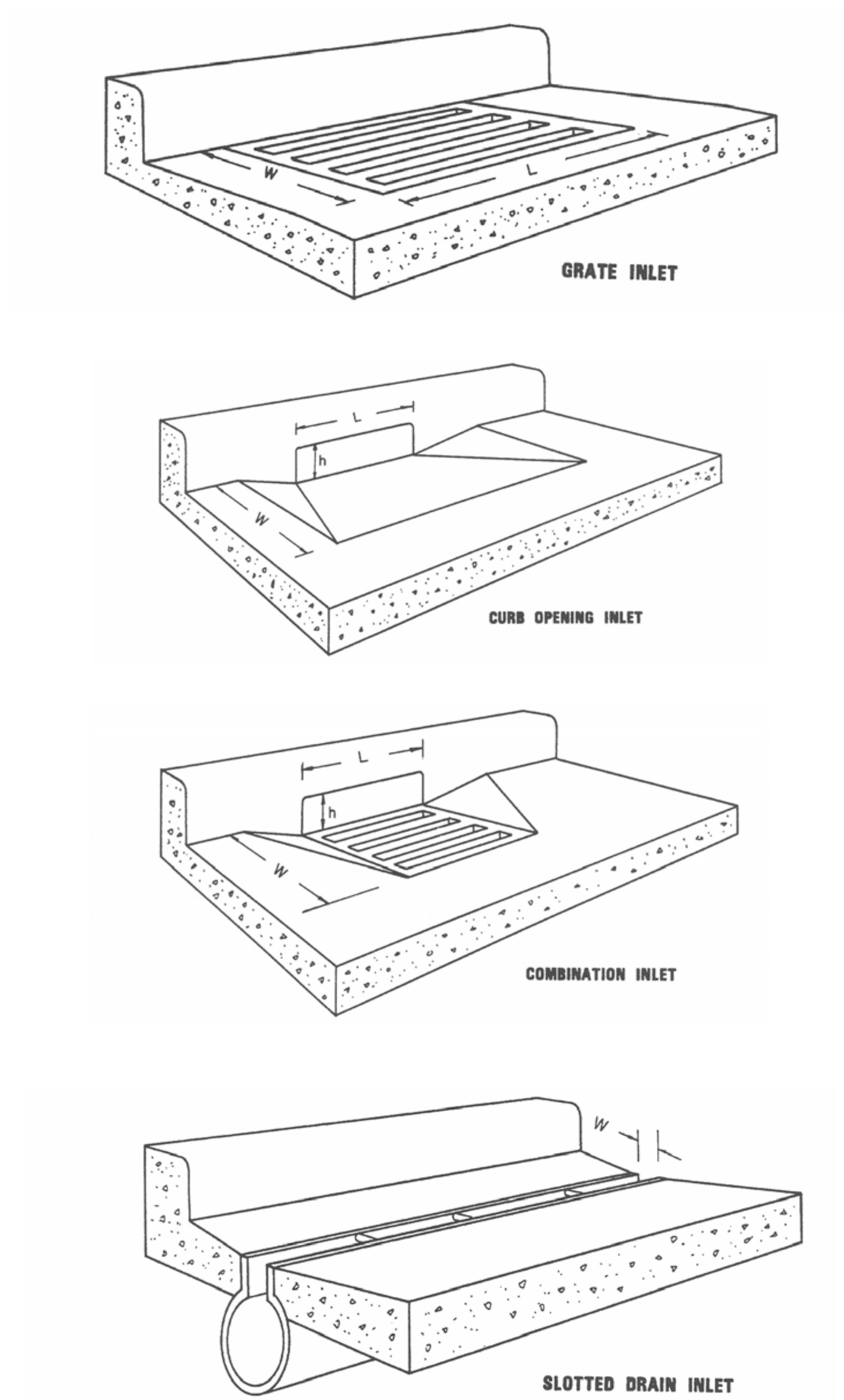


Figure ST-4—Perspective Views of Grate and Curb-Opening Inlets

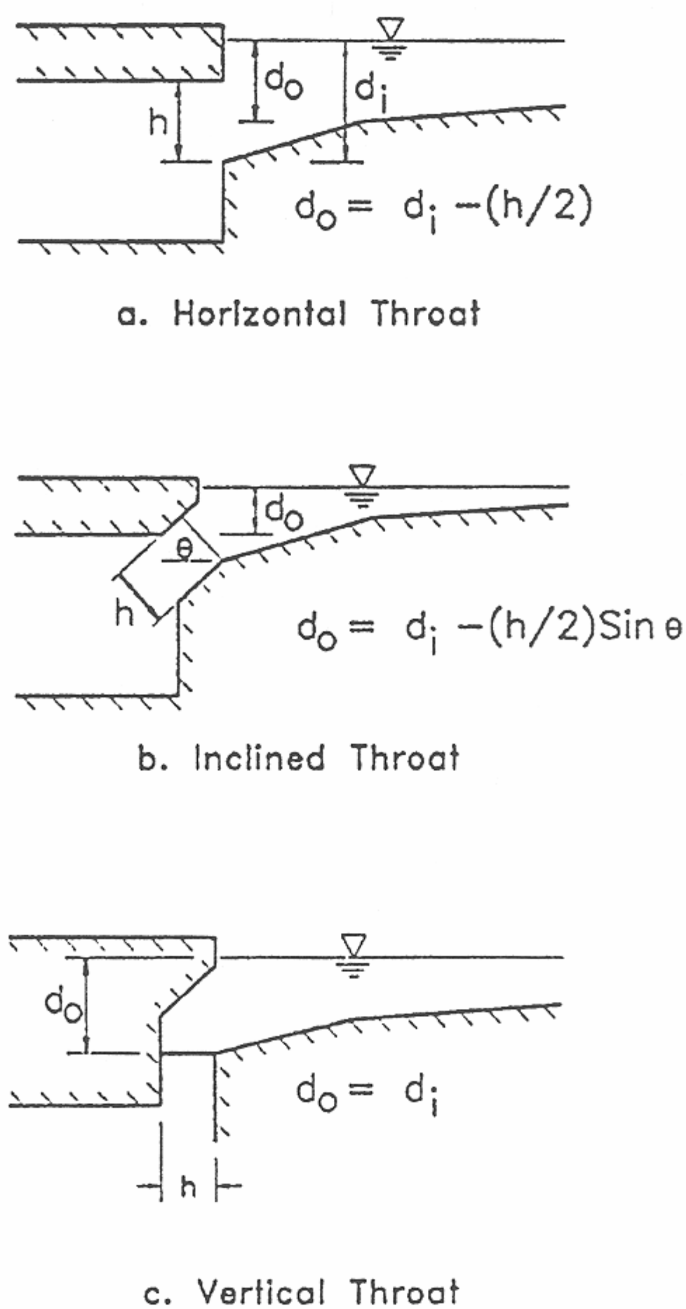


Figure ST-5—Curb-Opening Inlets

4.0 STORM SEWERS

4.1 Introduction

Once stormwater is collected from the street surface by an inlet, it is directed into the storm sewer system. The storm sewer system is comprised of inlets, pipes, manholes, bends, outlets, and other appurtenances. The stormwater passes through these components and is discharged into a stormwater management device (e.g., infiltration trench, stormwater pond, constructed wetland, etc.) to mitigate adverse downstream effects or discharged directly to a natural or constructed watercourse. Stormwater management devices are constructed to reduce the peak discharge, decrease the volume of runoff, and/or improve the water quality.

Apart from inlets, manholes are the most common appurtenance in storm sewer systems. Their primary functions include:

- Providing maintenance access.
- Providing ventilation.
- Serving as junctions when two or more pipes merge.
- Providing flow transitions for changes in pipe size, slope, and alignment.

Manholes are generally made of pre-cast or cast-in-place reinforced concrete. They are typically 4 to 5 feet in diameter and are required at regular intervals, even in straight sections, for maintenance reasons. Standard size manholes cannot accommodate large pipes, so junction chambers are used for that application.

Other appurtenances are not as common as manholes, but serve vital functions. Occasionally, bends and transitions are accomplished without manholes, particularly for large pipe sizes. These sections provide gradual transitions in size or alignment to minimize energy losses. Outlet structures are transitions from pipe flow into open channel flow or still water (e.g., ponds, lakes, etc.). Their primary function is to minimize erosion in the receiving water body. Flow splitters separate incoming flow and send it in two or more directions. Flow deflectors are used to minimize energy losses in manholes, junction chambers, and flow splitters. Flap gates are placed on outlets to prevent backflow in areas subject to high tailwater or flood flow.

4.2 Design Process, Considerations, and Constraints

The design of a storm sewer system requires a large data collection effort. The data requirements in the proposed service area include topography, drainage boundaries, soil types, and locations of any existing storm sewers, inlets, and manholes. In addition, identification of the type and location of other utilities is necessary. Alternative layouts of a new system (or modifications to an existing system) can be investigated using this data.

Alternative system layouts rely largely on street right-of-ways and topography. Most layouts are dendritic (tree) networks that follow the street pattern. Dendritic networks collect stormwater from a broad area and tend to converge in the downstream direction. Looping networks shall be avoided because of their complex hydraulics and potentially higher cost. Each layout should contain inlet and manhole locations, drainage boundaries serviced by the inlets, storm sewer locations, flow directions, and outlet locations. A final layout selection is made from the viable alternatives based on likely system performance and cost.

Once a final layout is chosen, storm sewers are sized using hydrologic techniques (to determine peak flows) and hydraulic analysis (to determine pipe capacities). This is accomplished by designing the upstream pipes first and moving downstream. Pipes sizes smaller than 15 inches are not recommended for storm sewers. Pipes generally increase in size moving downstream since the drainage area is increasing. It is not good design practice to decrease the pipe size moving downstream, even if a steeper slope is encountered that will provide sufficient capacity with a smaller pipe. The potential for clogging is always a concern.

Storm sewers are typically sized to convey the minor storm without surcharging using normal flow techniques. In other words, the flow is in a pipe that is flowing *just full* determined by open channel hydraulics calculations.

The minor storm is defined by the return interval that usually varies from the 2-year to the 10-year storm depending on the importance of the infrastructure being served. Refer to the POLICY chapter for guidance regarding selection of the design storm.

Manholes are located in the system prior to and in conjunction with pipe design. Most manhole locations are dictated by proper design practices. For example, manholes are required whenever there is a change in pipe size, alignment, or slope. In addition, manholes are required at pipe junctions. Manholes are also required along straight sections of pipe for maintenance purposes. The distance between manholes is dependent on pipe size. The invert of a pipe leaving a manhole should be at least 0.1 foot lower than the incoming pipe to ensure positive low flows through the manhole. Whenever possible, match the crown of the pipe elevations when the downstream pipe is larger to minimize backwater effects on the upstream pipe.

Once storm sewers are sized and manhole locations are determined, the performance of the sewer system must be evaluated using energy grade line calculations starting at the downstream terminus of the system. As stormwater flows through the storm sewer system, it encounters many flow transitions. These transitions include changes in pipe size, slope, and alignment, as well as entrance and exit conditions. All of these transitions produce energy losses, usually expressed as head losses. These losses must be accounted for to ensure that inlets and manholes do not surcharge to a significant degree (i.e., produce street flooding). This is accomplished using hydraulic grade line (HGL) calculations as a check on pipe sizes and system losses. If significant surcharging occurs, the pipe sizes should be increased. High tailwater conditions at the storm sewer outlet may also produce surcharging. This can also be accounted for using HGL calculations.

4.3 Storm Sewer Hydrology

4.3.1 Peak Runoff Prediction

The Rational method is commonly used to determine the peak flows that storm sewers must be able to convey. It is an appropriate method due to the small drainage areas typically involved. It is also relatively easy to use and provides reasonable estimates of peak runoff. The total drainage area contributing flow to a particular storm sewer is often divided up into smaller subcatchments. The Rational method is described in the RUNOFF chapter of this *Manual*.

The first pipe in a storm sewer system is designed using Equation ST-33 to determine the peak flow. Downstream pipes receive flow from the upstream pipes as well as local inflows. The Rational equation applied to the downstream pipes is:

$$Q_p = I \sum_{j=1}^{n} C_j A_j \tag{ST-34}$$

in which:

I = design rainfall average intensity, over the time of concentration T_c (in/hr)

n = number of subareas above the stormwater pipe

C_j = runoff coefficient of subarea j

A_j = drainage area of subarea j (acres)

In using this equation, it is evident that the peak flow changes at each design point since the time of concentration, and thus the average intensity, changes at each design point. It is also evident that the time of concentration coming from the local inflow may differ from that coming from upstream pipes. Normally, the longest time of concentration is chosen for design purposes. If this is the case, all of the subareas above the design point will be included in Equation ST-34, and it usually produces the largest peak flow. On rare occasions, the peak flow from a shorter path may produce the greater peak discharge if the downstream areas are heavily developed. It is good practice to check all alternative flow paths and tributary areas to determine the tributary zone that produces the biggest design flow.

4.4 Storm Sewer Hydraulics (Gravity Flow in Circular Conduits)

4.4.1 Flow Equations and Storm Sewer Sizing

Storm sewer flow is usually unsteady and non-uniform. However, for design purposes it is assumed to be steady and uniform at the peak flow rate. Therefore, Manning's equation is appropriate, which can be stated as:

$$Q = \frac{1.49}{n} A R^{2/3} S_f^{1/2} \qquad \text{(ST-35)}$$

in which:

Q = flow rate (cfs)

n = Manning's roughness factor

A = flow area (ft^2)

R = hydraulic radius (ft)

S_f = friction slope (normally the storm sewer slope) (ft/ft)

For full flow in a circular storm sewer,

$$A = A_f = \frac{\pi D^2}{4} \qquad \text{(ST-36)}$$

$$R = R_f = \frac{D}{4} \qquad \text{(ST-37)}$$

in which:

D = pipe diameter

A_f = flow area at full flow (ft^2)

R_f = hydraulic radius at full flow (ft)

If the flow is pressurized (i.e., surcharging at the inlets or manholes is occurring), $S_f \neq S_o$ where S_o is the longitudinal bottom slope of the storm sewer. Design of storm sewers assumes *just full flow*, a reference condition referring to steady, uniform flow with a flow depth, y, nearly equal to the pipe diameter, D. Just full flow discharge, Q_f , is calculated using:

$$Q_f = \frac{1.49}{n} A_f R_f^{2/3} S_o^{1/2} \qquad \text{(ST-38)}$$

Computations of flow characteristics for partial depths in circular pipes are tedious. Design aids like Figure ST-6 are very helpful when this is necessary.

Storm sewers are sized to flow *just full* (i.e., as open channels using nearly the full capacity of the pipe). The design discharge is determined first using the Rational equation as previously discussed, then the Manning's equation is used (with $S_f = S_o$) to determine the required pipe size. For circular pipes,

$$D_r = \left[\frac{2.16 n Q_p}{\sqrt{S_o}} \right]^{3/8} \qquad \text{(ST-39)}$$

in which D_r is the minimum size pipe required to convey the design flow and Q_p is peak design flow. However, the pipe diameter that should be used in the field is the next standard pipe size larger than D_r.

The typical process proceeds as follows. Initial storm sewer sizing is performed first using the Rational equation in conjunction with Manning's equation. The Rational equation is used to determine the peak discharge that storm sewers must convey. The storm sewers are then initially sized using Manning's equation assuming uniform, steady flow at the peak. Finally, these initial pipe sizes are checked using the energy equation by accounting for all head losses. If the energy computations detect surcharging at manholes or inlets, the pipe sizes are increased.

4.4.2 Energy Grade Line and Head Losses

Head losses must be accounted for in the design of storm sewers in order to find the energy grade line (EGL) and the hydraulic grade line (HGL) at any point in the system. The FHWA (1996) gives the following equation as the basis for calculating the head losses at inlets, manholes, and junctions (h_{LM}, in feet):

$$h_{LM} = K_o C_D C_d C_Q C_p C_B \left(\frac{V_o^2}{2g} \right)$$

in which:

K_o = initial loss coefficient

V_o = velocity in the outflow pipe (ft/sec)

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

C_D, C_d, C_Q, C_p, and C_B = correction factors for pipe size, flow depth, relative flow, plunging flow and benching

However, this equation is valid only if the water level in the receiving inlet, junction, or manhole is above the invert of the incoming pipe. Otherwise, another protocol has to be used to calculate head losses at manholes. What follows is a modified FHWA procedure that the engineer can use to calculate the head losses and the EGL along any point in a storm sewer system.

The EGL represents the energy slope between the two adjacent manholes in a storm sewer system. A manhole may have multiple incoming sewers, but only one outgoing sewer. Each sewer and its downstream and upstream manholes form a *sewer-manhole* unit. The entire storm sewer system can be decomposed into a series of sewer-manhole units that satisfy the energy conservation principle. The computation of the EGL does this by repeating the energy-balancing process for each sewer-manhole unit.

As illustrated in Figure ST-6, a *sewer-manhole* unit has four distinctive sections. Section 1 represents the downstream manhole, Section 2 is the point at the exit of the incoming sewer just as enters this manhole, Section 3 is at the entrance to this sewer at the upstream manhole, and Section 4 represents the upstream manhole. For each *sewer-manhole* unit, the head losses are determined separately in two parts as:

- Friction losses through the sewer pipe, and
- Juncture losses at the manhole.

The discussion that follows explains how to apply energy balancing to calculate the EGL through each *sewer-manhole* unit.

4.4.2.1 Losses at the Downstream Manhole—Section 1 to Section 2

The continuity of the EGL is determined between the flow conditions at centerline of the downstream manhole, Section 1, and the exit of the incoming sewer, Section 2, as illustrated in Figure ST-6 and an idealized EGL and HGL profiles in Figure ST-7.

At Section 2 there may be pipe-full flow, critical/supercritical open channel flow, or sub-critical open channel flow. If the sewer crown at the exit is submerged, the EGL at the downstream manhole provides a tailwater condition; otherwise, the manhole drop can create a discontinuity in the EGL. Therefore, it is necessary to evaluate the two possibilities, namely:

$$E_2 = Max(\frac{V_2^{\,2}}{2g} + Y_x + Z_2, E_1) \qquad \text{(ST-40)}$$

in which:

E_2 = EGL at Section 2

V_2 = sewer exit velocity in fps

Y_2 = flow depth in feet at the sewer exit

Z_2 = invert elevation in feet at the sewer exit

E_1 = tailwater at Section 1

Equation ST-40 states that the highest EGL value shall be considered as the downstream condition. If the manhole drop dictates the flow condition at Section 2, a discontinuity is introduced into the EGL.

4.4.2.2 Losses in the Pipe, Section 2 to Section 3.

The continuity of the EGL upstream of the manhole depends on the friction losses through the sewer pipe. The flow in the sewer pipe can be one condition or a combination of open channel flow, full flow, or pressurized (surcharge) flow.

When a free surface exists through the pipe length, the open channel hydraulics apply to the backwater surface profile computations. The friction losses through the sewer pipe are the primary head losses for the type of water surface profile in the sewer. For instance, the sewer pipe carrying a subcritical flow may have an M-1 water surface profile if the water depth at the downstream manhole is greater than normal depth in the sewer or an M-2 water surface profile if the water depth in the downstream manhole is lower than normal depth. Under an alternate condition, the pipe carrying a supercritical flow may have an S-2 water surface profile if the pipe entering the downstream manhole is not submerged; otherwise, a hydraulic jump is possible within the sewer.

When the downstream sewer crown is submerged to a degree that the entire sewer pipe is under the HGL, the head loss for this full flow condition is estimated by pressure flow hydraulics.

When the downstream sewer crown is slightly submerged, the downstream end of the sewer pipe is surcharged, but the upstream end of the sewer pipe can have open channel flow. The head loss through a surcharge flow depends on the flow regime. For a subcritical flow, the head loss is the sum of the friction losses for the full flow condition and for the open channel flow condition. For a supercritical flow, the head loss may involve a hydraulic jump. To resolve which condition governs, culvert hydraulic principles can be used under both inlet and outlet control conditions and the governing condition is the one that produces the highest HGL at the upstream manhole.

Having identified the type of flow in the sewer pipe, the computation of friction losses begins with the determination of friction slope. The friction loss and energy balance are calculated as:

$$h_f = LS_f \qquad \text{(ST-41)}$$

$$E_3 = E_2 + \sum h_f \qquad \text{(ST-42)}$$

in which:

h_f = friction loss

L = length in feet of sewer pipe

S_f = friction slope in the pipe in ft/ft

E_3 = EGL at the upstream end of sewer pipe

4.4.2.3 Losses at the Upstream Manhole, Section 3 to Section 4

Additional losses may be introduced at the sewer entrance. The general formula to estimate the entrance loss is:

$$h_E = K_E \frac{V^2}{2g} \tag{ST-43}$$

in which:

h_E = entrance loss in feet

V = pipe-full velocity in feet per second in the incoming sewer

K_E = entrance loss coefficient between 0.2 to 0.5

In the modeling of sewer flow, the sewer entrance coefficients can be assumed to be part of the bend loss coefficient.

The energy principle between Sections 3 and 4 is determined by:

$$E_4 = E_3 + h_E \tag{ST-44}$$

in which E_4 = EGL at Section 4.

4.4.2.4 Juncture and Bend Losses at the Upstream Manhole, Section 4 to Section 1

The analysis from Section 4 of the downstream *sewer-manhole* unit to Section 1 of the upstream *sewer-manhole* unit consists only of juncture losses through the manhole. To maintain the conservation of energy through the manhole, the outgoing energy plus the energy losses at the manhole have to equal the incoming energy. Often a manhole is installed for the purpose of maintenance, deflection of the sewer line, change of the pipe size, and as a juncture for incoming laterals. Although there are different causes for juncture losses, they are often, rightly or wrongly, considered as a minor loss in the computation of the EGL. These juncture losses in the sewer system are determined solely by the local configuration and geometry and not by the length of flow in the manhole.

4.4.2.4.1 Bend/Deflection Losses

The angle between the incoming sewer line and the centerline of the exiting main sewer line introduces a bend loss to the incoming sewer. Bend loss is estimated by:

$$h_b = K_b \frac{V^2}{2g} \tag{ST-45}$$

in which:

h_b = bend loss in feet

V = full flow velocity in feet per second in the incoming sewer

K_b = bend loss coefficient

As shown in Figure ST-8 and Table ST-9, the value of K_b depends on the angle between the exiting sewer line and the existence of manhole bottom shaping. A shaped manhole bottom or a deflector guides the flow and reduces bend loss. Figure ST-9 illustrates four cross-section options for the shaping of a manhole bottom. Only sections "c. Half" and "d. Full" can be considered for the purpose of using the bend loss coefficient for the curve on Figure ST-9 labeled as "Bend at Manhole, Curved or Shaped."

Because a manhole may have multiple incoming sewers, Equation ST-45 shall be applied to each incoming sewer based on its incoming angle, and then the energy principle between Sections 4 and 1 is calculated as:

$$E_1 = E_4 + h_b \tag{ST-46}$$

4.4.2.4.2 Lateral Juncture Losses

In addition to the bend loss, the lateral juncture loss is also introduced because of the added turbulence and eddies from the lateral incoming flows. The lateral juncture loss is estimated as:

$$h_j = \frac{V_o^2}{2g} - K_j \frac{V_i^2}{2g} \tag{ST-47}$$

in which:

h_j = lateral loss in feet

V_o = full flow velocity in feet per second in the outgoing sewer

K_j = lateral loss coefficient

V_i = full flow velocity in feet per second in the incoming sewer

In modeling, a manhole can have multiple incoming sewers, one of which is the main (i.e., trunk) line, and one outgoing sewer. As shown in Table ST-9, the value of K_j is determined by the angle between the lateral incoming sewer line and the outgoing sewer line.

Table ST-9—Bend Loss and Lateral Loss Coefficients (FHWA 1996)

Angle in Degree	Bend Loss Coefficient for Curved Deflector in the Manhole	Bend Loss Coefficient for Non-shaping Manhole	Lateral Loss Coefficient on Main Line Sewer
Straight Through	0.05	0.05	Not Applicable
22.50	0.10	0.13	0.75
45.00	0.28	0.38	0.50
60.00	0.48	0.63	0.35
90.00	1.01	1.32	0.25

At a manhole, the engineer needs to identify the main incoming sewer line (the one that has the largest inflow rate) and determine the value of K_j for each lateral incoming sewer line. To be conservative, the smallest K_j is recommended for Equation ST-44, and the lateral loss is to be added to the outfall of the incoming main line sewer as:

$$E_1 = E_4 + h_b + h_j$$ (h_j is applied to main sewer line only) (ST-48)

The difference between the EGL and the HGL is the flow velocity head. The HGL at a manhole is calculated by:

$$H_1 = E_1 - \frac{V_o^{\ 2}}{2g} \qquad \text{(ST-49)}$$

The energy loss between two manholes is defined as:

$$\Delta E = (E_1)_{upstream} - (E_1)_{downstream} \qquad \text{(ST-50)}$$

in which ΔE = energy loss between two manholes. It is noted that ΔE includes the friction loss, juncture loss, bend loss, and manhole drop.

4.4.2.5 Transitions

In addition to *sewer-manhole* unit losses, head losses in a storm sewer can occur due to a transition in the pipe itself, namely, gradual pipe expansion. Transition loss, h_{LE}, in feet, can be determined using:

$$h_{LE} = K_e \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right) \qquad \text{(ST-51)}$$

in which K_e is the expansion coefficient and subscripts 1 and 2 refer to upstream and downstream of the transition, respectively. The value of the expansion coefficient, K_e, may be taken from Table ST-10 for free surface flow conditions in which the angle of cone refers to the angle between the sides of the tapering section (see Figure ST-10).

Table ST-10—Head Loss Expansion Coefficients in Non-Pressure Flow (FHWA 1996)

D_2/D_1	Angle of Cone						
	10°	20°	30°	40°	50°	60°	70°
1.5	0.17	0.40	1.06	1.21	1.14	1.07	1.00
3	0.17	0.40	.86	1.02	1.06	1.04	1.00

Head losses due to gradual pipe contraction, h_{LC}, in feet, are determined using:

$$h_{LC} = K_c \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g} \right) \quad \text{(ST-52)}$$

in which K_c = contraction coefficient. Typically, K_c = 0.5 provides reasonable results.

This *Manual* does not recommend pipe contractions for storm sewers.

4.4.2.6 Curved Sewers

Head losses due to curved sewers (sometimes called radius pipe), h_{Lr}, in feet, can be determined using:

$$h_{Lr} = K_r \frac{V^2}{2g} \quad \text{(ST-53)}$$

in which K_r = curved sewer coefficient from Figure ST-8.

4.4.2.7 Losses at Storm Sewer Exit

Head losses at storm sewer outlets, h_{LO}, are determined using:

$$h_{LO} = \frac{V_o^2}{2g} - \frac{V_d^2}{2g} \quad \text{(ST-54)}$$

in which V_o is the velocity in the outlet pipe, and V_d is the velocity in the downstream channel. When the storm sewer discharges into a reservoir or into air because there is no downstream channel, V_d = 0 and one full velocity head is lost at the exit.

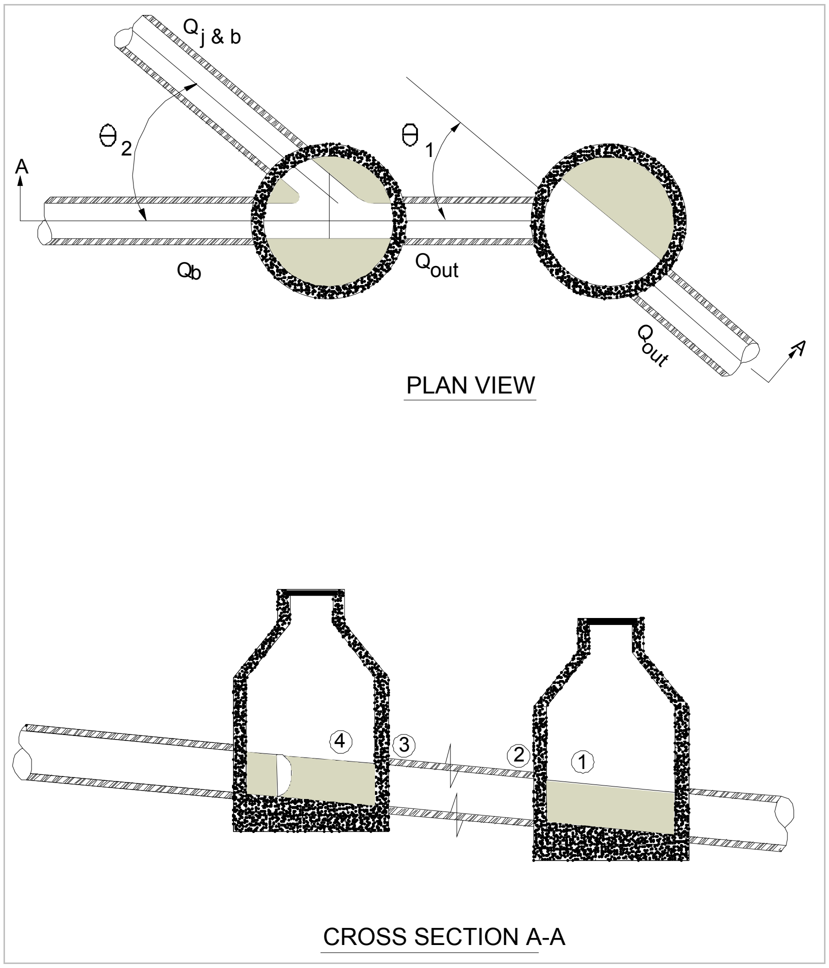


Figure ST-6—A Manhole-Sewer Unit

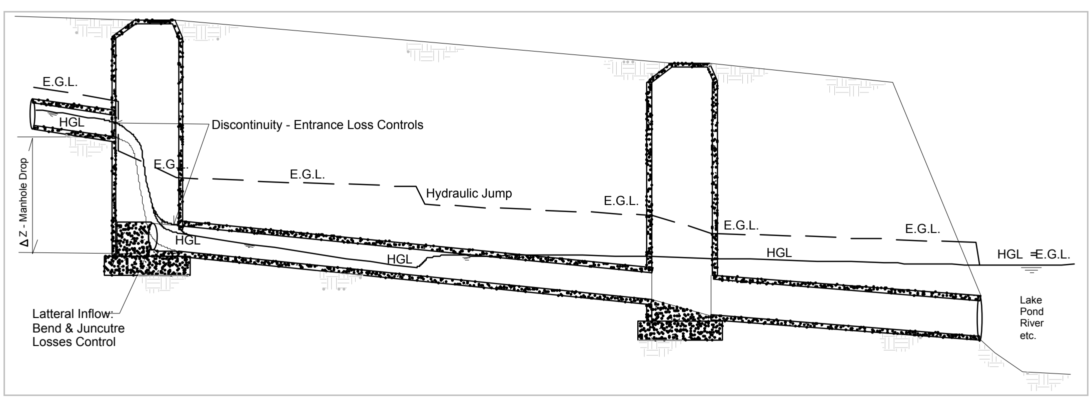


Figure ST-7—Hydraulic and Energy Grade Lines

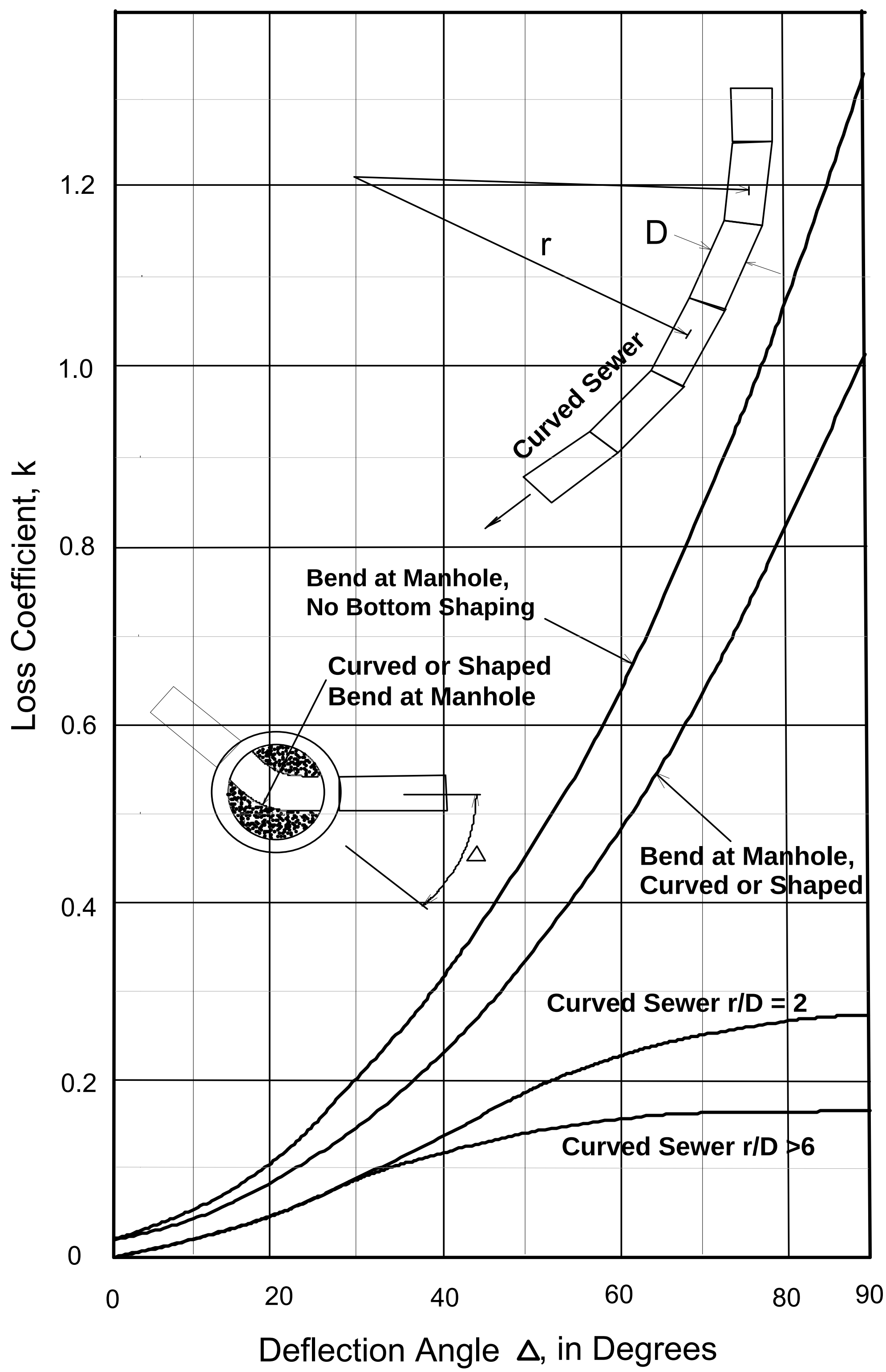


Figure ST-8—Bend Loss Coefficients

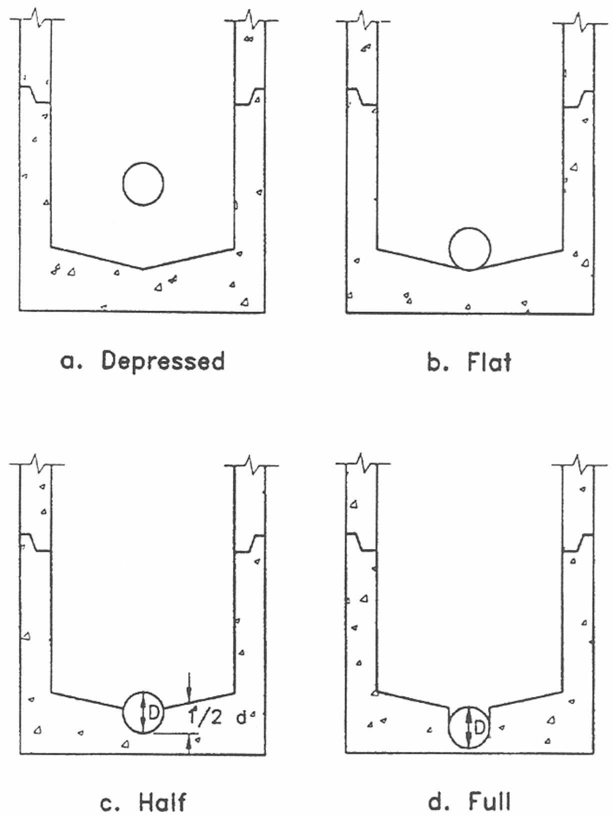


Figure ST-9—Access Hole Benching Methods

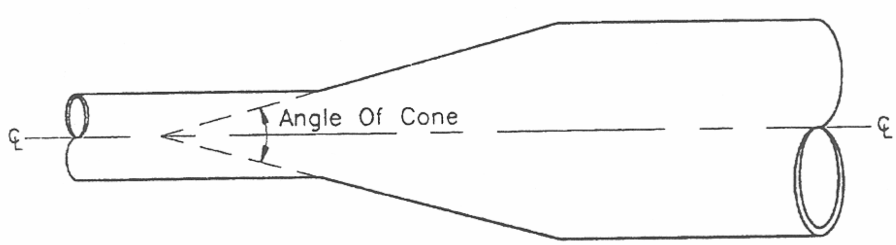


Figure ST-10—Angle of Cone for Pipe Diameter Changes

5.0 SPREADSHEETS

The UD-Inlet Spreadsheet provides quick solutions for many of the computations described in this chapter. A brief summary of each worksheet of the spreadsheet is provided below. Please note that some of the symbols and nomenclature in the worksheet do not correspond exactly with the nomenclature of the text. The text and the spreadsheets are computationally equivalent.

1. The **Q-Major Worksheet** calculates the gutter capacity for major storm events.
2. The **Q-Minor Worksheet** calculates the gutter capacity for minor storm events.
3. The **Flow Worksheet** provides Rational method hydrologic computations for streets and inlets.
4. The **Street Hy Worksheet** calculates gutter conveyance capacity ***and must be used in conjunction with any of the inlet capacity worksheets***.
5. The **Grate-G Worksheet** calculates the capacity of grate inlets on a grade.
6. The **Curb-G Worksheet** calculates the capacity of curb opening inlets on a grade.
7. The **Slot-G Worksheet** calculates the capacity of slotted inlets on a grade.
8. The **Grate G Worksheet** calculates the capacity of grate inlets in a sump.
9. The **Curb-G Worksheet** calculates the capacity of curb opening inlets in a sump.
10. The **Slot-G Worksheet** calculates the capacity of slotted inlets in a sump.

6.0 EXAMPLES

6.1 Example—Triangular Gutter Capacity

A triangular gutter has a longitudinal slope of S_L = 0.01, cross slope of S_x = 0.02, and a curb depth of 6 inches. Determine the flow rate and flow depth if the spread is limited to 9 feet.

Using Equation ST-1,

$$Q = [(0.56)(0.02)^{5/3}(0.01)^{1/2}(9.0)^{8/3}]/(0.016) = 1.81 \text{ cfs}$$

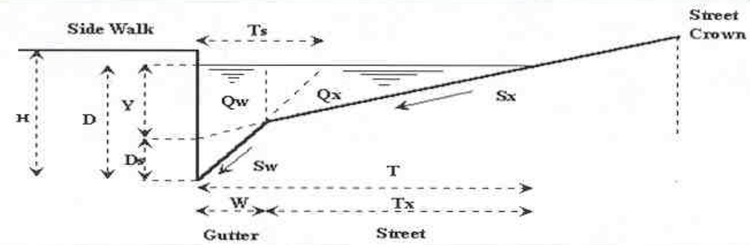
This is the theoretical flow rate. Then by using Equation ST-2,

$$y = (9.0)(0.02) = 0.18 \text{ ft}$$

Note that the computed flow depth is less than the curb height of 6 inches (0.5 feet). If it was not, the spread and associated flow rate would need to be reduced. A solution of this example using the **Q-Minor Worksheet** of the UD-Inlet Spreadsheet is included below.

Gutter Storm Water Conveyance Capacity for Minor Event

Project: Example 6.1 - Triangular Gutter Capacity
Street ID:



Gutter Geometry			
Curb Height	H =	6.00	inches
Gutter Width	W =	2.00	ft
Gutter Depression	Ds =	0.00	inches
Street Transverse Slope	Sx =	0.0200	ft/ft
Street Longitudinal Slope	So =	0.0100	ft/ft
Manning's Roughness	N =	0.016	
Maximum Allowable Water Spread for Minor Event	T =	9.00	ft
Gutter Conveyance Capacity Based On Maximum Water Spread			
Gutter Cross Slope:	Sw =	0.0200	ft/ft
Water Depth without Gutter Depression	Y =	0.18	ft
Water Depth with a Gutter Depression	D =	0.18	ft
Spread for Side Flow on the Street	Tx =	7.00	ft
Spread for Gutter Flow along Gutter Slope	Ts =	9.00	ft
Flow Rate Carried by Width Ts	Qws =	1.8	cfs
Flow Rate Carried by Width (Ts - W)	Qww =	0.9	cfs
Gutter Flow	Qw =	0.9	cfs
Side Flow	Qx =	0.9	cfs
Maximum Spread Capacity	**Q-Tm =**	**1.8**	**cfs**
Gutter Full Conveyance Capacity Based on Curb Height			
Spread for Side Flow on the Street	Tx =	25.00	ft
Spread for Gutter Flow along Gutter Slope	Ts =	25.00	ft
Flow Rate Carried by Width Ts	Qws =	27.5	cfs
Flow Rate Carried by Width (Ts - W)	Qww =	22.0	cfs
Gutter Flow	Qw =	5.5	cfs
Side Flow	Qx =	27.5	cfs
Gutter Full Capacity	**Q-full =**	**33.0**	**cfs**
Gutter Design Conveyance Capacity Based on Min(Q-Tm, R*Q-full)			
Reduction Factor for Minor Event	R-min =	1.00	
Gutter Design Conveyance Capacity for Minor Event	**Q-min =**	**1.8**	**cfs**

6.2 Example—Composite Gutter Capacity

Determine the discharge in a composite gutter section if the allowable spread is 9.0 feet, the gutter width, W, is 2 feet, and the gutter depression is 1.5 inches. The street's longitudinal slope is 0.01, the cross slope is 0.02, and the curb height is 6 inches.

Equation ST-8 yields the cross slope of the depressed gutter as:

$$S_w = 0.02 + (1.5/12)/2 = 0.083$$

Using Figure ST-1a, W = 2 feet, T_s = 7 feet. Equation ST-1 can now be used to find the flow in the street section.

$$Q_s = [(0.56)(0.02)^{5/3}(0.01)^{1/2}(7.0)^{8/3}]/(0.016) = 0.92 \text{ cfs}$$

Now with S_w/S_x = 0.083/0.02 = 4.1, T/W = 9.0/2.0 = 4.5, and T/W - 1 = 3.5, by using Equation ST-7,

$$E_o = \frac{1}{\left\{1 + \dfrac{4.1}{\left[1 + \dfrac{4.1}{3.5}\right]^{8/3} - 1.0}\right\}} = 0.63$$

Now the theoretical flow rate can be found using Equation ST-6 as:

$$Q = [(0.92)/(1 - 0.63)] = 2.49 \text{ cfs}$$

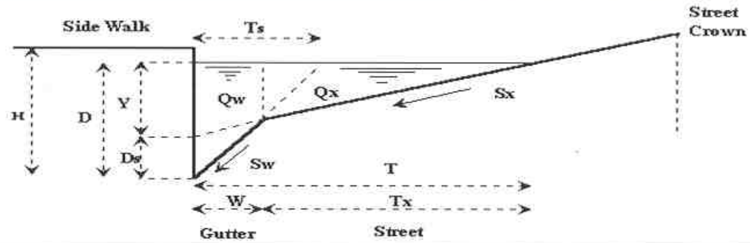
Then by using Equation ST-9,

$$y = 2/12 + (9.0)(0.02) = 0.35 \text{ feet}$$

Note that the computed flow depth is less than the curb height of 6 inches (0.5 feet). Also note that this is the same gutter section as Example 6.1 except for the depressed gutter section. This change has increased the gutter capacity by 38% and almost doubled the depth of flow. A spreadsheet solution of this example problem using the **Q-Minor Worksheet** of the UD-Inlet Spreadsheet is included below.

Gutter Storm Water Conveyance Capacity for Minor Event

Project: Example 6.2 - Composite Gutter Capacity
Street ID:



Gutter Geometry			
Curb Height	H =	6.00	inches
Gutter Width	W =	2.00	ft
Gutter Depression	Ds =	1.50	inches
Street Transverse Slope	Sx =	0.0200	ft/ft
Street Longitudinal Slope	So =	0.0100	ft/ft
Manning's Roughness	N =	0.016	
Maximum Allowable Water Spread for Minor Event	T =	9.00	ft
Gutter Conveyance Capacity Based On Maximum Water Spread			
Gutter Cross Slope:	Sw =	0.0825	ft/ft
Water Depth without Gutter Depression	Y =	0.18	ft
Water Depth with a Gutter Depression	D =	0.31	ft
Spread for Side Flow on the Street	Tx =	7.00	ft
Spread for Gutter Flow along Gutter Slope	Ts =	3.70	ft
Flow Rate Carried by Width Ts	Qws =	1.8	cfs
Flow Rate Carried by Width (Ts - W)	Qww =	0.2	cfs
Gutter Flow	Qw =	1.6	cfs
Side Flow	Qx =	0.9	cfs
Maximum Spread Capacity	**Q-Tm =**	**2.5**	**cfs**
Gutter Full Conveyance Capacity Based on Curb Height			
Spread for Side Flow on the Street	Tx =	18.75	ft
Spread for Gutter Flow along Gutter Slope	Ts =	6.06	ft
Flow Rate Carried by Width Ts	Qws =	6.7	cfs
Flow Rate Carried by Width (Ts - W)	Qww =	2.3	cfs
Gutter Flow	Qw =	4.4	cfs
Side Flow	Qx =	12.8	cfs
Gutter Full Capacity	**Q-full =**	**17.1**	**cfs**
Gutter Design Conveyance Capacity Based on Min(Q-Tm, R*Q-full)			
Reduction Factor for Minor Event	R-min =	1.00	
Gutter Design Conveyance Capacity for Minor Event	**Q-min =**	**2.5**	**cfs**

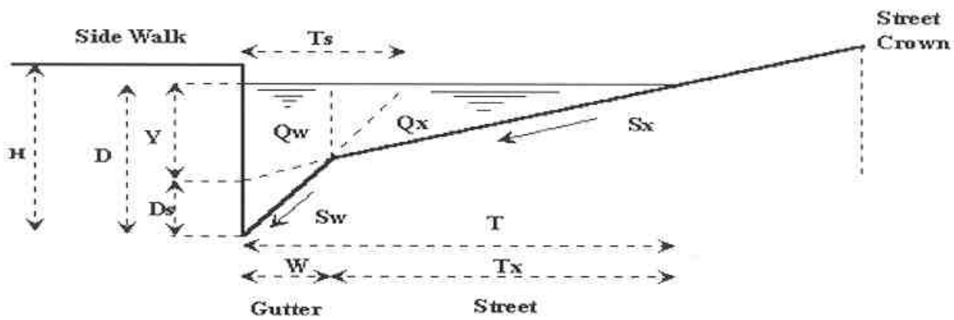
6.3 Example—Composite Gutter Spread

A composite gutter section has S_x = 0.02, S_L = 0.01, a = 2 inches, n = 0.016 and W = 2 feet. Determine the spread, T, at Q = 2.5 cfs (Akan and Houghtalen 2002).

Solving this problem by using Equations ST-6 and ST-7 requires a trial-and-error procedure since the equations are implicit in T. In the trial-and-error procedure, the value of T is guessed, and Q is calculated using Equations ST-6 and ST-7. If the calculated Q is the same as the given Q, the guessed value of T is correct. Otherwise, the procedure is repeated using another guess for T. In this case, a guessed value of a spread equal to 8.5 feet yields the correct flow of 2.5 cfs. A direct solution is possible by using the **Street Hy Worksheet** of the UD-Inlet Spreadsheet.

GUTTER CONVEYANCE CAPACITY

Project = Example 6.3 - Composite Gutter Spread
Street ID =



Street Geometry (Input)			
Design Discharge in the Gutter	**Qo =**	**2.5**	**cfs**
Curb Height	H =	6.00	inches
Gutter Width	W =	2.00	ft
Gutter Depression	Ds =	2.00	inches
Street Transverse Slope	Sx =	0.0200	ft/ft
Street Longitudinal Slope	So =	0.0100	ft/ft
Manning's Roughness	N =	0.016	

Gutter Conveyance Capacity			
Gutter Cross Slope	Sw =	0.10	ft/ft
Water Spread Width	T =	8.58	ft
Water Depth without Gutter Depression	Y =	0.17	ft
Water Depth with a Gutter Depression	D =	0.34	ft
Spread for Side Flow on the Street	Tx =	6.58	ft
Spread for Gutter Flow along Gutter Slope	Ts =	3.27	ft
Flowrate Carried by Width Ts	Qws =	1.9	cfs
Flowrate Carried by Width (Ts - W)	Qww =	0.2	cfs
Gutter Flow	Qw =	1.7	cfs
Side Flow	Qx =	0.8	cfs
Total Flow (Check against Qo)	**Qs =**	**2.5**	**cfs**
Gutter Flow to Design Flow Ratio	Eo =	0.69	
Equivalent Slope for the Street	Se =	0.08	ft/ft
Flow Area	As =	0.90	sq ft
Flow Velocity	Vs =	2.77	fps
VsD product	**VsD =**	**0.94**	**ft^2/s**

6.4 Example—V-Shaped Swale Capacity

Determine the maximum discharge and depth of flow in a V-shaped, roadside swale with the following characteristics: S_{x1} = 0.08, S_{x2} = 0.06, n = 0.016, S_L = 0.02, and T = 6 feet.

Equations ST-13, ST-1, and ST-3 are used to determine the adjusted slope, the flow, and the flow depth.

$$S_x = (0.08)(0.06)/(0.08 + 0.06) = 0.034$$

$$Q = [(0.56)(0.034)^{5/3}(0.02)^{1/2}(6.0)^{8/3}]/(0.016) = 2.09 \text{ cfs}$$

$$y = (0.034)(6.0) = 0.20 \text{ feet}$$

6.5 Example—V-Shaped Swale Design

Design a V-shaped swale to convey a flow of 1.8 cfs. The available swale top width is 8 feet, the longitudinal slope is 0.01, and the Manning's roughness factor is 0.016. Determine the cross slopes and the depth of the swale.

Solving Equation ST-1 for S_x (i.e., average side slope) yields:

$$S_x = [(1.8)(0.016)/(0.56)(0.01)^{1/2}(8.0)^{8/3}]^{3/5} = 0.024$$

Now Equation ST-13 is used to solve for the actual cross slope if $S_{x1} = S_{x2}$. Then,

$$0.024 = (S_{x1})^2/2S_{x1} = S_{x1}/2, \text{ and } S_{x1} = 0.048$$

Then using Equation ST-2 yields

$$y = (0.024)(8.0) = 0.19 \text{ ft}$$

The swale is 8-feet wide with right and left side slopes of 0.048 ft/ft.

6.6 Example—Major Storm Street Capacity

Determine the flow capacity of an arterial street during the major storm if the street is 60-feet wide (gutter to gutter) with a cross slope of 0.025 ft/ft, a curb height of 6 inches, and a longitudinal slope of 0.03. A 12-foot-wide sidewalk is adjacent to the curb. Flow capacity beyond the sidewalk cannot be relied upon because buildings often abut the sidewalk in this commercial district.

Table ST-3 shows the limitations on the stormwater depth during the major storm (100-year) event. The depth cannot exceed the crown elevation, nor can it exceed 12 inches over the gutter flow line. If the flow depth was at the street crown elevation, the corresponding depth of flow at the curb would be (0.025)(30) = 0.75 feet. Therefore, assume that the crown elevation controls the flood depth (i.e., the entry level into the buildings will assumed to be high enough not to control the flood depth).

Since the street cross section is symmetric, determine the capacity on one side of the street crown and multiply by 2 to get the total capacity and break the flow section up into prismatic shapes. Flow occurs in a triangular section in the street and a rectangular section above the sidewalk (at a depth of 0.75 – 0.5 = 0.25 ft). The street section has a Manning's value of 0.016, and the sidewalk has a value of 0.013. The triangular flow area of the street is (1/2)(30)(0.75) = 11.25 ft^2 and a wetted perimeter of approximately 30 + 0.5 = 30.5 feet (assuming the slope length is roughly equal to the width plus the curb height). The sidewalk section has a flow area of (12)(0.25) = 3.00 ft^2 and a wetted perimeter of 12 feet (ignoring the vertical sides of buildings). Thus, Equation ST-14 yields

$$Q = (1.49/0.016)(11.25)(11.25/30.5)^{2/3}(0.03)^{1/2} = 93.3 \text{ cfs (street section)}$$

$$Q = (1.49/0.013)(3.0)(3.0/12.0)^{2/3}(0.03)^{1/2} = 23.6 \text{ cfs (sidewalk section)}$$

$$Q = 2(93.3 + 23.6) = 234 \text{ cfs (total flow capacity of the section)}$$

Oftentimes, the 100-year flow rate will be available and the flow depth will need to be determined, or the flow cross section will not be prismatic. Fortunately, proprietary software is available to perform normal depth computations (i.e., Manning's depth) for irregular cross sections, rendering these problems trivial.

6.7 Example—Grate Inlet Capacity

Determine the efficiency of a curved vane grate (W = 2 feet and L = 2 feet) when placed in a composite gutter with the following characteristics: S_x = 0.02, S_L = 0.01, a = 0.167 feet, and n = 0.016. The flow rate in the gutter is 2.5 cfs with a spread of 8.5 feet. Note: The depressed section of the composite gutter has a width equal to the width of the grate (Akan and Houghtalen 2002).

Find the gutter slope using Equation ST-8:

$$S_w = 0.02 + \frac{0.167}{2} = 0.1033$$

By using Equation ST-7:

$$E_o = \frac{1}{1 + \left[\dfrac{0.1033/0.02}{\left[1 + \dfrac{(0.1033/0.02)}{(8.5/2) - 1}\right]^{2.67} - 1}\right]} = 0.69$$

The side flow Q_s is calculated using Equation ST-6:

$$Q_S = 2.5(1\text{-}0.69) = 0.77 \text{ cfs}$$

The frontal flow Q_w is calculated using Equation ST-5:

$$Q_W = 2.5 - 0.77 = 1.73 \text{ cfs}$$

Next, find the flow area using Equation ST-10 and velocity using the continuity equation $V = Q/A$.

$$A = ½(0.02)(8.5)^2 + ½(0.167)(2) = 0.89 \text{ ft}^2$$

$$V = Q/A = 2.5/0.89 = 2.81 \text{ ft/sec}$$

The splash-over velocity V_o is determined from Equation ST-20:

$$V_o = 0.30 + 4.85(2) - 1.31(2)^2 + 0.15(2)^3 = 5.96 \text{ ft/sec}$$

Because $V_o > V$, $R_f = 1.0$ from Equation ST-20.

Using Equation ST-21, the side-flow capture efficiency is calculated as:

$$R_s = \frac{1}{1+\dfrac{0.15\left(2.81\right)^{1.8}}{0.02\left(2\right)^{2.3}}} = 0.093$$

Finally, the overall capture efficiency is calculated using Equation ST-22:

$$E = 1.0\left(\frac{1.73}{2.5}\right) + \quad 0.093\left(\frac{0.77}{2.5}\right) = 0.72$$

$$= 72\%$$

Alternatively, the **Grate-G Worksheet** of the UD-Inlet Spreadsheet also performs the calculations and calculates a capture percentage of 71.94%.

6.8 Example—Curb-Opening Inlet Capacity

Determine the amount of flow that will be captured by a 6-foot-long curb-opening inlet placed in the composite gutter described in Example Problem 6.2. The composite gutter in that example had the following characteristics: T = 9.0 ft., W = 2.0 ft, S_L = 0.01, a = 1.5 inches, S_x = 0.02 and a Manning's roughness factor of n = 0.016. In Example Problem 6.2, it was determined that the frontal to total flow ratio was E_o = 0.63 and the total gutter discharge was Q = 2.49 cfs.

Equations ST-25 and ST-26 are used to determine the equivalent slope and the length of inlet required to capture 100% of the gutter flow.

$$S_e = 0.02 + [(1.5/12)/2]0.63 = 0.059$$

$$L_T = 0.60(2.49)^{0.42}(0.01)^{0.3}\left[\frac{1.0}{(0.016)(0.059)}\right]^{0.6} = 14.4\text{ ft}$$

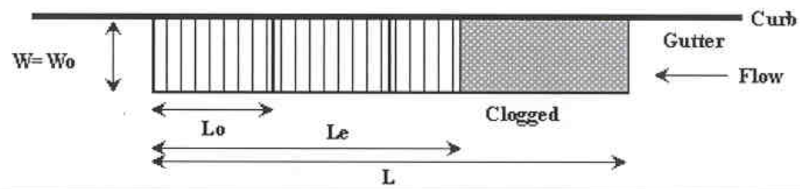
Then, by using Equation ST-23,

$$E = 1.0\left(1.0 - \frac{6.0}{14.4}\right)^{1.8} = 0.62$$

Therefore, $Q_i = EQ = (0.62)(2.49) = 1.54$ cfs will be intercepted by the curb-opening inlet. Note that this problem was performed using the theoretical gutter capacity from Example Problem 6.2. The **Curb-G Worksheet** of the UD-Inlet Spreadsheet also performs these calculations.

GRATE INLET ON A GRADE

Project: Example 6.8 - Grate Inlet Capacity
Inlet ID:



Design Information (Input)			
Design Discharge on the Street (from *Street Hy*)	**Qo =**	**2.5**	**cfs**
Type of Grate	Type =	Vane Grate	
Length of a Unit Grate	Lo =	2.00	ft
Width of a Unit Grate	Wo =	2.00	ft
Clogging Factor for a Unit Grate	Co =	0.00	
Water Depth for Design Condition	Yd =		inches
Number of Grates	No =	1	

Analysis (Calculated)			
Total Length of Grate Inlet	L =	2.00	ft
Ratio of Gutter Flow to Design Flow Eo (from *Street Hy*)	Eo =	0.69	
Equivalent Slope Se (from *Street Hy*)	Se =	0.0800	ft/ft
Flow Velocity Vs (from *Street Hy*)	Vs =	2.77	fps
Spash-over Velocity	Vo =	5.96	fps
Under No-Clogging Condition			
Interception Rate of Gutter Flow	Rf =	1.00	
Effective Length of Grate Inlet	L =	2.00	ft
Interception Rate of Side Flow Rx (from *Street Hy*)	Rx =	0.09	
Interception Capacity	Qi =	1.8	cfs
Under Clogging Condition			
Interception Rate of Gutter Flow	Rf =	1.00	
Clogging Coefficient for Multiple-unit Grate Inlet	Coef =	0.00	
Clogging Factor for Multiple-unit Grate Inlet	Clog =	0.00	
Effective (unclogged) Length of Multiple-unit Grate Inlet	Le =	2.00	ft
Interception Rate of Side Flow Rx (from *Street Hy*)	Rx =	0.09	
Actual Interception Capacity	**Qa =**	**1.8**	**cfs**
Carry-Over Flow = Qo-Qa =	**Q-co =**	**0.7**	**cfs**
Capture Percentage = Qa/Qo =	**C% =**	**71.94**	**%**

6.9 Example—Curb-Opening Inlet Capacity

Determine the amount of flow that will be captured by the 6-foot-long curb-opening inlet of Example Problem 6.8 if the gutter did not have a depressed curb section.

Since the cross slope is given (S_x = 0.02), an equivalent slope does not have to be determined. Equation ST-24 is used to determine the length of inlet required to capture 100% of the gutter flow.

$$L_T = 0.60(2.49)^{0.42}(0.01)^{0.3}\left[\frac{1.0}{(0.016)(0.02)}\right]^{0.6} = 27.6 \text{ ft}$$

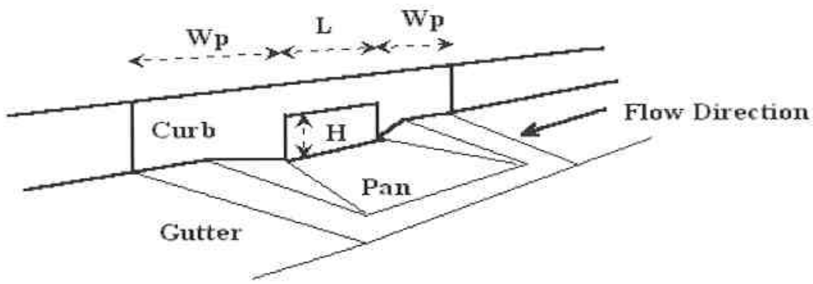
Then, by using Equation ST-23,

$$E = 1.0 - \left(1.0 - \frac{6.0}{27.6}\right)^{1.8} = 0.36$$

Therefore, $Q_i = EQ$ = (0.36)(2.49) = 0.90 cfs will be intercepted by the curb-opening inlet. Note that the curb-opening inlet is far less effective without a depressed curb section. The **Curb-G Worksheet** of the UD-Inlet Spreadsheet also performs these calculations.

CURB OPENING INLET ON A GRADE

Project:	Example 6.9 - Curb Opening Inlet Capacity
Inlet ID:	



Design Information (Input)			
Design Discharge on the Street (from *Street Hy*)	**Qo =**	**2.5**	**cfs**
Gutter Flow to Design Flow Ratio (from *Street Hy*)	Eo =	0.63	
Length of a Single Inlet Unit	Lu =	6.00	ft
Clogging Factor for a Single Unit Inlet	Co =	0.00	
Number of Inlet Units in Curb Opening	No =	1	
Analysis (Calculated)			
Total Length of Curb Opening Inlet	L =	6.00	ft
Equivalent Slope Se (from *Street Hy*)	Se =	0.0600	ft/ft
Required Length Lo to Have 100% Interception	Lo =	14.32	ft
Clogging Coefficient	C-coeff =	1.00	
Clogging Factor for Multiple-unit Curb Opening Inlet	Clog =	0.00	
Effective (Unclogged) Length	Le =	6.00	ft
Under No-Clogging Condition			
Effective Length of Curb Opening Inlet (must be $\leq$ Lo)	L =	6.00	ft
Interception Capacity	Qi =	1.6	cfs
Under Clogging Condition			
Effective Length of Curb Opening Inlet (must be $\leq$ Lo)	Le =	6.00	ft
Interception Capacity	**Qa =**	**1.6**	**cfs**
Carryover flow = Qo - Qa =	**Q-co =**	**0.9**	**cfs**
Capture Percentage for this Inlet = Qa / Qo =	**C% =**	**62.37**	**%**

6.10 Example—Combination Inlet Capacity

A combination inlet is installed in a triangular gutter carrying a discharge of 7 cfs. The gutter is characterized by S_L = 0.01, S_x = 0.025, and n = 0.016. The curb opening is 10 feet long and the grate is a 2-foot by 2-foot reticuline grate. An 8-foot-long portion of the curb opening is upstream of the grate. Determine the flow intercepted by this combination inlet (Akan and Houghtalen 2002).

First consider the upstream curb-opening portion of the combination inlet. By using Equations ST-24 and ST-23, respectively,

$$L_T = (0.6)(7.0)^{0.42}(0.01)^{0.3}\left[\frac{1.0}{(0.016)(0.025)}\right]^{0.6} = 37\text{ ft}$$

$$E = 1.0 - \left[1.0 - \frac{8.0}{37}\right]^{1.8} = 0.36$$

Thus, the 8-foot-long portion of the curb opening intercepts (0.36)(7.0) = 2.5 cfs. The remaining flow is 7.0 - 2.5 = 4.5 cfs. The spread corresponding to this discharge is calculated using Equation ST-1 as:

$$T = \left[\frac{(4.5)(0.016)}{(0.56)(0.025)^{1.67}(0.01)^{0.5}}\right]^{3/8} = 11\text{ ft}$$

Now the flow intercepted by the grate can be computed. By using Equation ST-17,

$$Q_w = 4.5\left[1 - \left(1 - \frac{2.0}{11}\right)^{2.67}\right] = 1.9\text{ cfs}$$

and $Q_s = Q - Q_w$ = 4.5 - 1.9 = 2.6 cfs. The splash-over velocity for the grate (Equation ST-20) is 0.28 + $2.28(2) – 0.18(2)^2 + 0.01(2)^3$ = 4.2 ft/sec. Also, by using Equation ST-4, the flow area just upstream from the grate is $A = (0.5)(0.025)(11)^2 = 1.5\text{ ft}^2$. Likewise, $V = Q/A$ = 4.5/1.5 = 3.0 ft/sec. Because $V < V_o$, R_f = 1.0 by using Equation ST-19. Next, by using Equation ST-21,

$$R_s = \frac{1}{1 + \frac{(0.15)(3.0)^{1.8}}{(0.025)(2.0)^{2.3}}} = 0.10$$

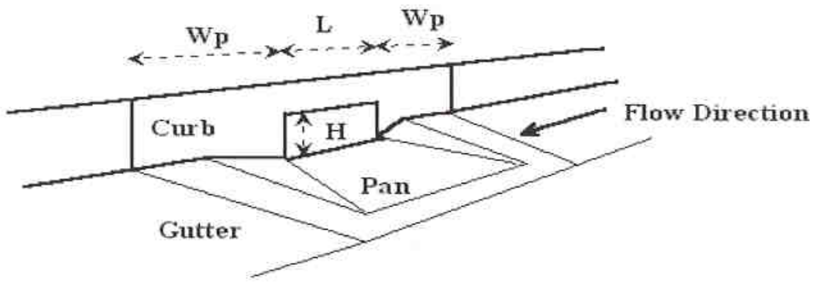
Then by using Equation ST-22, the efficiency of the grate is:

$$E = 1.0\left(\frac{1.9}{4.5}\right) + 0.10\left(\frac{2.6}{4.5}\right) = 0.48$$

The flow intercepted by the grate becomes (0.48)(4.5)= 2.2 cfs. The total flow intercepted by the combination inlet is then 2.5 + 2.2 = 4.7 cfs. The overall efficiency is 4.7/ 7.0 = 0.67 and the bypass flow is 7.0 - 4.7 = 2.3 cfs.

CURB OPENING INLET ON A GRADE

Project: Example 6.10 - Curb Opening Inlet Capacity
Inlet ID:



Design Information (Input)			
Design Discharge on the Street (from *Street Hy*)	**Qo =**	**2.5**	**cfs**
Gutter Flow to Design Flow Ratio (from *Street Hy*)	Eo =	0.44	
Length of a Single Inlet Unit	Lu =	6.00	ft
Clogging Factor for a Single Unit Inlet	Co =	0.00	
Number of Inlet Units in Curb Opening	No =	1	
Analysis (Calculated)			
Total Length of Curb Opening Inlet	L =	6.00	ft
Equivalent Slope Se (from *Street Hy*)	Se =	0.0200	ft/ft
Required Length Lo to Have 100% Interception	Lo =	27.68	ft
Clogging Coefficient	C-coeff =	1.00	
Clogging Factor for Multiple-unit Curb Opening Inlet	Clog =	0.00	
Effective (Unclogged) Length	Le =	6.00	ft
Under No-Clogging Condition			
Effective Length of Curb Opening Inlet (must be $\leq$ Lo)	L =	6.00	ft
Interception Capacity	Qi =	0.9	cfs
Under Clogging Condition			
Effective Length of Curb Opening Inlet (must be $\leq$ Lo)	Le =	6.00	ft
Interception Capacity	**Qa =**	**0.9**	**cfs**
Carryover flow = Qo - Qa =	**Q-co =**	**1.6**	**cfs**
Capture Percentage for this Inlet = Qa / Qo =	**C% =**	**35.58**	**%**

6.11 Example—Curb-Opening Inlet in a Sump Condition

Determine the flow depth and spread at a curb-opening inlet placed in a sump given the following conditions: L = 6 ft, h = 0.3 ft, S_x = 0.025, and Q_i = 5.8 cfs. Assume there is no clogging.

The flow condition must be assumed and then verified. Assuming orifice flow, Equation ST-28 yields

$$Q_i = C_o A_o (2gd)^{0.5}$$

Now, based on Table ST-7,

$$Q_i = 0.67(h)(L)[(2g)(d_i - h/2)]^{0.5}$$

and by substituting known values,

$$5.8 = (0.67)(0.3)(6)[(2)(32.2)(d_i - 0.3/2)]^{0.5}$$

which yields:

$$d_i = 0.51 \text{ ft}$$

Since $d_i > 1.4h$, the orifice equation is appropriate. Equation ST-2 yields T = 0.51/0.025 = 20.4 ft.

The **Curb-S Worksheet** performs these calculations.

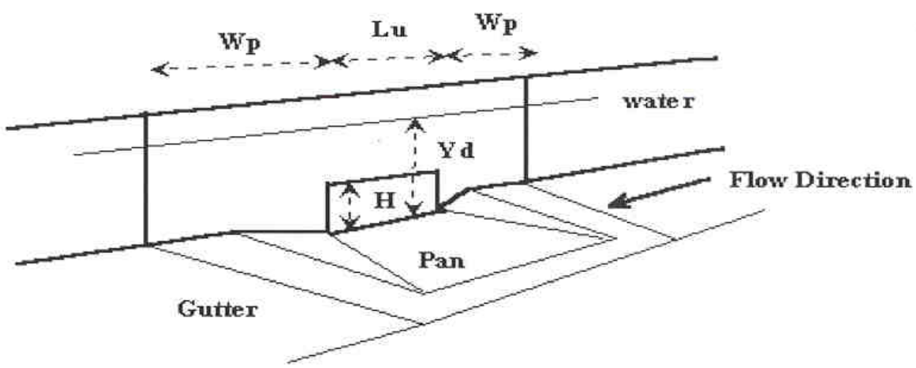
6.12 Example—Storm Sewer Hydraulics (Akan and Houghtalen 2002)

Determine the depth of flow, y, flow area, and flow velocity in a storm sewer (D = 2.75 ft, n = 0.013, and S_0 = 0.003) for a flow rate of 26.5 cfs.

Just full flow conditions are computed first. From Equations ST-34, ST-37 and ST-38, A_f = 5.94 ft^2, R_f = 0.69 ft, and Q_f = 29.1 cfs. Then, V_f = 29.1/5.94 = 4.90 ft/sec. Now, by using Figure ST-6 with Q/Q_f = 26.5/29.1 = 0.91, it is determined that y/D = 0.73, A/A_f = 0.79, and V/V_f = 1.13. Therefore, y = (0.73)(2.75) = 2.0 ft, A = (0.79)(5.94) = 4.69 ft^2, and V = (1.13)(4.90) = 5.54 ft/sec.

CURB OPENING INLET IN A SUMP

Project = Example 6.12 - Curb Opening Inlet in a Sump Condition
Inlet ID =



Design Information (Input)			
Design Discharge on the Street (from *Street Hy*)	**Qo =**	**2.5**	**cfs**
Length of a Unit Inlet	Lu =	6.00	ft
Side Width for Depression Pan	Wp =	3.00	ft
Clogging Factor for a Single Unit	Co =	0.00	
Height of Curb Opening in Inches	H =	3.60	inches
Orifice Coefficient	Cd =	0.67	
Weir Coefficient	Cw =	3.00	
Water Depth for the Design Condition	Yd =	0.51	ft
Angle of Throat (see USDCM Chapter 6, Figure ST-5)	Theta =	63.0	degrees
Number of Curb Opening Inlets	No =	1	

Curb Opening Inlet Capacity in a Sump			
As a Weir			
Total Length of Curb Opening Inlet	L =	6.00	ft
Capacity as a Weir without Clogging	Qwi =	12.5	cfs
Clogging Coefficient for Multiple Units	Clog-Coeff =	1.00	
Clogging Factor for Multiple Units	Clog =	0.00	
Capacity as a Weir with Clogging	Qwa =	12.5	cfs
As an Orifice			
Capacity as an Orifice without Clogging	Qoi =	5.9	cfs
Capacity as an Orifice with Clogging	Qoa =	5.9	cfs
Capacity for Design with Clogging	**Qa =**	**5.9**	**cfs**
Capture Percentage for this Inlet = Qa / Qo =	**C% =**	**100.00**	**%**

Note: Unless additional ponding depth or spilling over the curb is acceptable, a capture percentage of less than 100% in a sump may indicate the need for additional inlet units.

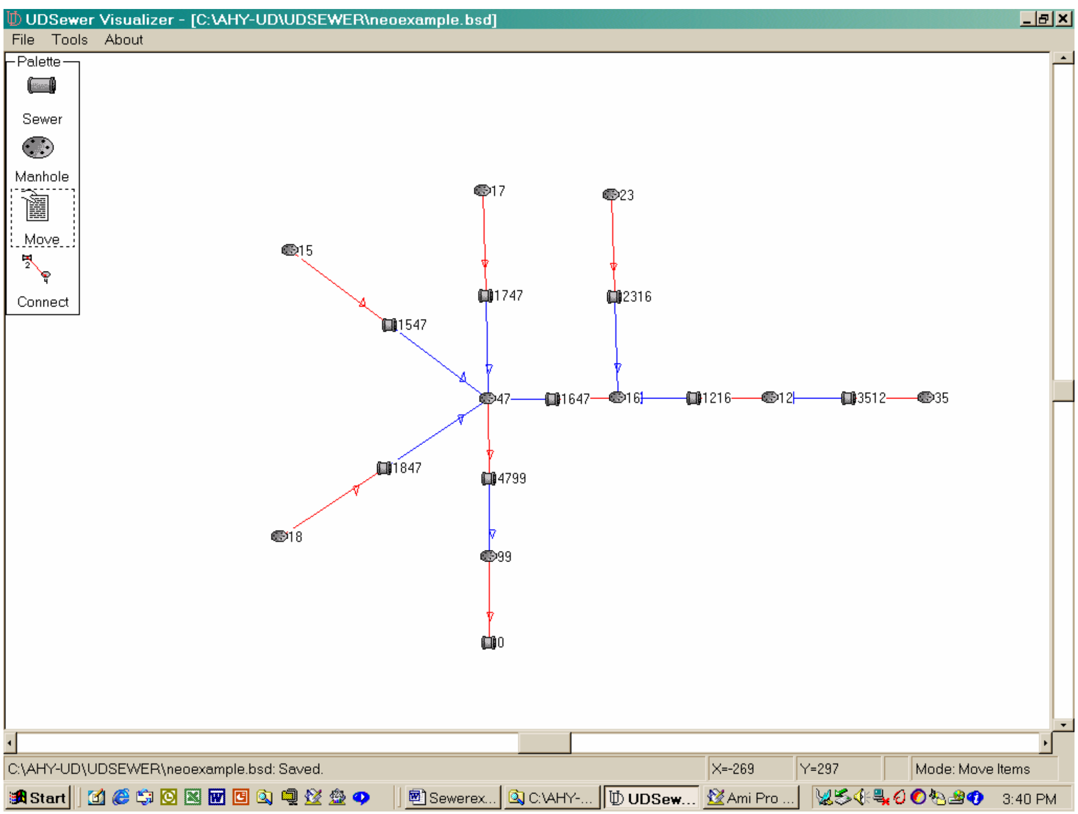
6.13 Example—Storm Sewer Hydrology

This example storm sewer system is based on the hydrology for the Denver, Colorado area. It is developed here to illustrate the solution using the NeoUDSEWER computer software. The storm sewer system is to be designed to fully convey the five-year runoff event. The following formula, taken from the Rainfall Chapter of this Manual, describes design rainfall intensity as a function of storm duration:

$$i = \frac{38.5}{(10 + T_d)^{0.768}}$$

in which i = rainfall intensity in inches per hour and T_d = rainstorm duration in minutes.

The illustration below depicts a layout of the storm sewer system. It is a copy of the input screen from the NeoUDSEWER software. An ID number is assigned to each manhole and to each sewer segment. The ID numbers have to be unique among the manholes in a system and cannot be duplicated, as is the case for sewer ID numbers among the sewers. At a manhole, NeoUDSEWER can accommodate one outgoing sewer and up to four incoming sewers.



Example Storm Sewer System Using Computer Model: NeoUDSEWER

NeoUDSEWER is a storm sewer system sizing and analysis software package. It calculates rainfall and runoff using the Rational Formula method and then sizes circular sewers using Manning's equation. It has a graphical interface for data entry and editing. NeoUDSEWER can handle a storm sewer system having up to 100 manholes and up to 100 sewers.

Data entry includes project title, rainfall statistics, manhole information, basin hydrology, and sewer network information. Rainfall IDF information can be entered as a table or calculated using the equation given above by entering only a value for the 1-hour depth, P1. The user needs to check all of the default design constraints and criteria and make all necessary changes to these values as needed.

The input parameters for each manhole include the manhole identification number, ground elevation, and incoming and outgoing sewer identification numbers. The hydrologic parameters for the tributary area at a manhole include tributary area, runoff coefficients, overland flow length and slope, local tributary gutter flow length, and gutter flow velocity.

When the local runoff flow rate at a manhole is known, it may be entered (along with non-zero values for local tributary area and local runoff coefficient) to override the flows calculated by the Rational Equation for the local area. NeoUDSEWER will combine the local flow with the upstream flow to calculate the design discharge at the manhole. When the design discharge at a manhole is known for the entire upstream area, the user must enter this discharge (along with total tributary area) and the weighted runoff coefficient to have the program then analyze the EGL and HGL for the system.

A storm sewer is described by its length, slope, upstream crown elevation, Manning's roughness coefficient, shape, bend loss coefficient, and lateral loss coefficient. An existing sewer is identified by the user-defined size and shape. Use of noncircular sewers such as box sewers and arch pipes can be achieved by prescribing their dimensions. However, all new sewers are sized using circular pipes. The program provides suggested commercial sewer sizes for both new and existing sewers. Sewers with flat or negative slope may be analyzed as existing sewers with user-defined sizes provided to the program, along with user-defined tailwater surface elevation at the outlet end. NeoUDSEWER applies open channel hydraulics, culvert hydraulics, and pressure flow hydraulics to calculate the EGL and HGL along the predefined sewer system.

For this example, Table ST-11 provides the watershed hydrologic parameters for the determination of peak design flow rates at the manholes in the system. The design flow can be changed only at a manhole. Sewers 3512, 1216, 1647, and 1547 are treated as existing sewer and their sizes are given in Table ST-12. Other sewer segments are new and will be sized by NeoUDSEWER using circular pipes. In a case that a box conduit is preferred, the sewer may be treated as an existing sewer with a known width. NeoUDSEWER will calculate the water depth and recommend the height for a box sewer. All manholes must have an outgoing pipe except the system outfall pipe (i.e., Manhole 99 in this example) whose outgoing sewer has a pre-assigned ID of zero. For this example, the global Manning's roughness coefficient n = 0.013 was used, and the tailwater surface elevation was set at an elevation of 87 feet.

Table ST-11—Hydrologic Parameters at Manholes

Manhole ID Number	Ground Elevation Feet	Tributary Area acres	Runoff Coeff.	Overland Slope percent	Overland Length Feet	Gutter Slope percent	Gutter Length Feet
35.00	111.00	3.00	0.90	0.15	250.00	0.49	150.00
12.00	109.00	6.45	0.85	0.25	180.00	1.00	450.00
23.00	110.00	5.00	0.90	1.00	275.00	1.00	450.00
16.00	101.50	0.00	0.00	0.00	0.00	0.00	0.00
15.00	104.00	5.00	0.85	0.50	285.00	2.25	450.00
47.00	99.00	3.00	0.80	0.40	250.00	1.56	255.00
99.00	97.50	0.00	0.00	0.00	0.00	0.00	0.00
17.00	99.90	1.00	0.65	0.10	200.00	0.36	300.00
18.00	99.75	1.20	0.45	0.40	300.00	0.00	0.00

Table ST-12—Vertical Profile Information of Sewers

Sewer ID	Length (feet)	Slope (percent)	Upstream Crown Elevation (feet)	Diameter (inches)	Height or Rise (inches)	Width or Span (inches)	Bend Loss Coef.	Lateral Loss Coef.
3512 (round)	450.00	0.50	104.50	24			0.05	
1216 (arch)	360.00	0.80	97.05		20.00	28.00	0.05	0.25
2316	460.00	1.20	105.50				1.00	
1647 (round)	380.00	- 0.10	94.25	27			0.05	0.25
1547 (round)	295.00	1.50	101.10	18			0.40	
4799 (box)	410.00	0.25	93.32		48.00	48.00	0.05	
1747	200.00	2.00	96.80				1.00	
1847	350.00	0.75	94.00				1.00	

For the input parameters in Tables St-11 and ST-12, Neo-UDSEWER produced the following outputs:

NeoUDS Results Summary

Project Title: CASE STUDY : EXAMPLE ONE

Project Description: STORM SEWER SYSTEM DESIGN: NEW SEWERS WITH EXISTING SEWERS

Output Created On: 8/2/2002 at 9:08:16 AM

Using NeoUDSewer Version 1.1.

Rainfall Intensity Formula **Used.**

Return Period of Flood is 5 **Years.**

A. Sub Basin Information

		Time of Concentration				
Manhole ID #	**Basin Area * C**	**Overland (Minutes)**	**Gutter (Minutes)**	**Basin (Minutes)**	**Rain I (Inch/Hour)**	**Peak Flow (CFS)**
35	2.70	12.2	0.0	0.0	3.36	9.1
12	3.83	5.0	0.0	0.0	6.33	24.2
23	4.50	13.0	0.0	0.0	3.28	14.7
16	0.00	0.0	0.0	0.0	0.00	35.6
15	4.25	14.1	0.0	0.0	3.16	13.4
47	2.40	19.1	0.0	0.0	2.72	6.5
99	1.70	17.2	0.0	0.0	2.87	4.9
17	0.65	12.8	0.0	0.0	3.30	2.1
18	0.54	11.7	0.0	0.0	3.43	1.9

The shortest design rainfall duration is 5 minutes.

For rural areas, the catchment time of concentration is always => 10 minutes.

For urban areas, the catchment time of concentration is always => 5 minutes.

At the first design point, the time constant is <= (10+Total Length/180) in minutes.

When the weighted runoff coefficient => 0.2, then the basin is considered to be urbanized.

When the Overland Tc plus the Gutter Tc does not equal the catchment Tc, the above criteria supercedes the calculated values.

B. Summary of Manhole Hydraulics

Manhole ID #	Contributing Area * C	Rainfall Duration (Minutes)	Rainfall Intensity (Inch/Hour)	Design Peak Flow (CFS)	Ground Elevation (Feet)	Water Elevation (Feet)	Comments
35	2.7	12.2	3.36	9.1	111.00	106.60	
12	6.52	9.6	3.71	24.2	109.00	105.08	
23	4.5	13.0	3.28	14.7	110.00	105.17	
16	11.02	13.4	3.23	35.6	101.50	99.61	
15	4.25	14.1	3.16	13.4	104.00	101.46	
47	18.86	15.6	3.00	56.7	99.00	91.66	
99	0	0.0	0.00	0.0	97.50	87.00	
17	0.65	12.8	3.30	2.1	99.90	95.88	
18	0.54	11.7	3.43	1.9	99.75	93.03	

C. Summary of Sewer Hydraulics

Note: The given depth to flow ratio is 1.

	Manhole ID Number			Calculated	Suggested	Existing	
Sewer ID #	Upstream	Downstream	Sewer Shape	Diameter (Rise) (Inches) (FT)	Diameter (Rise) (Inches) (FT)	Diameter (Rise) (Inches) (FT)	Width (FT)
3512	35	12	Round	19.4	21	24	N/A
1216	12	16	Arch	25.7	27	20	28
2316	23	16	Round	19.8	21	21	N/A
1647	16	47	Round	27.0	27	27	N/A
1547	15	47	Round	18.3	21	18	N/A
4799	47	99	Box	2.3	2	4	4
1747	17	47	Round	8.7	18	18	N/A
1847	18	47	Round	9.9	18	18	N/A

Round and arch sewers are measured in inches.

Box sewers are measured in feet.

Calculated diameter was determined by sewer hydraulic capacity.

Suggested diameter was rounded up to the nearest commercially available size

All hydraulics where calculated using the existing parameters.

If sewer was sized mathematically, the suggested diameter was used for hydraulic calculations.

Sewer ID	Design Flow (CFS)	Full Flow (CFS)	Normal Depth (Feet)	Normal Velocity (FPS)	Critical Depth (Feet)	Critical Velocity (FPS)	Full Velocity (FPS)	Froude Number	Comment
3512	9.1	16.0	1.08	5.3	1.08	5.2	2.9	1	
1216	24.2	20.3	2.00	7.7	1.73	8.4	7.7	N/A	
2316	14.7	17.4	1.24	8.1	1.42	7.0	6.1	1.34	
1647	35.6	35.6	2.25	8.9	2.00	9.5	8.9	N/A	
1547	13.4	12.9	1.50	7.6	1.35	8.0	7.6	N/A	
4799	56.7	91.7	2.34	6.0	1.84	7.7	3.5	0.7	
1747	2.1	14.9	0.38	6.0	0.58	3.4	1.2	2.02	
1847	1.9	9.1	0.46	4.0	0.53	3.3	1.0	1.24	

A Froude number = 0 indicated that a pressured flow occurs.

D. Summary of Sewer Design Information

		Invert Elevation		Buried Depth		
Sewer ID	Slope %	Upstream (Feet)	Downstream (Feet)	Upstream (Feet)	Downstream (Feet)	Comment
3512	0.50	102.50	100.25	6.50	6.75	
1216	0.80	95.37	92.49	11.96	7.34	
2316	1.20	103.75	98.23	4.50	1.52	Sewer Too Shallow
1647	-0.10	92.00	92.38	7.25	4.37	
1547	1.50	99.60	95.17	2.90	2.33	
4799	0.25	89.32	88.29	5.68	5.21	
1747	2.00	95.30	91.30	3.10	6.20	
1847	0.75	92.50	89.88	5.75	7.62	

E. Summary of Hydraulic Grade Line

			Invert Elevation		Water Elevation		
Sewer ID #	Sewer Length (Feet)	Surcharged Length (Feet)	Upstream (Feet)	Downstream (Feet)	Upstream (Feet)	Downstream (Feet)	Condition
3512	450	450	102.50	100.25	106.60	105.08	Pressured
1216	360	360	95.37	92.49	105.08	99.61	Pressured
2316	460	256.58	103.75	98.23	105.17	99.61	Jump
1647	380	380	92.00	92.38	99.61	91.66	Pressured
1547	295	295	99.60	95.17	101.46	91.66	Pressured
4799	410	0	89.32	88.29	91.66	87.00	Subcritical
1747	200	0	95.30	91.30	95.88	91.66	Jump
1847	350	118.29	92.50	89.88	93.03	91.66	Jump

F. Summary of Energy Grade Line

	Upstream Manhole			Juncture Losses				Downstream Manhole	
Sewer ID #	Manhole ID #	Energy Elevation (Feet)	Sewer Friction (Feet)	Bend K Coefficient	Bend Loss (Feet)	Lateral K Coefficient	Lateral Loss (Feet)	Manhole ID #	Energy Elevation (Feet)
3512	35	106.73	0.72	0.05	0.01	0.00	0.00	12	106.00
1216	12	106.00	4.10	0.05	0.05	0.25	1.01	16	100.85
2316	23	105.94	4.51	1.00	0.58	0.00	0.00	16	100.85
1647	16	100.85	8.51	0.05	0.06	0.25	0.05	47	92.23
1547	15	102.35	9.77	0.40	0.36	0.00	0.00	47	92.23
4799	47	92.23	5.23	0.05	0.00	0.00	0.00	99	87.00
1747	17	96.06	3.81	1.00	0.02	0.00	0.00	47	92.23
1847	18	93.20	0.96	1.00	0.02	0.00	0.00	47	92.23

Bend loss = Bend K * Flowing full vhead in sewer.
Lateral loss = Outflow full vhead - Junction Loss K * Inflow full vhead.
A friction loss of 0 means it was negligible or possible error due to jump.
Friction loss includes sewer invert drop at manhole.
Notice: Vhead denotes the velocity head of the full flow condition.
A minimum junction loss of 0.05 Feet would be introduced unless Lateral K is 0.
Friction loss was estimated by backwater curve computations.

G. Summary of Earth Excavation Volume for Cost Estimate

The user given trench side slope is 1.

Manhole ID #	Rim Elevation (Feet)	Invert Elevation (Feet)	Manhole Height (Feet)
35	111.00	102.50	8.50
12	109.00	95.37	13.63
23	110.00	103.75	6.25
16	101.50	92.00	9.50
15	104.00	99.60	4.40
47	99.00	89.32	9.68
99	97.50	88.29	9.21
17	99.90	95.30	4.60
18	99.75	92.50	7.25

	Upstream Trench Width		**Downstream Trench Width**				
Sewer ID #	**On Ground (Feet)**	**At Invert (Feet)**	**On Ground (Feet)**	**At Invert (Feet)**	**Trench Length (Feet)**	**Wall Thickness (Inches)**	**Earth Volume (Cubic Yards)**
3512	16.5	4.5	17.0	4.5	450	3.00	1347
1216	27.8	4.8	18.5	4.8	360	3.00	1981
2316	12.3	4.2	6.3	4.2	460	2.75	562
1647	18.2	4.8	12.4	4.8	380	3.25	1031
1547	8.9	3.9	7.7	3.9	295	2.50	272
4799	16.4	6.9	15.5	6.9	410	5.51	1409
1747	9.3	3.9	15.5	3.9	200	2.50	358
1847	14.6	3.9	18.3	3.9	350	2.50	988

Total earth volume for sewer trenches = 7947.8 Cubic Yards. The earth volume was estimated to have a bottom width equal to the diameter (or width) of the sewer plus two times either 1 foot for diameters less than 48 inches or 2 feet for pipes larger than 48 inches.

If the bottom width is less than the minimum width, the minimum width was used.

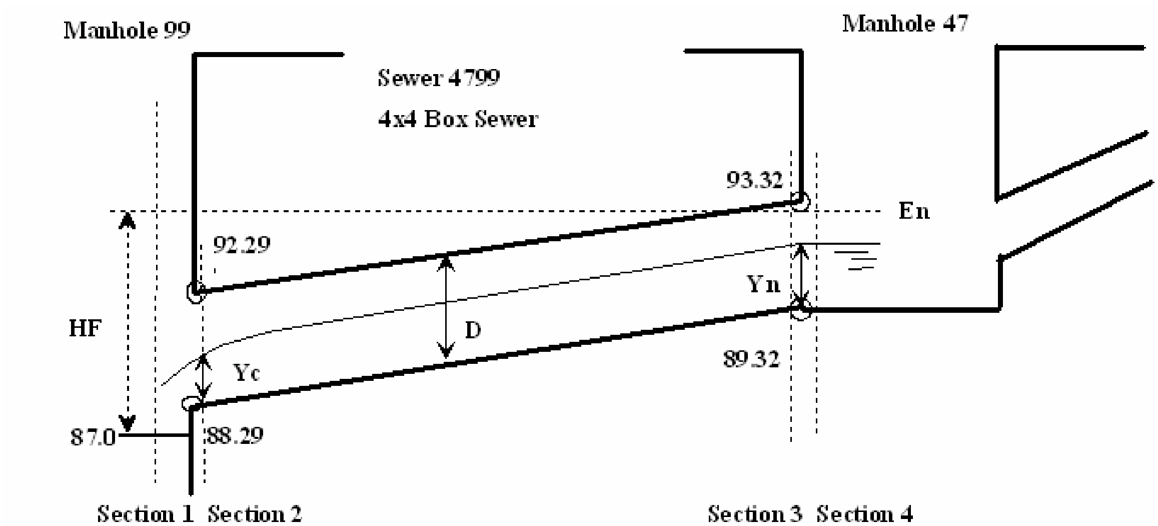
The backfill depth under the sewer was assumed to be 1 foot.

The sewer wall thickness is equal to: (equivalent diameter in inches/12)+1

The following two cases illustrate how the HGL and EGL were calculated by NeoUDSEWER:

Case 1. Energy Grade Line Calculation for Sewer 4799 in Example 6.13

The profile for Sewer 4799 is shown below:



Calculation of an EGL requires the knowledge of flow hydraulics in the sewer and in the downstream manhole. The following parameters are extracted from the NeoUDSEWER output:

Q cfs	*Yn* ft	*Vn* fps	*Ss* ft/ft	*Yc* ft	*Vc* fps	*Sc* ft/ft	*N*	*Fr*	*Ls* ft
56.70	2.34	6.04	0.25%	1.84	7.71	0.48%	0.013	0.70	410

in which:

Q = design flow

N = Manning's roughness coefficient

Fr = Froude number for normal flow

Ss = sewer slope

Ls = length of sewer

S = energy slope

V = flow velocity

Y = flow depth

The subscript of *n* represents the normal flow condition and *c* represents the critical flow condition.

The calculations of energy balance for this example include three ports: (A) juncture loss at the downstream manhole, (B) friction losses along Sewer 4799, and (C) energy balance between upstream and downstream manholes. They are conducted separately as follows:

A. Juncture Loss at Manhole 99

Manhole 99 is the system exit. There is no bend loss and lateral loss at Manhole 99. As a result, the known tailwater surface elevation of 87 feet serves for both the EGL and HGL at Manhole 99.

B. Along Sewer 4799

Sewer 4799 carries a discharge of 56.70 cfs. The water surface profile in Sewer 4799 is an M-2 curve produced by a subcritical flow with a Froude number of 0.70.

<u>Section 1</u>

With EGL = HGL= 87 feet at Manhole 99, the EGL and HGL at Section 1 are:

E_1 = 87 feet and W_1 = 87 feet

<u>Section 2</u>

With an unsubmerged condition at the sewer exit, an M-2 water surface profile is expected. Therefore, the EGL at Section 2 is dictated by the critical flow condition. Let $Y_2 = Yc$, $V_2 = Vc$. According to Equation ST-40, the EGL at Section 2 is:

$$E_2 = Max(\frac{7.71^2}{2*32.2} + 1.84 + 88.29,\ 87.0) = 91.05 \text{ feet}$$

and the HGL at Section 2 is:

W_2 = 1.82 + 88.29 = 90.13 feet

<u>Section 3</u>

The determination of the EGL from Section 2 to Section 3 is essentially the backwater profile calculation using the direct step method. Assuming that the flow depth at Section 3 is the normal flow depth, the energy equation is written as:

$E_c = E_n + h_f$

in which:

E_c = 92.23 feet which is the EGL of the critical flow at Section 2

E_n = 91.05 feet which is the EGL of the normal flow at Section 3

h_f = friction loss which is related to the critical energy slope, S_c

S_c = 0.0048

S_s = 0.0025 which is the normal flow energy slope

Both energy slopes can be calculated by Manning's equation. Using the direct step method, the length of the M-2 water surface profile, *X*, between the critical flow section and normal flow section is calculated as:

$$X = \frac{E_n - E_c}{0.5(S_n + S_c)} = \frac{92.23 - 91.05}{0.5(0.0025 + 0.0048)} = 322.45 \text{ feet}$$

Because the length of the M-2 curve is shorter than the length of Sewer 4799, the assumption of normal flow at Section 3 is acceptable. Therefore, the EGL and HGL at Section 3 are:

E_3 = 92.23 feet (normal flow condition)

W_3 = 2.34 + 89.23 = 91.66 feet

<u>Section 4</u>

Assuming that the loss at the entrance of Sewer 4799 is negligible, the EGL and HGL at Section 4 are the same as those at Section 3, namely:

E_4 = 92.23 feet

W_4 = 91.66 feet

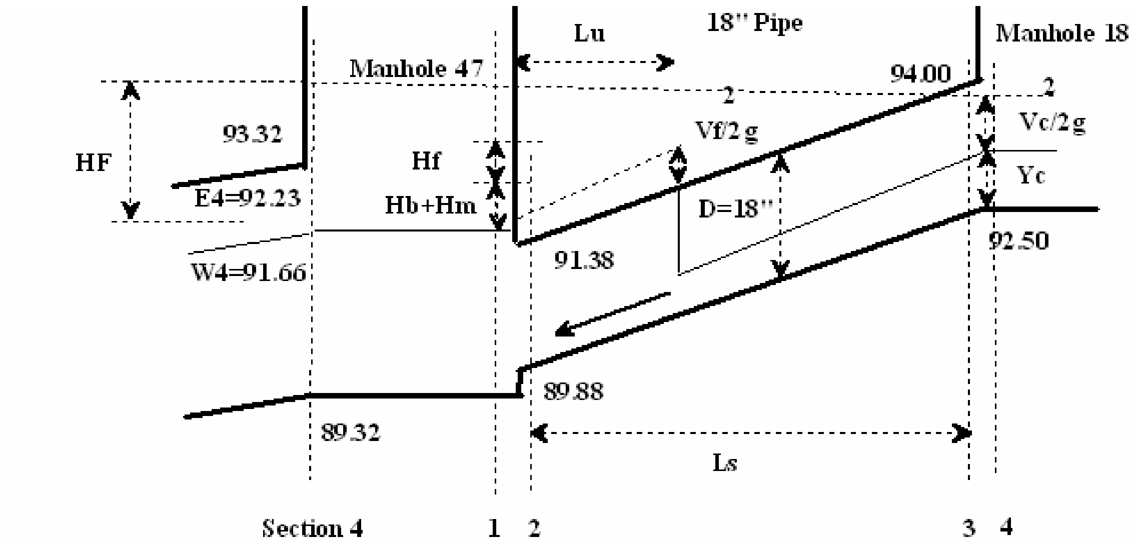
C. Energy Balance Between Manholes.

The calculations of the EGL along Sewer 4799 and across Manhole 99 do not include manhole drop and possible losses due to hydraulic jump. Therefore, it is necessary to perform energy balancing between Manholes 47 and 99 as:

$92.23 = 87.0 + H_b + H_m + H_f$

$H_f = 92.23 – 87.0 – 0 – 0 = 5.23$ feet

Case 2. Energy Grade Line for Sewer 1847 in Example 6.13



The flow parameters along Sewer 1847 and at Manhole 47 can be found in the NeoUDSEWER output file. They are summarized as follows:

Q	*Yn*	*Vn*	*Yc*	*Vc*	V_f	*N*	*Fr*	*Ss*	*Ls*	*Kb*
cfs	ft	fps	ft	fp	ft			ft/ft	ft	
1.85	0.46	4.05	0.53	3.33	1.05	0.01	1.05	.75%	350	1.00

in which V_f = full-flow velocity and the definitions of other flow parameters can be found in Example 12-1.

A. Juncture Loss at Manhole 47

Sewer 1847 carries a discharge of 1.85 cfs, which is a supercritical flow with a Froude number of 1.05. At Manhole 47, the EGL and HGL have been calculated as E_4 = 92.23 and W_4 = 91.66 feet.

To cross Manhole 47 (i.e., from Section 4 to Section 1) the bend loss is:

$$H_b = K_b \frac{V_f^{\ 2}}{2g} = 1.0 \frac{1.05^2}{2*32.2} = 0.017 \text{ feet}$$

Because Sewer 1847 is not on the main line, it does not have a lateral loss (i.e., K_m = 0.0). Between Sections 4 and 1, the energy principle is written as:

$$E_1 = E_4 + H_b + H_m = 92.23 + 0.017 + 0 = 92.25 \text{ feet}$$

$$H_1 = E_1 - \frac{V_f^{\ 2}}{2g} = 92.25 - \frac{1.05^2}{2*32.2} = 92.23 \text{ feet}$$

B. Friction Losses through Sewer 1847

<u>Section 1</u>

With EGL = 92.25 feet and HGL= 92.23 feet at Manhole 99, the EGL and HGL at Section 1 are:

E_1 = 92.25 feet and W_1 = 92.23 feet

The downstream end of Sewer 1847 is submerged.

<u>Section 2</u>

With a submerged exit, the EGL for the full-flow condition is:

$$E_F = \frac{1.05^2}{2*32.2} + 1.5 + 89.88 = 91.39 \text{ feet}$$

The EGL at Section 2 is chosen as the higher one between the one for the full-flow condition and the EGL at Section 1, thus:

$$E_2 = Max(E_F, \ E_1) = 92.25 \text{ feet}$$

and the resulting HGL at Section 2 is:

W_2 = 92.23 feet

<u>Section 3</u>

The lower portion of Sewer 1847 is surcharged because of the exit submergence. According to Manning's equation, the friction slope for the full flow condition in Sewer 1847 is 0.003 ft/ft. According to the direct step method, the surcharge length near the downstream end of Sewer 1847 can be approximated by W_2 and the sewer crown elevation, *Crown*, as:

$$L_u = \frac{W_2 - Crown}{(S_s - S_f)} = \frac{92.23 - 91.38}{(0.0075 - 0.003)} = 118.1 \text{ feet}$$

The friction loss through the surcharged length is:

$H_f = S_f * L_u$ = 0.003 * 118.1 = 0.35 feet

The EGL at Section 3 is controlled by either the friction loss through the surcharged length or the critical flow condition at the entrance. Considering the friction loss, we have:

$$E_{31} = E_1 + H_f = 92.25 + 0.35 = 92.6 \text{ feet}$$

Considering the critical flow condition at the entrance, we have:

$$E_{32} = \frac{V_c^{\ 2}}{2g} + Y_c + 92.50 = 93.20 \text{ feet}$$

In comparison, the EGL at Section 3 is determined as:

$$E_3 = Max(E_{31}, E_{32}) = 93.20 \text{ feet}$$

This process is similar to the culvert hydraulics under a possible hydraulic jump. The headwater depth at the entrance of Sewer 1847 shall consider both inlet and outlet controls; whichever is higher dictates the answer. As a result, the HGL at Section 3 is:

$$W_3 = E_3 - \frac{V_c^{\ 2}}{2g} = 93.03 \text{ feet}$$

Section 4

Considering that the entrance loss is negligible for Sewer 1847, we have:

$E_4 = E_3$ and $W_4 = W_3$

C. Between Manholes 47 and 18

The energy balance between Manhole 18 and Manhole 47 is:

$93.20 = 92.23 + H_b + H_m + H_f$

$H_f = 93.20 - 92.23 - 0.017 - 0 = 0.96$ feet

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MAJOR DRAINAGE

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TABLES

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1.0 INTRODUCTION

1.1 General

Major drainage is the cornerstone of an urban storm runoff system. The major drainage system will exist whether or not it has been planned and designed, and whether or not urban development is wisely located in respect to it. Thus, major drainage must be given high priority when considering drainage improvements.

The major drainage system may include many features such as natural and artificial channels, culverts, long underground conduits and outfalls, streets, property line drainage easements, and others. It is closely allied to, but separate from, the initial drainage system consisting of storm sewers, curbs and gutters, swales, and minor drainageways. The two separate systems should generally be planned together. In many cases, a good major system can reduce or eliminate the need for an underground storm sewer system. An ill-conceived major system can make a storm sewer system very costly. The 2-, 5- or 10-year or other smaller runoff event can flow in the major system, but only a portion of the 100-year and larger runoff events will flow in the initial drainage system.

Photograph MD-1—An engineered wetland channel can serve as a filter for low flows and yet carry the major flood event without permanent damage.

While the primary function of a major drainageway is conveyance of runoff, many design decisions contribute to the role of the drainageway in the urban environment in terms of stability, multiple use benefits, social acceptance, aesthetics, resource management, and channel maintenance. It is important for the engineer to be involved from the very start of a land development project, so that the criteria in this *Manual* have bearing on the critical planning decisions involved in route selection for the major drainage system. The importance of route selection cannot be overstated since the route selected will influence every element of the major drainage project from the cost to the type of channel to use to the benefits derived to the community for years to come.

1.2 Types of Major Drainage Channels

The types of major drainage channels available to the designer are numerous, depending upon good hydraulic practice, environmental considerations, sociological/community impact and needs, permitting limitations, and basic project requirements. Section 3.3.1 describes in detail the following types of channels engineers can consider as potential major drainage channels in urban areas and then select the ones that address the considerations listed above the best:

- Natural channels
- Grass-lined channels
- Composite channels
- Concrete-lined channels
- Riprap-lined channels
- Bioengineered channels
- Channels with manufactured liners
- Boatable channels

As discussed in the rest of this chapter, the selection of the channel type for any given reach of a major drainageway is a complex function of hydraulic, hydrologic, structural, financial, environmental, sociological, public safety, and maintenance considerations and constraints.

1.3 Overview of Chapter

This chapter addresses the major topics related to major drainage design, beginning with essential background on the issues of major drainage planning and engineering (Section 1.4) and fluvial geomorphology (Section 1.5). Section 2.0 addresses planning for major drainage systems, including route selection and requirements for drainage master planning. General open channel hydraulics and preliminary design criteria are presented in Section 3.0. It is assumed that the designer is knowledgeable of open channel hydraulics, and, therefore, the key principles and equations are reviewed without extensive background of the subject matter, theoretical considerations, etc. Section 4.0 contains specific design criteria for a variety of channel types and includes example calculations, typical cross sections, and other representative design details. Sections 5.0 and 6.0 address rectangular conduits and large pipes, respectively, and Section 7.0 provides information on the use of riprap and boulders for major drainage applications. Section 8.0 addresses sediment.

1.4 Issues in Major Drainage Planning and Engineering

The planner and engineer have great opportunities when working on major channels to help provide a better urban environment for all citizens. The challenge is particularly great for those having the opportunity to plan and design works in the core areas of cities. The most fundamental function of a major drainageway is conveyance of the major storm runoff event, and an important characteristic is its stability during minor and major storms. Stability must be examined in the context of the future urbanized condition, in terms of both runoff events and altered base flow hydrology. Urbanization in the Denver metropolitan area commonly causes base flows to increase, and the planner and engineer must anticipate and design for this increase.

In addition to stability issues, there are many planning and engineering decisions that contribute to the role of the drainageway in the urban environment, in terms of multiple use benefits, social acceptance, aesthetics, and resource management. The choices of the type and layout of the major drainage system and the type of flow conveyance elements are of prime importance.

Types of major drainageways can generally be characterized as open (i.e., open channel) or closed (i.e., below-ground rectangular conduits and large pipes). Open channels for transporting major storm runoff are more desirable than underground conduits in urban areas, and use of such channels is encouraged. Open channels offer many opportunities for creation of multiple use benefits such as incorporation of parks and greenbelts along the channel and other aesthetic and recreational uses that closed-conveyance drainageway designs preclude. Channel layout affords many opportunities for creation of multiple uses in addition to the channel's fundamental function of conveyance of the major event. Photograph MD-2 illustrates some of the multiple uses/benefits of well-planned major drainageways. Open channels are also usually less costly than closed conduits and they provide a higher degree of flood routing storage.

The function of open channels does not depend on a limited number of inlet points. Getting storm flows into a closed coduit system can be problematic since blockage of inflow points can be problematic and has been observed to occur during larger runoff events. Public safety is a major concern with closed conduits and the record of life loss is well documented when individuals were swept into a conduit.

Disadvantages of open channels include higher right-of-way needs and maintenance costs; however, maintenance of failed or failing closed conduits can be much more expensive. Careful planning and design can minimize the disadvantages and increase the benefits of open channel drainageways.

Photograph MD-2—Well-planned major drainageways provide biological diversity, recreational opportunities, and aesthetic benefits in addition to flood conveyance.

The choice of the type of open channel is a critical decision in planning and design of major drainageways. The ideal channel is a stable natural one carved by nature over a long period of time that can remain stable after urbanization. The benefits of such a channel can often include any or all of the following:

1. Relatively low-flow velocities sometimes resulting in longer concentration times and lower downstream peak flows.

2. Channel storage that tends to decrease peak flows.

3. Reasonable maintenance needs when the channel is somewhat stabilized.

4. A desirable greenbelt, which can support urban wildlife and recreation, adding significant social and environmental benefits. The REVEGETATION chapter provides guidance on vegetation selection, design, planning and maintenance for wetland and upland settings along naturalized man-made or stabilized natural channels.

5. Support of a variety of processes that preserve and/or enhance water quality, ranging from microbial activity in the bed and water column to the pollution prevention afforded by a stable channel's resistance to erosion.

Photograph MD-3—Integrating major drainageways into neighborhoods is critical for success.

Generally, the closer an artificial channel's character can be made to that of a natural channel, the more functional and attractive the artificial channel will be. In an urban area, however, it is rarely feasible to leave a natural channel untouched since urbanization alters the hydrology of the watershed. Consequently, some level of stabilization is usually necessary to prevent the channel from degrading and eroding.

Design of the major drainage system should consider the features and functions of the existing drainage system. Natural drainageways should be used for storm runoff waterways when feasible, and floodplains along drainageways should be preserved when feasible and practicable. Open channel planning and design objectives are often best met by using natural-like vegetated channels, which characteristically have slower velocities and large width-to-depth ratios. Efforts must be made to reduce peak flows and control erosion so that the natural channel regime is preserved, to the extent practical.

1.5 Fluvial Geomorphology

Any person who has witnessed the rise in stage of a river during spring snowmelt or who has observed the swelled banks of a river after an intense thunderstorm has a sense of the dynamic nature of waterways. Relatively simple hydraulic calculations can be performed to define flow conditions for a given set of specific, well-defined parameters; these techniques have been a highly effective basis of open channel design for many years. Walking along the bank of a channel, however, one quickly realizes that the actual behavior of the channel is far more complex than a simple, unchanging geometric cross section and a battery of design flow conditions. Fluvial geomorphology provides an approach to understanding the dynamic nature of a stream and the interactions between the water and the channel.

A drainage system within a watershed involves flowing water or movement of water, thus the term *fluvial*. When flowing water develops a drainage pattern or surface forms, the process is identified as *fluvial geomorphology*. Surface form characteristics represented by stream channels behave in a complex manner dependent on watershed factors such as geology, soils, ground cover, land use, topography, and hydrologic conditions. These same watershed factors contribute to the sediment eroded from the watershed and transported by the stream channel. The sediments moved by the flowing water also influence channel hydraulic characteristics. The natural-like channel and stabilization systems recommended in this *Manual* are based on fluvial geomorphology principles.

1.5.1 Stream Channel Characterization

At the start of the design process for on-site major drainageways, the designer should carefully characterize all existing channels on a reach-by-reach basis, documenting parameters including bank slope, bank cover, trees, bank line, sediment deposits, and scour areas in addition to geomorphic characteristics related to channel planform and hydraulics, such as sinuosity, riffle characteristics, cross-sectional geometry, and slope. For larger, complex channels, assistance from a specialist in stream channel behavior is recommended. Biologists can provide valuable input, as well, regarding existing wetland and upland vegetation, wildlife habitat, revegetation considerations, and other factors that indirectly relate to channel stability considerations.

Methods of channel assessment should utilize aerial photography, interviews with nearby residents, master plans, and other information available for the existing channel. Inspection of channels in areas of urbanization that, prior to development, had similar characteristics to the area planned for development can provide valuable foresight into channel changes likely to occur because of urbanization. By understanding channel behavior historically, currently, and in the future, the designer will focus on the optimal strategies for attaining channel stability. Detailed information regarding field data to collect for channel assessment is provided by Leopold (1994).

1.5.2 Effects of Urbanization on Stream Channels

In response to urbanization, stream channels can undergo substantial changes, especially if channel stabilization measures are not instituted in the early stages of urbanization. Urbanization causes (1) significant increases in peak discharges, total runoff volume, and frequency of bank-full discharges; (2) the steepening of channel slopes if and where natural channels are straightened to accommodate new development (this practice is discouraged by the District); (3) reduction in sediment bed load from fully developed areas; and (4) eroding and degrading natural channels. These factors, in combination, create conditions that are conducive to channel instability—widening (erosion) and deepening (degradation) in most reaches and debris and sediment accumulation (aggradation) in others. Photograph MD-4 illustrates severe channel degradation in response to increased flows caused by urbanization.

Photograph MD-4—Channel degradation in an unstable channel.

To fully evaluate the proper channel morphological processes when undertaking a basic design or protective measure project, it is necessary to have some knowledge of channel stability concepts. The normal objective of channel stability evaluation is identification of principal channel hydraulic parameters influencing the stability of the channel. After identifying these parameters under existing channel conditions, the values of these parameters under future conditions are estimated. For areas undergoing urbanization, one of the most important changes is an increase in the volume, frequency, and flow rates of water in main channels. Stability analysis is then performed based on hydraulic parameters for anticipated future conditions, and stabilization measures are planned to minimize potential channel erosion under future conditions. There are a number of quantitative methods of channel stability analysis available to the designer including allowable velocity methods (Fortier and Scobey 1926), tractive force calculations, and Leopold channel configuration relationships (Leopold 1994), among others.

1.5.3 Stable Channel Balance

A stable channel is usually considered an alluvial channel in equilibrium with no significant change in channel cross section with time. This is a *dynamic equilibrium* in which the stream has adjusted its width, depth, and slope so that the channel neither aggrades nor degrades. In this case, the sediment supply from upstream is equal to the sediment transport capacity of the channel. Under watershed conditions with normal hydrologic variations affecting runoff and sediment inflow, some adjustments in channel characteristics are inevitable.

An illustration, shown as Figure MD-1 (from USFISRWG 1998 [originally from Lane 1955a]), provides a visual depiction of a stable channel balance based on the relationship proposed by Lane (1955a) for the equilibrium concept whereby:

$$Q_w S \propto Q_s D_{50} \tag{MD-1}$$

in which:

Q_w = water discharge (cfs)

S = channel slope (ft/ft)

Q_s = bed material load (tons/day)

D_{50} = size of bed material (mm)

For a stable channel, these four parameters are balanced, and, when one or more of the parameters changes, the others adjust to restore the state of equilibrium. For example, if the stream flow increased with no change in channel slope, there would be an adjustment on the sediment side of the balance, with an increase in either bed material size or sediment load, or both.

1.5.4 References for Additional Information

Copious information exists on fluvial geomorphology ranging from the pioneering works of Lane, Leopold, and others to more recent compendiums on channel geomorphology and stability. References that may be useful to the designer include:

- *The Importance of Fluvial Morphology in Hydraulic Engineering* (Lane 1955b).
- *Progress Report on Results of Studies on the Design of Stable Channels: A Guide for Planners, Policymakers and Citizens* (Lane 1955a).
- *A View of the River* (Leopold 1994).
- *Restoring Streams in Cities* (Riley 1998).
- *Applied River Morphology* (Rosgen 1996).
- *Stream Corridor Restoration: Principles, Processes, and Practices* (USFISRWG 1998).
- *Sedimentation Engineering* (Vanoni (ed.) 1975).

- *Channel Rehabilitation: Process, Design, and Implementation* (Watson, Biedenharn, and Scott 1999).

Additional references can be found in the reference section of this chapter or in the extensive bibliographies of the references listed above.

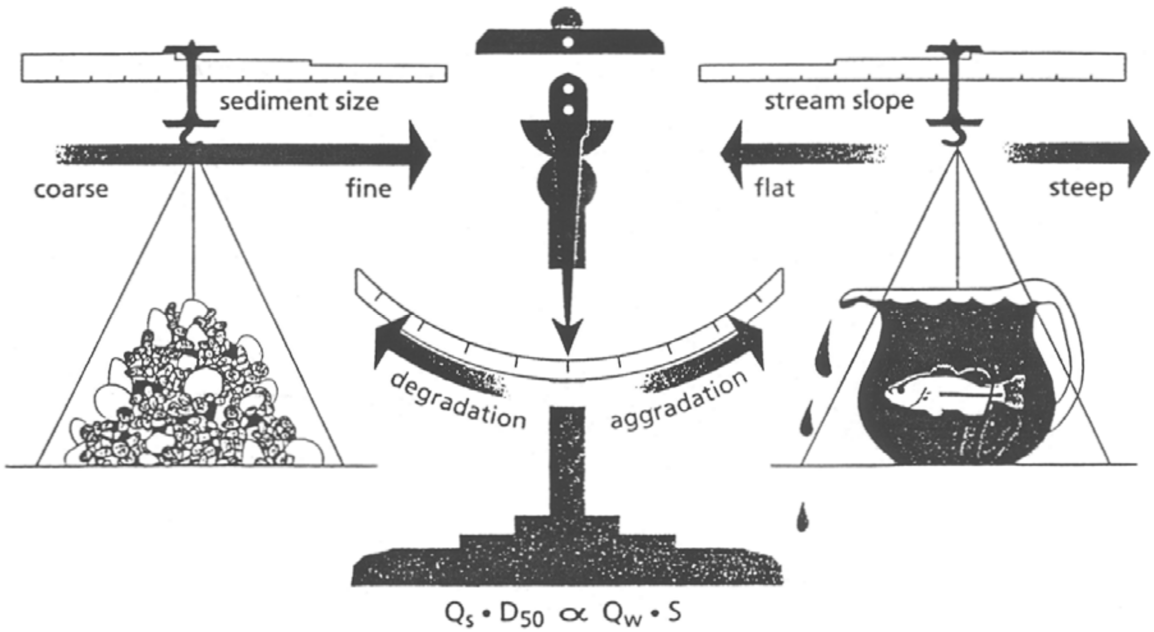


Figure MD-1—Illustration of the Stable Channel Balance Based on the Relationship Proposed by Lane (1955)

Note: This graphical interpretation of Lane's Equation was reprinted from *Applied River Hydrology*, with written permission from Mr. David Rosgen of Wildland Hydrology (and author of the book) in Pagosa Springs, Colorado.

2.0 PLANNING

2.1 General

A major drainage system that is thoughtfully planned can provide adequate conveyance of the major runoff event in addition to other benefits to the urban area that it serves. A basic policy of the District is that the major drainage system, regardless of type, should be capable of conveying water without flooding buildings and remain relatively stable during the major runoff event (e.g., the 100-year flood). A study of the POLICY and PLANNING chapters is suggested to provide a foundation for understanding this section.

By respecting natural drainage patterns and existing floodplains in planning, appropriate major drainageway systems, including natural-like open-channel drainageways, can be created that provide flood capacity, that are stable, cost effective, and environmentally sensitive, and that offer multiple use benefits to surrounding urban areas.

2.2 Impacts of Urbanization and Associated Effects

The hallmark of urbanization is increased imperviousness. Planning of a major drainage system must account for changes in hydrology, hydraulics, and channel stability that urbanization produces. As a result, the design of the major drainage system must be based on fully urbanized conditions to assure adequate capacity for conveyance of the major (e.g., 100-year) flood event. It is also important to recognize that the higher sediment loads during the process of urbanization (during construction) may shift the channel toward an equilibrium state that is different from the desired stable channel balance for the urbanized basin.

2.3 Special Considerations for Semi-Arid Climates

Major drainage planning and design efforts along natural waterways in the Denver area must consider the region's semi-arid climate. Special considerations include:

1. Streams that have historically been ephemeral or intermittent often develop base flow because of the increased volume of water from impervious areas and infiltration of lawn and garden irrigation water, water line leakage, car-wash rinse water, and other factors. In addition, the increase in impervious area from urbanization can result in dramatic increases in the volume, discharge, and frequency of surface runoff, especially relative to base flow (if any), resulting in channel instability.

2. Availability of water for support of vegetation must be evaluated when considering types of major drainage channels utilizing vegetation including grass-lined channels, channels with wetland bottoms, and bioengineered channels. This is especially important for channel types using wetland vegetation since the high productivity of wetland plants

results in a high level of water consumption. See the REVEGETATION chapter for additional information on vegetation selection and water use.

2.4 Route Considerations

A preliminary estimate of the design rate of flow is necessary to roughly approximate the channel or conduit capacity and size. This estimate can be made by comparing to other similar watersheds where unit rates of discharge have been computed, or using the design flow rates published in master plans.

Routing of the outfall is usually a relatively straightforward matter of following the natural valley (thalweg) and defining it on a map. In many urbanized and agricultural areas, however, there is no thalweg, or the thalweg has been filled and/or built upon. For these cases, it is necessary to determine many factors before the route is chosen. Representative items to determine for routing the outfall are discussed below, many of which apply even when the thalweg is defined.

2.4.1 Present Flow Path

Fully examine topographic mapping to determine where the storm runoff would go without any further work or modification to the ground surface.

2.4.2 Historic Flow Path

Determine, by using old mapping and aerial photographs, where the water would have flowed prior to any man-made changes.

2.4.3 Permitting and Regulations

Major drainage planning and design along existing natural channels are multi-jurisdictional processes and, therefore, must comply with regulations and requirements ranging from local ordinances to federal laws. The concept of floodplain regulation recognizes, and is premised upon, governmental responsibility for administration of publicly owned rights-of-way and flood-related prescriptive easements. At the local level, floodplain management is accomplished through zoning ordinances and land use regulations and/or requirements. On a regional level, floodplain management and drainage policies are identified in the POLICY chapter of this Manual.

All construction within the 100-year floodplain must comply with the National Flood Insurance Program (NFIP) regulations. Permits for all development in the 100-year floodplain and the Special Flood Hazard Area (SFHA) must be acquired from local governments. The policy of the District is to encourage the preservation and enhancement of natural floodplains whenever feasible. Filling floodplain fringes is generally discouraged because discharge and flood stoorage capacity in the flood fringe is important and filling tends to increase water surface elevations, velocity of flow, and downstream peak flows. All filling in the floodplain fringe should be undertaken with caution and in accordance with Federal Emergency

Management Agency (FEMA) and local regulations. Modifications to the 100-year floodplain related to the major drainage system must be documented through the FEMA map revision process.

Wetland regulations and permitting issues are also relevant to the major drainage system. A permit under Section 404 of the federal Clean Water Act (CWA) is required for any activities impacting "waters of the U.S. and jurisdictional wetlands." Construction of major drainage improvements along existing natural drainageways typically requires a Section 404 permit from the U.S. Army Corps of Engineers (USACE). In addition, routine maintenance activities along established major drainage channels and in wetlands may also require a Section 404 permit. Always check with the USACE to determine if the proposed channel work or maintenance activities require a 404 permit. In addition to federal wetland regulations, construction of major drainage improvements along existing natural drainageways may be subject to the federal Endangered Species Act. Early and regular discussions and coordination with permitting authorities is encouraged from start through final mitigation activities. Refer to Section 3.3.3 for additional information on permitting.

2.4.4 Public Safety

Public safety is fundamental to the major drainage system. One purpose of the major drainage system is to protect an urban area from extensive property damage and loss of life from flooding. However, there are also "day-to-day" safety considerations in design such as the use of railings at vertical walls and avoiding vertical drops and use of steep side slopes adjacent to public trails.

2.4.5 Public Acceptance

Planning and design are of primary importance in gaining public acceptance. Public acceptance of the major drainage system depends on many factors such as public perception of flood protection, channel aesthetics, right-of-way, open space preservation, and channel maintenance. The use of open channels, especially those utilizing vegetation and other natural material and natural-like planform and morphology can create aesthetic and recreational amenities for the public and often are congruous with community open space goals. The general principle that the closer an artificial channel's character is to that of a natural channel, the better the artificial channel will be, often holds true for public acceptance, as well.

2.4.6 Alternate Routes

Choose various routes on maps and examine them in the field from engineering viewpoints. Also, determine social impacts on neighborhoods and general environmental design restraints.

2.4.7 Maintenance

Identify points of access along alternate routes based on existing and proposed roads and public rights-of-way. Adequate right-of-way is necessary to provide maintenance access for a major drainageway.

2.4.8 Route Costs

Prepare profiles of apparently satisfactory routes and make rough cost estimates of each, using approximations as to character and location of channel or conduit. Include costs of bridges, culverts, drop structures, special structures and facilities, etc.

2.4.9 Recreational Use Potential

Identify areas with potential for recreational use. Factors to consider include proximity to residential areas, access to channel via roads and trails, areas suitable for creation of multi-use areas along channel, and location of potentially hazardous areas.

2.4.10 Environmental Considerations

Examine advantages and disadvantages of routes with an environmental design team normally consisting of an urban planner, biologist, and landscape architect, and, in some cases, an urban sociologist and drainage attorney. Include USACE regulatory personnel in these examinations to identify permitting issues that need to be addressed and to avoid 404 permitting problems later. Choose the best route based upon maximum total advantages and benefits.

2.4.11 Presentation of Choice

A meeting should be held between project sponsors and affected parties to discuss the routes studied and to select the final route. At the same time, the types of channel or conduit being considered should be presented and suggestions or concurrence should be obtained. A dialogue with citizen groups where various alternates are explained is encouraged.

2.4.12 Underground Conduits

Open channels for transporting major storm runoff are more desirable than underground conduits in urban areas because they are closer in character to natural drainageways and offer multiple use benefits. However, right-of-way constraints in urbanized areas (in the case of redevelopment, for example) may necessitate the use of underground conduits. District does not support the practice of putting major drainageways into underground conduits unless there is an overwhelming need to reduce flooding in already developed areas. The primary considerations when selecting underground conduits are public safety concerns of people being swept into them and the fact that underground conduits are extremely susceptible to having their inflow points clogged, especially when equipped with safety or trash racks. Once clogged, they fail to provide the intended flood protection. For this reason, overflow paths should be provided for to have little or no flood damage when the inlet end of a long conduit is clogged.

2.4.13 Two-Stage Channels

In some cases, it may be desirable to distribute the 100-year flow between a formal channel and the adjacent floodplain. These two-stage channels are acceptable as long as they are designed so that velocity and depth criteria stated in this chapter are satisfied for the 100-year event. Freeboard must still be provided between the 100-year water surface profile and the lowest point of building entry or first floor elevation, whichever is lower, and all applicable roadway overtopping criteria must be considered.

2.5 Layout

The approximate centerline should be laid out on topographic mapping and adjustments made for best fit. At a minimum, the following factors should be taken into consideration:

- Land form (including topography and historic and existing thalwegs)
- Right-of-way
- Curvature
- Existing or future streets
- Ability to drain adjacent land

2.5.1 Working Map

The outfall should be surveyed with adequate detail. An aerial photographic contour map with 2-foot contours at a scale of 1 inch to 50 feet or 100 feet is desirable. In the case of an outfall conduit, a centerline field survey often suffices if adequate adjacent conditions are reflected in the survey.

2.5.2 Preliminary Plan and Profile

The existing ground surface, street grades, conflicting utilities, and other pertinent data can be plotted in plan and profile. Grades should be noted and analyzed and thought should be given to hydraulic requirements. Adjustments to the centerline should be made where needed to alleviate problem areas when possible and to provide the maximum total benefits.

2.6 Master Planning or Preliminary Design

The preliminary design portion of the planning phase is second in importance only to route selection and the concept stage. Here major decisions are made as to design velocities, location of structures, means of accommodating conflicting utilities, and potential alternate uses in the case of an open channel. Decisions on the use of downstream detention storage or upstream storage also need to be made. The planning and preliminary design should include evaluation of the full spectrum of channel improvements for application in each major drainage management project.

2.6.1 Criteria for Final Hydrology

The characteristics of the outfall are defined after the master planning is underway. At this time, the final hydrological analyses should be performed for additional refinements and use as the proposed conveyance geometries can affect the peak flows in the total system.

2.7 The Master Plan

The master major drainage plan must both provide thorough attention to engineering detail and be suitable for day-to-day use by local and regional governmental administrators. Drainage facility designers should check relevant major drainageway/outfall master plans to assure that the facilities they are designing are consistent with the intent of these master plans. The significant parts of a master plan are described below.

2.7.1 Report

The report shall include a description of the basin, the present and future ultimate development (both on-site and in the upstream drainage area), rainfall data, unit hydrograph derivations, major runoff quantities, engineering criteria used in planning, alternate plans, environmental design considerations, legal opinions, and recommendations. The ability of the major drainage system to serve the total tributary basin must be demonstrated.

2.7.2 Drawings

The drawings shall be prepared on full-size plan and profile sheets at a scale of 1 inch to 50, 100, 200, or even 400 feet, as appropriate for the plan being developed. Detail must be shown in regard to bottom elevations, the approximate hydraulic grade line, bridge and culvert opening criteria, and typical cross sections. Adequate information is needed to provide a guide to land acquisition.

3.0 OPEN CHANNEL DESIGN PRINCIPLES

This section is intended to provide the designer with information necessary to perform open channel hydraulic analysis related to channel geometry, channel lining, and flow characteristics. This section includes preliminary design criteria and identifies considerations in selection of channel type.

3.1 General Open Channel Flow Hydraulics

Whether using a natural or constructed channel, hydraulic analyses must be performed to evaluate flow characteristics including flow regime, water surface elevations, velocities, depths, and hydraulic transitions for multiple flow conditions. Open channel flow analysis is also necessary for underground conduits to evaluate hydraulics for less-than-full conditions. Hydraulic grade lines and energy grade lines should be prepared on all design projects.

The purpose of this section is to provide the designer with an overview of open channel flow hydraulics principles and equations relevant to the design of open channels. Many excellent references address open channel hydraulics in great detail, including Chow (1959), Daugherty and Franzini (1977), and King and Brater (1963). Water surface profile computations are not addressed herein, and the reader is referred to these references for discussion of this topic.

3.1.1 Types of Flow in Open Channels

Open channel flow can be characterized in many ways. Types of flow are commonly characterized by variability with respect to time and space. The following terms are used to identify types of open channel flow:

- *Steady flow*—conditions at any point in a stream remain constant with respect to time (Daugherty and Franzini 1977).
- *Unsteady flow*—flow conditions (e.g., depth) vary with time.
- *Uniform flow*—the magnitude and direction of velocity in a stream are the same at all points in the stream at a given time (Daugherty and Franzini 1977). If a channel is uniform and resistance and gravity forces are in exact balance, the water surface will be parallel to the bottom of the channel for uniform flow.
- *Varied flow*—discharge, depth, or other characteristics of the flow change along the course of the stream. For a steady flow condition, flow is termed *rapidly varied* if these characteristics change over a short distance. If characteristics change over a longer stretch of the channel for steady flow conditions, flow is termed *gradually varied*.

For the purposes of open channel design, flow is usually considered steady and uniform. For a channel with a given roughness, discharge, and slope, there is only one possible depth for maintaining a uniform flow. This depth is the *normal depth*. When roughness, depth, and slope are known at a channel section, there can only be one discharge for maintaining a uniform flow through the section. This discharge is the *normal discharge*.

Manning's Equation describes the relationship between channel geometry, slope, roughness, and discharge for uniform flow:

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \qquad \text{(MD-2)}$$

in which:

Q = discharge (cfs)

n = roughness coefficient

A = area of channel cross section (ft^2)

R = hydraulic radius = Area/Wetted Perimeter, P (ft)

P = wetted perimeter (ft)

S = channel bottom slope (ft/ft)

Manning's Equation can also be expressed in terms of velocity by employing the continuity equation, $Q = VA$, as a substitution in Equation MD-2, where V is velocity (ft/sec).

For wide channels of uniform depth, where the width, b, is at least 25 times the depth, the hydraulic radius can be assumed to be equal to the depth, y, expressed in feet, and, therefore:

$$Q = \frac{1.49}{n} b y^{5/3} S^{1/2} \qquad \text{(MD-3)}$$

$$y = \frac{Q^{0.6} n^{0.6}}{1.27 b^{0.6} S^{0.3}} \qquad \text{(MD-4)}$$

$$S = \frac{(Qn)^2}{2.2 b^2 y^{3.33}} \qquad \text{(MD-5)}$$

Since solution of Equation MD-2 for depth is iterative, a number of techniques are useful to quickly obtain the solution without having to perform iterations. Figure MD-2 can be used to determine normal depth graphically based on convenient dimensionless parameters. In addition, the ***UD-Channels*** spreadsheet available through the www.udfcd.org website can be used to perform normal flow calculations for trapezoidal channels and can help with the design of such channels. Example MD-1, provided at the end of this chapter, illustrates application of this spreadsheet for finding the normal depth of a trapezoidal channel.

The designer should realize that uniform flow is more often a theoretical abstraction than an actuality (Calhoun, Compton, and Strohm 1971), namely, true uniform flow is difficult to find. Channels are sometimes designed on the assumption that they will carry uniform flow at the normal depth, but because of ignored conditions the flow actually has depths that can be considerably different. Uniform flow computation provides only an approximation of what will occur

3.1.2 Roughness Coefficients

When applying Manning's Equation, the choice of the roughness coefficient, n, is the most subjective parameter. Table MD-1 provides guidance on values of roughness coefficients to use for channel design. Both maximum and minimum roughness coefficients should be used for channel design to check for sufficient hydraulic capacity and channel lining stability, respectively. When using the retardance curves for grass-lined channels and swales, use Retardance C for finding Manning's n for finding the depth in a mature channel and Retardance D for finding the controlling velocity in a newly constructed channel.

The designer should be aware that roughness greater than that assumed will cause the same discharge to flow at a greater depth, or conversely that flow at the computed depth will result in less discharge. Obstructions in the channel will cause an increase in depth above normal depth and must be taken into account. Sediment and debris in channels increase roughness coefficients, as well, and should be accounted for.

For additional information on roughness coefficients, the reader is referred to the U.S. Geological Survey Water Supply Paper 1849 (Barnes, Jr. 1967).

3.1.3 Flow Regime

Another important characteristic of open channel flow is the state of the flow, often referred to as the flow regime. Flow regime is determined by the balance of the effects of viscosity and gravity relative to the inertia of the flow. The Froude number, F_r, is a dimensionless number that is the ratio of inertial forces to gravitational forces that defines the flow regime. The Froude number is given by:

$$F_r = \frac{V}{\sqrt{gd}} \tag{MD-6}$$

in which:

V = mean velocity (ft/sec)

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

d = hydraulic depth (ft) = A/T, cross-sectional area of water/width of free surface

Table MD-1—Roughness Coefficients ("*n*") for Channel Design

(After Chow 1959)

Channel Type	Roughness Coefficient (*n*) Minimum	Typical	Maximum
I. Excavated or Dredged			
1. Earth, straight and uniform			
a. Gravel, uniform section, clean	0.022	0.025	0.030
b. With short grass, few weeds	0.022	0.027	0.033
2. Earth, winding and sluggish			
a. Grass, some weeds	0.025	0.030	0.033
b. Dense weeds or aquatic plants	0.030	0.035	0.040
c. Earthy bottom and rubble/riprap sides	0.028	0.030	0.035
3. 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
II. Natural streams (top width at flood stage 100 ft)			
1. Streams on plain			
a. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
b. Clean, winding, some pools and shoals, some weeds and stones	0.035	0.045	0.050
c. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
III. Lined or Built-Up Channels			
1. Concrete			
a. Trowel/float finish	0.011	0.015	0.016
b. Shotcrete	0.016	0.020	0.025
2. Gravel bottom with sides of:			
a. Formed concrete	0.017	0.020	0.025
b. Random stone in mortar	0.020	0.023	0.026
c. Dry rubble or riprap	0.023	0.033	0.036
3. Wetland Bottom Channels	See Figure MD-9a		
4. Grass-Lined Channels and Swales	See Figure MD-9b		

When F_r = 1.0, flow is in a *critical* state. When F_r < 1.0, flow is in a *subcritical* state. When F_r > 1.0, flow is in a *supercritical* state. The following sections describe these flow regimes and associated criteria for channel design.

The *specific energy* of flow in a channel section is defined as the energy per pound of water measured with respect to the channel bottom. Specific energy, E (expressed as head in feet), is given by:

$$E = y + \frac{V^2}{2g} = y + \frac{Q^2}{2gA^2} \qquad \text{(MD-7)}$$

in which:

y = depth (ft)

V = mean velocity (ft/sec)

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

Q = discharge (cfs)

A = area of channel cross section (ft^2)

For all subcritical channels, check the Froude number using the *minimum* value of n. When performing hydraulic computations for grassed channels, the n values for the 0.1-foot to 1.5-feet flow depth range are generally suitable for calculating the wetted channel portion for the initial storm runoff. For major runoff computations, however, the greater than 3.0-feet depth values are more appropriate since flows will tend to lay the grass down to form a smoother bottom surface.

3.1.3.1 Critical Flow

Critical flow in an open channel or covered conduit with a free water surface is characterized by several conditions (Fletcher and Grace 1972):

1. The specific energy is a minimum for a given discharge.
2. The discharge is a maximum for a given specific energy.
3. The specific force is a minimum for a given discharge.
4. The velocity head is equal to half the hydraulic depth in a channel of small slope.
5. The Froude number is equal to 1.0 (see Equation MD-6.)
6. The velocity of flow in a channel of small slope is equal to the celerity of small gravity waves in shallow water.

If the critical state of flow exists throughout an entire reach, the channel flow is critical flow, and the channel slope is at critical slope, S_{cr}. A slope less than S_{cr} will cause subcritical flow, and a slope greater than S_{cr} will cause supercritical flow. A flow at or near the critical state may not be stable. In design, if the depth is found to be at or near critical, the shape or slope should be changed to achieve greater hydraulic stability.

To simplify the computation of critical flow, dimensionless curves have been given for rectangular, trapezoidal, and circular channels in Figure MD-3. Critical velocity, V_c, can be calculated from the critical hydraulic depth, d_c. For a rectangular channel, the flow depth is equal to hydraulic depth, ($y_c = d_c$), and the critical flow velocity is:

$$V_c = \sqrt{g y_c} \qquad \text{(MD-8)}$$

In addition, the **Critical** worksheet from the UD-Channels Spreadsheet performs critical depth calculations.

3.1.3.2 Subcritical Flow

Flows with a Froude number less than 1.0 are *subcritical* flows and have the following characteristics relative to critical flows (Maricopa County 2000):

1. Flow velocity is lower.
2. Flow depth is greater.
3. Hydraulic losses are lower.
4. Erosive power is less.
5. Behavior is easily described by relatively simple mathematical equations.
6. Surface waves can propagate upstream.

Most stable natural channels have *subcritical* flow regimes. Consistent with the District's philosophy that the most successful artificial channels utilize characteristics of stable natural channels, major drainage design should seek to create channels with *subcritical* flow regimes.

A concrete-lined channel should not be used for subcritical flows except in unusual circumstances where a narrow right-of-way exists. A stabilized natural channel, a wide grass-lined channel, a channel with a wetland bottom, or a bioengineered channel is normally preferable in the Denver region. Do not design a subcritical channel for a Froude number greater than 0.8 using the velocity and depth calculated with the lowest recommended range for Manning's n. When designing a concrete-lined channel for subcritical flow, use a Manning's n = 0.013 for capacity calculations and 0.011 to check whether the flow could go supercritical. If significant sediment deposition or sediment transport is likely, a Manning's n greater than 0.013 may be necessary for capacity calculations.

3.1.3.3 Supercritical Flow

Flows with a Froude number greater than 1.0 are supercritical flows and have the following characteristics relative to critical flows (Maricopa County 2000):

1. Flows have higher velocities.
2. Depth of flow is shallower.
3. Hydraulic losses are higher.
4. Erosive power is greater.
5. Surface waves propagate downstream only.

Supercritical flow in an open channel in an urban area creates hazards that the designer must consider.

From a practical standpoint, it is generally not practical to have curvature in such a channel. Careful attention must be taken to prevent excessive oscillatory waves, which can extend down the entire length of the channel from only minor obstructions upstream. Imperfections at joints can cause rapid deterioration of the joints, which may cause a complete failure of the channel. In addition, high velocity flow at cracks or joints creates an uplift force by creating zones of flow separation with negative pressures and converts the velocity head to pressure head under the liner which can virtually tear out concrete slabs. It is evident that when designing a lined channel with supercritical flow, the designer must use utmost care and consider all relevant factors.

In the Denver region, all channels carrying supercritical flow shall be lined with continuously reinforced concrete linings, both longitudinally and laterally. There shall be no diminution of wetted area cross section at bridges or culverts. Freeboard shall be adequate to provide a suitable safety margin. Bridges or other structures crossing the channel must be anchored satisfactorily to withstand the full dynamic load that might be imposed upon the structure in the event of major trash plugging.

The concrete linings must be protected from hydrostatic uplift forces that are often created by a high water table or momentary inflow behind the lining from localized flooding. A perforated underdrain pipe, designed to be free draining, is required under the lining. For supercritical flow, minor downstream obstructions do not create any backwater effect. Backwater computation methods are applicable for computing the water surface profile or the energy gradient in channels having a supercritical flow; however, the computations must proceed in a downstream direction. The designer must take care to prevent the possibility of unanticipated hydraulic jumps forming in the channel. Flows at Froude numbers between 0.8 and 1.2 are unstable and unpredictable and should be avoided.

Roughness coefficients for lined channels are particularly important when dealing with supercritical flow. Once a particular roughness coefficient is chosen, the construction inspection must be carried out in a manner to insure that the particular roughness is obtained. Because of field construction limitations, the designer should use a Manning's n roughness coefficient equal to 0.013 for a well-trowelled concrete finish. Other finishes should have proportionately larger n values assigned to them.

3.2 Preliminary Design Criteria

3.2.1 Design Velocity

Minimum and maximum velocities must be considered in the design of major drainage systems. From structural and stability standpoints, maximum velocities are of concern; however, minimum velocities should also be considered in design with respect to sediment accumulation and channel maintenance. For channels with high velocity flows, drop structures, suitable channel lining, check dams or other velocity controls will be necessary to control erosion and maintain channel stability. *Subcritical* flow is desirable since the velocity for *subcritical* flow is less than that of critical or *supercritical* flow for a given discharge. Froude number criteria also restrict velocity.

The flow velocity during the major design storm (i.e., 100-year) must recognize the scour potential of the channel, whether natural, grassed, bioengineered, riprapped or concrete-lined. Average velocities need to be determined using backwater calculations, which account for water drawdowns at drops, expansions, contractions and other structural controls. Velocities must be kept sufficiently low to prevent excessive erosion in the channel. As preliminary design criteria, flow velocities should not exceed velocities and Froude numbers given in Table MD-2 for non-reinforced channel linings and, in general, should not exceed 18 ft/sec for reinforced channel linings. Channel-specific velocity criteria depend greatly on the channel lining and slope and are presented in more detail in Section 4.0 of this chapter for various types of open channels.

For estimating maximum velocities for erosive or hazard considerations or localized scour in a channel, relying only upon the HEC-2 or HEC-RAS (USACE 1991, 1995) outputs for the cross section is not acceptable. Instead, more detailed hydraulic analysis of the specific cross section, which accounts for variable velocities across the channel, is necessary.

3.2.2 Design Depths

The maximum design depths of flow should also recognize the scour potential of the channel lining and the bank materials. Scouring power of water increases in proportion to the third to fifth power of flow depth and is also a function of the length of time flow is occurring (USBR 1984). As criteria, the design depth of flow for the major storm runoff flow during a 100-year flood should not exceed 5.0 feet in areas of the channel cross section outside the low-flow channel area, and less depth is desirable for channel stability. Low-flow channel depth should be between 3.0 and 5.0 feet.

3.2.3 Design Slopes

3.2.3.1 Channel Slope

The slope of a channel affects flow velocity, depth, and regime and can have a significant impact on erosion and channel stability. Channel slope criteria vary based on the type of channel; however, the slope of a channel should not be so steep as to result in a Froude number greater than 0.5 or 0.8, depending on soil erodibility characteristics (see Table MD-2), for the 100-year event. Slopes for channels with vegetative linings should not exceed 0.6% and should be less than 1% for channels with reinforced concrete linings. For steep-gradient drainageways, drop structures are necessary to meet slope criteria. An important consideration in channel slope is sinuosity of the channel—straightening of a natural channel inevitably results in an increase in slope. Conversely, for a constructed channel, a design incorporating meanders can be used to satisfy slope criteria, potentially reducing the number of drop structures required.

3.2.3.2 Side Slopes

The flatter the side slopes, the more stable are the banks. For grassed channels, channels with wetland bottoms, and bioengineered channels, side slopes should not be steeper than 4H:1V. Under special conditions in areas of existing development (i.e., not new development) and where right-of-way is a problem, the slopes may be as steep as 3H:1V; however, the designer is cautioned that operation of mowing equipment may not be safe on side slopes that are steeper than 4H:1V. Channels that require minimal slope maintenance such as concrete channels may have side slopes as steep as 1.5H:1V, although public safety issues must be taken into account. For riprap-lined channels, side slopes should not be steeper than 2.5H:1V.

For vegetated channels with underlying riprap, slopes must accommodate maintenance. For example, a grassed channel with underlying riprap should have side slopes no steeper than 4H:1V, as required for a grassed channel.

Local standards or conditions may require flatter side slopes. Side slopes steeper than 3H:1V are not recommended in residential areas or areas with frequent foot traffic. Fencing or railings may need to be considered if side slopes will be steeper than 3H:1V in these areas.

3.2.4 Curvature and Transitions

Generally, the gentler the curves, the better the channel will function. Channel alignments should not be selected to maximize land-use opportunities for lot layout; instead, lot layouts should be selected based on channel alignment. The centerline curvature of the channel shall have a radius of at least twice the top width of the 100-year flow channel. The exception to this axiom is for concrete channels that may experience *supercritical* flow conditions. From a practical standpoint, it is generally not advisable to have any curvature in a channel conveying *supercritical* flow, since minor perturbations can be amplified as they move downstream.

Superelevation must also be considered with respect to curvature. Curves in a channel cause the flow velocity to be greater on the outside of the curve, and the depth of flow is also greater on the outside of a curve due to centrifugal force. This rise in water surface on the outside of a curve is referred to as superelevation. For *subcritical* flows, superelevation can be estimated by:

$$\Delta y = \frac{V^2 T}{2 g r_c} \qquad \text{(MD-9)}$$

in which:

Δy = increase in water surface elevation above average elevation due to superelevation (ft)

V = mean flow velocity (ft/sec)

T = top width of the channel under design flow conditions (ft)

g = gravitational constant = 32.2 ft/sec^2

r_c = radius of curvature (ft)

Transitions (expansions and contractions) are addressed in Section 4.4 (riprap-lined channels) and in Section 5.0 of the HYDRAULIC STRUCTURES chapter.

3.2.5 Design Discharge Freeboard

Residual discharge freeboard is necessary to ensure that a design developed using idealized equations will perform as desired under actual conditions. The amount of residual freeboard that must be allowed depends on the type of channel and the location and elevation of structures adjacent to the channel. Preserving existing floodplains maximizes "natural" freeboard. Freeboard requirements are addressed for a number of specific channel types in Section 4.0 of this chapter; however, in general, a minimum residual freeboard of 1 to 2 feet should be allowed between the water surface and top of bank.

3.2.6 Erosion Control

Erosion control pertains to major drainage channels on the watershed scale as well as the drainage corridor scale. On the watershed scale, erosion and sediment control is critical in areas of urbanization, especially active construction areas, to prevent loading of initial and major drainageways with excessive sediment from disturbed areas in the watershed. Poor control of erosion on the watershed scale can result in increased maintenance and decreased capacity of major drainageways. Watershed erosion and sediment control is beyond the scope of this *Manual* but is regulated at the federal, state, regional, and local levels. In the State of Colorado, the Colorado Department of Public Health and Environment administers the National Pollutant Discharge Elimination System (NPDES) of the CWA, which requires stormwater management measures including erosion and sediment controls for construction sites larger than 1 acre under the Stormwater Permitting Regulations. In addition, most localities in Colorado require erosion and sediment control measures for construction sites.

For major drainage channels, protection against erosion is key to maintaining channel stability. Unless hard-lined and vigilantly maintained, most major drainage channels are susceptible to at least some degree of erosion. The concave outer banks of stream bends are especially susceptible to erosion and may require armoring with riprap for grassed, bioengineered, or wetland bottom channels. While high sediment loads to a channel may occur as a result of active construction in the watershed, once an area is fully urbanized, the channel behavior changes. Flows increase significantly due to the increase in imperviousness in the watershed, and the runoff from these fully urbanized areas contains relatively low levels of sediment. As a result, the potential for erosion in the channel increases.

In the Denver area, most waterways will need the construction of drops and/or erosion cutoff check structures to control the channel slope. Typically, these grade control structures are spaced to limit channel degradation to what is expected to be the final stable longitudinal slope after full urbanization of the tributary watershed. The designer should also be aware of the erosion potential created by constriction and poorly vegetated areas. An example is a bridge crossing over a grassed major drainage channel, where velocities increase as a result of the constriction created by the bridge, and bank cover is poor due to the inability of grass to grow in the shade of the bridge. In such a situation, structural stabilization, such as riprap, may be needed.

Another aspect of erosion control for major drainage channels is controlling erosion during and after construction of channel improvements. Construction of channel improvements during times in the year that are typically dryer can reduce the risk of erosion from storm runoff. Temporary stabilization measures including seeding and mulching and erosion controls such as installation and maintenance of silt fencing should be used during construction of major drainage improvements to minimize erosion.

3.2.7 Summary of Preliminary Design Guidance

Table MD-2 summarizes the guidance for the preliminary design of man-made channels discussed above. This guidance is for simple trapezoidal shapes to approximate alignment and geometry. Final design of man-made channels of a more complex nature will be discussed in Section 4.0.

Table MD-2—Trapezoidal Channel Design Guidance/Criteria

Design Item	Major Drainage Chapter Section	Criteria for Various Types of Channel Lining			
		Grass: Erosive Soils	Grass: Erosion Resistant Soils	Riprap	Concrete
Maximum 100-yr velocity	3.2.1	5.0 ft/sec	7.0 ft/sec	12.0 ft/sec	18.0 ft/sec
Minimum Manning's *n*—stability check	Table MD-3	0.03	0.03	0.03	0.011
Maximum Manning's *n*—capacity check	Table MD-3	0.035	0.035	0.04	0.013
Maximum Froude number	3.2.1	0.5	0.8	0.8	N/A
Maximum depth outside low-flow zone	3.2.2	5.0 ft	5.0 ft	n/a	N/A
Maximum channel longitudinal slope	3.2.3.1	0.6%	0.6%	1.0%	N/A
Maximum side slope	3.2.3.2	4H:1V	4H:1V	2.5H:1V	1.5H:1V [4]
Minimum centerline radius for a bend	3.2.4	2 x top width	2 x top width	2 x top width	2 x top width
Minimum freeboard [3]	3.2.5	1.0 ft [1]	1.0 ft [1]	2.0 ft [1]	2.0 ft [2]

[1] Suggested freeboard is 2.0 ft to the lowest adjacent habitable structure's lowest floor.

[2] For supercritical channels, use the freeboard recommended in Section 4.3.1.5 for final design.

[3] Add superelevation to the normal water surface to set freeboard at bends.

[4] Side slopes may be steeper if designed as a structurally reinforced wall to withstand soil and groundwater forces.

3.2.8 Maintenance Eligibility

The minimum design criteria requirements below must be satisfied as of June 2001 for a major drainage channel to be eligible for District maintenance assistance. Note that the District's *Maintenance Eligibility Guidelines* may change with time. The reader is directed to the District's Web site (www.UDFCD.org) for the latest version of the *Maintenance Elegibility Guidelines*.

3.2.8.1 Natural Channels (Open Floodplain Design)

When a developer chooses to stay out of the 100-year floodplain, the following requirements must be met:

1. If the total flow of the channel and floodplain is confined to an incised channel and erosion can be expected to endanger adjacent structures, 100-year check structures are required to control erosion and degradation of the channel area. See the HYDRAULIC STRUCTURES chapter of this *Manual* for more information. In addition, sufficient right-of-way shall be reserved to install the equivalent of a trapezoidal grass-lined channel that satisfies the velocity criteria specified in Table MD-2. Extra width shall be reserved where drop structures are needed, in which locations a 20-foot-wide maintenance access bench shall be provided along one side of the channel.

2. If the floodplain is wide and the low-flow channel represents a small portion of the floodplain area, low-flow check structures are usually required, unless it can be demonstrated that the channel will remain stable as the watershed urbanizes.

3. Consult the applicable Urban Drainage and Flood Control District's master plan document for guidance on the design event and stable steam or waterway longitudinal slope.

4. For either of the above cases, a maintenance access trail shall be provided. It should be designed according to the guidelines for grass-lined channels in Section 3.2.8.3, below.

3.2.8.2 Open Floodway Design (Natural Channel With Floodplain Encroachment)

Although floodplain preservation is preferable, when the design involves preserving the floodway while filling and building on the fringe area, the developer must meet the requirements in Section 3.2.8.1, and the fill slopes must be adequately protected against erosion with:

1. Fill slopes of 4H:1V or flatter that are vegetated according to the criteria in the REVEGETATION chapter.

2. Fill slopes protected by rock (not broken concrete or asphalt) riprap meeting District criteria with up to 2.5H:1V slopes.

3. Retaining walls, no taller than 3.5 feet, with adequate foundation protection.

3.2.8.3 Grass-Lined Channel Design

The design for a grass-lined channel must meet the following criteria to be eligible for District maintenance:

1. Side slopes should be 4H:1V or flatter.

2. Continuous maintenance access, such as with a trail, must be provided. The stabilized trail surface must be at least 8 feet wide with a clear width of 12 feet. It shall be located above the minor event water surface elevation (usually 2- to 10-year event, as directed by local government), but never less than 2-feet (3-feet for streams with perennial flow). Trail profiles need to be shown for all critical facilities such as roadway crossings, stream crossings and drop structures. All access trails shall connect to public streets. Maintenance trials need not be paved, but must be of all-weather construction such as aggregate base course, crusher fines, recycled concrete course or Aggregate Turf Reinforced Grass Pavement (RGP) described in Volume 3 of this *Manual* and capable of sustaining loads associated with large maintenance equipment. Paved trails are encouraged to allow for recreational use of the trails. When paved, pavement should be 5-inches minimum thickness of concrete (not asphalt). Maximum longitudinal slope for maintenance-only trails is 10%, but less than 5% when used as multi-purpose recreational trails to meet the requirements of the *Americans with Disabilities Act*. The District may accept adjacent public local streets or parking lots in lieu of a trail.

3. A low-flow or trickle channel is desirable. See Section 4.1.5 of this chapter for criteria.

4. Wetland bottom and bioengineered channels are acceptable when designed according to District wetland bottom channel criteria in Section 4.2 of this chapter.

5. The channel bottom minimum cross slope for dry bottom channels shall be 1%.

6. Tributary inflow points shall be protected all the way to the low-flow channel or trickle channel to prevent erosion. Inflow facilities to wetland bottom channels shall have their inverts at least 2 feet above the channel bottom to allow for the deposition of sediment and shall be protected with energy dissipaters.

7. All roadway crossings of wetland bottom channels shall incorporate a minimum of a stblized 2-foot drop from the outlet to the bottom of the downstream channel in order to preserve hydraulic capacity as sediment deposition occurs ovcer time in the channel.

8. All drop structures shall be designed in accordance with the HYDRAULIC STRUCTURES chapter of this *Manual*. Underdrain and storm sewer outlets located below the stilling basin's end sills are not acceptable. Construction plans shall utilize District standard details.

9. Storm sewer outlets shall be designed in accordance with the criteria in Sections 5.0, 6.0, and 7.0 of this chapter. Alternatively, conduit outlet sstructures, including low tailwater riprap basins design described in Section 3.0 of the HYDRAULIC STRUCTURES chapter of the *Manual* shall be used when appropriate.

10. Grouted boulder rundowns and similar features shall be designed in accordance with Section 7.0

of the HYDRAULIC STRUCTURES chapter of the *Manual*.

11. Grass seeding specifications provided by the District (see the REVEGETATION chapter of this *Manual*) are recommended unless irrigated blue grass is used. The District will not maintain irrigated blue grass (due to cost constraints), but other elements of such a channel (i.e., drop structures, trickle channel) can still qualify for maintenance eligibility.

3.3 Choice of Channel Type and Alignment

3.3.1 Types of Channels for Major Drainageways

The types of major drainage channels available to the designer are almost infinite, depending only upon good hydraulic practice, environmental design, sociological impact, and basic project requirements. However, from a practical standpoint, it is useful to identify general types of channels that can be used by the designer as starting points in the design process. The following types of channels may serve as major drainage channels for the 100-year runoff event in urban areas:

Natural Channels—Natural channels are drainageways carved or shaped by nature before urbanization occurs. They often, but not always, have mild slopes and are reasonably stable. As the channel's tributary watershed urbanizes, natural channels often experience erosion and degrade. As a result, they require grade control checks and stabilization measures. Photograph MD-5 shows a natural channel serving as a major drainageway for an urbanized area.

Photograph MD-5—Natural channel (open floodplain design) serving as a major drainageway. Note the preservation of riparian vegetation, absence of floodplain encroachment and the use of grade control structures to arrest thalweg downcutting (i.e., channel incising/degradation)

Grass-Lined Channels—Among various types of constructed or modified drainageways, grass-lined channels are some of the most frequently used and desirable channel types. They provide channel storage, lower velocities, and various multiple use benefits. Grass-lined channels in urbanizing watersheds should be stabilized with grade control structures to prevent downcutting, depression of the water table, and degradation of natural vegetation. Low-flow areas may need to be armored or otherwise stabilized to guard against erosion. Photograph MD-6 shows a grass-lined major drainage channel.

Photograph MD-6—Engineered grass-lined major drainageway with low-flow channel with bioengineered components integrated into the design.

Composite Channels—Composite channels have a distinct low-flow channel that is vegetated with a mixture of wetland and riparian species. A monoculture of vegetation should be avoided. In composite channels, dry weather (base) flows are encouraged to meander from one side of the low-flow channel to the other. The low-flow channel banks need heavy-duty biostabilization that includes rock lining to protect against undermining and bank erosion. Photograph MD-6 shows a composite channel.

Concrete-Lined Channels—Concrete-lined channels are high velocity artificial drainageways that are not recommended for use in urban areas. However, in retrofit situations where existing flooding problems need to be solved and where right-of-way is limited, concrete channels may offer advantages over other types of open drainageways. A concrete-lined channel is shown in Photograph MD-8.

Photograph MD-7—Composite channel.

Photograph MD-8—Concrete-lined channel.

Riprap-Lined Channels—Riprap-lined channels offer a compromise between grass-lined channels and concrete-lined channels. Riprap-lined channels can somewhat reduce right-of-way needs relative to grass-lined channels and can handle higher velocities and greater depths than grass-lined channels. Relative to concrete-lined channels, velocities in riprap-lined channels are generally not as high. Riprap-lined channels are more difficult to keep clean and maintain than other types of channels and are recommended for consideration only in retrofit situations where existing urban flooding problems are being addressed. Riprap may also be useful for bank line protection along sections of channels susceptible to erosion such as outer banks of bends. Photograph MD-9 shows a riprap-lined major drainage channel.

Photograph MD-9—Riprap channel. Burying and revegetation of the rock (i.e., soil riprap) could make this site blend into the adjacent terrain very nicely.

Bioengineered Channels—Bioengineered channels utilize vegetative components and other natural materials in combination with structural measures to stabilize existing channels in existing urban areas, area undergoing urbanization and to construct natural-like channels that are stable and resistant to erosion. Bioengineered channels provide channel storage, slower velocities, and various multiple use benefits. Photographs MD-10 and 11 show examples of bioengineered major drainage channels. Wetland bottom channels are an example of one type of bioengineered channel.

Photograph MD-10—Bioengineered major drainage channel using low-grade control structure provides long-term structural integrity and diverse ecology.

Photograph MD-11—Bioengineered major drainageway with dense and diverse vegetation and energy dissipator.

Channels with Manufactured Liners—A variety of artificial channel liners are on the market, intended to protect the channel banks and bottom from erosion at higher velocities. These include gabions, interlocked concrete blocks, concrete revetment mats formed by injecting concrete into double layer fabric forms, and various types of synthetic fiber liners. All of these types are best considered for helping to solve existing urban flooding problems and are not recommended for new developments. Each type of channel lining has to be scrutinized for its merits, applicability, ability to meet other community needs, long term integrity, maintenance needs and maintenance costs.

Boatable Channels—Larger, natural, perennial waterways such as the South Platte River, Clear Creek, and Boulder Creek in the Denver metropolitan area are regularly used for boating and, because of their size and capacity, are subject to more comprehensive hydraulic analyses and considerations. Unless there is evidence of erosion, suitable natural armoring of the channel should not be disturbed; however, boater-friendly drop structures and diversion structures are often necessary. Refer to the discussion on boatable channels in the HYDRAULIC STRUCTURES chapter of this *Manual*.

3.3.2 Factors to Consider in Selection of Channel Type and Alignment

The choice of channel type and alignment must be based upon a variety of multi-disciplinary factors and complex considerations that include, among others:

Hydraulic Considerations

Slope of thalweg

Right-of-way

Capacity needs

Basin sediment yield

Topography

Ability to drain adjacent lands

Structural Considerations

Cost

Availability of material

Areas for wasting fill

Seepage and uplift forces

Shear stresses

Pressures and pressure fluctuations

Momentum transfer

Environmental Considerations

Neighborhood character

Neighborhood aesthetic requirements

Street and traffic patterns

Municipal or county policies

Need for new green areas

Wetland mitigation

Character of existing channel

Wildlife habitat

Water quality enhancement

Sociological Considerations

Neighborhood social patterns

Neighborhood children population

Public safety of proposed facilities for storm and non-storm conditions

Pedestrian traffic

Recreational needs

Right-of-way corridor needs

Maintenance Considerations

Life expectancy

Repair and reconstruction needs

Maintainability

Proven performance

Accessibility

Regulatory constraints to maintenance

Prior to choosing the channel type, the planner should consult with experts in related fields in order to choose the channel that will create the greatest overall benefits. Whenever practical, the channel should have slow flow characteristics, be wide and shallow, and be natural in its appearance and functioning (Bohan 1970).

3.3.3 Environmental Permitting Issues

Environmental permitting, in particular wetland permitting, must be considered in selection of the type of major drainage channel. To assist with the selection of type of channel or drainageway improvements to be used, a flow chart is presented in Figure MD-4. The flow chart contains a series of questions to be considered in light of the requirements in this *Manual* and the requirements of the CWA, Section 404 (dredge and fill in jurisdictional wetlands and "Waters of the United States").

Following along with the chart, the first step is to determine whether channelization is needed or desired. In many cases, a well-established natural drainageway and its associated floodplain can be preserved and protected from erosion damage. Therefore, before deciding to channelize, assess whether the value of reclaimed lands will justify the cost of channelization and whether a new channel will provide greater community and environmental benefits than the existing drainageway.

If the decision is to neither channelize nor re-channelize an existing drainageway, investigate the stability of the natural drainageway and its banks, design measures to stabilize the longitudinal grade and banks, if needed, in selected areas, and obtain, if necessary, Section 404 permits and other approvals for these improvements. However, it is suggested that the reader review the latest Maintenance Eligibility Guidelines available at the District's Web site before deciding what improvements to natural channels are needed to qualify for the District's maintenance eligibility.

If the decision is to channelize, then determine whether the existing natural drainageway has a perennial flow, evidence of wetland vegetation, or is a well-established ephemeral channel. This will often require the assistance of a biologist with wetland training. If any of these conditions exist, then the project is likely to be subject to individual or nationwide Section 404 permitting requirements. Regardless, it is suggest that the designer check with the local USACE office early to determine which permit will be needed. Keep in mind that it is the responsibility of the proponent to comply with all applicable federal and state laws and regulations. Approvals by the local authorities do not supercede or waive compliance with these federal laws.

3.3.4 Maintenance

All major drainage channels in urban areas will require maintenance to ensure that they are capable of conveying their design flow, such as the 100-year flow (as well as more frequently occurring flows) and to ensure that channels do not become a public nuisance and eyesore. Routine maintenance (i.e., mowing for weed control or annual or seasonal clean-outs), unscheduled maintenance (i.e., inspection and clean-out after large events) and restorative maintenance after some years of operation should be expected.

Native tall grasses may require mowing three to six times a year or on a less frequent schedule, depending on the type of channel and setting. Mowing cuts down the presence of "standing dead" grasses and place them on the ground where decomposition can take place. Often mowing of dry-land native grasses during the growing season may not be necessary, except for weed control.

A maintenance access platform with a minimum passage width of 12 feet shall be provided along the entire length of all major drainageways except at drop structures, where a 20 foot maintenance platform is needed. The local government may require the road to be surfaced with 6-inches of Class 2 road base or a 5-inch-thick concrete slab.

Channels may be eligible for District maintenance assistance if they are designed and constructed in accordance with the criteria in this *Manual*, are under some form of public ownership, and meet the District Maintenance Eligibility Guidelines that are stated in Section 3.2.8 of this chapter (see District Web site for periodic updates).

3.4 Design Flows

The major drainage system, including residual floodplain, must be able to convey the flow from a fully urbanized watershed for the event with a 100-year recurrence interval without significant damage to the system. Methods for calculating this flow are described in the RAINFALL and RUNOFF chapters of this *Manual*. In addition to consideration of the 100-year event, the designer must also consider events of lesser magnitudes. For the low-flow channel, ⅓ to ½ of the 2-year flow for fully developed conditions, assuming no upstream detention, is recommended for design. Base flow must also be assessed, especially for grassed channels, channels with wetland bottoms, and bioengineered channels. Base flows are best estimated by examining already-urbanized watersheds that are similar to the planned urban area in terms of imperviousness, land use, and hydrology.

3.5 Choice of Channel Lining

Where the project requires a waterway for storm runoff to be lined because of either hydraulic, topographic, or right-of-way needs, there are a number of choices for linings including grass and other types of vegetation (see the REVEGETATION chapter), other natural materials, riprap, concrete, and manufactured lining materials. The major criterion for choosing a lining is that the lining selected must be designed to withstand the various forces and actions that tend to overtop the bank, damage the lining, and erode unlined areas.

Natural-like channel linings are encouraged; however, in some situations where right-of-way is limited within the constraints of an already-urbanized area, hard-lined channels (i.e., riprap or concrete) may be necessary to assure a stable drainageway. Hard-lined channels are most applicable in solving existing urban flooding problems and are not recommended for new developments.

Natural-like channel linings need to have gentle to mild slopes and are especially desirable for residential areas and areas with public access.

Manufactured channel linings such as gabions, interlocked concrete blocks, synthetic linings, etc., are not recommended for new developments. As with concrete- and riprap-lined channels, all of these types are best considered for helping to solve existing urban flooding problems where right-of-way is very limited. Manufactured channel linings should be used with caution, and each type of channel lining must be scrutinized for its merits, applicability, ability to meet other community needs, long term integrity, and maintenance needs and costs.

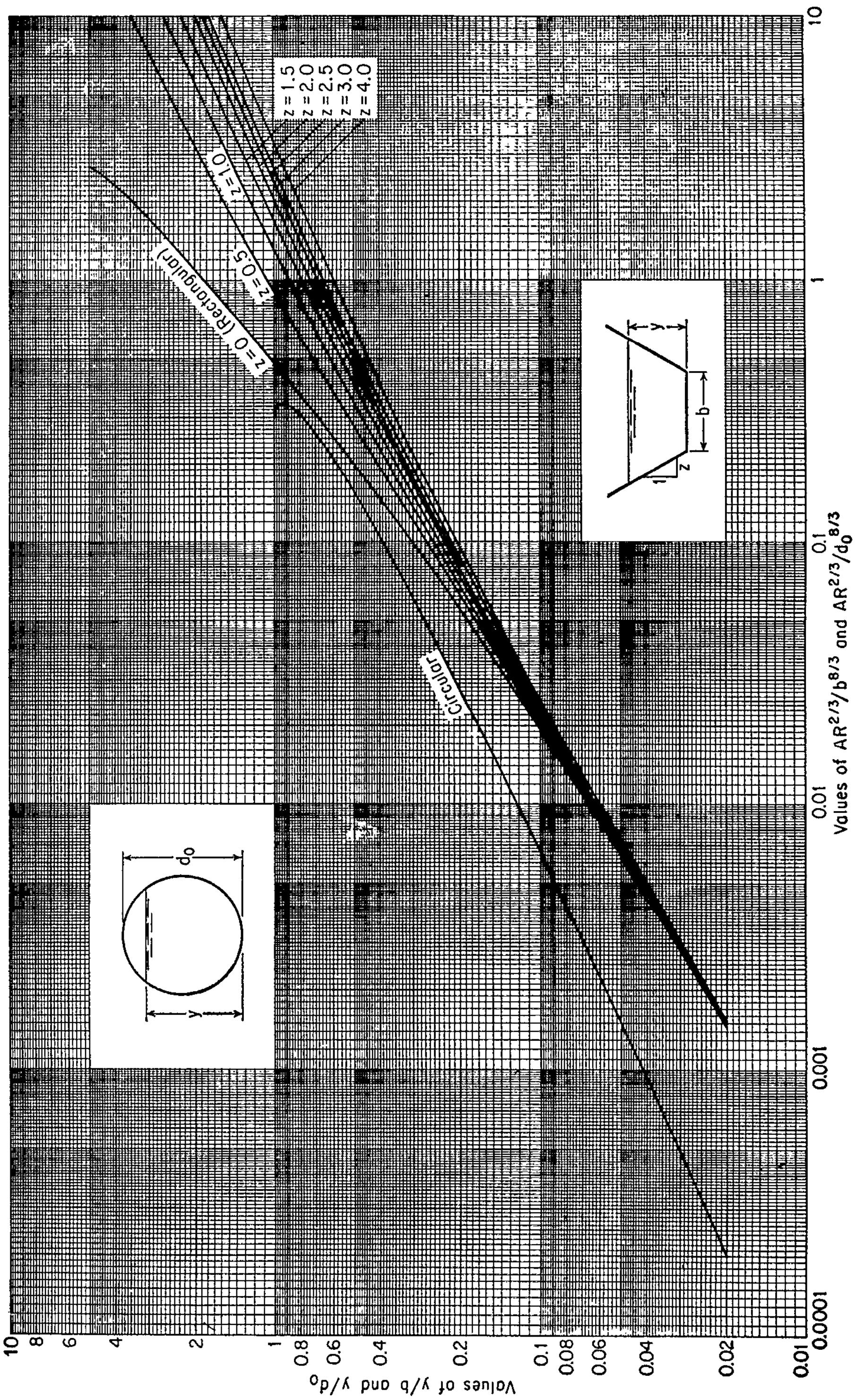


Figure MD-2—Normal Depth for Uniform Flow in Open Channels

(Fletcher and Grace 1972)

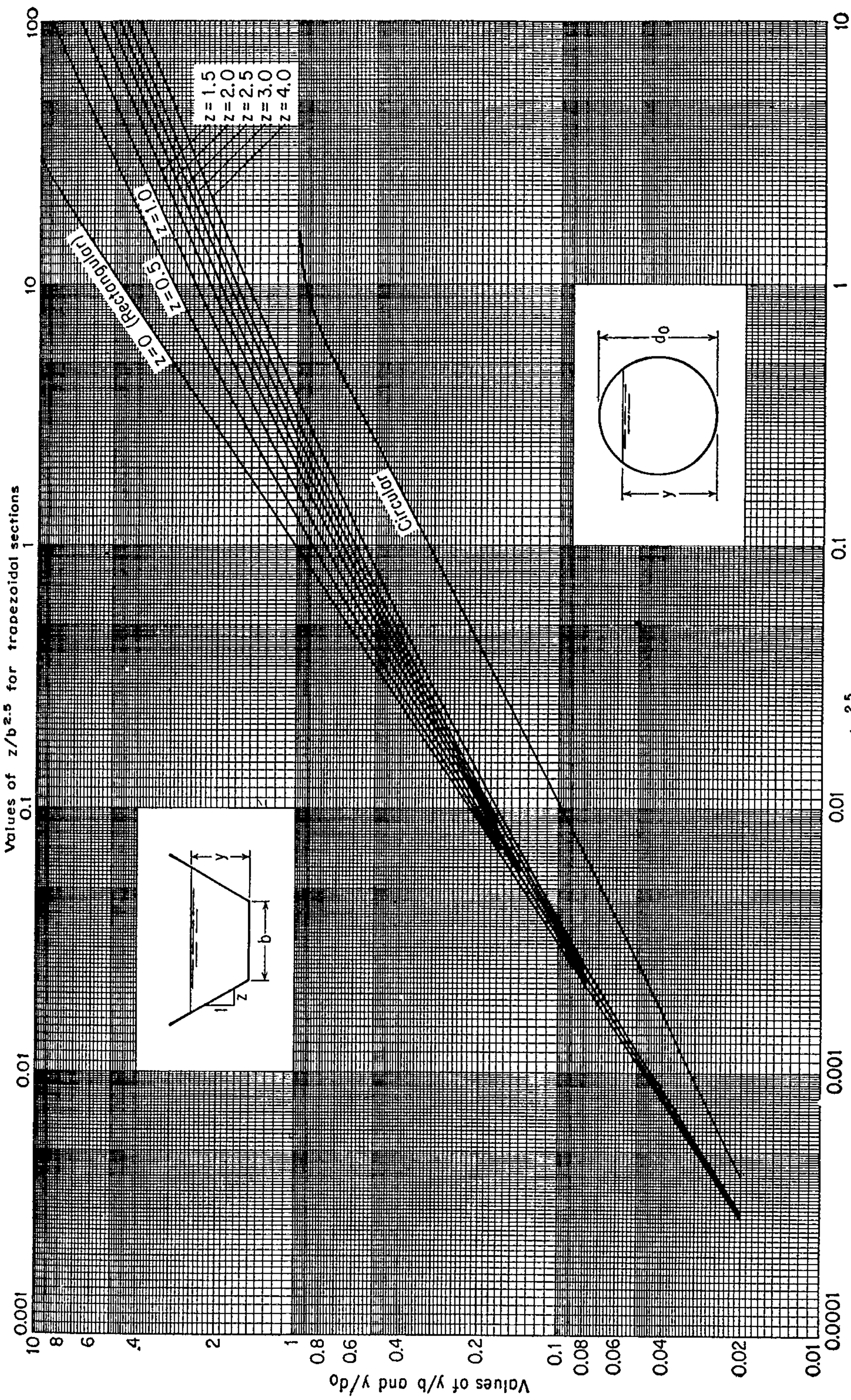


Figure MD-3—Curves for Determining the Critical Depth in Open Channels

(Fletcher and Grace 1972)

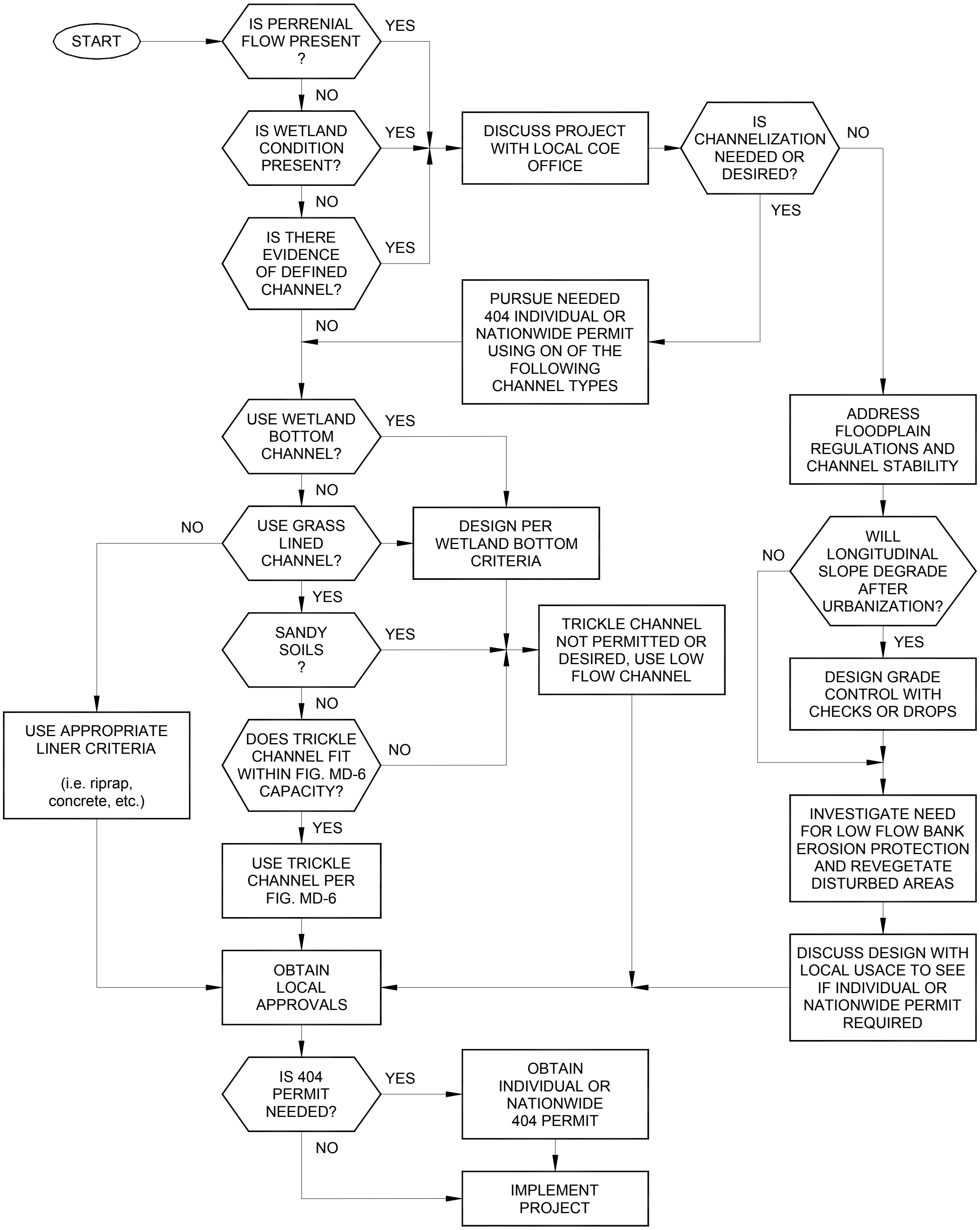


Figure MD-4—Flow Chart for Selecting Channel Type and Assessing Need for 404 Permit

4.0 OPEN-CHANNEL DESIGN CRITERIA

The purpose of this section is to provide design criteria for open channels, including grass-lined channels, composite channels, concrete-lined channels, riprap-lined channels, bioengineered channels, and natural channels. Open-channel hydraulic principles summarized in Section 3.0 can be applied using these design criteria to determine channel geometry and hydraulics.

4.1 Grass-Lined Channels

Grass-lined channels may be considered the most desirable type of artificial channels for new development where natural channels are absent or have limited environmental value. Channel storage, lower velocities, and wildlife, aesthetic, and recreational benefits create significant advantages over other types of channels. The design must fully consider aesthetics, sediment deposition, scouring, hydraulics, safety, and maintenance. Photograph MD-6 shows a grass-lined channel.

4.1.1 Design Criteria

These design criteria are particularly useful in preliminary design and layout work. Any final design that has parameters that vary significantly from those described below should be carefully reviewed for adequacy. Figures MD-5, MD-7, and MD-8 provide representative sketches for grass-lined channels.

4.1.1.1 Design Velocity and Froude number

In determining flow velocity during the major design storm (i.e., 100-year) the designer must recognize the scour potential of the soil-vegetative cover complex. Average velocities need to be determined using backwater calculations, which account for water draw-down at drops, expansions, contractions, and other structural controls. Velocities must be kept sufficiently low to prevent excessive erosion in the channel. The recommended maximum normal depth velocities and Froude numbers for 100-year flows are listed in Table MD-2.

4.1.1.2 Design Depths

The maximum design depths of flow should recognize the scour potential of the soil-vegetative cover complex. The scouring power of water increases in proportion to a third to a fifth power of depth of flow and is a function of the length of time flow is occurring. As preliminary criteria, the design depth of flow for the major storm runoff flow should not exceed 5.0 feet in areas of the channel cross section outside the low-flow or trickle channel area.

4.1.1.3 Design Slopes

To function without instability, grass-lined channels normally have longitudinal slopes ranging from 0.2 to 0.6%. Where the natural slope is steeper than desirable, drop structures should be utilized.

With respect to side slopes, the flatter the side slope, the more stable it is. For grassed channels, side slopes should not be steeper than 4H:1V. Under special conditions where development exists and right-of-way is a problem, the slopes may be as steep as 3H:1V; however, the designer is cautioned that operation of mowing equipment may not be safe on side slopes that are steeper than 4H:1V.

4.1.1.4 Curvature

The more gentle the curve, the better the channel will function. At a minimum, centerline curves shall have a radius that is greater than two times the top width (i.e., $2 \cdot T$) of the 100-year design flow (or other major flow) in the channel.

4.1.1.5 Design Discharge Freeboard

Bridge deck bottoms and sanitary sewers often control the freeboard along the channel in urban areas. Where such constraints do not control the freeboard, the allowance for freeboard should be determined by the conditions adjacent to the channel. For instance, localized overflow in certain areas may be acceptable and may provide flow storage benefits. In general, a minimum freeboard of 1 to 2 feet should be allowed between the water surface and top of bank. Along major streams such as the South Platte River, Clear Creek, Boulder Creek, and others where potential for much timber and other debris exists during a flood, a 3-foot freeboard is recommended.

For curves in the channel, superelevation should be evaluated using Equation MD-9 in Section 3.2.4 and should be included in addition to freeboard.

4.1.2 Grass and Vegetation Selection and Use

Please refer to the REVEGETATION chapter.

4.1.3 Channel Cross Sections

The channel shape may be almost any type suitable to the location and environmental conditions. Often the shape can be chosen to suit open space and recreational needs, to create wildlife habitat, and/or to create additional sociological benefits (Murphy 1971). Typical cross sections suitable for grass-lined channels are shown in Figure MD-5.

4.1.3.1 Side Slopes

The flatter the side slopes, the better. Side slopes should not be steeper than specified in Section 4.1.1.3 of this chapter.

4.1.3.2 Depth

The maximum depth should not exceed the guidelines in Section 4.1.1.2 of this chapter. For known channel geometry and discharge, normal water depth can be calculated using Manning's Equation from Section 3.1.1 of this chapter.

4.1.3.3 Bottom Width

The bottom width should be designed to satisfy the hydraulic capacity of the cross section recognizing the limitations on velocity, depth, and Froude number. For a given discharge, the bottom width can be calculated using the depth, velocity, and Froude number constraints in Sections 4.1.1.1 and 4.1.1.2 using Manning's Equation from Section 3.1.1 of this chapter.

4.1.3.4 Trickle and Low-Flow Channels

When base flow is present or is anticipated as the drainage area develops, a trickle or low-flow channel is required. Steady base flow will affect the growth of grass in the bottom of the channel, create maintenance needs, and can cause erosion. A trickle channel with a porous bottom (i.e., unlined or riprapped) or a low-flow channel is required for all urban grass-lined channels. In some cases, a traditional concrete trickle channel may be necessary, but should be limited to headland tributary channels created in areas where no natural channel previously existed. However, low-flow/trickle channels with natural-like linings are preferable, especially for larger major drainageways, streams and rivers, or for channels located on sandy soils. Criteria for low-flow/trickle channels are presented in Section 4.1.5 of this chapter.

4.1.3.5 Outfalls Into Channel

Outfalls into grass-lined, major channels should be at least 1 foot (preferably 2 feet) above the channel invert with adequate erosion protection provided.

4.1.4 Roughness Coefficients

The hydraulic roughness of man-made grass-lined channels depends on the length of cutting (if any), the type of grass, and the depth of flow (Steven, Simons, and Lewis 1971). Table MD-1 summarizes typical roughness coefficients for grass-lined channels, and Table MD-2 provides guidance for the coefficients for simple trapezoidal channels.

4.1.5 Trickle and Low-Flow Channels

The low flows, and sometimes base flows, from urban areas must be given specific attention. Waterways which are normally dry prior to urbanization will often have a continuous base flow after urbanization because of lawn irrigation return flow and other sources, both overland and from groundwater inflow. Continuous flow over grass or what used to be ephemeral waterways will cause the channel profile to degrade, its cross-section to widen, its meanders to increase, destroy a healthy grass stand and may create boggy nuisance conditions.

These new perennial flows in previously ephemeral waterways change the composition of vegetation. However, it may be possible to plant species adapted to the new hydrologic regime. More mesic species could be planted as flows increase to establish a better-adapted native vegetation type. In some cases, namely in man-made channels, a concrete-lined trickle channel may guard against erosion; however, low-flow/trickle channels with natural-like linings are more attractive visually. Low-flow channels shall be used for larger major drainageways, streams, and rivers and for channels located on sandy soils. Trickle channels with natural-like linings offer and advantage over concrete-lined trickle channels because they more closely mimic natural channels, have greater aesthetic appeal, and provide habitat benefits and vegetative diversity. These linings are best when porous and allow exchange of water with adjacent groundwater table and sub-irrigate vegetation along the channel. In addition, a vegetated low-flow channel provides a degree of water quality treatment, unlike concrete lined channels that tend to flush pollutants accumulated on the impervious lining downstream during runoff events. Low-flow channels with natural-like linings must be carefully designed to guard against erosion.

Low flows must be carried in a trickle channel, a low-flow channel, or an underdrain pipe. The capacity of a trickle channel should be approximately 2.0% of the major (i.e., 100-year) design flow for the fully developed condition assuming no upstream detention. If an underdrain pipe is used, it should be at least 24 inches in diameter, have access manholes at least every 200 feet, and have a slope so that a velocity of at least 3 ft/sec is maintained at ½ full pipe depth. Underdrains are subject to sediment deposition and are very expensive to maintain. As a result, the District does not recommend, nor will consider them for maintenance eligibility.

Figure MD-6 should be used to estimate the required capacity of a trickle flow channel based on the percent of impervious area, I_a. For flows exceeding the limits in Figure MD-6 or where a natural gulch or stream exists, a separate low-flow channel having stabilized banks should be used. A low-flow channel should have a minimum capacity of ⅓ to ½ of the 2-year peak flow under the fully developed watershed conditions. To the extent practicable, a low-flow channel should be gently sloped and shallow to promote flow through the channel's vegetation. See Figure MD-7 and MD-8 for typical details of grass-lined channels with trickle and low-flow channels.

Using a soil-riprap mix for the low-flow channel lining can provide a stable, vegetated low-flow channel for grass-lined wetland bottom and bioengineered channels. Soil and riprap should be mixed prior to placement for these low-flow channels. Soil-riprap low-flow channels should have a cross slope of 1% to 2% (they may be "dished out"). It's longitudinal slope should be consistent with the channel type used.

4.1.6 Erosion Control

Grassed channels are erodible to some degree. Experience has shown that it is uneconomical to design a grassed channel that is completely protected from erosion during a major storm. It is far better to provide reasonably erosion-resistant design with the recognition that additional erosion-control measures and corrective steps will be needed after a major runoff event. The use of drops and checks at regular intervals in a grassed channel is almost always needed to safeguard the channel from serious degradation and erosion by limiting velocities in the channel and dissipating excess energy at these structures. Take advantage of other infrastructure crossing the channel, such as a concrete-encased sewer crossing the channel that can be designed to also serve the function of a grade control structure or a drop structure. Erosion tends to occur at the edges and immediately upstream and downstream of a drop. Proper shaping of the crest and the use of riprap at all drops is necessary. Grade control structures will also protect healthy and mature native vegetation (i.e., trees, shrubs, grasses, wetlands) and reduce long-term maintenance needs.

Under bridges, grass will not grow; therefore, the erosion tendency is larger. A cut-off wall at the downstream edge of a bridge is a good practice.

4.1.6.1 Erosion at Bends

Often special erosion control measures are often needed at bends, (see Section 4.1.1.4). An estimate of protection and velocity along the outside of the bend needs to be made using the following guidelines:
When $r_c/T \geq 8.0$, no riprap protection is needed for the bank on the outside of the bend for channels meeting the velocity and depth criteria specified in this *Manual* for grass-lined channels.
When $r_c/T < 8.0$, protect the bank on the outside of the bend with riprap sized per Section 4.4.2.3 using an adjusted channel velocity determined using Equation MD-10.

$$V_a = (-0.147\,\frac{r_c}{T} + 2.176)\,V \qquad \text{(MD-10)}$$

in which:

V_a = adjusted channel velocity for riprap sizing along the outside of channel bends

V = mean channel velocity for the peak flow of the major design flood

r_c = channel centerline radius

T = Top width of water during the major design flood

Riprap should be applied to the outside ¼ of the channel bottom and to the channel side slope for the entire length of the bend plus a distance of $2 \cdot T$ downstream of the bend. As an alternative to lining the channel bottom, extend the riprap liner at the channel side slope to 5-feet below the channel's bottom.

Construction of channels, should be accomplished in a manner that retards erosion of bare soil areas. Downstream streams, channels, culverts and storm sewers experience severe silting problems if erosion is not controlled during construction by use of contour furrows and aggressive mulching during and after construction. In addition, to control erosion from construction site runoff all concentrated flows have be intercepted and conveyed across or around the construction site in a pipe or a lined open channel. Consult Volume 3 of this *Manual* for detailed guidance on erosion control.

4.1.6.2 Riprap Lining of Grass-lined Channels

For long-term maintenance needs, it is recommended that riprap channel linings be used only in the low-flow channel portion of a composite channel, but not on the banks above the low-flow channel section, nor on the banks of other grass-lined channels, with the exception of use of riprap at bends as discussed above. For this reason whenever soil-riprap linings are used above the low-flow section, a side-slope typically used for grass-line channels is recommended (i.e., 4H:1V), with certain exceptions in retrofit situations in older urbanized areas with limited right of way, where a maximum steepness of 3H:1V may be used.

4.1.7 Water Surface Profile

Water surface profiles should be computed for all channels, typically for the 10-year and 100-year events. Computation of the water surface profile should include standard backwater methods, taking into consideration all losses due to changes in velocity, drops, bridge openings, and other obstructions. Computations should begin at a known point and extend in an upstream direction for subcritical flow. It is for this reason that the channel should be designed from a downstream direction to an upstream direction. It is necessary to show the hydraulic and energy grade lines on all preliminary drawings to help ensure against errors. Whether or not the energy grade line is shown on the final drawings is an option of the reviewing agency, although the District encourages this.

The designer must remember that open-channel flow in urban settings is usually non-uniform because of bridge openings, curves, and structures. This necessitates the use of backwater computations for all final channel design work.

4.1.8 Maintenance

Grass-lined channels must be designed with maintainability in mind. See Section 3.2.8 for the District's Maintenance Eligibility Guidelines, which also provide guidance for elements of design that permit good maintenance of these installations. A stable maintenance access road with a minimum passage width of 12 feet shall be provided along the entire length of all major drainageways. The local government may require the road to have an all weather surface such as a 5-inch-thick concrete pavement.

4.1.9 Calculation Tool

Calculations for sizing of a grass-lined channel using hydraulic equations from Section 3.0 and criteria from Section 4.1 can be performed using the **Grass Ch Worksheet** of the UD-Channels Spreadsheet. The **Composite Design Worksheet** of the UD-Channels Spreadsheet can be used for the design of a grass-lined channel with a low-flow channel. An example of this tool is provided in Example MD-2, which is located at the end of this chapter.

4.1.10 Design Submittal Checklist

Table MD-3 provides a design submittal checklist for a grass-lined channel.

Table MD-3—Design Submittal Checklist for Grass-Lined Channel

Criterion Requirements	✓
Maximum velocity for 100-year event: ≤ 5.0 ft/sec for erosive soils ≤ 7.0 ft/sec for erosion-resistant soils	
Manning's *n* ≥ 0.035 used to check capacity Froude Number	
Manning's *n* ≤ 0.030 used to check velocity and maximum Froude Number	
Froude number: < 0.5 for erosive soils and < 0.8 for non-erosive soils	
Maximum depth for 100-year event ≤ 5.0 ft outside of trickle channel	
Longitudinal channel slope ≥0.2% and ≤ 0.6%	
Side slopes no steeper than 4H:1V	
Channel bottom cross-slope 1% to 2%	
Centerline curve radius > 2 x top width for 100-year event	
Channel bends checked for needed erosion protection (see Section 4.1.6.1 Erosion Control" of the Major Drainage Chapter).	
Channel bend protection, use Type V or VL soil riprap lining extended below channel bottom, buried and vegetated when called for at bends (see Section 4.1.6.1).	
Outfalls into channel ≥ 1 foot above channel invert (use pipes, concrete-lined rundowns or grouted boulder rundowns)	
Adequate freeboard provided, including superelevation	
Grass species appropriate (drought resistant, sturdy, easily established, turf forming)	
Trickle channel (if any) sized for 2.0% of 100-year design flow for fully developed, undetained condition in u/s watershed.	
Underdrain pipe (if any) diameter ≥ 24 inches [**<u>Note</u>: not recommended or endorsed**.]	
Underdrain pipe (if any) includes manhole access every 200 ft	
Underdrain pipe (if any) velocity ≥ 3.0 ft/sec when one-half full	
Erosion protection measures included where necessary	
District Maintenance Eligibility Guidelines satisfied	
Continuous maintenance access road provided (minimum 8-foot stable surface with 12-foot clear width, 20-feet at drop structures)	
Energy and hydraulic grade lines calculated, plotted, design discharges annotated	

4.2 Composite Channels

When the trickle flow channel capacity limits are exceeded as discussed earlier, the use of a composite channel is required, namely a channel with a stabilized low-flow section and an overflow section above it to carry major flow. It is best to assume that wetland and other flow-retarding vegetation will develop in the los-flow section over time. A fact that needs to be accounted for when designing a composite channel. Under certain circumstances, such as when existing wetland areas are affected or natural channels are modified, the USACE's Section 404 permitting process may mandate the use of composite channels that will have wetland vegetation in their bottoms (see Photograph MD-7 for representative example). In other cases, a composite channel with a wetland bottom low-flow channel may better suit individual site needs if used to mitigate wetland damages elsewhere or if used to enhance urban stormwater runoff quality. Composite channels can be closely related to bioengineered and natural channels. Composite channel can provide aesthetic benefits, habitat for aquatic, terrestrial and avian wildlife and water quality enhancement as base flows come in contact with vegetation.

Wetland bottom vegetation within a composite channel will trap sediment and, thereby, reduce the low-flow channel's flood carrying capacity over time. To compensate for this the channel roughness factor used for design must be higher than for a grass-lined channel. As a result, more right-of-way is required for composite channels that have the potential for developing wetlands in their bottom. In developed areas, where right-of-way is limited, mitigating flood damages should take precedence over other considerations during project design. In cases where existing wetlands are eliminated or otherwise impacted, off-site wetland mitigation may be required by the USACE's 404 Permit.

4.2.1 Design Criteria

The simplified design procedures in this *Manual* are based on assumptions that the flow depth is affected by the maturity of vegetation in the low-flow channel, affects the channel roughness, and the rate of sediment deposition on the bottom. These assumptions are based on state-of-the-art literature, observed sediment loads in stormwater (USEPA 1983, DRCOG 1983) and locally observed sediment buildup (District 1996) in several existing wetland bottom and composite channels in the Denver area.

The recommended criteria parallel the criteria for the design of grass-lined channels (Section 4.1), with several notable differences. Composite channels are, in essence, grass-lined channels in which more dense vegetation (including wetland-type) is encouraged to grow on the bottom and sides of the low-flow channel. From a design perspective, these types of channels are differentiated from smaller grass-lined channels by (1) the absence of an impermeable trickle channel, (2) gentler longitudinal slopes and wider bottom widths that encourage shallow, slow flows, (3) greater presence of hydrophytic vegetation along the channel's bottom and lower banks, and (4) non-applicability of the 1% to 2% cross-slope criterion. Another major difference is that a wetland bottom channel should be designed as a low-flow channel having a capacity to carry the 2-year flood peak, instead of the ⅓ to ½ of the 2-year peak required for low-flow channels. Figures MD-8 illustrates a representative wetland bottom composite channel. The use on an appropriate Manning's *n* in its design is critical and guidance for one can be found in Figure MD-9. More detailed design guidance for wetland bottom channels may be found in Volume 3 of this *Manual*.

In designing low-flow channels, the engineer must account for two flow roughness conditions. To ensure vertical stability, the longitudinal slope of the channel should be first calculated and fixed assuming there is no wetland vegetation on the bottom (i.e., "new channel"). Next, in order to ensure adequate flow capacity after the low-flow channel vegetation matures and some sedimentation occurs, the channel's bottom is widened to find the channel cross section needed to carry the design flow using roughness coefficients under the "mature channel" condition. To allow for the "mature channel" condition and potential sediment accumulation, outfalls into channels with low-flow channels should be at least 2 feet above the low-flow channel invert. Guidance for the design of a wetland bottom channel for water quality purposes is given in the STRUCTURAL BMPs chapter in Volume 3 of this *Manual*. A typical cross-sections for composite channels is shown in Figure MD-7.

4.2.2 Design Procedure

If a wetland bottom channel is to be used, the designer may utilize the *CWC Worksheet* from the *Design Forms Spreadsheet* provided for Volume 3 of this *Manual* to assist in these calculations. Otherwise use the *Open Channel Design* workbook. Both may be downloaded from the www.udfcd.org web site.

After the low-flow channel has been designed, complete the design by providing additional channel capacity for the major flows in accordance with the grass-lined channel design requirement. The final Manning's *n* for the composite channel shall be determined using Equation MD-11.

$$n_c = \frac{P \cdot R^{5/3}}{\dfrac{P_L \cdot R_L^{5/3}}{n_L} + \dfrac{P_M \cdot R_M^{5/3}}{n_M} + \dfrac{P_R \cdot R_R^{5/3}}{n_R}} \qquad \text{(MD-11)}$$

In which:

n_c = Manning's n for the composite channel

n_L = Manning's n for the left overbank

n_R = Manning's n for the right overbank

n_M = Manning's n for the middle area (low-flow)

P_L = Wetted perimeter of the left overbank

P_R = Wetted perimeter of the right overbank

P_M = Wetted perimeter of the middle area

R_L = Hydraulic radius of the left overbank

R_R = Hydraulic radius of the right overbank

R_M = Hydraulic radius of the middle area

Figure MD-9 is provided to assist the designer in determining Manning's n for the low-flow section of a composite channel when the design water depth is known.

Whenever a composite bottom channel is crossed by a road, railroad, or a trail requiring a culvert or a bridge, a drop structure should be provided immediately downstream of such a crossing. This will help reduce sediment deposition in the crossing. A 1-foot to 2-foot drop is recommended (a larger drop may be preferred in larger systems) on the downstream side of each culvert and crossing of a wetland bottom channel (see Figure MD-10).

Water surface profiles must be computed, typically for the 10- and 100-year events. Computation of the water surface profile should utilize standard backwater methods, taking into consideration all losses due to changes in velocity, drops, bridge openings, and other obstructions. Computations begin at a known point and extend in an upstream direction for subcritical flow. It is for this reason that the channel should be designed from a downstream direction to an upstream direction. It is necessary to show the energy gradient on all preliminary drawings to help prevent errors. Whether or not the energy gradient line is shown on the final drawings is the option of the reviewing agency but is encouraged by the District.

The designer must remember that open-channel flow in urban drainage is usually non-uniform because of bridge openings, curves, and structures. This necessitates the use of backwater computations for all final channel design work.

Guidance regarding vegetation selection, planting, and maintenance is provided in the REVEGETATION chapter.

4.2.3 Life Expectancy and Maintenance

The low-flow channel can serve as a productive ecosystem and can also be highly effective at trapping sediment. Wetland vegetation bottom channels are expected to fill with sediment over time. Some sediment accumulation is necessary for a wetland channel's success to provide organic matter and nutrients for growth of biological communities. The life expectancy of such a channel will depend primarily on the land use of the tributary watershed. However, life expectancy can be dramatically reduced to as little as 2 to 5 years, if land erosion in the tributary watershed is not controlled. Therefore, land erosion control practices need to be strictly enforced during land development and other construction within the watershed, and all facilities should be built to minimize soil erosion to maintain a reasonable economic life for the wetland bottom channel. In addition, sediment traps or forebays located at stormwater runoff points of entry can trap a significant portion of the sediment arising at the wetland channel and, if used, could decrease the frequency of major channel dredging.

A maintenance access road with a minimum passage width of 12 feet shall be provided along the entire length of all major drainageways. The local government may require the road to be surfaced with 6 inches of Class 2 roadbase or a 5-inch-thick concrete slab.

4.2.4 Calculation Example for Wetland Bottom Channel

See Volume 3 of this *Manual* for a design example.

4.2.5 Design Submittal Checklist

Table MD-4, below, provides a design checklist for a composite channel.

Table MD-4—Design Submittal Checklist for Composite Channel

Criterion/Requirement	✓
Maximum velocity in main channel outside of the low-flow or wetland low-flow section for the 100-year event: ≤ 5.0 ft/sec for erosive soils ≤ 7.0 ft/sec for non-erosive soils	
"New channel" roughness condition used to set longitudinal slope	
"Mature channel" roughness condition used to evaluate capacity	
Composite Manning's n calculated for channel and used in hydraulic computations	
Froude number: < 0.5 for erosive soils; < 0.8 for non-erosive soils	
Maximum depth for 100-year event ≤ 5.0 ft outside of low-flow channel	
Side slopes in low-flow section, no steeper than 2.5H:1V for soil riprap lined (i.e., rock mixed with topsoil, covered with topsoil and revegetated)	
Side slopes above low-flow channel: no steeper than 4H:1V	
Centerline curve radius: > 2 x top width for 100-year event	
Channel bends: check for need for erosion protection in accordance with recommendation of section "4.1.6.1 Erosion at Bends" of the *USDCM*	
Channel bend protection, use Type V or VL soil riprap lining extended below channel or low-flow channel bottom, buried and vegetated if called for at bends (see Section 4.1.6.1).	
Outfalls into channel: ≥ 1 foot above channel invert (use pipes, concrete-lined rundowns or grouted boulder rundowns)	
Adequate freeboard provided, including superelevation	
Vegetation Species appropriate for anticipated hydroperiod, water levels, zonation on banks (see the REVEGETATION chapter)	
No impermeable lining present	
Drop downstream of each culvert or bridge crossing: 1-foot to 2-foot for wetland bottom channels	
Low-flow channel size: ⅓ to ½ of the 2-year flow for the fully developed watershed flows	
Low-flow channel depth: ≥ 3.0 ft and ≤ 5.0 ft	
Erosion protection measures included where necessary (at crossing, drops, bend, etc.)	
District Maintenance Eligibility Guidelines satisfied	
Continuous maintenance access road provided (minimum 8-foot stable surface with 12-foot clear width, 20-foot at drop structures)	
Energy and hydraulic grade lines calculated and plotted (min. 2- and 100-year flows)	

4.3 Concrete-Lined Channels

Although not recommended for general use because of safety and structural integrity and aesthetic reasons; hydraulic, topographic, or right-of-way constraints may necessitate the use of a concrete-lined channel in some instances. A common constraint requiring a concrete-lined channel is the need to convey high velocity, sometimes supercritical, flow. Whether the flow will be supercritical or subcritical, the concrete lining must be designed to withstand the various forces and actions that cause overtopping of the bank, damage to the lining, and erosion of unlined areas. Concrete-lined channels will typically not be eligible for District's maintenance eligibility.

Concrete-lined channels can be used for conveyance of both subcritical and supercritical flows. In general, however, other types of channels such as grass-lined channels or channels with wetland bottoms are preferred for subcritical flows. The use of a concrete-lined channel for subcritical flows should not be used except in unusual circumstances where a narrow right-of-way exists. Vegetated channels are normally preferable in the Denver region because available thalweg slopes are generally steep enough.

Channels conveying supercritical flows must be carefully designed due to many potential hazards. Imperfections at joints can cause their rapid deterioration, in which case a complete failure of the channel can occur. In addition, high-velocity flow at cracks or joints creates an uplift force by creating zones of flow separation with negative pressures and conversion of the velocity head to pressure head under the liner, which can virtually tear out concrete slabs. When designing a lined channel with supercritical flow, the designer must use utmost care and consider all relevant factors.

In the Denver region, all channels carrying supercritical flow shall be lined with continuously reinforced concrete linings, both longitudinally and laterally. There shall be no diminution of wetted area cross sections at bridges or culverts. Adequate freeboard shall be provided to have a suitable safety margin. Bridges or other structures crossing the channel must be anchored satisfactorily to withstand the full dynamic load that might be imposed upon the structure in the event of major trash plugging.

The concrete linings must be protected from hydrostatic uplift forces, which are often created by a high water table or momentary inflow behind the lining from localized flooding. A perforated underdrain pipe is required under the lining, and the underdrain must be designed to be free draining. At supercritical flow, minor downstream obstructions do not create any backwater effect. Backwater computation methods are applicable for computing the water surface profile or the energy gradient in channels having a supercritical flow; however, the computations must proceed in a downstream direction.

Roughness coefficients for lined channels are particularly important when dealing with supercritical flow. Once a particular roughness coefficient is chosen, the construction inspection must be carried out in a manner to ensure that the particular roughness is obtained. Because of field construction limitations, the designer should use a Manning's n roughness coefficient equal to 0.013 for a well-trowelled concrete finish. Other finishes should have proportionately larger n values assigned to them. A value of n higher than 0.013 may be applicable for a concrete channel with subcritical flow if deposition of sediment or transport of sediment as bedload is expected.

Small concrete channels that function as rundowns are addressed in the HYDRAULIC STRUCTURES chapter.

4.3.1 Design Criteria

4.3.1.1 Design Velocity and Froude Number

Concrete channels can be designed to convey supercritical or subcritical flows; however, the designer must take care to prevent the possibility of unanticipated hydraulic jumps forming in the channel. For concrete channels, flows at *Froude Numbers* between 0.7 and 1.4 are unstable and unpredictable and should be avoided at all flow levels in the channel. When a concrete channel is unavoidable, the maximum velocity at the peak design flow shall not exceed 18 feet per second.

To calculate velocities, the designer should utilize Manning's Equation from Section 3.1.1 of this chapter with roughness values from Table MD-5. When designing a concrete-lined channel for subcritical flow, use a Manning's n = 0.013 for capacity calculations and 0.011 to check whether the flow could go supercritical. Do not design a subcritical channel for a Froude number greater than 0.7 using the velocity and depth calculated with a Manning's n = 0.011. Also, do not design supercritical channel with a Froude Number less than 1.4 when checking for it using a Manning's n = 0.013

Table MD-5—Roughness Values for Concrete-Lined Channels

Type of Concrete Finish	Roughness Coefficient (n)		
	Minimum	Typical	Maximum
Concrete			
Trowel finish*	0.011	0.013	0.015
Float finish*	0.013	0.015	0.016
Finished, with gravel on bottom*	0.015	0.017	0.020
Unfinished*	0.014	0.017	0.020
Shotcrete, trowelled, not wavy	0.016	0.018	0.023
Shotcrete, trowelled, wavy	0.018	0.020	0.025
Shotcrete, unfinished	0.020	0.022	0.027
On good excavated rock	0.017	0.020	0.023
On irregular excavated rock	0.022	0.027	0.030

* For a *subcritical* channel with these finishes, check the Froude number using n = 0.011

4.3.1.2 Design Depths

There are no specific limits set for depth for concrete-lined channels, except as required for low-flow channels of a composite section where the low-flow channel is concrete lined.

4.3.1.3 Curvature

Curvature is not allowed for channels with supercritical flow regimes. For concrete-lined channels with subcritical flow regimes, the centerline radius of curvature should be at least two times the top width, and superelevation should be evaluated for all bends using Equation MD-9 in Section 3.2.4 and included in determining freeboard.

4.3.1.4 Design Discharge Freeboard

Freeboard above the design water surface shall not be less than that determined by the following:

$$H_{fb} = 2.0 + 0.025V(y_0)^{1/3} + \Delta y \qquad \text{(MD-12)}$$

in which:

H_{fb} = freeboard height (ft)

V = velocity of flow (ft/sec)

y_o = depth of flow (ft)

Δy = increase in water surface elevation due to superelevation at bends (see Equation MD-9) (no bends allowed in supercritical channels)

In addition to H_{fb}, add height of estimated standing waves, superelevation and/or other water surface disturbances to calculate the total freeboard. In all cases, the freeboard shall be no less than 2 feet and the concrete lining shall be extended above the flow depth to provide the required freeboard.

4.3.2 Concrete Lining Specifications

4.3.2.1 Concrete Lining Section

All concrete lining shall be designed to withstand the anticipated hydrodynamic and hydrostatic forces, and the minimum thickness shall be no less than 7 inches for supercritical channels and no less than 5 inches for subcritical channels. A free draining granular bedding shall be provided under the concrete liner and shall be no less than 6-inches thick for channels with Froude number ≤ 0.7 and 9-inches thick for channels with Froude number ≥ 1.4.

The side slopes shall be no steeper than 1.5V:1H unless designed to act as a structurally reinforced wall to withstand soil and groundwater forces. In some cases, a rectangular cross section may be required. Rectangular cross sections are acceptable, provided they are designed to withstand potential lateral loads. In addition, fencing along concrete channels should be used to restrict access for safety reasons.

4.3.2.2 Concrete Joints

Concrete joints must satisfy the following criteria:

1. Channels shall be constructed of continuously reinforced concrete without transverse joints.

2. Expansion/contraction joints shall be installed where new concrete lining is connected to a rigid structure or to existing concrete lining which is not continuously reinforced.

3. Longitudinal joints, where required, shall be constructed on the sidewalls at least 1 foot vertically

above the channel invert.

4. All joints shall be designed to prevent differential movement.

5. Construction joints are required for all cold joints and where the lining thickness changes. Reinforcement shall be continuous through the joint.

4.3.2.3 Concrete Finish

The surface of the concrete lining may be finished in any of the finishes listed in Table MD-5, provided appropriate finishing technique is used. Check with local authorities to determine which finishes are acceptable.

4.3.2.4 Underdrain

Longitudinal underdrains shall be provided along the channel bottom on 10-foot centers within a free-draining bedding under the channel lining, be free draining, and daylight at check drops (when applicable). A check valve or flap valve shall be provided at the outlet to prevent backflow into the drain. Appropriate numbers of weep holes and one-way valves shall be provided in vertical wall sections of the channel to relieve hydrostatic pressure.

4.3.3 Channel Cross Section

4.3.3.1 Side Slopes

The side slopes shall be no steeper than 1.5H:1V unless designed to act as a structurally reinforced wall to withstand soil and groundwater forces.

4.3.3.2 Depth

Maximum depth shall be consistent with Section 4.3.1.2. For known channel geometry and discharge, normal water depth can be calculated using Manning's Equation recommended in Section 3.1.1.

4.3.3.3 Bottom Width

The bottom width should be designed to satisfy the hydraulic capacity of the cross section recognizing the limitations on velocity, depth, and Froude number. For a given discharge, the bottom width can be calculated from depth, velocity, slope, and Froude number constraints in Sections 4.3.1.1, 4.3.1.2, and 4.3.1.3 using Manning's Equation.

4.3.3.4 Trickle and Low-Flow Channels

For a well-designed concrete-lined channel, a trickle or low-flow channel is not necessary since the entire channel is hard-lined. However, if a small base flow is anticipated, it is a good idea to incorporate a trickle flow swale or section to reduce occurrence of bottom slime, noxious odors and mosquito breeding.

4.3.3.5 Outfalls Into Channel

Outfalls into concrete-lined channels should be at least 1 foot above the channel invert.

4.3.4 Safety Requirements

A 6-foot-high chain-link or comparable fence shall be installed to prevent access wherever the 100-year channel concrete section depth exceeds 3 feet. Appropriate numbers of gates, with top latch, shall be placed and staggered where a fence is required on both sides of the channel to permit good maintenance access.

In addition, ladder-type steps shall be installed not more than 200 feet apart on alternating sides of the channel. A bottom rung shall be placed approximately 12 inches vertically above the channel invert.

4.3.5 Calculation Tools

Calculations for sizing of a concrete-lined channel using hydraulic equations from Section 3.0 and criteria from Section 4.3 can be performed using the *Basis Worksheet* of UD-Channels Spreadsheet.

4.3.6 Maintenance

Concrete channels require periodic maintenance including debris and sediment removal, patching, joint repair, and other such activities. Their condition should be periodically monitored, especially to assure that flows cannot infiltrate beneath the concrete lining. A maintenance access road with a minimum passage width of 12 feet shall be provided along the entire length of all major drainageways. The local government may require the road to have an all weather surface such as 5-inch-thick concrete pavement.

4.3.7 Design Submittal Checklist

Table MD-6 provides a design checklist for a concrete-lined channel.

Table MD-6—Design Submittal Checklist for Concrete-Lined Channel

Criterion/Requirement	✓
Maximum velocity for 100-year event ≤ 18 ft/sec	
Channel capacity and Froude Number ≥ 1.4: checked with Manning's n = 0.013	
Maximum velocity and Froude Number ≤ 0.7: checked using Manning's n = 0.011	
Froude number ≤ 0.7 and ≥ 1.4 under both Manning's n assumptions	
Side slopes no steeper than 1.5H:1V	
Centerline curve radius for subcritical channels: > 2 x top width for 100-year event	
Centerline curve radius for supercritical channels: NO CURVATURES PERMITTED	
Concrete lining designed to withstand hydrodynamic and hydrostatic forces (minimum thickness = 7.0 inches for supercritical channels, 5.0 inches for subcritical channels)	
Concrete joints meet Section 4.3.2.2 criteria	
Free draining granular bedding under concrete (6-inch minimum thickness for Fr ≤ 0.7, 9-inch minimum thickness for Fr ≥ 1.4	
Free draining longitudinal underdrains provided on 10-ft centers, including check or flap valve at outlet to prevent backflow	
Concrete finish from list in Table MD-5	
Outfalls into channel ≥ 1 ft above channel invert	
Adequate freeboard provided (see criteria in text)	
Standing waves included in freeboard for supercritical channels	
6-ft chain link fence (or comparable) provided when channel depth > 3.0 ft	
Ladder-type steps spaced no more than 200 ft apart on alternating sides of channel with lowest rung approximately 12 inches above channel invert	
District Maintenance Eligibility Guidelines satisfied	
Continuous maintenance access road provided (minimum 8-foot stable surface with 12-foot clear width, 20-foot at drops)	
Energy and hydraulic grade lines calculated and plotted for the channel and also annotate the design discharges	

4.4 Riprap-Lined Channels

Channel linings constructed from soil riprap, grouted boulders, or wire-encased rock to control channel erosion may be considered on a case-by-case basis, or may be required as the case may be, for the following situations:

1. Where major flows such as the 100-year flood are found to produce channel velocities in excess of allowable non-eroding values (5 ft/sec for sandy soil conditions and 7 ft/sec in erosion resistant soils) or when main channel depth is greater than 5 feet.

2. Where channel side slopes must be steeper than 3H:1V.

3. For low-flow channels.

4. Where rapid changes in channel geometry occur such as channel bends and transition s.

Design criteria applicable to these situations are presented in this section. Riprap-lined channels should only be used for subcritical flow conditions where the Froude number is 0.8 or less. When used, it is recommended that all riprap outside frequent flow zones have the voids filled with soil, the top of the rock covered with topsoil, and the surface revegetated with native grasses, namely, use soil riprap.

4.4.1 Types of Riprap

4.4.1.1 Ordinary and Soil Riprap

Ordinary riprap, or simply "riprap," refers to a protective blanket of large loose stones, which are usually placed by machine to achieve a desired configuration. The term ordinary riprap has been introduced to differentiate loose stones from grouted boulders and wire-enclosed rock. Photograph MD-9 shows a representative riprap-lined channel, while Figures MD-11 through MD-14 depict key design aspects of such channels.

Many factors govern the size of the rock necessary to resist the forces tending to move the riprap. For the riprap itself, this includes the size and weight of the individual rocks, shape of the stones, gradation of the particles, blanket thickness, type of bedding under the riprap, and slope of the riprap layer. Hydraulic factors affecting riprap include the velocity, current direction, eddy action, waves, and hydraulic uplift forces.

Experience has shown that riprap failures result from a variety of factors: undersized individual rocks in the maximum size range; improper gradation of the rock, which reduces the interlocking of individual particles; and improper bedding for the riprap, which allows leaching of channel particles through the riprap blanket.

Classification and gradation for riprap and boulders are given in Table MD-7, Table MD-8 and Figure MD-11 and are based on a minimum specific gravity of 2.50 for the rock. Because of its relatively small size and weight, riprap types VL, L and M must be mixed with native topsoil, covered with topsoil and revegetated. This practice also protects the rock from vandalism.

The type of riprap that is mixed with native soil as described above is called *soil riprap*. Soil Riprap consist of 35% by volume of native soil, taken from the banks of the channel, that is mixed in with 65% by volume of riprap on-site, before placement as channel liner. Soil riprap is recommended for all urban channels within District regardless of riprap size used. A typical section for soil riprap installation is illustrated in Figure MD-13b.

Table MD-7—Classification and Gradation of Ordinary Riprap

Riprap Designation	% Smaller Than Given Size by Weight	Intermediate Rock Dimensions (inches)	d_{50} (inches)*
Type VL	70-100	12	
	50-70	9	
	35-50	6	6**
	2-10	2	
Type L	70-100	15	
	50-70	12	
	35-50	9	9**
	2-10	3	
Type M	70-100	21	
	50-70	18	
	35-50	12	12**
	2-10	4	
Type H	70-100	30	
	50-70	24	
	35-50	18	18
	2-10	6	
Type VH	70-100	42	
	50-70	33	
	35-50	24	24
	2-10	9	

* d_{50} = mean particle size (intermediate dimension) by weight.

** Mix VL, L and M riprap with 35% topsoil (by volume) and bury it with 4 to 6 inches of topsoil, all vibration compacted, and revegetate.

Basic requirements for riprap stone are as follows:

- Rock shall be hard, durable, angular in shape, and free from cracks, overburden, shale, and organic matter.
- Neither breadth nor thickness of a single stone should be less than one-third its length, and rounded stone should be avoided.
- The rock should sustain a loss of not more than 40% after 500 revolutions in an abrasion test (Los Angeles machine—ASTM C-535-69) and should sustain a loss of not more than 10% after 12 cycles of freezing and thawing (AASHTO test 103 for ledge rock procedure A).
- Rock having a minimum specific gravity of 2.65 is preferred; however, in no case should rock have a specific gravity less than 2.50.

4.4.1.2 Grouted Boulders

Table MD-8 provides the classification and size requirements for boulders. When grouted boulders are used, they provide a relatively impervious channel lining which is less subject to vandalism than ordinary riprap. Grouted boulders require less routine maintenance by reducing silt and trash accumulation and are particularly useful for lining low-flow channels and steep banks. The appearance of grouted boulders is enhanced by exposing the tops of individual stones and by cleaning the projecting rocks with a wet broom right after the grouting operation. In addition, it is recommended that grouted boulders on channel banks and outside of frequent flow areas be buried with topsoil and revegetated with native grasses, with or without shrubs depending on the local setting. Boulders used for grouting should meet all the properties of rock for ordinary riprap, and rock of uniform size should be used. The boulder sizes are categorized in Table MD-8.

Table MD-8—Classification of Boulders

Boulder Classification	Nominal Size and [Range in Smallest Dimension of Individual Rock Boulders (inches)]	Maximum Ratio of Largest to Smallest Rock Dimension of Individual Boulders
B18	18 [17 – 20]	2.5
B24	24 [22 – 26]	2.0
B30	30 [28 – 32]	2.0
B36	36 [34 – 38]	1.75
B42	42 [40 – 44]	1.65
B48	48 [45 – 51]	1.50

Grouted boulders should be placed directly on subbase without granular bedding. The top one-half of the boulders shall be left ungrouted and exposed. Weep holes should be provided at the toe of channel slopes and channel drops to reduce uplift forces on the grouted channel lining. Underdrains should be provided if water is expected to be present beneath the liner. Grouted boulders on the banks should be buried and vegetated with dry-land grasses and shrubs. Cover grouted boulders with slightly compacted topsoil, filling depressions and covering the top of the tallest rocks to a height of no less than 4-inches (6-inches of more preferred) to establish dry-land vegetation. Recommended grass seed mixtures and how to plant and much them are provided in the REVEGETATION chapter of this *Manual*. Shrubs also may be planted, but will not grow well over grouted boulders unless irrigated.

Two types of grout are recommended for filling the voids for the grouted boulders. The technical specifications for two types of structural grout mix are given as a part of Figures HS-7a4 and HS-7b4 of the HYDRAULIC STRUCTURES chapter of this *Manual*. Type A can be injected using a low-pressure grout pump and can be used for the majority of applications. Type B has been designed for use in streams and rivers with significant perennial flows where scouring of Type A grout is a concern. It requires a concrete pump for injection.

Full penetration of grout around the lower one-half of the rock is essential for successful grouted boulder performance. Inject grout in a manner that ensures that no air voids between the grout, subbase, and boulders will exist. To accomplish this, inject the grout by lowering the grouting nozzle to the bottom of the boulder layer and build up the grout from the bottom up, while using a vibrator or aggressive manual rodding. Inject the grout to a depth equal to one-half of the boulders being used and keep the upper one-half ungrouted and clean. Remove all grout splatters off the exposed boulder portion immediately after grout injection using wet brooms and brushes.

4.4.1.3 Wire-Enclosed Rock (Gabions)

Wire-enclosed rock, or gabions, refers to rocks that are bound together in a wire basket so that they act as a single unit. The durability of wire-enclosed rock is generally limited by the life of the galvanized binding wire that has been found to vary considerably under conditions along waterways. Water carrying sand or gravel will reduce the service life of the wire dramatically. Water that rolls or otherwise moves cobbles and large stones breaks the wire with a hammer-and-anvil action, considerably shortening the life of the wire. The wire has been found to be susceptible to corrosion by various chemical agents and is particularly affected by high-sulfate soils. Wire-enclosed rock installations have been found to attract vandalism, and flat mattress surfaces seem to be particularly susceptible to having wires cut and stones removed. For these reasons, the District discourages the use of wire-enclosed rock. If the designer chooses to utilize gabions, they should be placed above the low-flow channel or 2-year water surface elevation. All flat mattresses must be filled with topsoil and then covered with a 6-inch layer of topsoil.

4.4.2 Design Criteria

The following sections present design criteria for riprap-lined channels. Additional information on riprap can be found in Section 7.0 of this chapter.

4.4.2.1 Design Velocity

Riprap-lined channels should only be used for subcritical flow conditions where the Froude number is 0.8 or less.

4.4.2.2 Design Depths

There is no maximum depth criterion for riprap-lined channels. Wire-enclosed rock sections shall be used on banks only above the low-flow channel or 2-year flood water surface, placed on a stable foundation.

4.4.2.3 Riprap Sizing

The stone sizing for ordinary riprap can be related to the channel's longitudinal slope, flow velocity, and the specific gravity of the stone using the relationship:

$$\frac{VS^{0.17}}{d_{50}^{0.5}(G_s-1)^{0.66}} = 4.5 \qquad \text{(MD-13)}$$

in which:

V = mean channel velocity (ft/sec)

S = longitudinal channel slope (ft/ft)

d_{50} = mean rock size (ft)

G_s = specific gravity of stone (minimum = 2.50)

Note that Equation MD-13 is applicable for sizing riprap for channel lining. This equation is not intended for use in sizing riprap for rundowns or culvert outlet protection. Information on rundowns is provided in Section 7.0 of the HYDRAULIC STRUCTURES chapter of this *Manual*, and protection downstream of culverts is discussed in Section 7.0 of this chapter, as well as in the HYDRAULIC STRUCTURES chapter, Section 3.0.

Table MD-10 shall be used to determine the minimum size of rock type required. Note that rock types for ordinary riprap, including gradation, are presented in Table MD-7 and Figure MD-11.

Table MD-10—Riprap Requirements for Channel Linings*

$\frac{VS^{0.17}}{(G_s - 1)^{0.66}}$ **	Rock Type
< 3.3	VL** (d_{50} = 6 inches)
≥ 3.3 to < 4.0	L** (d_{50} = 9 inches)
≥ 4.0 to < 4.6	M (d_{50} = 12 inches)
≥ 4.6 to < 5.6	H (d_{50} = 18 inches)
≥ 5.6 to 6.4	VH (d_{50} = 24 inches)

* Applicable only for a Froude number of < 0.8 and side slopes no steeper than 2H:1V.

** Use G_s = 2.5 unless the source of rock and its density are known at time of design.

Table MD-10 indicates that rock size does not need to be increased for steeper channel side slopes, provided the side slopes are no steeper than 2.5H:1V (District 1982). Rock-lined side slopes steeper than 2.5H:1V are considered unacceptable under any circumstances because of stability, safety, and maintenance considerations. Proper bedding is required both along the side slopes and the channel bottom for a stable lining. The riprap blanket thickness should be at least 1.75 times d_{50} (at least 2.0 times d_{50} in sandy soils) and should extend up the side slopes at least 1 foot above the design water surface. At the upstream and downstream termination of a riprap lining, the thickness should be increased 50% for at least 3 feet to prevent undercutting.

4.4.2.4 Riprap Toes

Where only the channel sides are to be lined and the channel bottom remains unlined, additional riprap is needed to protect such lining. In this case, the riprap blanket should extend at least 3 feet below the channel thalweg (invert) in erosion resistant soils, and the thickness of the blanket below the existing channel bed should be increased to at least 3 times d_{50} to accommodate possible channel scour during higher flows. The designer should compute the scour depth for the 100-year flow and, if this scour depth exceeds 3 feet, the depth of the riprap blanket should be increased accordingly (see Figure MD-12). As an alternative, a thinner layer of riprap (i.e., 1.75 to 2.0 d_{50}) may be used in the toe provided it is extended to 5.0 feet below the channel bottom. For sandy soils, it will be necessary to extend the riprap toe to even greater depths (5-foot minimum) and site-specific scour calculations are recommended.

4.4.2.5 Curves and Bends

The potential for erosion increases along the outside bank of a channel bend due to acceleration of flow velocities on the outside part of the bend. Thus, it is often necessary to provide erosion protection in channels that otherwise would not need protection; riprap is commonly used for this. The need for protection of the bank on the outside of the bend has been discussed in Section 4.1.6 for channel bends that have a radius less than 8 times the top width of the channel cross section.

Whenever an outside bend in a grass-lined channel needs protection, soil riprap should be used, then covered with native topsoil and revegetated to provide a grassed-line channel appearance. Note that buried soil riprap may lose its cover in a major event if vegetation has not fully matured, requiring re-burial and revegetation.

The minimum allowable radius for a riprap-lined bend is 2.0 times the top width of the design flow water surface. The riprap protection should be placed along the outside of the bank and should be extended downstream from the bend a distance of not less than 2.0 times the top width of the channel. The riprap does not need to be extended upstream of the point of curvature (start of the bend).

Where the mean channel velocity exceeds the allowable non-eroding velocity so that riprap protection is required for straight channel sections, increase the rock size using the adjusted flow velocity found using Equation MD-10. Use the adjusted velocity in Table MD-10 to select appropriate riprap size.

4.4.2.6 Transitions

Scour potential is amplified by turbulent eddies near rapid changes in channel geometry such as transitions and bridges. Table MD-10 may be used for selecting riprap protection for subcritical transitions (Froude numbers 0.8 or less) by using the maximum velocity in the transition and then increasing the velocity by 25%.

Protection should extend upstream from the transition entrance at least 5 feet and downstream from the transition exit for a distance equal to at least 5 times the design flow depth.

4.4.2.7 Design Discharge Freeboard

Freeboard above the design water surface shall not be less than that determined by Equation MD-12 in Section 4.3.1.5.

In addition to the freeboard height calculated using Equation MD-12, add the height of estimated standing waves and/or other water surface disturbances and calculate total freeboard. In all cases, the riprap lining shall be extended above the flow depth to provide freeboard.

4.4.3 Roughness Coefficient

The Manning's roughness coefficient, n, for a riprap-lined channel may be estimated for ordinary riprap using:

$$n = 0.0395 d_{50}^{1/6} \tag{MD-14}$$

In which, d_{50} = the mean stone size in feet.

This equation does not apply to grouted boulders or to very shallow flow (where hydraulic radius is less than, or equal to 2.0 times the maximum rock size). In those cases the roughness coefficient will be greater than indicated by Equation MD-14.

4.4.4 Bedding Requirements

The long-term stability of riprap erosion protection is strongly influenced by proper bedding conditions. A large percentage of all riprap failures is directly attributable to bedding failures.

Properly designed bedding provides a buffer of intermediate-sized material between the channel bed and the riprap to prevent channel particles from leaching through the voids in the riprap. Two types of bedding are in common use: (1) a granular bedding filter and (2) filter fabric.

4.4.4.1 Granular Bedding

Two methods for establishing gradation requirements for granular bedding are described in this section. The first method, a single or two-layer bedding that uses Type I and II gradations, is shown in Table MD-11 and is adequate for most ordinary riprap and grouted riprap applications. The second utilizes a design procedure developed by Terzaghi, which is referred to as the T-V (Terzaghi-Vicksburg) design (Posey 1960, USACE 1970). The T-V filter criteria establish an optimum bedding gradation for a specific channel soil. The latter requires channel soil information, including a gradation curve, while the Type I and Type II bedding specifications given in Table MD-11 and Figure MD-13 are applicable whether or not soil information is available.

Table MD-11—Gradation for Granular Bedding

U.S. Standard Sieve Size	Percent Weight by Passing Square-Mesh Sieves: Type I CDOT Sect. 703.01	Type II CDOT Sect. 703.09 Class A
3 inches	-----	90-100
1½ inches	-----	-----
¾ inches	-----	20-90
$^3/_8$ inches	100	-----
#4	95-100	0-20
#16	45-80	-----
#50	10-30	-----
#100	2-10	-----
#200	0-2	0-3

The Type I and Type II bedding specifications shown in Table MD-11 were developed using the T-V filter criteria and the fact that bedding which will protect an underlying non-cohesive soil with a mean grain size of 0.045 mm will protect anything finer. Since the T-V filter criterion provides some latitude in establishing bedding gradations, it is possible to make the Type I and Type II bedding specifications conform with Colorado Division of Highways' aggregate specifications. The Type I bedding in Table MD-11 is designed to be the lower layer in a two-layer filter for protecting fine-grained soils and has a gradation identical to Colorado Department of Transportation's (CDOT's) concrete sand specification AASHTO M-6 (CDOT Section 703.01). Type II bedding, the upper layer in a two-layer filter, is equivalent to Colorado Division of Highways' Class A filter material (Section 703.09 Class A) except that it permits a slightly larger maximum rock fraction. When the channel is excavated in coarse sand and gravel (50% or more of coarse sand and gravel retained on the #40 sieve by weight), only the Type II filter is required. Otherwise, a two-layer bedding (Type I topped by Type II) is required. Alternatively, a single 12-inch layer of Type II bedding can be used, except at drop structures. For required bedding thickness, see Table MD-12. At drop structures, a combination of filter fabric and Type II bedding is acceptable as an alternative to a two-layer filter.

Table MD-12—Thickness Requirements for Granular Bedding

Riprap Designation	Minimum Bedding Thickness (inches)		
	Fine-Grained Soils*		Coarse-Grained Soils**
	Type I	Type II	Type II
VL (d_{50} = 6 in), *L* (d_{50} = 9 in)	4	4	6
M (d_{50} = 12 in)	4	4	6
H (d_{50} = 18 in)	4	6	8
VH (d_{50} = 24 in)	4	6	8

* May substitute one 12-inch layer of Type II bedding. The substitution of one layer of Type II bedding shall not be permitted at drop structures. The use of a combination of filter fabric and Type II bedding at drop structures is acceptable.

** Fifty percent or more by weight retained on the # 40 sieve.

The specifications for the T-V reverse filter relate the gradation of the protective layer (filter) to that of the bed material (base) by the following inequalities:

$$D_{15(filter)} \leq 5d_{85(base)} \quad \text{(MD-15)}$$

$$4d_{15(base)} \leq D_{15(filter)} \leq 20d_{15(base)} \quad \text{(MD-16)}$$

$$D_{50(filter)} \leq 25d_{50(base)} \quad \text{(MD-17)}$$

in which, the capital "D" and lower case "d" refer to the filter and base grain sizes, respectively. The subscripts refer to the percent by weight, which is finer than the grain size denoted by either D or d. For example, 15% of the filter material is finer than $D_{15(filter)}$ and 85% of the base material is finer than $d_{85(base)}$. Application of the T-V filter criteria is best described using an example provided in Section 4.4.8.

4.4.4.2 Filter Fabric

Filter fabric is not a substitute for granular bedding. Filter fabric provides filtering action only perpendicular to the fabric and has only a single equivalent pore opening between the channel bed and the riprap. Filter fabric has a relatively smooth surface, which provides less resistance to stone movement. As a result, it is recommended that the use of filter fabric be restricted to slopes no steeper than 3H:1V. Tears in the fabric greatly reduce its effectiveness; therefore, direct dumping of riprap on the filter fabric is not allowed, and due care must be exercised during construction. Nonetheless, filter fabric has proven to be a workable supplement to granular bedding in many instances, provided it is properly selected, installed and not damaged during installation.

At drop structures and sloped channel drops, where seepage forces may run parallel to the fabric and cause piping along the bottom surface of the fabric, special care is required in the use of filter fabric. Seepage parallel with the fabric must be reduced by folding the edge of the fabric vertically downward about 2 feet (similar to a cutoff wall) at 12-foot intervals along the installation, particularly at the entrance and exit of the channel reach. Filter fabric has to be lapped a minimum of 12 inches at roll edges, with upstream fabric being placed on top of downstream fabric at the lap.

Fine silt and clay has been found to clog the openings in filter fabric. This prevents free drainage, increasing failure potential due to uplift. For this reason, a double granular filter is often more appropriate bedding for fine silt and clay channel beds. See Figure MD-14 for details on acceptable use of filter fabric as bedding.

4.4.5 Channel Cross Section

4.4.5.1 Side Slopes

For long-term maintenance needs, it is recommended that riprap channel linings be used only as toe protection in natural channel and in low-flow channel portion of an engineered channel, but not on the banks above the low-flow channel section. For this reason whenever soil-riprap linings are used above the low-flow section or above what is needed for toe protection, a slope typically used for grass-line channels is recommended (i.e., 4H:1V), with certain exceptions in retrofit situations with limited right of way, where a maximum steepness of 3H:1V may be used.

Riprap-lined and soil riprap-lined side slopes when used as described above that are steeper than 2.5H:1V are considered unacceptable because of stability, safety, and maintenance considerations. In some cases, such as under bridges and in retrofit situations where right-of-way is very limited, use of 2H:1V may be considered.

4.4.5.2 Depth

The maximum depth should be consistent with the guidelines in Section 4.4.2.2 of this chapter. For known channel geometry and discharge, normal water depth can be calculated using Manning's Equation from Section 3.1.1 of this chapter.

4.4.5.3 Bottom Width

The bottom width should be designed to satisfy the hydraulic capacity of the cross section, recognizing the limitations on velocity, depth, and Froude number. For a given discharge, the bottom width can be calculated from depth, velocity, slope, and Froude number constraints in Sections 4.4.2.1, 4.4.2.2, and 4.4.2.3 using Manning's Equation from Section 3.1.1 of this chapter.

4.4.5.4 Outfalls Into Channel

Outfalls into riprap-lined channels should be at least 1 foot (preferably 2 feet) above the channel invert.

4.4.6 Erosion Control

For a properly bedded and lined riprap channel section, in-channel erosion should not generally be a problem. As with concrete channels, the primary concern with erosion is control of erosion in the watershed tributary to the channel. Good erosion control practices in the watershed will reduce channel maintenance. In addition, accumulation of debris in the channel, especially after a large event, may be of concern due to the potential for movement of riprap and damming.

4.4.7 Maintenance

A maintenance access road with a minimum passage width of 12 feet shall be provided along the entire length of all major drainageways. The local government may require the road to have an all weather surface such as 5-inch-thick concrete pavement. Requirements for District maintenance eligibility are reviewed in Section 3.2.8 of this chapter. Of particular concern is the long-term loss of riprap, particularly due to the public removing rock. If grouted rock is used, follow the criteria for grouted boulders (i.e., use of grouted riprap is not an acceptable practice). Grout can deteriorate with time, and this should be monitored, as well. Improper grout installation creates long-term maintenance problems.

4.4.8 Calculation Example

Calculations for sizing a riprap-lined channel using hydraulic equations from Section 3.0 and criteria from Section 4.4 are shown in Example MD-3 using the **Riprap Worksheet** of the UD-Channels Spreadsheet. This example is located at the end of this chapter.

4.4.9 Design Submittal Checklist

Table MD-13 provides a design checklist for a riprap-lined channel.

Table MD-13—Design Submittal Checklist for Riprap-Line d Channel

Criterion Requirement	✓
Maximum normal depth velocity for 100-year event ≤12 ft/sec	
Channel capacity checked with Manning's n = 0.041	
Maximum velocity checked using Manning's n = 0.030	
Froude number ≤ 0.8	
Side slopes in low-flow channel and for toe protection in natural channel: no steeper than 2.5H:1V (see section 4.4.5.1).	
Use of soil riprap, buried with topsoil and revegetated, if type VL, L or M riprap in grass-lined channel is used. (Use of soil riprap is suggested for larger stones as well)	
Rock specific gravity ≥ 2.50 and meets other requirements in Section 4.1.1.1	
Riprap size determined using Equation MD-13 and Table MD-10	
Riprap blanket thickness ≥ 2.0 x d_{50}	
Blanket thickness increased at least 50% for ≥ 3 ft at upstream & downstream ends of lining	
Toe protection provided in accordance with Section 4.4.2.4	
Scour depth calculated for 100-yr flow to assure adequate toe thickness	
Outfalls into channel 1 to 2 ft above channel invert	
Riprap lined bend curve radius of the channel's centerline ≥ 2.0 x top width for 100-year event	
Channel bends size riprap using adjusted velocity in accordance with recommendations in section "4.1.6 Erosion Control" of the *USDCM*	
Riprap protection for outer bank of bend extended downstream at least 2 x 100-yr top width	
Minimum of 2.0 ft freeboard, including superelevation, for adjacent structures	
Riprap at transitions extended upstream by 5 ft and downstream by 5 x design flow depth	
Riprap sized for transitions using 1.25 times maximum transition velocity	
Appropriate gradation of granular bedding material per Section 4.4.4.1	
Adequate thickness for granular bedding Section 4.4.4.1	
District Maintenance Eligibility Guidelines satisfied	
Continuous maintenance access road provided (8-foot surface with 12-foot clear width, 20-foot at drop structures)	
Energy and hydraulic grade lines calculated and plotted for channel, with annotated design discharges shown	

4.5 Bioengineered Channels

Bioengineered channels (see Photographs MD-10 and MD-11) emphasize the use of vegetative components in combination with structural measures to stabilize and protect stream banks from erosion. The District advocates the integration of bioengineering techniques into drainage planning, design, and construction when the use of such channels is consistent with the District's policies concerning flow carrying capacity, stability, maintenance, and enhancement of the urban environment and wildlife habitat. The following discussion on bioengineered channels interfaces closely with Section 4.2, Wetland Bottom Channels, and Section 4.6, Natural Channels; designers are encouraged to read Sections 4.2, 4.5 and 4.6, concurrently. In addition, because bioengineered channels require some structural assistance to maintain stability in urban settings, the designer is referred to guidance on drop structures in the HYDRAULIC STRUCTURES chapter.

4.5.1 Components

Vegetation is the basic component of what is known as "bioengineering" (Schiechtl 1980). Schiechtl (1980) states that, "bioengineering requires the skills of the engineer, the learning of the biologist and the artistry of the landscape architect."

It has been hypothesized that vegetation can function as either armor or indirect protection, and, in some applications, can function as both simultaneously (Biedenharn, Elliot, and Watson 1997 and Watson, Biedenharn, and Scott 1999). Grassy vegetation and the roots of woody vegetation may function as armor, while brushy and woody vegetation may function as indirect protection; the roots of the vegetation may also add a degree of geotechnical stability to a bank slope through reinforcing the soil (Biedenharn, Elliot, and Watson 1997 and Watson, Biedenharn, and Scott 1999), but these premises have not yet been technically substantiated through long-term field experience in urban settings. Each species of grass or shrub has differing ecological requirements for growth and differing characteristics such as root strength and density. Species should be selected based on each site's individual characteristics. Bioengineered channels must be designed with care and in full recognition of the physics and geomorphic processes at work in urban waterways and changing watersheds. Representative components of bioengineered channels include:

1. Planted riprap
2. Planted, grouted boulders
3. Brush layering
4. Fiber rolls
5. Fascines
6. Live willow stakes (with and without joint plantings in soil filled rock)
7. Live plantings in conjunction with geotextile mats
8. Wide ranges of planting of wetland and upland vegetation

9. Wrapped soil lifts for slope stability

See Photographs MD-10 and MD-13 and Figures MD-15 through MD-18 for more guidance.

Photograph MD-12—Willow plantings and vegetation along bioengineered channel.

Photograph MD-13—Integration of open water areas with major drainageways provides habitat and aesthetic benefits in addition to providing storage.

4.5.2 Applications

Bioengineered channels are applicable when channel designs are firmly grounded in engineering principles and the following conditions are met:

1. Hydrologic conditions are favorable for establishment and successful growth of vegetation.

2. Designs are conservative in nature, and bioengineered features are used to provide redundancy.
3. Maintenance responsibilities are clearly defined.
4. Adequate structural elements are provided for stable conveyance of the major runoff flow.
5. Species are selected based on individual site characteristics.

4.5.3 Bioengineering Resources

The purpose of this section is to provide the designer with an overview of bioengineering and basic guidelines for the use of bioengineered channels on major drainage projects within the District. There are many sources of information on bioengineering that the designer should consult for additional information when planning and designing a bioengineered channel (Watson, Biedenharn, and Scott 1999; USFISRWG 1998; Riley 1998; and Biedenharn, Elliot, and Watson 1997).

4.5.4 Characteristics of Bioengineered Channels

The following characteristics are generally associated with bioengineered channels:

1. Their design must address the hydrologic changes associated with urbanization (increased peak discharges, increased runoff volume, increased base flow, and increased bank-full frequency). These changes typically necessitate the use of grade control structures. In the absence of grade control structures, especially in the semi-arid climate of the Denver area, purely bioengineered channels will normally be subject to bed and bank erosion, channel instability, and degradation.

2. In addition to grade controls, most bioengineered channels require some structural methods to assist the vegetation with maintaining channel stability. Examples include buried riprap at cannel toes and at outer channel banks (see Figures MD-16, MD-17 and MD-18).

3. The designer must ensure that there will be sufficient flow in the channel (or from other sources, such as locally high groundwater) to support the vegetation. A complicating factor is that, in newly developing areas, base flows will *not* be present; whereas, if the tributary drainage area is large enough, base flows will often materialize after substantial urbanization has occurred. Therefore, it is important to match the channel stabilization technique to the water available at the time of construction, whether naturally or from supplemental water sources.

4. The extent to which vegetative techniques for channel stabilization will need to be supplemented with structural measures is a function of several factors:

   a. Slope
   b. Maximum velocity during 5-year event
   c. Maximum velocity during 100-year event
   d. Froude number during 5-year event
   e. Froude number during 100-year event

f. Tractive force
g. Sinuosity
h. Timing of period of construction relative to the growing season
i. Other site-specific factors

In general, slight channel slopes, lower velocities, lower Froude numbers, lower tractive force values, and higher sinuosity are conducive to channel stabilization approaches that emphasize bioengineering. These factors indicate that park-like settings (areas of open space, parks, office parks, etc.) are often conducive to bioengineered projects because they provide space for the channel to have a meander pattern that increases flow length and decreases channel slope, velocities, and tractive forces.

A technique that can be utilized is stabilization of the outer banks of a defined low-flow channel to withstand the major storm. Within the defined low-flow channel, base flows and small storm flows can then assume their own flow path (meander pattern). This pattern can either be pre-established (with a "pilot" channel) or the flows can move freely from one side of the hardened low-flow channel to the other, thereby establishing their own pattern.

Figure MD-19 shows examples of details for boulder toe protection (grouted and ungrouted, for one- and two-boulder high toe walls) that can be used to define a hardened, low-flow channel within which base flows and small storm flows can freely meander. Boulders should be placed on a Type L riprap foundation, and boulders should be aligned so that they are wider than they are tall. Boulders should be placed so that the top of the toe protection wall is flat. If stacking is stable, grouting may not be necessary. In areas where the channel is easily accessible to the public, the top row of boulders may be grouted in place so that vandals cannot remove them.

4.5.5 Advantages of Bioengineered Channels

Public reaction to bioengineered channels is generally favorable, not only in metropolitan Denver, but also regionally and nationally. In contrast to major drainageway stabilization projects that focus on structural measures, such as concrete-lined or riprap-lined channels, bioengineered channels:

1. Appear more natural in character and, often, more like a channel prior to urbanization. When post-urbanization hydrology permits, riparian areas may be created where there previously was little vegetation. Also, wetlands can often be created in conjunction with bioengineered channels.
2. Have a "softer" appearance and are generally judged by most to be more aesthetic.
3. Are often found where space is not a limitation, such as in public parks and open space areas.
4. Generally, provide wildlife habitat.
5. Provide other benefits such as passive recreational opportunities for the public (like bird watching), open space creation/preservation, potentially water temperature moderation, and/or water quality enhancement.

6. Create a living system that may strengthen over time.
7. Can facilitate obtaining 404 permits.

4.5.6 Technical Constraints

The following constraints are associated with bioengineered channels:

1. There is only limited experience to rely on for successful design of urban channels. The majority of the experience with bioengineering techniques relates to channels in nonurban settings.
2. The semi-arid conditions that characterize Denver can be at odds with the need for an adequate water supply for maintaining the vegetation. Careful species selection that reflects the site's soils and water availability characteristics is essential.
3. A basic design criterion within the District is to demonstrate channel stability during the major (100-year) storm, due to public safety and property protection concerns within urban areas. There is little evidence (locally, regionally, or nationally) as to whether purely bioengineered channels can withstand 100-year (or lesser) flood forces.
4. Significant space can be required for bioengineered channels, yet space is often at a premium in urban areas.
5. Bioengineered facilities can be more expensive than their traditional counterparts.
6. Bioengineered channels can be maintenance intensive, particularly in their early years.
7. During the early years while the vegetation is becoming established, if a significant storm occurs, the probability of significant damage to the facility and adjacent infrastructure and properties (i.e., economic loss) is high.

Additional potential constraints of vegetative stabilization methods are summarized by Biedenharn, Elliot, and Watson (1997), as follows:

- Even well executed vegetative protection cannot be planned and installed with the same degree of confidence, or with as high a safety factor, as structural protection. Vegetation is especially vulnerable to extremes of weather, disease, insects, and inundation before it becomes well established.
- Most vegetation has constraints on the season of the year that planting can be performed.
- Growth of vegetation can cause a reduction in flood conveyance or erosive increases in velocity in adjacent un-vegetated areas.
- Vegetation can deteriorate due to mismanagement by adjacent landowners or natural causes.
- Trunks of woody vegetation or clumps of brushy vegetation on armor revetments can cause local flow anomalies, which may damage the armor.
- Large trees can threaten the integrity of structural protection by root invasion, by toppling and damaging the protection works, by toppling and directing flow into an adjacent unprotected bank,

or by leaving voids in embankments due to decomposition.

- Roots can infiltrate and interfere with internal bank drainage systems or cause excess infiltration of water into the bank.
- Many of these problems may be avoided through selection of the appropriate type and species of vegetation. Such selections and expert advice must be obtained from qualified individuals in revegetation and bioengineering. Invasion by other species is quite likely over the years the bioengineered channel is in operation.

4.5.7 Design Guidelines

To provide the designer with guidelines for the applicability of bioengineered channels, a comparison of hydraulic characteristics is provided in Table MD-14 for four types of channels, ranging from a fully bioengineered channel to a structural channel. To allow for growth of vegetation and accumulation of sediment, outfalls into bioengineered channels should be 1 to 2 feet above the channel invert.

Table MD-14—Guidelines for Use of Various Types of Channels

(Note: All channel types typically require grade control structures.)

Design Parameter	Fully Bioengineered Channel	Bioengineered Channel Including Structural Elements	Structural Channel With Bioengineered Elements	Structural Channel
Maximum Slope	0.2%	0.5%	0.6%	1.0%
Is base flow necessary?	Yes	Yes	Yes	No
V_{max} for $Q_{5\text{-}year}$*	3.5 ft/sec (2.5)	4.0 ft/sec (3.0)	5.0 ft/sec (3.5)	**
V_{max} for $Q_{100\text{-}year}$*	5.0 ft/sec (3.5)	6.0 ft/sec (4.5)	7.0 ft/sec (5.0)	**
$F_{r5\text{-}year}$	0.4 (0.3)	0.6 (0.4)	0.7 (0.5)	**
$F_{r100\text{-}year}$	0.4 (0.3)	0.8 (0.5)	0.8 (0.5)	**
Maximum tractive force (100-year event)	0.30 lb/ft^2	0.60 lb/ft^2	1.00 lb/ft^2	1.30 lb/ft^2
Maximum sinuosity	1.6	1.2	1.2	1.0

* Values presented for both non-erosive and erosive soils. Erosive soil values are in parenthesis ( ).

** With a purely structural channel, such as a reinforced concrete channel, allowable velocities and allowable Froude numbers, F_r, are based on site-specific design calculations.

4.6 Natural Channels

Natural waterways in the Denver region are sometimes in the form of steep-banked gulches, which have eroding banks and bottoms. On the other hand, many natural waterways exist in urbanized and to-be-urbanized areas, which have mild slopes, are reasonably stable, and are not currently degrading. If the channel will be used to carry storm runoff from an urbanized area, it can be assumed that the changes in the runoff regime will increase channel erosion and instability. Careful hydraulic analysis is needed to address this projected erosion. In most cases, stabilization of the channel will be required. Stabilization using bioengineering techniques, described in Section 4.5 of this chapter, has the advantage of preserving and even enhancing the natural character and functions of the channel. Some structural stabilization measures will also be required in combination with the bioengineered stabilization measures.

In the Denver area, most natural waterways will need drops and/or erosion cutoff check structures to maintain a mild channel slope and to control channel erosion. Typically, these grade control structures are spaced to limit channel degradation to what is expected to be the final stable longitudinal slope after full urbanization of the tributary watershed. In the Denver area, this slope, depending on watershed size and channel soils, has been observed to range from 0.2 to 0.6%, with the South Platte River itself approaching a slope of 0.1%. Whenever feasible, natural channels should be kept in as near a natural condition as possible by limiting modifications to those necessary to protect against the destabilizing hydrologic forces caused by urbanization.

Investigations needed to ensure that the channel is stable will differ for each waterway; however, generally, it will be necessary to measure existing cross sections, investigate the bed and bank material, determine soil particle size distribution, and study the stability of the channel under future conditions of flow. At a minimum, the designer should consider the concept of the stable channel balance discussed in Section 1.5.3 of this chapter, complete tractive force analysis, and apply the Leopold equations to evaluate channel stability and changes in channel geometry. Oftentimes, more sophisticated analysis will be required. When performing stability and hydraulic analyses, keep in mind that supercritical flow normally does not exist in natural-earth channels. During backwater computations, check to ensure that the computations do not reflect the presence of consistent supercritical flow (Posey 1960).

Because of the many advantages of natural channels to the community (e.g., preservation of riparian habitat, diversity of vegetation, passive recreation, and aesthetics), the designer should consult with experts in related fields as to method of development. Nowhere in urban hydrology is it more important to convene an environmental design team to develop the best means for using a natural waterway. It may be concluded that park and greenbelt areas should be incorporated into the channel design. In these cases, the usual rules of freeboard, depth, curvature, and other rules applicable to artificial channels often will need to be modified to better suit the multipurpose objectives. For instance, there are advantages that may accrue if the formal channel is designed to overtop, resulting in localized flooding of adjacent floodplain areas that are laid out for the purpose of being inundated during larger (i.e., > 10-year) flood events. See the STORAGE chapter of this *Manual*.

The following design criteria are recommended when evaluating natural channels:

1. The channel and overbank floodplain should have adequate capacity for the 100-year flood.
2. A water surface profile should be defined in order to identify the 100-year floodplain, to control

earthwork, and to build structures in a manner consistent with the District's and local floodplain regulations and ordinances.

3. Use roughness factors (n) representative of un-maintained channel conditions for analysis of water surface profiles. Roughness factors for a variety of natural channel types are presented in Table MD-1.
4. Use roughness factors (n) representative of maintained channel conditions to analyze effects of velocities on channel stability. Roughness factors for a variety of natural channel types are presented in Table MD-1.
5. Prepare plan and profile drawings of the channel and floodplain.
6. Provide erosion-control structures, such as drop structures or grade-control checks, to control channel erosion and/or degradation as the tributary watershed urbanizes.
7. Outfalls into natural channels should be 2 feet above the channel invert to account for vegetation and sediment accumulation. The engineer should visit the site of any outfalls into natural drainageways to examine the actual ground surface condition.

4.7 Retrofitting Open-Channel Drainageways

Many projects involving major drainage system design will occur in areas that have already been developed, rather than in newly urbanizing areas. Design of major drainageways in these areas can be challenging due to limitations of the existing major drainage system, right-of-way constraints, community desires, and public acceptance. While underground conduits or hard-lined channels may be required in some situations, the designer should first consider the option of retrofitting a channel to provide flood conveyance and other recreational, aesthetic, environmental, and/or water quality benefits. Retrofitting a major drainage channel may be appropriate when:

1. The retrofitted channel will be capable of conveying the major flow event in a stable manner.
2. The retrofitted channel will provide recreational, aesthetic, environmental, and/or water quality benefits that other design options (i.e., an underground conduit or concrete channel) would not provide.
3. The retrofitted channel will not pose an increased public health or safety risk and, preferably, will be a safer alternative than other design options.

4.7.1 Opportunities for Retrofitting

Opportunities for retrofitting exist in many projects occurring in areas that have already been developed. Retrofitting is well suited to areas such as urban parks and designated open space areas where right-of-way is not too restricted by existing development. Retrofitting is especially favorable for redevelopment projects in urban areas that seek to incorporate the major drainageway as a feature of the development, providing aesthetic, recreational, and/or water quality benefits.

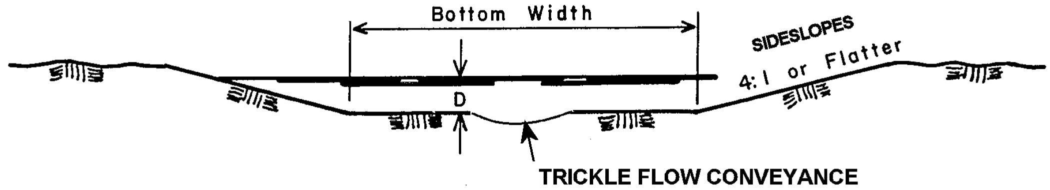
4.7.2 Objectives of Retrofitting

The foremost objective of retrofitting a drainageway must be to provide stable conveyance of the major flow event for the future developed condition of the watershed. Other objectives of retrofitting include:

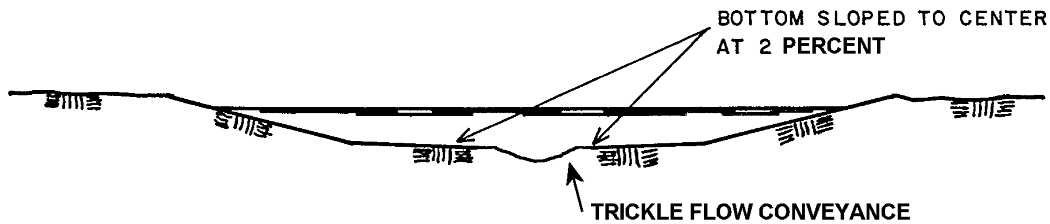
1. Creating multi-use areas. Uses that may be compatible with a well-designed retrofitted major drainageway include recreation, open space, parks and trails, wildlife corridors, restoration of vegetation for diversity and habitat, and others.
2. Enhancing channel aesthetics. Revegetation and landscaping can provide a riparian corridor that is attractive to the public as well as wildlife.
3. Enhancing water quality. Improved channel stabilization resulting from a major drainage channel retrofit has a direct benefit to water quality in reducing erosion and sediment transport. In addition, retrofitting can create aquatic habitat, and riparian vegetation and soil microorganisms can provide a degree of water quality treatment. Retrofitting can also be designed to limit access to some portions of the channel or to encourage access in specific areas that are more frequently maintained and/or equipped with trash cans.
4. Increasing benefit-to-cost ratio. For retrofitting to be acceptable, in most cases, it must be cost effective. Retrofitting an open channel may often be less expensive than constructing an underground conduit. Even when retrofitting costs are comparable to or higher than the costs of other design options, the multi-use potential for a retrofitted channel may justify the additional cost by providing benefits that otherwise would require separate facilities for each use.

4.7.3 Natural and Natural-Like Channel Creation and Restoration

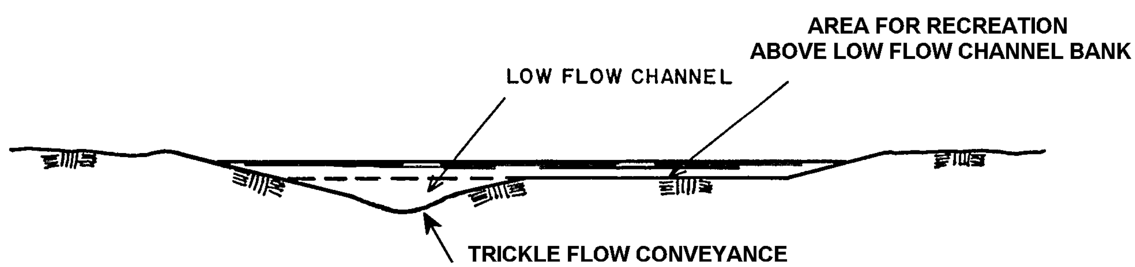
The designer should refer to Sections 4.1, 4.2, and 4.5 for guidance and criteria for creation of grass-lined channels, channels with wetland bottoms, and bioengineered channels, respectively.



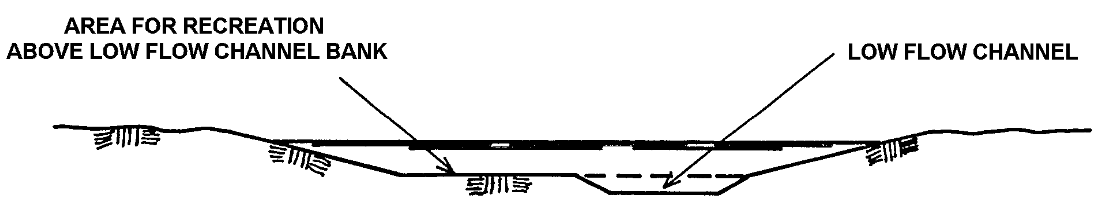
CROSS SECTION WITH OVAL OR SLOPED BOTTOM WITH TRICKLE CHANNEL



CROSS SECTION WITH OVAL OR SLOPED BOTTOM WITH TRICKLE CHANNEL

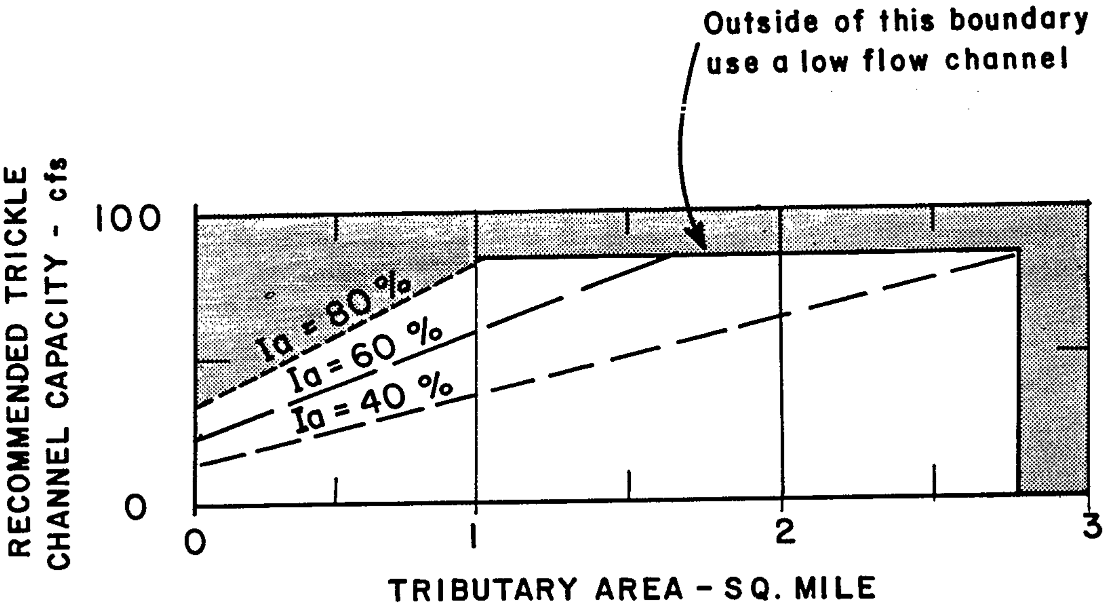


CROSS SECTION WITH LOW FLOW CHANNEL WITH TRICKLE CHANNEL
AREA FOR MAJOR DRAINAGE RUNOFF



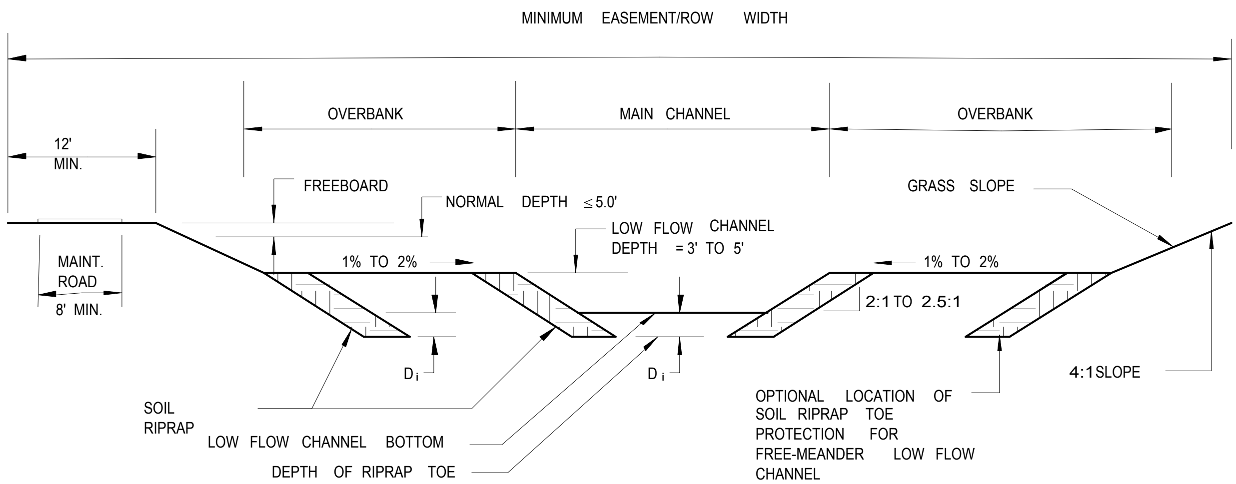
CROSS SECTION WITH LOW FLOW CHANNEL WITH
OVERFLOW AREA FOR MAJOR DRAINGAE RUNOFF

Figure MD-5—Typical Grassed Channels



Note: I_a = tributary basin impervious area percentage using full basin development condition.

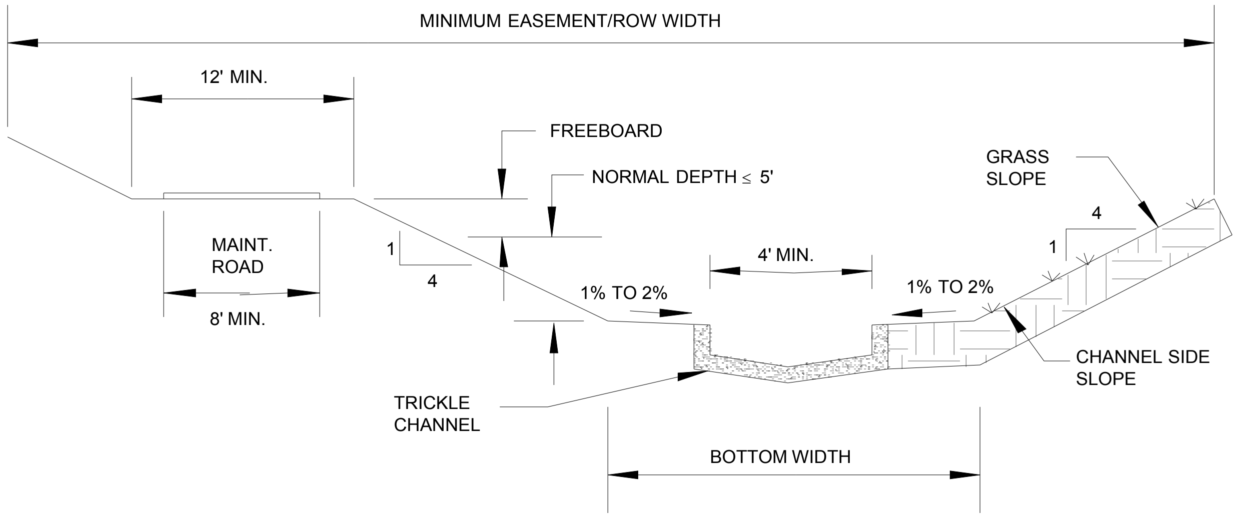
Figure MD-6—Minimum Capacity Requirements for Trickle Channels



NOTE:

1. LOW FLOW CHANNEL: CAPACITY TO BE THE EQUIVALENT OF $\frac{1}{3}$ TO $\frac{1}{2}$ OF THE 2-YEAR FLOW BASED ON FULLY DEVELOPED TRIBUTARY WATERSHED PEAK FLOW.
2. NORMAL DEPTH: FLOW DEPTH FOR 100-YEAR FLOW SHALL NOT EXCEED 5 FEET, NOT INCLUDING THE LOW FLOW CHANNEL DEPTH. 100-YEAR FLOW VELOCITY AT NORMAL DEPTH SHALL NOT EXCEED 7 FT/S FOR CHANNELS WITH EROSION RESISTANT SOILS OR 5 FT/S FOR CHANNELS WITH EROSIVE SOILS.
3. FREEBOARD: FREEBOARD TO BE A MINIMUM OF 1 FOOT.
4. MAINTENANCE ACCESS ROAD: MINIMUM STABLE WIDTH TO BE 8 FEET WITH A CLEAR WIDTH OF 12 FEET.
5. ROW WIDTH: MINIMUM WIDTH TO INCLUDE FREEBOARD AND MAINTENANCE ACCESS ROAD.
6. OVERBANK: FLOW IN EXCESS OF MAIN CHANNEL TO BE CARRIED IN THIS AREA. AREA MAY BE USED FOR RECREATION PURPOSES.
7. D_i = 3-FOOT MINIMUM FOR EROSION RESISTANT SOILS.
   D_i = 5-FOOT MINIMUM FOR SANDY SOILS
8. CHANNEL SIDESLOPE ABOVE LOW-FLOW CHANNEL 4H:1V OR FLATTER, EVEN IF LINED WITH SOIL RIPRAP.
9. FROUDE NUMBER FOR ALL FLOWS SHALL NOT EXCEED 0.8 FOR EROSION RESISTANT SOILS AND 0.5 FOR EROSIVE SOILS.
10. THE CHANNEL CAN BE DESIGNED TO HAVE THE LOW-FLOW SECTION TO HAVE A WETLAND BOTTOM.

Figure MD-7—Composite Grass-line Channel with a Low-Flow Channel, including a Wetland Bottom Low-Flow Channel



NOTES:

1. BOTTOM WIDTH: CONSISTENT WITH MAXIMUM ALLOWABLE DEPTH AND VELOCITY REQUIREMENTS, SHALL NOT BE LESS THAN TRICKLE CHANNEL WIDTH.

2. TRICKLE CHANNEL: CAPACITY TO BE APPROXIMATELY 2% OF 100-YEAR FLOW FOR THE FULLY DEVELOPED, UNDETAINED CONDITION TRIBUTARY WATERSHED PEAK FLOW. USE NATURAL LINING WHEN PRACTICAL.

3. NORMAL DEPTH: NORMAL DEPTH AT 100-YEAR FLOW SHALL NOT EXCEED 5 FEET. MAXIMUM 100-YEAR FLOW VELOCITY AT NORMAL DEPTH SHALL NOT EXCEED 7 FT/S FOR CHANNELS WITH EROSION RESISTANT SOILS OR 5 FT/S FOR CHANNELS WITH EROSIVE SOILS.

4. FREEBOARD: FREEBOARD TO BY A MINIMUM OF 1 FOOT.

5. MAINTENANCE ACCESS ROAD: MINIMUM STABLE WIDTH TO BE 8 FEET WITH CLEAR WIDTH OF 12 FEET.

6. EASEMENT/ROW WIDTH: MINIMUM WIDTH TO INCLUDE FREEBOARD AND MAINTENANCE ACCESS ROAD.

7. CHANNEL SIDE SLOPE: MAXIMUM SIDE SLOPE FOR GRASSED CHANNELS TO BE NO STEEPER THAN 4:1.

8. FROUDE NUMBER: MAXIMUM VALUE FOR MINOR AND MAJOR FLOODS SHALL NOT EXCEED 0.8 FOR CHANNELS WITH EROSION RESISTANT SOILS OR 0.5 FOR CHANNELS WITH EROSIVE SOILS.

Figure MD-8—Grass-lined Channel with a Trickle Channel

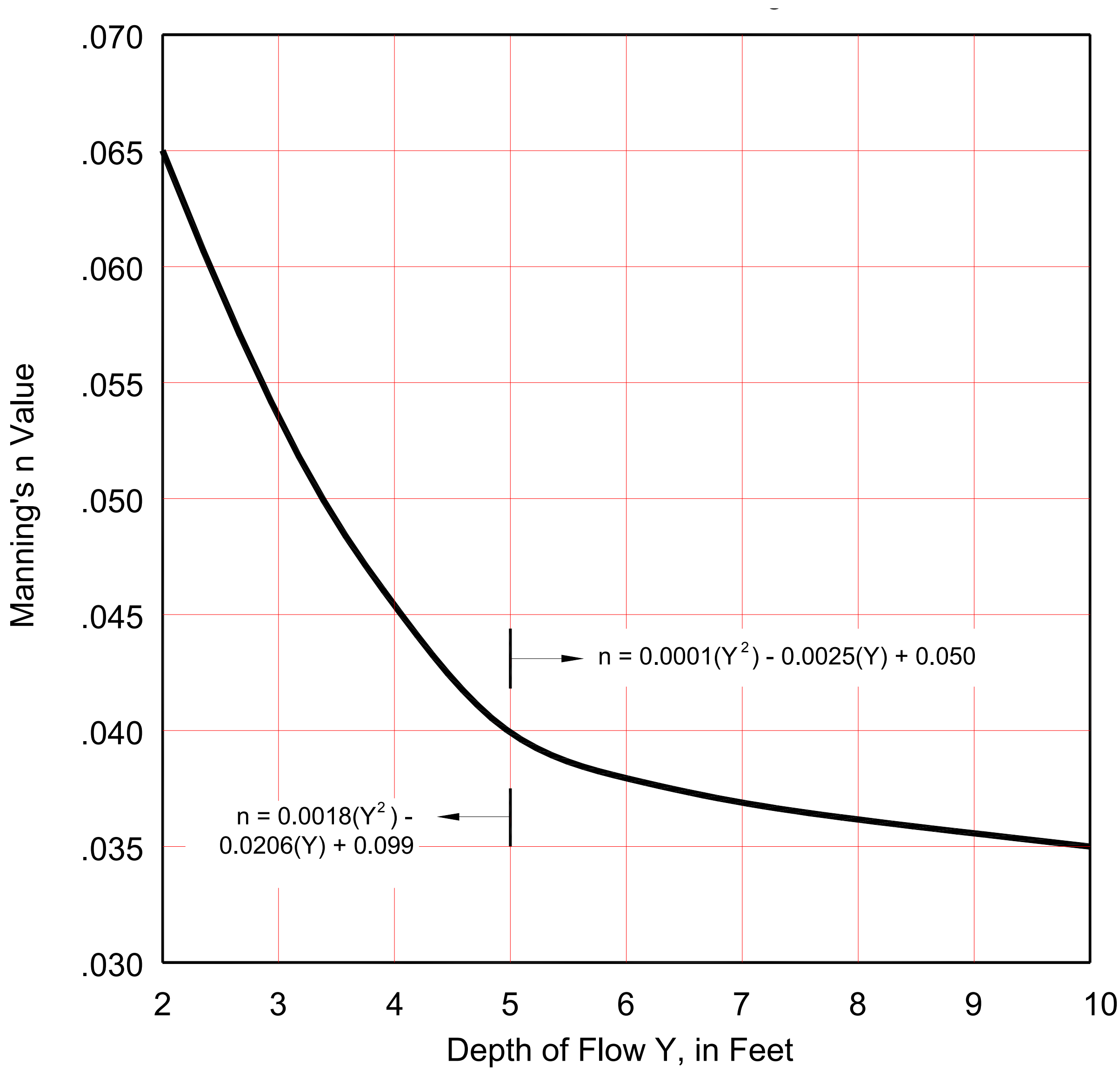
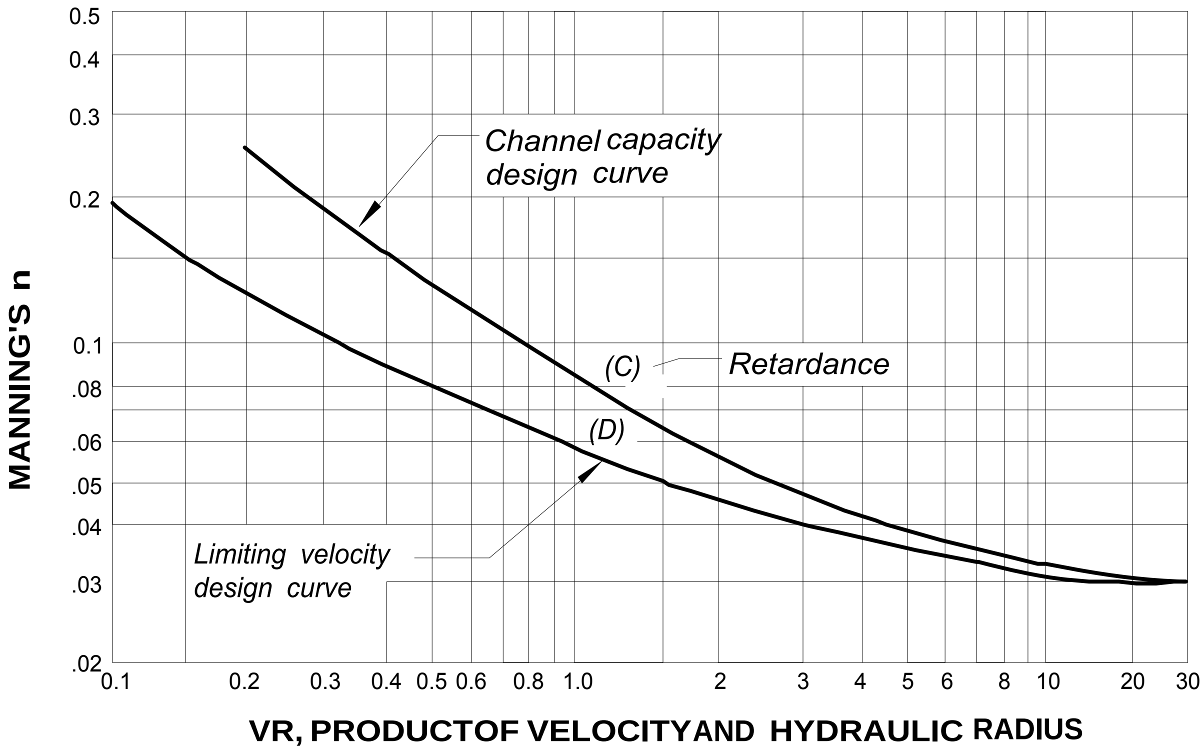


Figure MD-9a—Manning's *n* vs. Depth for Low-Flow Section in a Composite Channel.



From "Handbook of Channel Design For Soil and Water Conservation,: U.S. Department of Agriculture, Soils Conservation Service, No. SCS-TP-61 March, 1947, Rev. June, 1954

Figure MD-9b—Manning's *n* vs. *VR* for Two Retardances in Grass-Lined Channels.

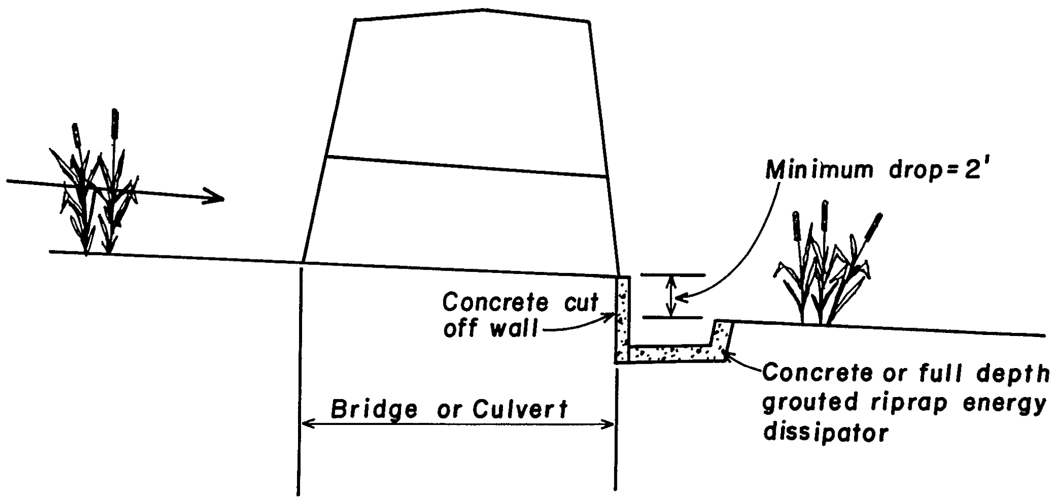


Figure MD-10—Composite (Wetland Bottom) Channel At Bridge or Culvert Crossing

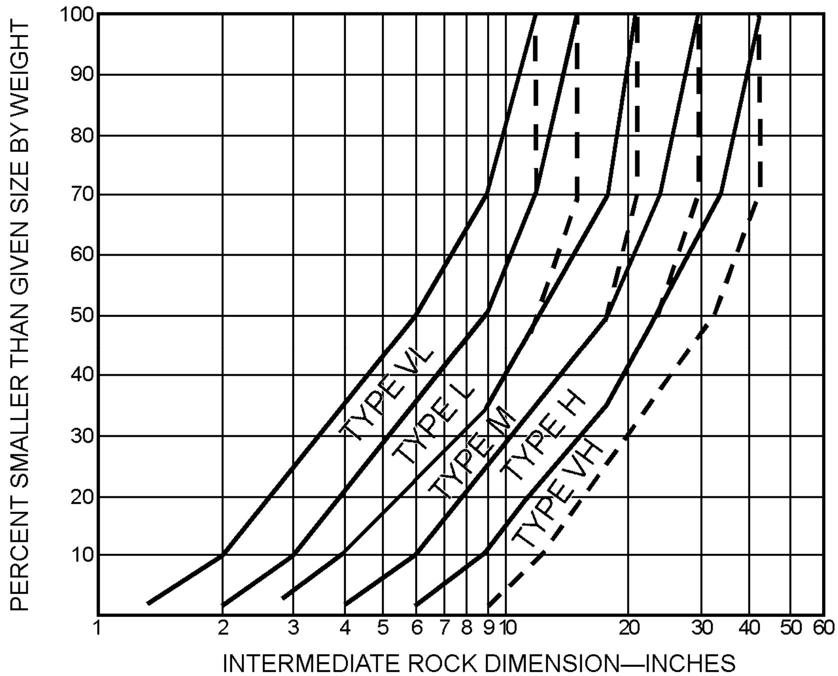


Figure MD-11—Gradation of Ordinary Riprap

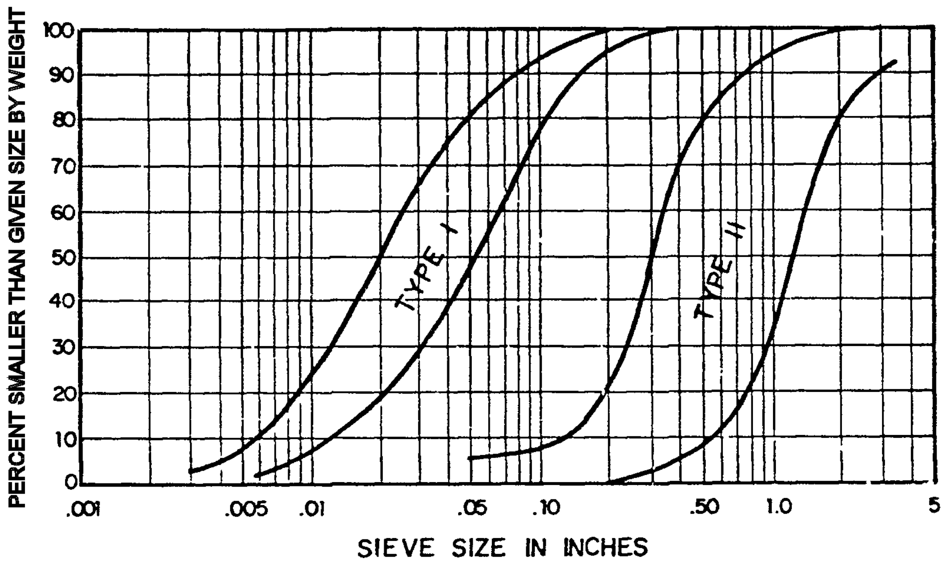
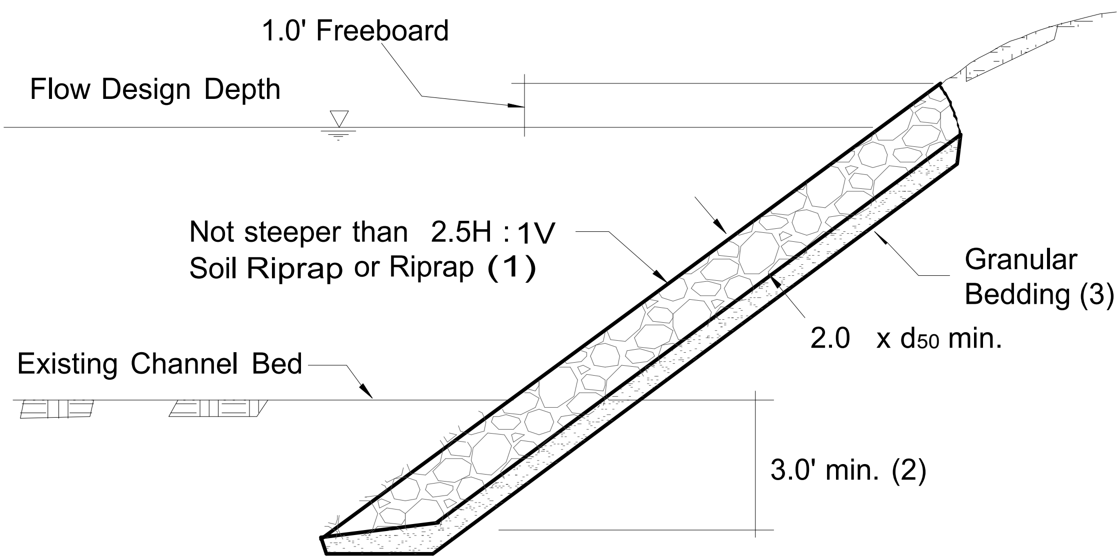


Figure MD-12—Gradation Curves for Granular Bedding

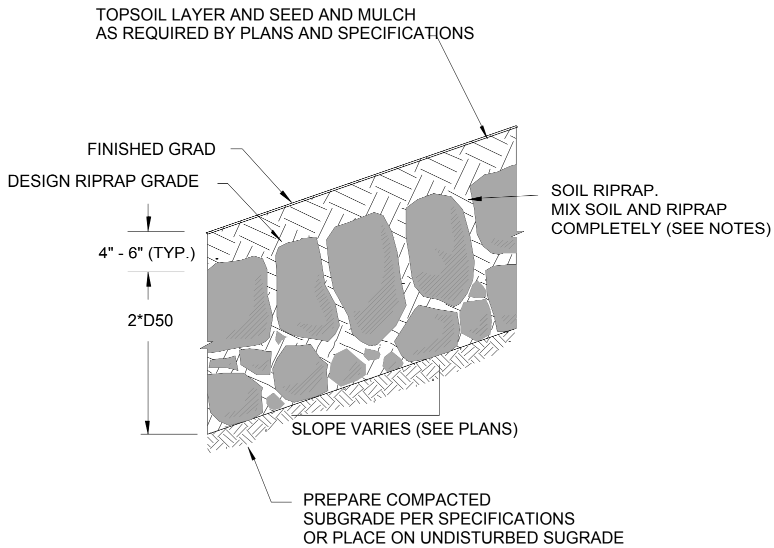


(1) Use Soil Riprap when d_{50} is less than or equal to Type M.
(Suggest use of Soil Riprap for larger riprap sizes as well)

(2) 5 - feet minimum in sandy or erosive soils.

(3) Eliminate granular bedding when soil-riprap is used.

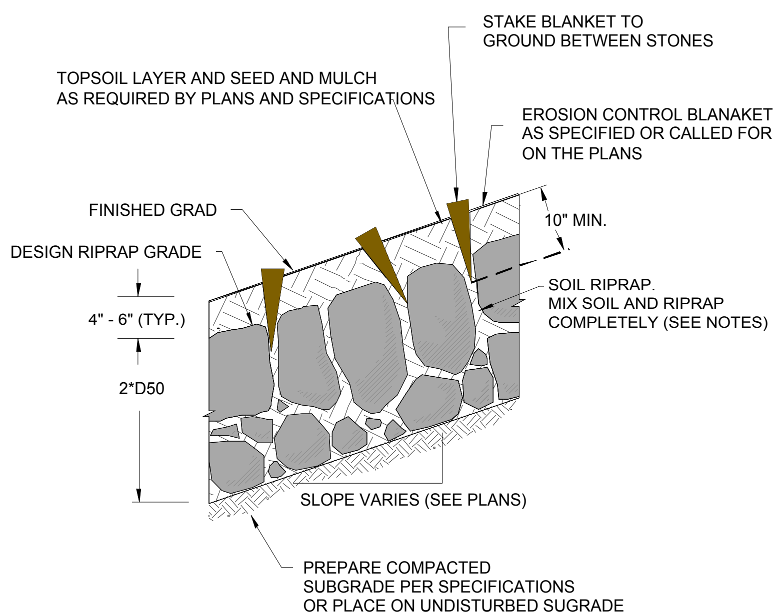
Figure MD-13a—Riprap Channel Bank Lining, Including Toe Protection

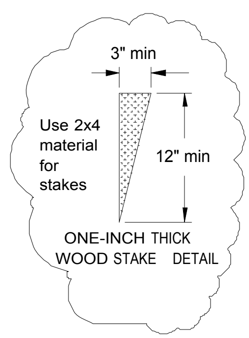


TYPICAL SECTION -
SOIL RIPRAP WITH MUCLH

NOTES:

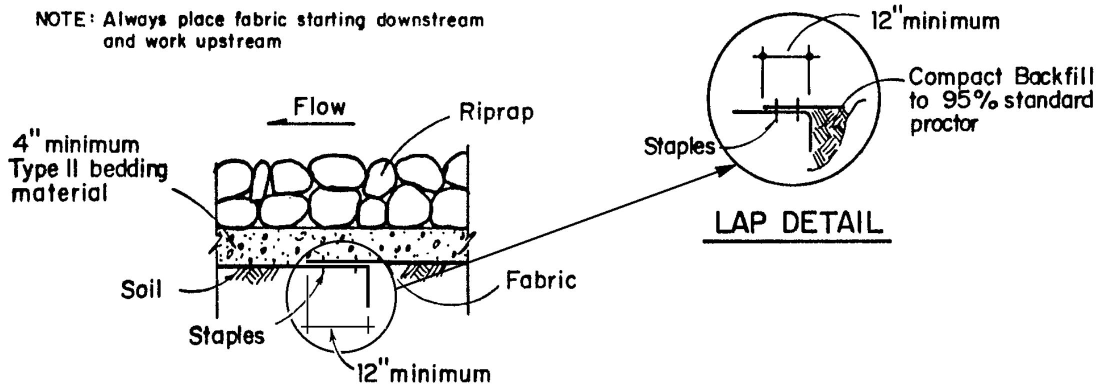
1. SOIL RIPRAP DETAILS ARE APPLICABLE TO SLOPED AREAS. REFER TO THE SITE PLAN ACTUAL LOCATION AND LIMITS.
2. MIX UNIFORM ALLY 65% RIPRAP BY VOLUME WITH 35% OF APPROVED SOIL BY VOLUME PRIOR TO PLACEMENT.
3. PLACE STONE-SOIL MIX TO RESULT IN SECURELY INTERLOCKED ROCK AT THE DESIGN THICKNESS AND GRADE. COMPACT AND LEVEL TO ELIMINATE ALL VOIDS AND ROCKS PROJECTING ABOVE DESIGN RIPRAP TOP GRADE.
4. CRIMP OR TACKIFY MULCH OR USE APPROVED HYDROMULCH AS CALLED FOR IN THE PLANS AND SPECIFICATIONS.



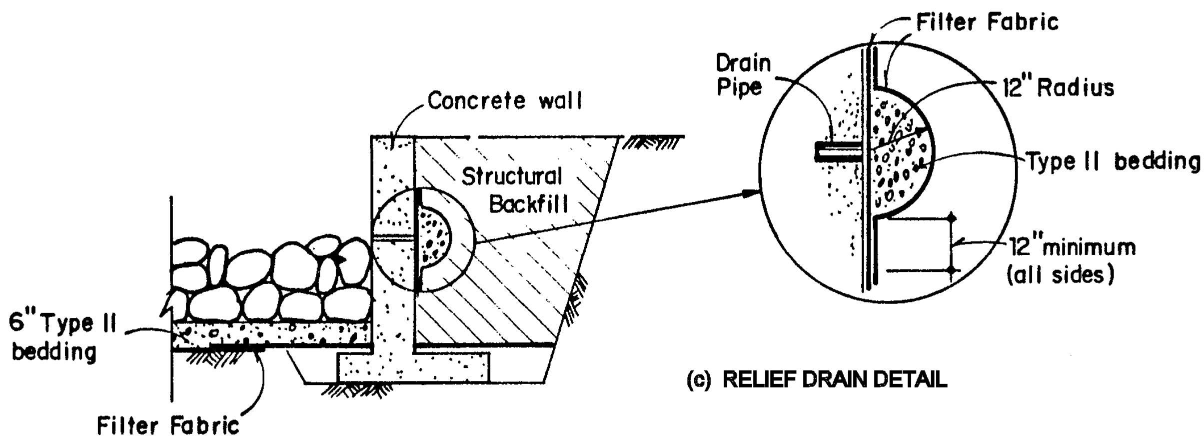


TYPICAL SECTION -
SOIL RIPRAP WITH EROSION CONTROL FABRIC

Figure MD-13b—Soil Riprap Typical Details



(a) TYPICAL LAP DETAIL AND FILTER FABRIC PLACEMENT



(c) RELIEF DRAIN DETAIL

Figure MD-14—Filter Fabric Details

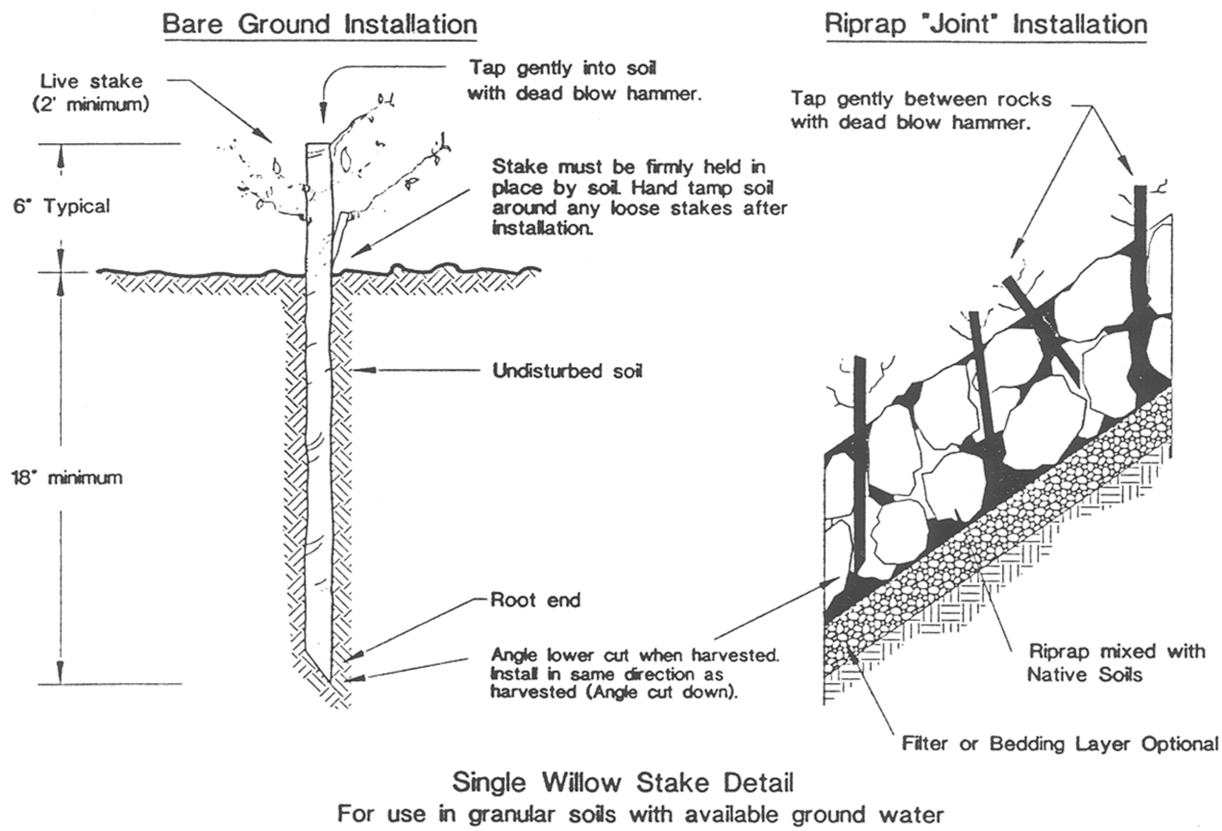
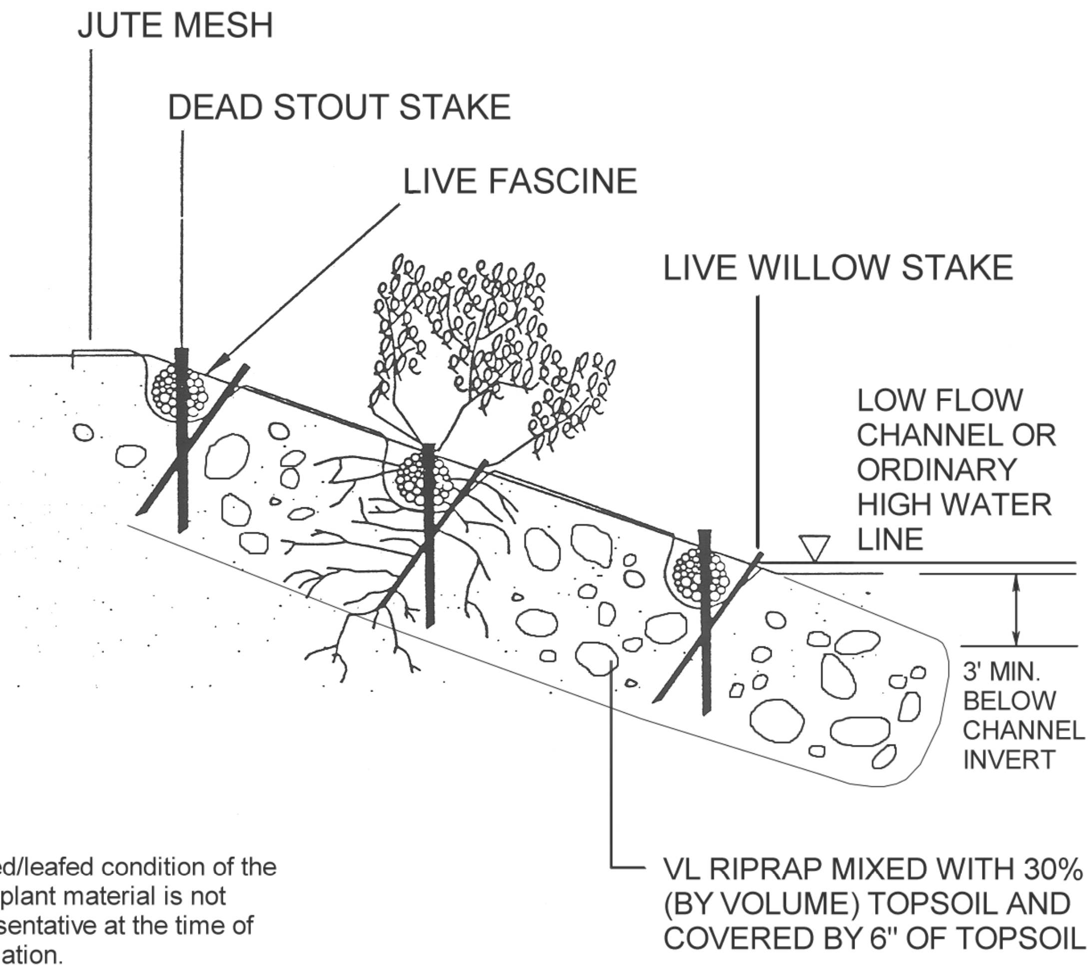


Figure MD-15—Live Willow Staking for Bare Ground and Joint Installation

SECTION



NOTES:
1. Rooted/leafed condition of the living plant material is not representative at the time of installation.
2. Refer to the contract documents for further specifications.

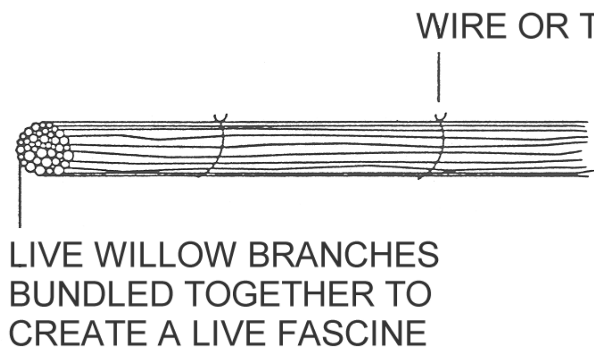
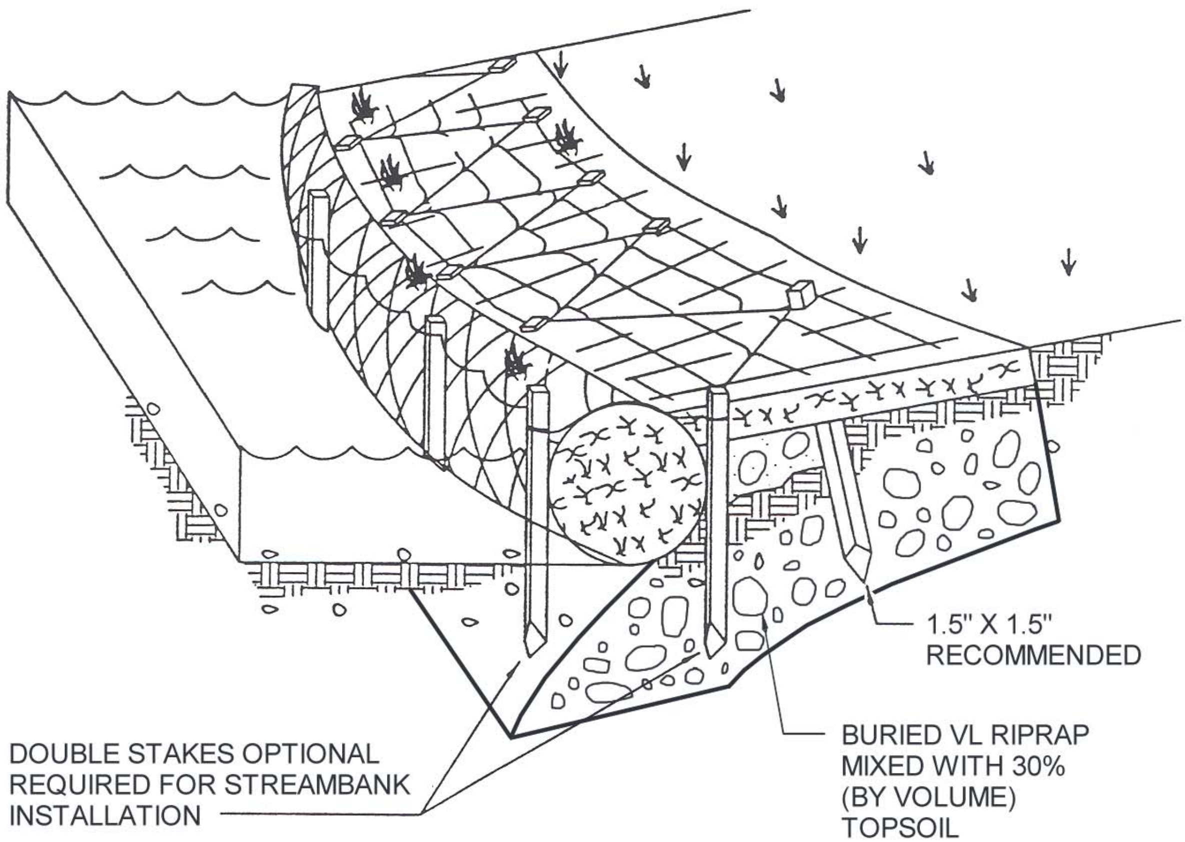


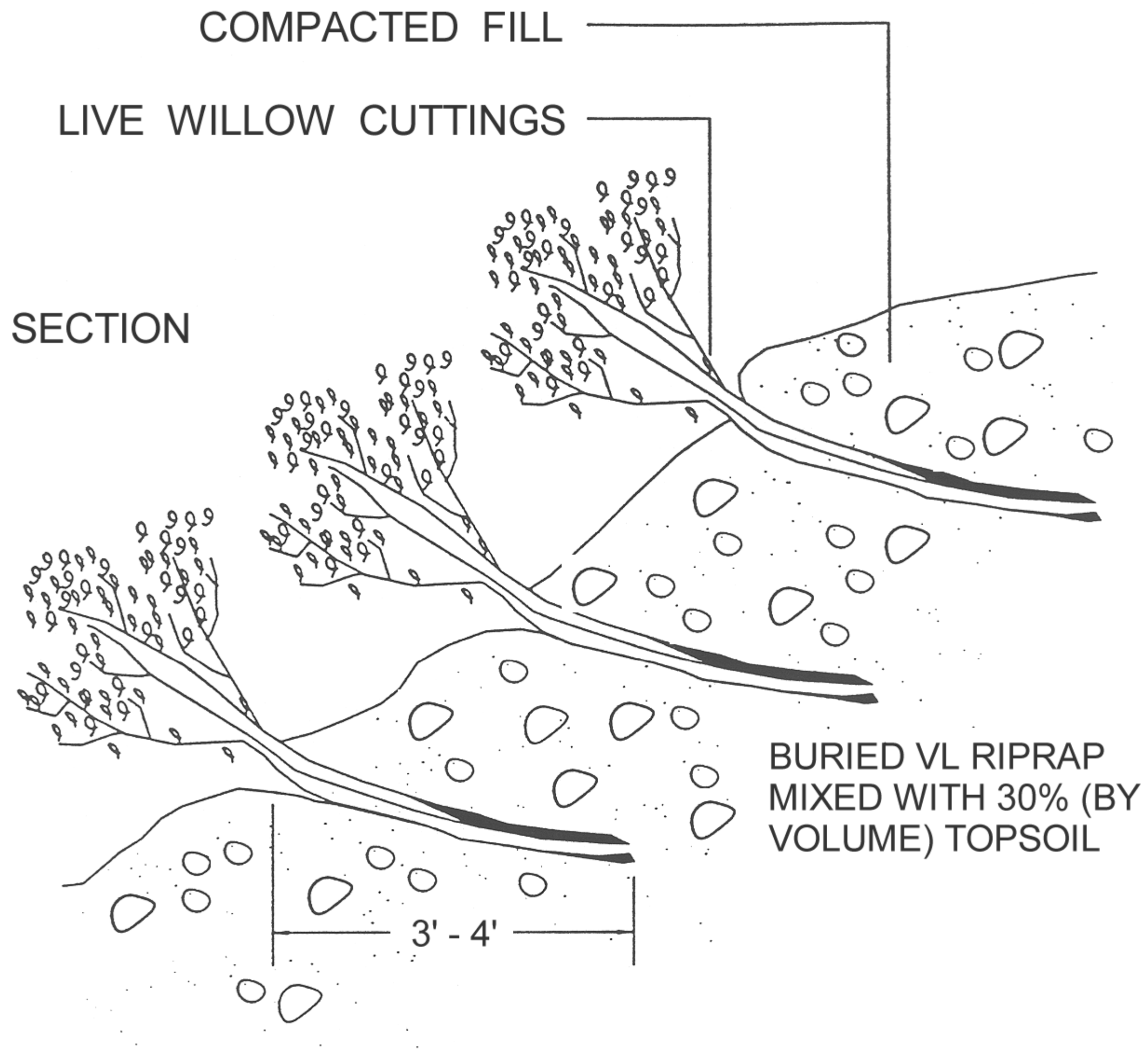
Figure MD-16—Fascine in Conjunction With Jute Mesh Mat



NOTES:
1. LENGTH OF STAKE DETERMINED BY THE SUBSTRATE.
2. REFER TO CONTRACT DOCUMENTS FOR FURTHER DETAILS.

Reprinted from Salix Applied Earthcare, Erosion Draw 2.0, 1996

Figure MD-17—Fiber Roll

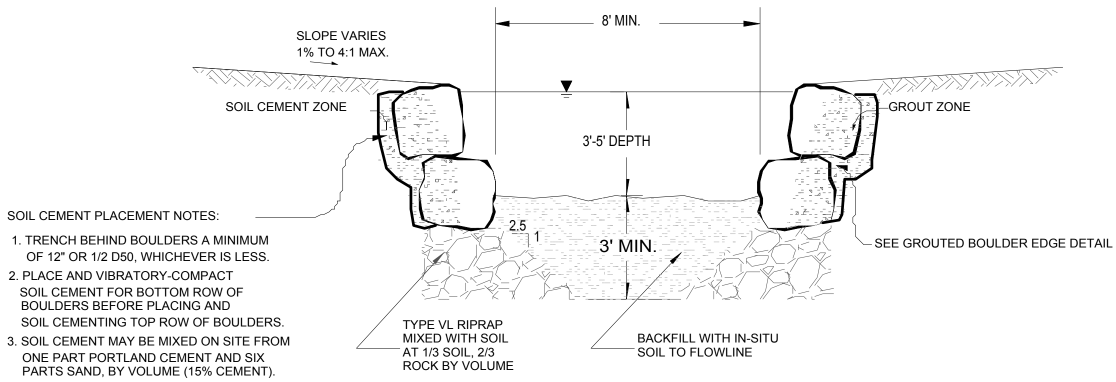


NOTES:

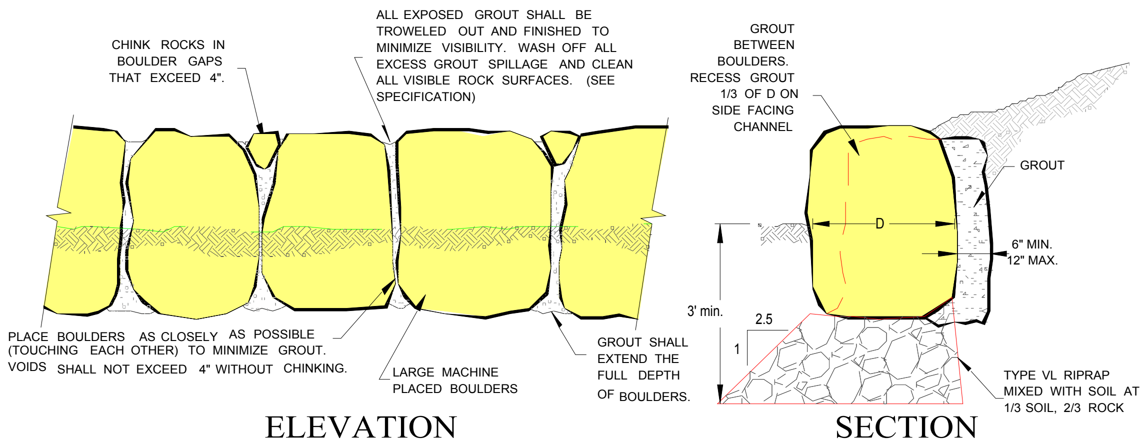
1. Rooted/leafed condition of the living plant material is not representative at the time of installation.

2. Refer to the contract documents for further details.

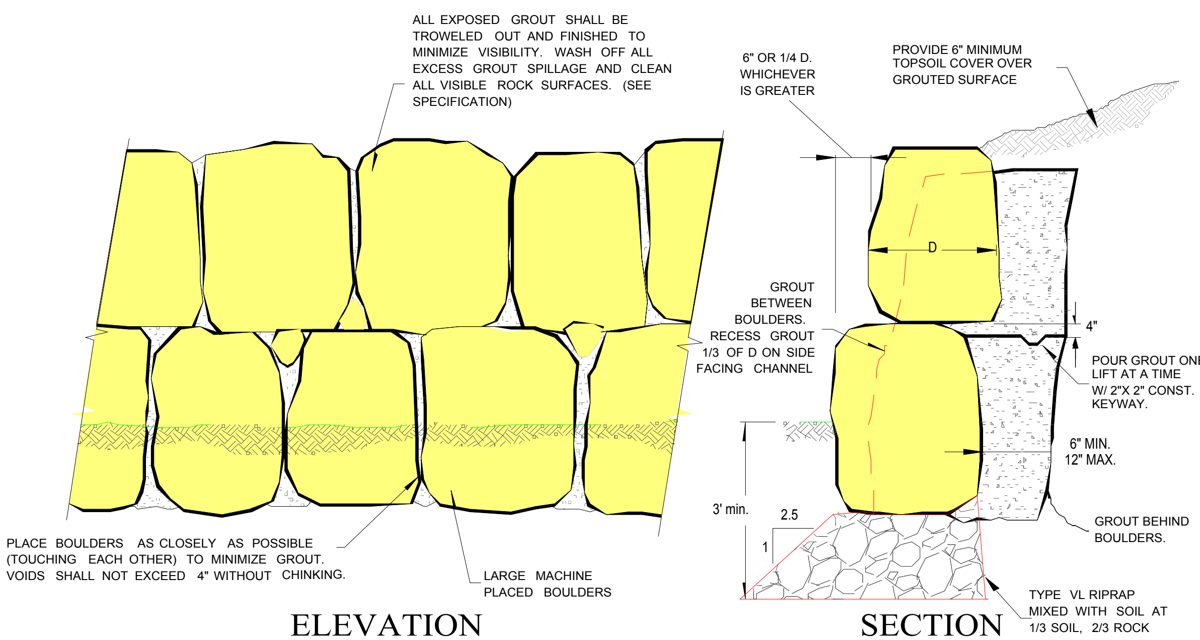
Figure MD-18—Brush Layering with Willow Cuttings



BOULDER EDGED LOW FLOW CHANNEL CROSS-SECTION



GROUTED BOULDER EDGE DETAIL



GROUTED BOULDER STACKED WALL EDGE

Figure MD-19—Details for Boulder Edge Treatment of a Low-Flow Channel

5.0 RECTANGULAR CONDUITS

The use of rectangular conduits of larger capacity can sometimes have cost advantages over large-diameter pipe. Furthermore, because they can be poured in place, advantages accrue in being able to incorporate conflicting utilities into the floor and roof of the structure.

Major disadvantages of rectangular conduits as storm sewers are:

1. The conduit's capacity drops significantly when the water surface reaches its roof since the wetted perimeter dramatically increases. The drop is 20% for a square cross section and more for a rectangular cross section where the width is greater than the height.

2. Normal structural design, because of economics, usually does not permit any significant interior pressures, meaning that if the conduit reached a full condition and the capacity dropped, there could be a failure due to interior pressures caused by a choking of the capacity (Murphy 1971).

It is apparent that the use of long rectangular conduits for outfall purposes requires a high standard of planning and design involving complex hydraulic considerations.

The chapters on CULVERTS and HYDRAULIC STRUCTURES in this *Manual* contain information that should be used to supplement this section in development of designs.

5.1 Hydraulic Design

Rectangular conduits are often considered as a covered free-flow conduit. They are open channels with a cover (Smith 1974). Computational procedures for flow in rectangular conduits are essentially the same as for canals and lined channels, except that special consideration is needed in regard to rapidly increasing flow resistance when a long conduit becomes full. The reader is referred to the chapters on CULVERTS and HYDRAULIC STRUCTURES for additional information.

An obstruction, or even a confluence with another conduit, may cause the flow in a near-full rectangular conduit to strike the roof and choke the capacity. The capacity reduction may then cause the entire upstream reach of the conduit to flow full, with a resulting surge and pressure head increase of sufficient magnitude to cause a structural failure. Thorough design is required to overcome this inherent potential problem. Structural design must account for internal pressure if pressure will exist.

Structural requirements and efficiency for sustaining external loads, rather than hydraulic efficiency, usually control the shape of the rectangular conduit. In urban drainage use, a rectangular conduit should usually have a straight alignment and should not decrease in size or slope in a downstream direction. It is desirable to have a slope that increases in a downstream direction as an added safety factor against it flowing full. This is particularly important for supercritical velocities that often exist in long conduits. For flatter-sloped conduits, the sediment deposition problem must be considered to prevent loss of capacity.

Roughness coefficients from Table MD-15 should be chosen carefully because of their effect on proper operation of the conduit. Quality control is important during construction; attention must be paid to grinding off projections and keeping good wall alignment. When using precast box sections, joint alignment, sealing and grouting are especially important.

Bedding and cover on conduits are structural considerations, and specifications for bedding and cover are closely allied to the loads and forces used in the structural design.

Table MD-15—Roughness Coefficients for Large Concrete Conduits

Type of Concrete Conduit	Roughness Coefficient
Precast concrete pipe, excellent joint alignment	0.012
Precast concrete pipe, ordinary joint alignment	0.013
Poured-in-place steel forms, projections 1/8” or less	0.013
Poured-in-place smooth wood forms, projections 1/8” or less	0.013
Poured-in-place ordinary work with steel forms	0.014
Poured-in-place ordinary work with wood forms	0.015

5.1.1 Entrance

Because a long rectangular conduit is costly, as well as for other reasons, the hydraulic characteristics at the entrance are particularly important. A conduit that cannot flow at the design discharge because of an inadequate or clogged inlet represents wasted investment and can result in flooding of homes, buildings, structures, and other urban infrastructure.

The entrances take on a special degree of importance for rectangular conduits, however, because the flow must be limited to an extent to ensure against overcharging the conduit. Special maximum-flow limiting entrances are often used with rectangular conduits. These special entrances should reject flow over the design discharge so that, if a runoff larger than the design flow occurs, the excess water will flow via other routes, often overland. A combined weir-orifice design is useful for this purpose. Model tests are needed for dependable design (Murphy 1971).

A second function of the entrance should be to accelerate the flow to the design velocity of the conduit, usually to meet the velocity requirements for normal depth of flow in the upstream reach of the conduit.

Air vents are needed at regular intervals to obviate both positive and negative pressures and to permit released entrained air to readily escape from the conduit.

5.1.2 Internal Pressure

The allowable internal pressure in a rectangular conduit is limited by structural design. Often, internal pressures are limited to no more than 2 to 4 feet of head before structural failure will commence, if structural design has not been based on internal pressure. Surges or conduit capacity choking cannot normally be tolerated.

5.1.3 Curves and Bends

The analysis of curves in rectangular conduits is critical to insure its hydraulic capacity. When water surface (normal, standing or reflecting waves) reaches the roof of the conduit hydraulic losses increase significantly and the capacity drops. Superelevation of the water surface must also be investigated, and allowances must be made for a changing hydraulic radius, particularly in high-velocity flow. Dynamic loads created by the curves must be analyzed to assure structural integrity for the maximum flows. See the HYDRAULIC STRUCTURES chapter of this *Manual*.

5.1.4 Transitions

Transitions provide complex hydraulic problems and require specialized analyses. Transitions, either contracting or expanding, are important with most large outfall conduits because of high-velocity flow. The development of shock waves that continue downstream can create significant problems in regard to proper conduit functioning. The best way to study transitions is through model tests (Fletcher and Grace 1972). Analytical procedures can only give approximate results. Poor transitions can cause upstream problems with both subcritical and supercritical flow, and can cause unnecessary flooding. Criteria given in the HYDRAULIC STRUCTURES chapter of this *Manual* may be used as a guide to certain limitations.

5.1.5 Air Entrainment

Entrained air causes a swell in the volume of water and an increase in depth than can cause flow in the conduit to reach the height of the roof with resulting loss of capacity; therefore, hydraulic design must account for entrained air. In rectangular conduits and circular pipes, flowing water will entrain air at velocities of about 20 ft/sec and higher. Additionally, other factors such as entrance condition, channel roughness, distance traveled, channel cross section, and volume of discharge all have some bearing on air entrainment. Volume swell can be as high as 20% (Hipschman 1970).

5.1.6 Major Inlets

Major inlets to a rectangular conduit at junctions or large storm inlets should receive a rigorous hydraulic analysis to assure against mainstream conduit flow striking the top of the rectangular conduit due to momentum changes in the main flow body as a result of the introduction of additional flow. Model tests may be necessary.

5.1.7 Sedimentation

The conduit must be designed to obviate sediment deposition problems during storm runoff events that have a frequency of occurrence of about twice each year. That is, at least twice per year, on average, the storm runoff velocity should be adequate to scour deposited sediment from the box section.

5.2 Appurtenances

The appurtenances to a long rectangular conduit are dictated by the individual needs of the particular project. Most appurtenances have some effect upon the overall operation of the system; the designer must consider all of these effects.

5.2.1 Energy Dissipators

Long conduits usually have high exit velocities that must be slowed to avoid downstream problems and damage. Energy dissipators are nearly always required. See the HYDRAULIC STRUCTURES chapter of this *Manual*.

5.2.2 Access Manholes

A long rectangular conduit should be easy to inspect, and, therefore, access manholes are desirable at various locations. If a rectangular conduit is situated under a curb, the access manholes may be combined with the storm sewer system inlets. Manholes should be aligned with the vertical wall of the box to allow rungs in the riser and box to be aligned.

Access manholes and storm inlets are useful for permitting air to flow in and out of a rectangular conduit as filling and emptying of the conduit occurs. They might also be considered safety water ejection ports should the conduit ever inadvertently flow full and cause a pileup of water upstream. The availability of such ejection ports could very well save a rectangular conduit from serious structural damage.

5.2.3 Vehicle Access Points

A large rectangular conduit with a special entrance and an energy dissipater at the exit may need an access hole for vehicle use in case major repair work becomes necessary. A vehicle access point might be a large, grated opening just downstream from the entrance. This grated opening can also serve as an effective air breather for the conduit. Vehicles may be lowered into the conduit by a crane or A-frame.

5.2.4 Safety

See discussion on public safety design consideration in the CULVERTS chapter.

5.2.5 Air Venting

Whenever it is suspected the conduit could operate at Froude Number higher than 0.7 during any flow that is at the design flow and flows lower than the design flow, or when the headwater at the conduits entrance is above the top of the conduit, the engineer has to consider installation of adequate air vents along the conduit. These are necessary to minimize major pressure fluctuations that can occur should the flow becomes unstable. When instabilities occur, air is trapped and less-than-atmospheric pressures have been shown to occur intermittently which air vents can mitigate and reduce structural loads and fluctuating hydraulic capacity in the conduit.

6.0 LARGE PIPES

Large pipes are often used as underground outfall conduits. An advantage of using pipes (circular conduits) rather than rectangular conduits is that pipes can withstand internal pressure to a greater degree than rectangular conduits can. Thus, the hydraulic design is not as critical, and a greater safety factor exists from the structural standpoint. Unless the designer is competent, experienced in open-channel hydraulics, and prepared to utilize laboratory model tests as a design aid, large pipes should be used rather than rectangular conduits. Cost differentials for the project should be carefully weighed before choosing the type of outfall conduit.

Disadvantages may include the fact that large pipes are less adaptable to an existing urban street where conflicts may exist with sanitary sewer pipes and other utilities.

6.1 Hydraulic Design

Large pipes are also considered as covered free-flow conduits; they are open channels with a cover (Steven, Simons, and Lewis 1971). Computational procedures for flow in large pipes are essentially the same as for canals and lined channels, except that consideration is given to diminishing capacity as the pipe flow nears the full depth.

Large pipes lend themselves to bends and slope changes more readily than do rectangular conduits. In a situation with a large pipe with the slope increasing in a downstream direction, there is no reason that the downstream pipe cannot be made smaller than the upstream pipe. However, the required transitional structure may rule out the smaller pipe from an economic standpoint. Improper necking down of large pipes has been a contributing factor in significant flooding of urban areas.

To aid in the solution of uniform flow computations for large pipes, see Table MD-16. The background and use of the table are similar to that given in Section 3.1.1 for open channels. Figures MD-2 and MD-3 are also useful aids for flow computations in pipes. Figure MD-20 is given as an additional design aid example. Curves presented in the STREETS/INLETS/STORM SEWERS and CULVERTS chapters of this *Manual* are also helpful in studying flow in large pipes.

Table MD-16—Uniform Flow in Circular Sections Flowing Partially Full

(Hipschman 1970)

y_0 = depth of flow
D = diameter of pipe
A = area of flow
R = hydraulic radius
Q = discharge in cfs by Manning formula
n = Manning coefficient
S_0 = slope of channel bottom and of the water surface

y_0/D	A/D^2	R/D	$Qn/(D^{8/3}S_0^{1/2})$	$Qn/(y_0^{8/3}S_0^{1/2})$	y_0/D	A/D^2	R/D	$Qn/(D^{8/3}S_0^{1/2})$	$Qn/(y_0^{8/3}S_0^{1/2})$
0.01	0.0013	0.0066	0.00007	15.040	0.51	0.4027	0.2531	0.23900	1.442
0.02	0.0037	0.0132	0.00031	10.570	0.52	0.4127	0.2562	0.24700	1.415
0.03	0.0069	0.0197	0.00074	8.560	0.53	0.4227	0.2592	0.25500	1.388
0.04	0.0105	0.0262	0.00138	7.380	0.54	0.4327	0.2621	0.26300	1.362
0.05	0.0147	0.0325	0.00222	6.550	0.55	0.4426	0.2649	0.27100	1.336
0.06	0.0192	0.0389	0.00328	5.950	0.56	0.4526	0.2676	0.27900	1.311
0.07	0.0242	0.0451	0.00455	5.470	0.57	0.4625	0.2703	0.28700	1.286
0.08	0.0294	0.0513	0.00604	5.090	0.58	0.4724	0.2728	0.29500	1.262
0.09	0.0350	0.0575	0.00775	4.760	0.59	0.4822	0.2753	0.30300	1.238
0.10	0.0409	0.0635	0.00967	4.490	0.60	0.4920	0.2776	0.31100	1.215
0.11	0.0470	0.0695	0.01181	4.250	0.61	0.5018	0.2799	0.31900	1.192
0.12	0.0534	0.0755	0.01417	4.040	0.62	0.5115	0.2821	0.32700	1.170
0.13	0.0600	0.0813	0.01674	3.860	0.63	0.5212	0.2842	0.33500	1.148
0.14	0.0668	0.0871	0.01952	3.690	0.64	0.5308	0.2862	0.34300	1.126
0.15	0.0739	0.0929	0.02250	3.540	0.65	0.5404	0.2882	0.35000	1.105
0.16	0.0811	0.0985	0.02570	3.410	0.66	0.5499	0.2900	0.35800	1.084
0.17	0.0885	0.1042	0.02910	3.280	0.67	0.5594	0.2917	0.36600	1.064
0.18	0.0961	0.1097	0.03270	3.170	0.68	0.5687	0.2933	0.37300	1.044
0.19	0.1039	0.1152	0.03650	3.060	0.69	0.5780	0.2948	0.38000	1.024
0.20	0.1118	0.1206	0.04060	2.960	0.70	0.5872	0.2962	0.38800	1.004
0.21	0.1199	0.1259	0.04480	2.870	0.71	0.5964	0.2975	0.39500	0.985
0.22	0.1281	0.1312	0.04920	2.790	0.72	0.6054	0.2987	0.40200	0.965
0.23	0.1365	0.1364	0.05370	2.710	0.73	0.6143	0.2998	0.40900	0.947
0.24	0.1449	0.1416	0.05850	2.630	0.74	0.6231	0.3008	0.41600	0.928
0.25	0.1535	0.1466	0.06340	2.560	0.75	0.6319	0.3017	0.42200	0.910
0.26	0.1623	0.1516	0.06860	2.490	0.76	0.6405	0.3024	0.42900	0.891
0.27	0.1711	0.1566	0.07390	2.420	0.77	0.6489	0.3031	0.43500	0.873
0.28	0.1800	0.1614	0.07930	2.360	0.78	0.6573	0.3036	0.44100	0.856
0.29	0.1890	0.1662	0.08490	2.300	0.79	0.6655	0.3039	0.44700	0.838
0.30	0.1982	0.1709	0.09070	2.250	0.80	0.6736	0.3042	0.45300	0.821
0.31	0.2074	0.1756	0.09660	2.200	0.81	0.6815	0.3043	0.45800	0.804
0.32	0.2167	0.1802	0.10270	2.140	0.82	0.6893	0.3043	0.46300	0.787
0.33	0.2260	0.1847	0.10890	2.090	0.83	0.6969	0.3041	0.46800	0.770
0.34	0.2355	0.1891	0.11530	2.050	0.84	0.7043	0.3038	0.47300	0.753
0.35	0.2450	0.1935	0.12180	2.000	0.85	0.7115	0.3033	0.47700	0.736
0.36	0.2546	0.1978	0.12840	1.958	0.86	0.7186	0.3026	0.48100	0.720
0.37	0.2642	0.2020	0.13510	1.915	0.87	0.7254	0.3018	0.48500	0.703
0.38	0.2739	0.2062	0.14200	1.875	0.88	0.7320	0.3007	0.48800	0.687
0.39	0.2836	0.2102	0.14900	1.835	0.89	0.7384	0.2995	0.49100	0.670
0.40	0.2934	0.2142	0.15610	1.797	0.90	0.7445	0.2980	0.49400	0.654
0.41	0.3032	0.2182	0.16330	1.760	0.91	0.7504	0.2963	0.49600	0.637
0.42	0.3130	0.2220	0.17050	1.724	0.92	0.7560	0.2944	0.49700	0.621
0.43	0.3229	0.2258	0.17790	1.689	0.93	0.7612	0.2921	0.49800	0.604
0.44	0.3328	0.2295	0.18540	1.655	0.94	0.7662	0.2895	0.49800	0.588
0.45	0.3428	0.2331	0.19290	1.622	0.95	0.7707	0.2865	0.49800	0.571
0.46	0.3527	0.2366	0.20100	1.590	0.96	0.7749	0.2829	0.49600	0.553
0.47	0.3627	0.2401	0.20800	1.559	0.97	0.7785	0.2787	0.49400	0.535
0.48	0.3727	0.2435	0.21600	1.530	0.98	0.7817	0.2735	0.49800	0.517
0.49	0.3827	0.2468	0.22400	1.500	0.99	0.7841	0.2666	0.48300	0.496
0.50	0.3927	0.2500	0.23200	1.471	1.00	0.7854	0.2500	0.46300	0.463

6.1.1 Entrance

The longer a pipe is, the more important is design of the entrance. A large pipe unable to flow at the design capacity represents wasted investment. Acceleration of flow, typically to the design velocity of the pipe reach immediately downstream, is often an important characteristic of the entrance. Typically air vents are necessary immediately downstream of the entrance to allow entrained air to escape and to act as breathers should less-than-atmospheric pressures develop in the pipe. Long pipes that depend on flow entering at upstream points other than street inlets need to be equipped with adequately sized safety/trash racks at the entrances. For guidance on sizing safety/trash racks, see guidance in the CULVERTS chapter.

6.1.2 Internal Pressure

The allowable internal pressure is limited by the structural design of the pipe; however, it is not as critical as with rectangular conduits, with up to perhaps 25 feet of head being permissible in some pipe designs before failure commences. It is evident, however, that large pipe outfalls cannot be designed for flow under any significant pressure because then inflow from other lines could not enter, and water would flow out of storm inlets rather than into these inlets. The internal pressure aspect is important only as a safety factor in the event of a choking of capacity or an inadvertent flow surcharge.

6.1.3 Curves and Bends

Curves and bends are permitted, but detailed analysis is required to ensure structural integrity and proper hydraulic functioning of the conduit. Maintenance access should be provided in the proximity of all bends. Hydraulic analyses are important at locations where hydraulic jumps may occur.

6.1.4 Transitions

Transitions are discussed in the HYDRAULIC STRUCTURES chapter of this *Manual*.

6.1.5 Air Entrainment and Venting

The reader is referred to Sections 5.1.5 and 5.2.5 of this chapter.

6.1.6 Major Inlets

Inflow to the conduit can cause unanticipated hydraulic variations; however, the analytical approach need not be as rigorous as with rectangular conduits.

6.2 Appurtenances

The reader is referred to Section 5.2 of this chapter.

6.3 Safety

See guidance in the CULVERTS chapter.

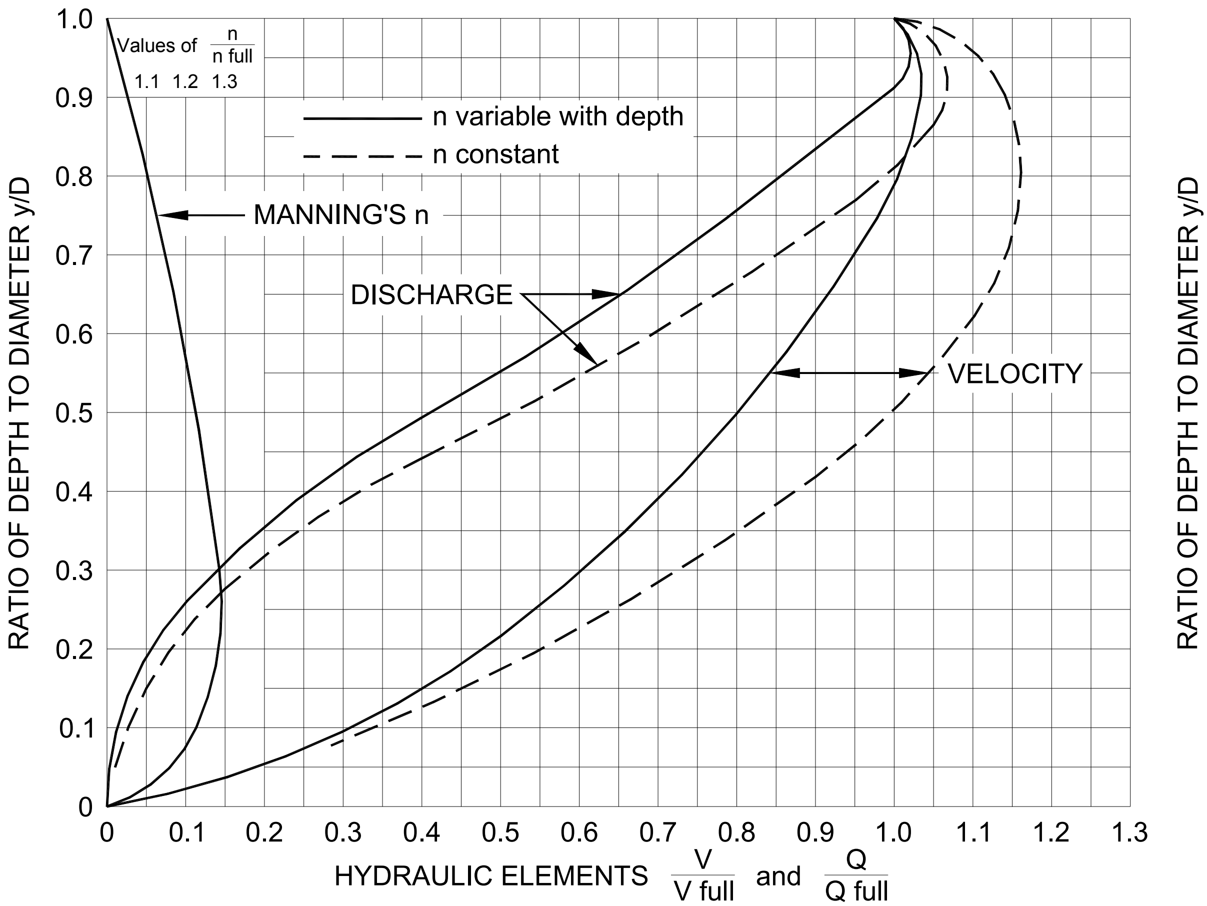


Figure MD-20—Hydraulic Properties of Pipes

(Steven, Simons, and Lewis 1976)

7.0 PROTECTION DOWNSTREAM OF PIPE OUTLETS

This section is intended to address the use of riprap for erosion protection downstream of conduit and culvert outlets that are in-line with major drainageway channels. Inadequate protection at conduit and culvert outlets has long been a major problem. The designer should refer to Section 4.4 for additional information on major drainage applications utilizing riprap. In addition, the criteria and guidance in Section 4.4 may be useful in design of erosion protection for conduit outlets. The reader is referred to Section 7.0 of the HYDRAULIC STRUCTURES chapter of this *Manual* for information on rundowns, and to Section 3.0 of the HYDRAULIC STRUCTURES chapter for additional discussion on culvert outfall protection.

Scour resulting from highly turbulent, rapidly decelerating flow is a common problem at conduit outlets. The riprap protection design protocol is suggested for conduit and culvert outlet Froude numbers up to 2.5 (i.e., Froude parameters $Q/d_0^{2.5}$ or $Q/WH^{1.5}$ up to 14 ft$^{0.5}$/sec) where the channel and conduit slopes are parallel with the channel gradient and the conduit outlet invert is flush with the riprap channel protection. Here, Q is the discharge in cfs, d_0 is the diameter of a circular conduit in feet and W and H are the width and height, respectively, of a rectangular conduit in feet.

7.1 Configuration of Riprap Protection

Figure MD-25 illustrates typical riprap protection of culverts and major drainageway conduit outlets. The additional thickness of the riprap just downstream from the outlet is to assure protection from flow conditions that might precipitate rock movement in this region.

7.2 Required Rock Size

The required rock size may be selected from Figure MD-21 for circular conduits and from Figure MD-22 for rectangular conduits. Figure MD-21 is valid for $Q/D_c^{2.5}$ of 6 or less and Figure MD-22 is valid for $Q/WH^{1.5}$ of 8.0 or less. The parameters in these two figures are:

1. $Q/D^{1.5}$ or $Q/WH^{0.5}$ in which Q is the design discharge in cfs, D_c is the diameter of a circular conduit in feet, and W and H are the width and height of a rectangular conduit in feet.

2. Y_t/D_c or Y_t/H in which Y_t is the tailwater depth in feet, D_c is the diameter of a circular conduit in feet, and H is the height of a rectangular conduit in feet. In cases where Y_t is unknown or a hydraulic jump is suspected downstream of the outlet, use $Y_t/D_t = Y_t/H = 0.40$ when using Figures MD-21 and MD-22.

3. The riprap size requirements in Figures MD-21 and MD-22 are based on the non-dimensional parametric Equations MD-18 and MD-19 (Steven, Simons, and Lewis 1971 and Smith 1975).

   Circular culvert:

$$\frac{\left(\frac{d_{50}}{D_c}\right)\left(\frac{Y_t}{D_c}\right)^{1.2}}{\left(\frac{Q}{D_c^{2.5}}\right)} = 0.023 \tag{MD-18}$$

   Rectangular culvert:

$$\frac{\left(\frac{d_{50}}{H}\right)\left(\frac{Y_t}{H}\right)}{\left(\frac{Q}{WH^{1.5}}\right)} = 0.014 \tag{MD-19}$$

The rock size requirements were determined assuming that the flow in the culvert barrel is not supercritical. It is possible to use Equations MD-18 and MD-19 when the flow in the culvert is supercritical (and less than full) if the value of D_c or H is modified for use in Figures MD-21 and MD-22. Whenever the flow is supercritical in the culvert, substitute D_a for D_c and H_a for H, in which D_a is defined as:

$$D_a = \frac{(D_c + Y_n)}{2} \tag{MD-20}$$

in which the maximum value of D_a shall not exceed D, and

$$H_a = \frac{(H + Y_n)}{2} \tag{MD-21}$$

in which the maximum value of H_a shall not exceed H, and:

D_a = parameter to use in place of D in Figure MD-21 when flow is supercritical

D_c = diameter of circular culvert (ft)

H_a = parameter to use in place of H in Figure MD-22 when flow is supercritical

H = height of rectangular culvert (ft)

Y_n = normal depth of supercritical flow in the culvert

7.3 Extent of Protection

The length of the riprap protection downstream from the outlet depends on the degree of protection desired. If it is necessary to prevent all erosion, the riprap must be continued until the velocity has been reduced to an acceptable value. For purposes of outlet protection during major floods, the acceptable velocity is set at 5.5 ft/sec for very erosive soils and at 7.7 ft/sec for erosion resistant soils. The rate at which the velocity of a jet from a conduit outlet decreases is not well known. For the procedure recommended here, it is assumed to be related to the angle of lateral expansion, θ, of the jet. The velocity is related to the expansion factor, $(1/(2\tan\theta))$, which can be determined directly using Figure MD-23 or Figure MD-24, assuming that the expanding jet has a rectangular shape:

$$L_p = \left(\frac{1}{2\tan\theta}\right)\left(\frac{A_t}{Y_t} - W\right) \qquad \text{(MD-22)}$$

where:

L_p = length of protection (ft)

W = width of the conduit in (ft) (use diameter for circular conduits)

Y_t = tailwater depth (ft)

θ = the expansion angle of the culvert flow

and:

$$A_t = \frac{Q}{V} \qquad \text{(MD-23)}$$

where:

Q = design discharge (cfs)

V = the allowable non-eroding velocity in the downstream channel (ft/sec)

A_t = required area of flow at allowable velocity (ft²)

In certain circumstances, Equation MD-22 may yield unreasonable results. Therefore, in no case should L_p be less than $3H$ or $3D$, nor does L_p need to be greater than $10H$ or $10D$ whenever the Froude parameter, $Q/WH^{1.5}$ or $Q/D^{2.5}$, is less than 8.0 or 6.0, respectively. Whenever the Froude parameter is greater than these maximums, increase the maximum L_p required by ¼ D_c or ¼ H for circular or rectangular culverts, respectively, for each whole number by which the Froude parameter is greater than 8.0 or 6.0, respectively.

7.4 Multiple Conduit Installations

The procedures outlined in Sections 7.1, 7.2, and 7.3 can be used to design outlet erosion protection for multi-barrel culvert installations by hypothetically replacing the multiple barrels with a single hydraulically equivalent rectangular conduit. The dimensions of the equivalent conduit may be established as follows:

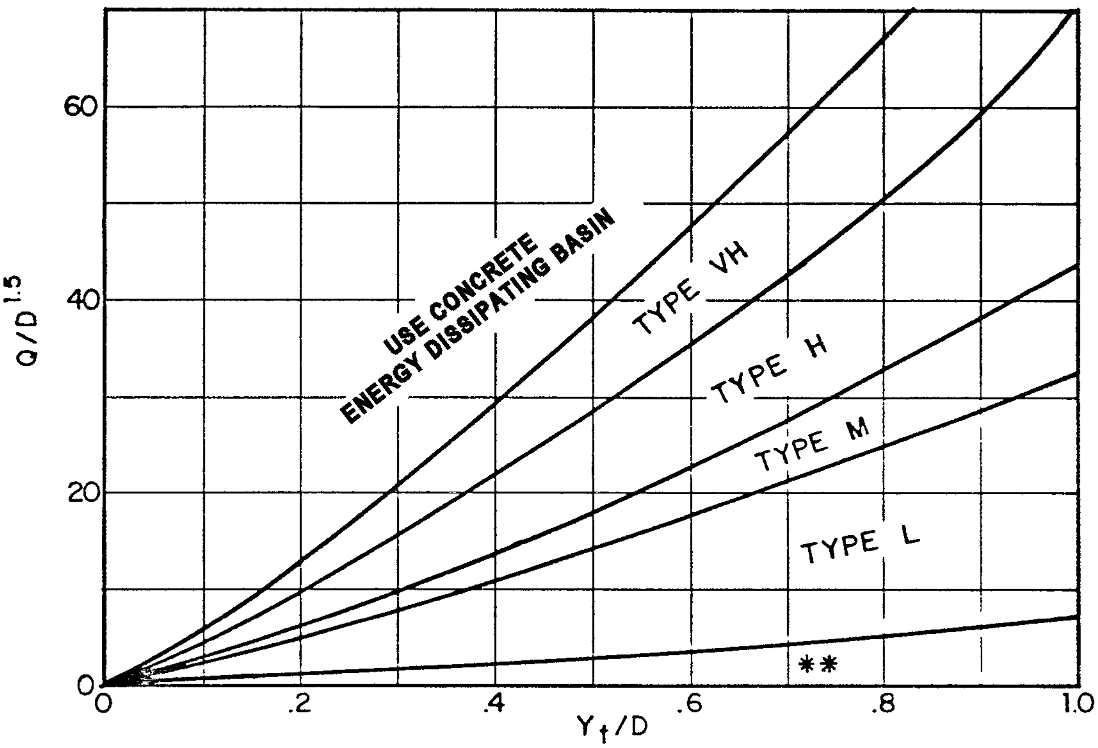
1. Distribute the total discharge, Q, among the individual conduits. Where all the conduits are hydraulically similar and identically situated, the flow can be assumed to be equally distributed; otherwise, the flow through each barrel must be computed.

2. Compute the Froude parameter $Q_i/D_{ci}^{2.5}$ (circular conduit) or $Q_i/W_iH_i^{1.5}$ (rectangular conduit), where the subscript i indicates the discharge and dimensions associated with an individual conduit.

3. If the installation includes dissimilar conduits, select the conduit with the largest value of the Froude parameter to determine the dimensions of the equivalent conduit.

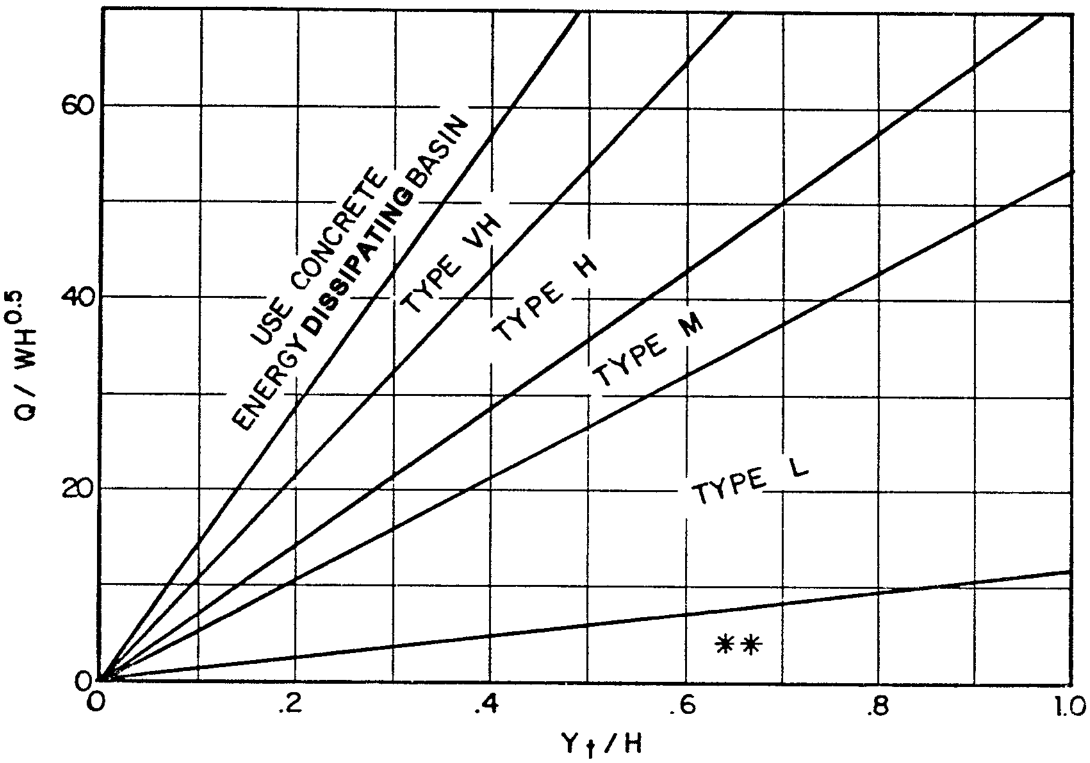
4. Make the height of the equivalent conduit, H_{eq}, equal to the height, or diameter, of the selected individual conduit.

5. The width of the equivalent conduit, W_{eq}, is determined by equating the Froude parameter from the selected individual conduit with the Froude parameter associated with the equivalent conduit, $Q/W_iH_{eq}^{1.5}$.



Use D_a instead of D whenever flow is supercritical in the barrel.
** Use Type L for a distance of 3D downstream.

Figure MD-21—Riprap Erosion Protection at Circular Conduit Outlet Valid for $Q/D^{2.5} \leq 6.0$



Use H_a instead of H whenever culvert has supercritical flow in the barrel.
**Use Type L for a distance of 3H downstream.

Figure MD-22—Riprap Erosion Protection at Rectangular Conduit Outlet Valid for $Q/WH^{1.5} \leq 8.0$

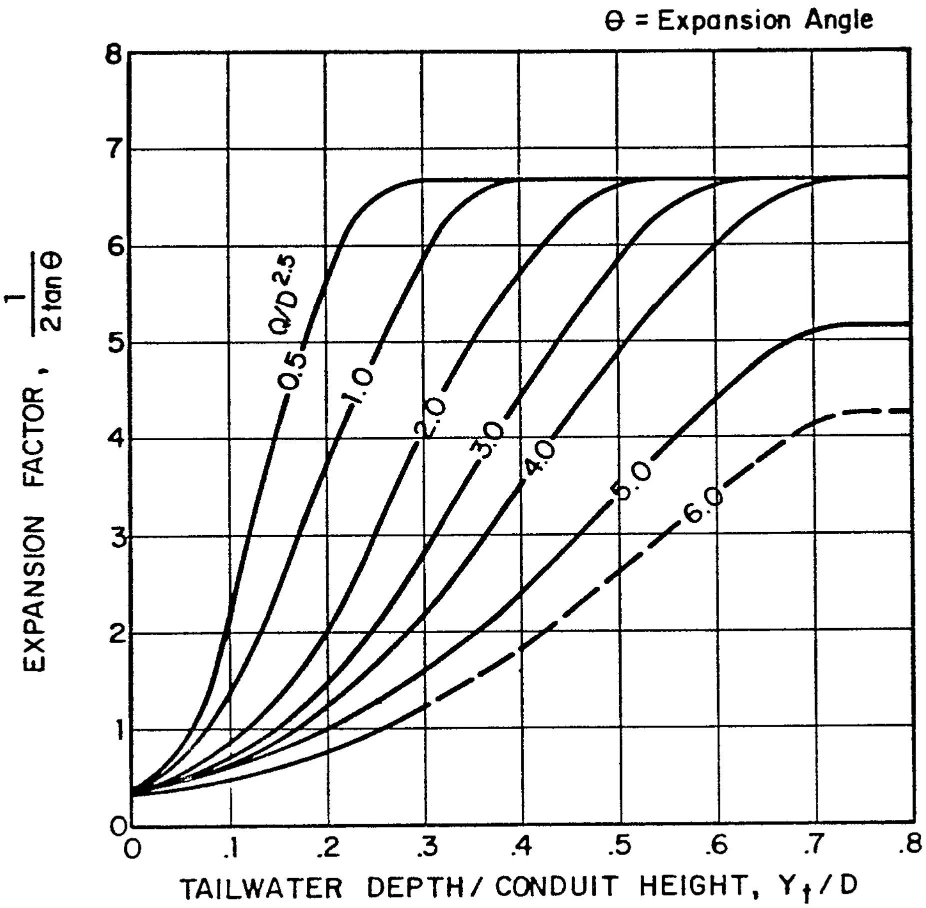


Figure MD-23—Expansion Factor for Circular Conduits

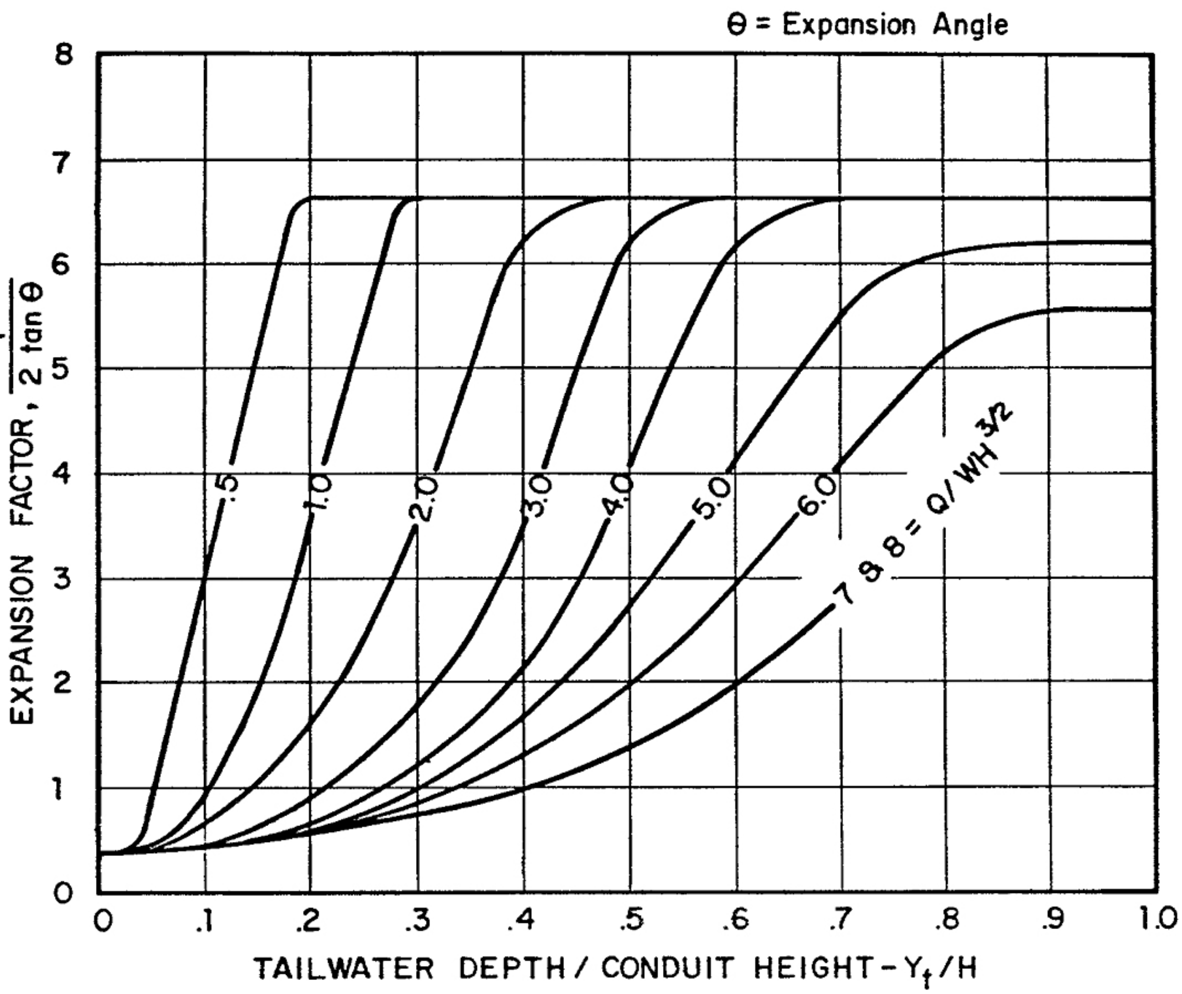
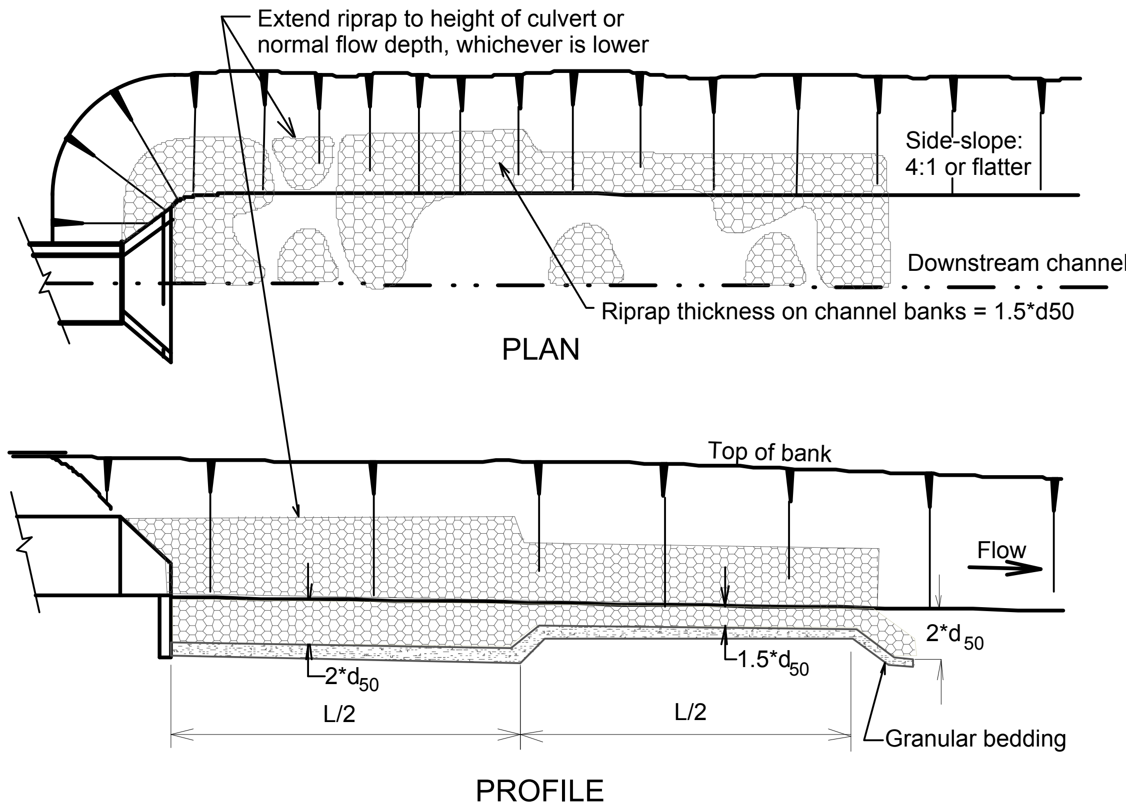


Figure MD-24—Expansion Factor for Rectangular Conduits



NOTES: 1. Headwall with wingwalls or flared end section required at all culvert outlets.
2. Cutoff wall required at end of wingwall aprons and end section.
Minimum depth of cutoff wall = 2*d50 or 3-feet, whichever is deeper.
3. Provide joint fasteners for flared end sections.

Figure MD-25—Culvert and Pipe Outlet Erosion Protection

8.0 SEDIMENT

Well-established urban areas are not significant sediment producers. However, winter sanding operations, new construction areas, and usual residential storm runoff will provide some sediment to the drainage system, which must be acknowledged. One of the greatest sedimentation problems occurs, however, when an area is undergoing urbanization. Furthermore, in a grass-lined channel or a natural channel, erosion will typically occur in some reaches of the channel and sediment will generally deposit in other reaches. Sedimentation is a problem in urban drainage hydrology in that, if the channel is made steep enough to transport all sediment, the velocities will also be high enough to cause erosion that would not otherwise occur if the channel was flatter. Often the designer must make the choice to have a well-planned and designed channel which will transport the minimal sediment yield in the future, realizing that the initial operation of the channel will result in sediment deposition during the process of urbanization.

The designer would be well advised to give full consideration to the sediment deposition problem and to utilize sediment deposition basins at selected locations along channels and at stormwater runoff entry points into channels for periodic sediment removal when it is obvious that there will be substantial sediment inflow, at least initially. In addition, the designer can include sediment storage and trap areas within flood detention basins and retention ponds to great advantage. See the chapter on STORAGE in this *Manual*.

In a grass-lined channel, particularly after the grass has obtained maturity, fine sediment will settle out regularly on top of the sod. Over a period of years, there will be a gradual buildup of the channel bottom, many times imperceptible, but nonetheless occurring. Because of the frequent use of drops in grass-lined channels as well as natural channels, the build-up rate will decrease with time. However, if aggradation tends to reduce the capacity of the channel, periodic restorative maintenance work will need to be performed to re-establish the design depth.

The subject of sedimentation design cannot be completely covered in this *Manual* because of its complexity. Volume 3 of the *Manual* addresses suspended sediment in greater detail, but little guidance is given for bedload since its presence is dependant on many factors (i.e., construction activities upstream, channel bank line erosion, channel bed degradation, use of erosion control practices, etc.). As a rule of thumb, velocities of 3.0 ft/sec will transport sediments up to the size of fine sands. However, being able to achieve these velocities during minor runoff events throughout the channel’s cross section may not be feasible.

9.0 EXAMPLES

9.1 Example MD-1: Normal Depth Calculation with Normal Worksheet

This example involves determination of channel capacity and other relevant hydraulic parameters for a grass-lined trapezoidal channel flowing at normal depth, given the following channel characteristics and constraints:

Channel Characteristics:

So = channel bottom slope (longitudinal slope) = 0.3%

Z = Z1 = Z2 = channel side slopes (left and right) = 4H:1V

n = Manning's n (grass-lined channel) = 0.035

B = bottom width = 10 ft

Constraints:

Y = maximum allowable depth of flow in channel = 5.0 ft

F = freeboard required = 2.0 ft

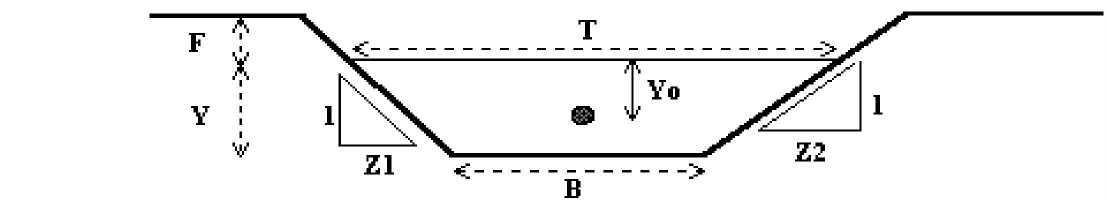
A sketch of the channel cross section, which defines these parameters, is included as a part of the worksheet and is illustrated for this example on the calculation sheet on the following page.

These channel characteristics and sizing constraints are entered into the input section of the *Basic* worksheet of the UD-Channels Spreadsheet to determine discharge using normal depth calculations. A worksheet demonstrating application of the *Basic Worksheet* titled "Normal Flow Analysis—Trapezoidal Channel" for "Project = Example MD-1" and "Channel ID = Normal Depth Example" is provided as an example of normal depth analysis.

Based on this analysis, the channel would be capable of carrying a flow of approximately 700 cfs, given a total bank height of 7 ft to allow for the required freeboard. In addition, the calculations indicate that flow will be subcritical under these conditions. Since the velocity is close to the 5.0 ft/sec maximum allowable 100-year velocity for grass-lined channels with erosive soils, the spreadsheet should be reapplied using a lower Manning's n value to see if the maximum velocity criterion is exceeded.

Normal Flow Analysis - Trapezoidal Channel

Project: **Example MD-1**
Channel ID: **Normal Depth Exampl**



Design Information (Input)			
Channel Invert Slope	So =	0.003	ft/ft
Manning's n	n =	0.035	
Bottom Width	B =	10	ft
Left Side Slope	Z1 =	4	ft/ft
Right Side Slope	Z2 =	4	ft/ft
Freeboard Height	F =	2	ft
Design Water Depth	Y =	5	ft
Normal Flow Condtion (Calculated)			
Discharge	**Q =**	**715.83**	cfs
Froude Number	**Fr =**	**0.49**	
Flow Velocity	**V =**	**4.77**	fps
Flow Area	A =	150.00	sq ft
Top Width	T =	50.00	ft
Wetted Perimeter	P =	51.23	ft
Hydraulic Radius	R =	2.93	ft
Hydraulic Depth	D =	3.00	ft
Specific Energy	Es =	5.35	ft
Centroid of Flow Area	Yo =	1.93	ft
Specific Force	Fs =	24.72	kip

9.2 Example MD-2: Composite Section Calculations Using Composite Design Worksheet

This example involves calculation of channel cross-section geometry parameters for a composite channel consisting of a low-flow channel with side slope protection for conveyance of frequent flows (up to 2-year) and vegetated overbanks to accommodate larger runoff events (up to the 100-year event). In this case, criteria for a grass-lined composite channel with side slope protection for the low-flow channel are applied for sizing. The channel sizing is based on hydraulic design parameters including:

Q-2yr = 2-year discharge = 600 cfs

Q-100yr = 100-year discharge (fully-developed, un-detained condition) = 3000 cfs

Qlf = design discharge for low flow channel = 300 cfs

Z1 = low flow channel left side slope = 3H:1V

Z2 = low flow channel right side slope = 3H:1V

Ym = low flow channel bank-full depth = 3 ft

ZL = left overbank side slope = 4H:1V

N-left = left overbank Manning's n = 0.040

ZR = right overbank side slope = 4H:1V

N-right = right overbank Manning's n = 0.040

Yob = overbank flow depth = 3.0 ft

Soil type = sandy

Left overbank width as a percentage of total overbank width = 50%

A sketch of the channel cross section, which defines these parameters, is included as a part of the worksheet and is illustrated for this example on the calculation sheet on the following page.

These hydraulic parameters are entered into the input section of the **Composite Design Worksheet** of the UD-Channels Spreadsheet to determine low flow, overbank, and composite channel characteristics for the low-flow design discharge and the 100-year discharge. A worksheet demonstrating application of the **Composite Design Worksheet** titled "Design of Composite Channel" for "Project = Example MD-2" and "Channel ID = Composite Channel Example" is provided as an example of this calculation tool.

The analysis demonstrates that a channel with the characteristics specified above, an invert slope of 0.49%, a 3-foot-deep low-flow channel with a bottom width of 20.7 ft, evenly distributed overbank benches to the left and right of the low-flow channel with width = 38.6 ft, and a total top width of 139.9 ft will meet the following design criteria:

1. The low-flow channel has the capacity to convey between ⅓ to ½ of the 2-year flow at a depth not exceeding 3 ft.

2. Flow is subcritical for all flow conditions evaluated, and $Fr < 0.8$, thereby satisfying Froude number criterion for non-erosive soils.

3. Longitudinal channel slope ≥ 0.2% and ≤ 0.6%.

4. Maximum depth of flow outside of low flow channel < 5.0 ft.

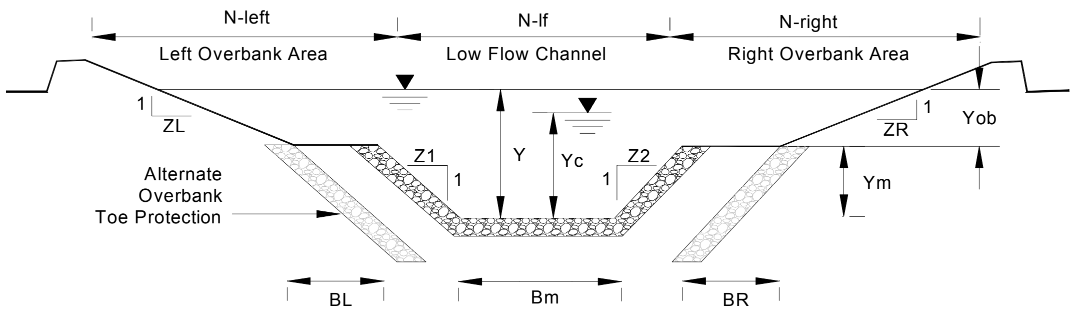
5. Composite cross section 100-year velocity < 7.0 ft/sec (non-erosive soils).

6. 4H:1V side slopes permit maintenance of vegetated banks.

Design of Composite Channel

Project: Example MD-2

Channel ID: Composite Section Calculations Using Composite Design Worksheet



Design Information (Input)

2-Year Discharge - Total	Q-2yr =	600 cfs				
100-Year Discharge - Total	Q-100yr =	3,000 cfs	**Check one of the following toe protection types**			
Design Discharge - Low Flow Channel	Qlf =	300 cfs		Low Flow Channel Sideslope Protection	X	check, OR
Low Flow Channel Left Side Slope	Z1 =	3.0 ft/ft		Overbank Toe Protection		check
Low Flow Channel Right Side Slope	Z2 =	3.0 ft/ft				
Low Flow Channel Bank-full depth	Ym =	3.00 ft		Left overbank width as a		
Left Overbank Side Slope	ZL =	4.0 ft/ft		percentage of total overbank width	50	%
Left Overbank Manning's n	n-left =	0.0400				
Right Overbank Side Slope	ZR =	4.0 ft/ft	**Check one of the following soil types**			
Right Overbank Manning's n	n-right =	0.0400		Sandy Soil	X	check, OR
Overbank Flow Depth Yob (Y - Ym)	Yob =	3.00 ft		Non-Sandy Soil		check

Flow Condition (Calculated)

Channel Invert Slope **So = 0.0049 ft/ft**

Low Flow Channel Condition for Qd			**Low Flow Channel Flow Condition for Q100**		
Channel Bottom Width	Blf =	20.7 ft	Low Flow Channel Bottom Width	Bm =	20.7 ft
Channel Normal Flow Depth	Ylf =	3.00 ft	Top width	Tm =	38.7 ft
Top width	Tlf =	38.7 ft	Flow area	Am =	204.9 sq ft
Flow area	Alf =	89.0 sq ft	Wetted perimeter	Pm =	39.6 ft
Wetted perimeter	Plf =	39.6 ft	Manning's n (Calculated)	n-m =	0.0386
Manning's n (Calculated)	n-lf =	0.0534	**Discharge**	**Qm =**	**1,667 cfs**
Discharge (Calculated)	**Qlf =**	**300 cfs**	Velocity	Vm =	8.1 fps
Velocity	Vlf =	3.4 fps	**Froude number**	**Frm =**	**0.62**
Froude number	Fr-lf =	0.39	100-Yr. Critical Velocity	Vmc =	11.2 fps
			100-Yr. Critical Depth	Ymc =	4.6 ft

Left Overbank Flow Condition for Q100			**Right Overbank Flow Condition for Q100**		
Overbank Bench Width	BL =	38.6 ft	Overbank Bench Width	BR =	38.6 ft
Normal Depth in Overbanks	YLob =	3.0 ft	Normal Depth in Overbanks	YRob =	3.0 ft
Top width	TL =	50.6 ft	Top width	TR =	50.6 ft
Flow area	AL =	133.7 sq ft	Flow area	AR =	133.7 sq ft
Wetted perimeter	PL =	50.9 ft	Wetted perimeter	PR =	50.9 ft
Discharge	**QL =**	**668 cfs**	**Discharge**	**QR =**	**668 cfs**
Velocity	**VL =**	**5.0 fps**	**Velocity**	**VR =**	**5.0 fps**
Froude number	**FrL =**	**0.54**	**Froude number**	**FrR =**	**0.54**
100-Yr. Critical Velocity	VLc =	7.7 fps	100-Yr. Critical Velocity	VRc =	7.7 fps
100-Yr. Critical Depth in Overbanks	YLc =	2.0 ft	100-Yr. Critical Depth in Overbanks	YRc =	2.0 ft

Composite Cross-Section Flow Condition for Q100

Top width	**T =**	**139.8 ft**	**Discharge**	**Q =**	**3,002 cfs**
Channel Depth Y	**Y =**	**6.00 ft**	**Velocity (average)**	**V =**	**6.4 fps**
Flow area	A =	472.2 sq ft	Froude number	Fr =	0.61
Wetted perimeter	P =	141.5 ft	100-Yr. Critical Velocity	Vc =	9.0 fps
Cross-Sectional Manning's n (Calculated)	n =	0.0392	100-Yr. Critical Depth in Overbanks	Yc =	1.97 ft

NOTE:

The sum of QL + QR + Qm will slightly overestimate the total composite channel discharge, and will not equal Q. These element values are used, however, to estimate critical velocity and critical depth for design purposes.

9.3 Example MD-3: Riprap Lined Channel Calculations Using Riprap Channel Worksheet

This example demonstrates application of the **Riprap Worksheet** of the UD-Channels Spreadsheet to determine riprap sizing for a trapezoidal channel. The worksheet calculates a riprap sizing parameter based on Equation MD-13, with adjustments for channel curvature, to determine the riprap type required for the channel lining. Calculations are based on the following channel characteristics provided by the user:

So = channel invert slope = 0.010 ft/ft

B = bottom width = 30.0 ft

Z1 = left side slope = 2.5H:1V

Z2 = right side slope = 2.5H:1V

Ss = specific gravity of rock = 2.5

Ccr = radius of channel centerline = 200 ft

Q = design discharge = 2500 cfs

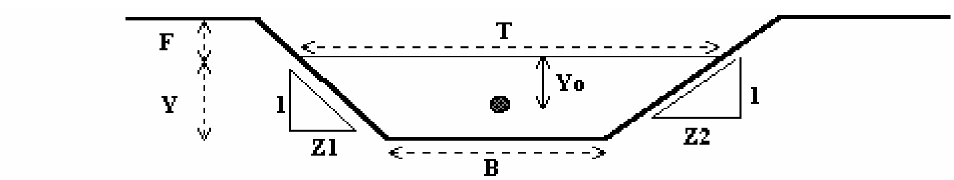
A sketch of the channel cross section, which defines these parameters, is included as a part of the worksheet and is illustrated for this example on the calculation sheet on the following page.

These parameters are entered into the input section of the **Riprap Worksheet** of the UD-Channels Spreadsheet to determine riprap type and channel hydraulic characteristics including Manning's *n*, the Froude number, velocity, and superelevation. A worksheet demonstrating application of the **Riprap Worksheet** titled "Design of Riprap Channel Cross Section" for "Project = Example MD-3" and "Channel ID = Riprap Channel Example" is provided as an example of this calculation tool.

Based on this analysis, Type H riprap is suitable for straight and curved sections of the channel and will meet the minimum K factor requirements. Calculations indicate that flow will be subcritical for the design discharge and that the Froude number is less than the maximum Froude number criterion for riprap channels of 0.8. Calculations also indicate that superelevation is not expected as a result of the channel curvature.

Design of Riprap Channel Cross Section

Project: Example MD-3
Channel ID: Rirap Channel Example



Design Information (Input)			
Channel Invert Slope	So =	0.0100	ft/ft
Bottom Width	B =	30.0	ft
Left Side Slope	Z1 =	2.5	ft/ft
Right Side Slope	Z2 =	2.5	ft/ft
Specific Gravity of Rock	Ss =	2.50	
Radius of Channel Centerline	Ccr =	200.0	ft
Design Disharge	**Q =**	**2,500.0**	**cfs**

Flow Condition (Calculated)			
Riprap Type (Straight Channel)	**Type =**	**H**	
Intermediate Rock Diameter (Straight Channel)	**D50 =**	**18**	**inches**
Calculated Manning's n (Straight Channel)	**n =**	**0.0423**	
Riprap Type (Outside Bend of Curved Channel)	**Type =**	**H**	
Intermediate Rock Dia. (O.B. of Curved Channel)	**D50 =**	**18**	**inches**
Calculated Manning's N (Curved Channel)	**n =**	**0.0423**	
Water Depth	Y =	5.97	ft
Top Width of Flow	T =	59.8	ft
Flow Area	A =	268.2	sq ft
Wetted Perimeter	P =	62.1	ft
Hydraulic Radius (A/P)	R =	4.3	ft
Average Flow Velocity (Q/A)	V =	9.3	fps
Hydraulic Depth (A/T)	D =	4.5	ft
Froude Number (max. = 0.8)	Fr =	0.78	
Channel Radius / Top Width	Ccr/T =	3.34	
Riprap Design Velocity Factor For Curved Channel	Kv =	1.69	
Riprap Sizing Velocity For Curved Channel	V_{Kv} =	15.8	fps
Riprap Sizing Paramenter for Straight Channel	**K =**	**3.27**	
Riprap Sizing Paramenter for Outside Bend of Curve	**K_{curve} =**	**5.51**	
*** Superelevation (dh)	dh =	0.41	ft
Discharge (Check)	**Q =**	**2,506.4**	**cfs**

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HYDRAULIC STRUCTURES

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1.0 USE OF STRUCTURES IN DRAINAGE

1.1 Introduction

Hydraulic structures are used to guide and control water flow velocities, directions and depths, the elevation and slope of the streambed, the general configuration of the waterway, and its stability and maintenance characteristics.

Careful and thorough hydraulic engineering is justified for hydraulic structures. Consideration of environmental, ecological, and public safety objectives should be integrated with hydraulic engineering design. The proper application of hydraulic structures can reduce initial and future maintenance costs by managing the character of the flow to fit the environmental and project needs.

Photograph HS-1—Denver's Harvard Gulch Flood Control Project introduced the baffle chute drop structure to urban flood control in 1966. Vegetation and time have made the structure part of the city's urban poetry.

Hydraulic structures include transitions, constrictions, channel drops, low-flow checks, energy dissipators, bridges, bends, and confluences. Their shape, size, and other features vary widely for different projects, depending upon the discharge and the function to be accomplished. Hydraulic design procedures must govern the final design of all structures. These may include model testing for larger structures when the proposed design requires a configuration that differs significantly from known documented guidelines or when questions arise over the character of the structure being considered.

This chapter deals with structures for drainage and flood control channels, in contrast to dam spillways or specialized conveyance systems. Specific guidance is given on drop structures for channels that match the District's guidelines for grass-lined and riprap-lined channels as given in the MAJOR DRAINAGE chapter of this *Manual*. In addition, guidance is provided for the design of energy dissipaters at conduit outlets. Sections on bridges, transitions, and constrictions primarily refer to other sources for more extensive design information.

Photograph HS-2—The Clear Creek I-25 vertical concrete drop structure was a "drowning machine" until it was retrofitted by CDOT with a 10:1 downstream face. (Photograph taken before retrofit.)

1.2 Channels Used for Boating

There are streams in the District in which rafting, canoeing, kayaking, and other water-based recreational activities occur. Design and construction of hydraulic structures in these waterways require a standard of care consistent with common sense safety concerns for the public that uses them. The ultimate responsibility for individual safety still resides with the boating public and their prudent use of urban waterways.

It is reasonable to retain a whitewater boating specialist to assist in the design criteria for a hydraulic structure on a boatable stream. In particular, reverse rollers are to be avoided (USACE 1985).

1.3 Channel Grade Control Structures

Grade control structures, such as check structures and drop structures, provide for energy dissipation and thereby result in a mild slope in the upstream channel reaches. The geometry at the crest of these structures can effectively control the upstream channel stability and, to an extent, its ultimate configuration.

A drop structure traverses the entire waterway, including the portion that carries the major flood. A check structure is similar, but is constructed to stabilize the low-flow channel (i.e., one carrying the minor or lesser flood) in artificial or natural drainageways. It crosses only the low-flow portion of the waterway or floodplain. During a major flood, portions of the flow will circumvent the check. Overall channel stability is maintained because degradation of the low-flow channel is prevented. Typically, the 2-year flows are contained in the protected zone so that the low-flow channel does not degrade downward, potentially undermining the entire waterway.

1.4 Wetland Channel Grade Control

Wetland channels, whether low-flow channels or from bank to bank, require modest slopes not exceeding about 0.3%. Grade control structures are often required for stability. Due to the environmental nature of the wetlands, the grade control structures are planned and designed to be compatible with a wetland environment. Wetland channels do not need a trickle channel, but where used, the trickle channel should not lower the wetland water table more than 12 inches.

1.5 Conduit Outlet Structures

Design criteria given in this chapter are for structures specifically designed to dissipate flow energy at conduit outlets to the open waterway. These types of structures are typically located at storm sewer outlets. Design criteria for culverts and storm sewers that discharge in-line with the receiving channel are described in the MAJOR DRAINAGE chapter of this *Manual*.

1.6 Bridges

Bridges have the advantage of being able to cross the waterway without disturbing the flow. However, for practical, economic, and structural reasons, abutment encroachments and piers are often located within the waterway. Consequently, the bridge structure can cause adverse hydraulic effects and scour potential that must be evaluated and addressed as part of each design project.

1.7 Transitions and Constrictions

Channel transitions are typically used to alter the cross-sectional geometry, to allow the waterway to fit within a more confined right-of-way, or to purposely accelerate the flow to be carried by a specialized high velocity conveyance. Constrictions can appreciably restrict and reduce the conveyance in a manner that is either detrimental or beneficial. For example, a bridge, box culvert, or constriction may increase the upstream flooding by encroaching too far into the floodplain conveyance, whereas in another situation a hydraulic control structure can be employed to purposely induce an upstream spill into an off-stream storage facility.

1.8 Bends and Confluences

General considerations for lined channels and conduits are discussed in the MAJOR DRAINAGE chapter of this *Manual*. Additional emphasis is added herein for certain situations. Channels and conduits that produce supercritical flow may require special structural or design considerations. This discussion is limited since these types of structures are generally associated with hydraulic performance exceeding the recommended criteria for grass-lined channels. Extensive study, specialized modeling and/or analysis may be required for these situations.

On the other hand, confluences are commonly encountered in design. Relative flow rates can vary disproportionately with time so that high flows from either upstream channel can discharge into the downstream channel when it is at high or low level. Depending on the geometry of the confluence, either condition can have important consequences, such as supercritical flow and hydraulic jump conditions, and result in the need for structures

1.9 Rundowns

A rundown is used to convey storm runoff from high on the bank of an open channel to the low-flow channel of the drainageway or into a detention facility. The purpose is to control erosion and head cutting from concentrated flow. Without such rundowns, the concentrated flow will create erosion.

1.10 Energy Dissipation

The energy of moving water is known as kinetic energy, while the stored energy due to elevation is potential energy. A properly sloped open channel will use up the potential energy in a uniform manner through channel roughness without the flow being accelerated. A grade control structure (i.e., drop and check) converts potential energy to kinetic energy under controlled conditions. Selection of the optimum spacing and vertical drop is the work of the hydraulic engineer. Many hydraulic structures deal with managing kinetic energy—to dissipate it in a reasonable manner, to conserve it at structures such as transitions and bridges, or occasionally to convert kinetic to potential energy using a hydraulic jump. Thus, managing energy involves understanding and managing the total energy grade line of flowing water.

1.11 Maintenance

Urban drainage facilities should not be built if they cannot be properly maintained on a long-term basis. This means that suitable access must be provided, a maintenance plan must be developed and funded, and the drainage facilities must be maintained in accordance with public works standards.

1.12 Structure Safety and Aesthetics

The design of structures must consider safety of flood control workers and the general public, especially when multiple uses are intended. Regulations and interpretations vary from community to community and may change with time. There are some inherent safety risks in any waterway that have to be recognized by the public, designers, and government officials. General suggestions are given in regard to safety; however, the designer must use a reasonable standard of care for the particular structure being designed or retrofitted that includes evaluation of present or likely future public access and uses such as recreation. The designer should give special consideration to structures located in waterways where boating is likely to occur. These structures need to be designed to avoid known hazards, such as reverse rollers (Leutheusser and Birk 1991), often referred to by some as "keepers."

Aesthetic appearance of structures in urban areas is also important. Structures can be designed with various configurations, different materials, and incorporation of adjacent landscaping to produce a pleasing appearance and good hydraulic function and to enhance the environmental and ecological character of the channel and floodplain. The incorporation of wetland vegetation, native grasses, and shrubs into the design adds to their aesthetics and provides erosion control and water quality functions.

Photograph HS-3—Stepped grouted sloping boulder drop structures such as in Denver's Bible Park can be safe, aesthetic, and provide improved aquatic habitat besides performing their primary hydraulic function of energy dissipation.

2.0 CHANNEL GRADE CONTROL STRUCTURES (CHECK AND DROP STRUCTURES)

2.1 Planning for the Future

Channel grade control structures (typically check structures and drop structures) should be designed for future fully developed basin conditions. In the use of a natural channel, the effects of future hydrology and potential down cutting must be included so that the natural channel is properly stabilized. Urbanization will create a base flow that, over time, will cause down cutting if not managed with grade control structures.

"Drop structures" are broadly defined. They establish a stable stream grade and hydraulic condition. Included are structures built to restore damaged channels, those that prevent accelerated erosion caused by increased runoff, and grade control drops in new channels. Drop structures provide special hydraulic conditions that allow a drop in water surface and/or channel grade. The supercritical flow may go through a hydraulic jump and then return to subcritical flow.

The focus of these criteria is on channel drops with primary emphasis on grass-lined channels. Check structures may be used to stabilize the natural low-flow channel in an unmodified floodplain. Thus, check structures also require additional consideration of the wider major flood path extending around the structure abutments.

Specific design guidance is presented for the following basic categories of drop structures: baffle chute drops (BCD), grouted sloping boulder drops (GSB), and vertical hard basin drops (VHB).

All drop structures should be evaluated after construction. Bank and bottom protection and adjustments may be needed when secondary erosion tendencies are revealed. It is advisable to establish construction contracts and budgets with this in mind. Use of standardized design methods for the types of drops suggested herein will reduce the need for secondary design refinements.

The design of the drop structure crest and provisions for the trickle or low-flow channel directly affect the ultimate configuration of the upstream channel. A shallow and/or dispersed trickle configuration will tend to result in some aggradation and a wetter channel bottom than might be associated with a wetland channel bottom. However, the wetland channel design would not contain a trickle channel because the low flows would be spread out uniformly across the entire channel bottom.

A higher unit flow will pass through the trickle or low-flow area than will pass through other portions of the channel cross section. This situation must be considered in design to avoid destabilization of the drop and the channel.

2.1.1 Outline of Section

The following section provides guidelines to aid in the selection of alternate types of drops, particularly those used for grass-lined channels. Drops for boatable channels are described separately.

Much of the section is oriented toward hydraulic design and criteria for drop structures. There are two levels of analysis given. One level of hydraulic analysis is “detailed.” All steps that are important are described, along with design aids. The other level is “simplified.” Layouts of typical drops, particularly the crest configuration and related channel, are given which result in grass-lined channel hydraulic performance at the maximum depths and velocities normally allowed by the District for these types of channels. The use of these charts allows a quicker start, but certain steps from the “detailed” analysis will still be necessary, particularly the effects of greater unit flows in the low-flow or trickle channel area.

Hydraulic analysis sections are followed by further details appropriate to each of the types of drops that are recommended for grass-lined channels and boatable channel drops. Then, further information on seepage analysis, construction concerns, and low-flow channel structures is given.

Photograph HS-4—This grade control structure on the South Platte River was a hazard to the boating public until it was retrofitted by the CDOT. Here, a rescue is supervised by Colorado Governor Richard Lamm who was enjoying a rafting trip with friends and the Denver Water Rescue Team.

2.1.2 Boatable Channels

Channels that are known to be boatable, either now or that will be in the future, and those others that are classified by the Colorado Water Quality Control Commission for Class 1 or 2 Recreation, but are not presently judged to be boatable, should have hydraulic structures designed with public safety as a special consideration. The designer should not set the stage for hazardous hydraulics that would trap a boater, such as at a drop structure having a reverse roller that may develop as the hydraulic jump becomes submerged.

Designs for boatable channels, grade control structures, and low-head dams have to prevent the development of submerged hydraulic jumps, have a gently sloped or stepped downstream face, and not have a deep stilling basin that would encourage the creation of a submerged hydraulic jump. One design approach is to direct the hydraulic momentum at the bottom of the drop at a relatively flat angle to help prevent a reverse roller. A downstream face on a drop having large grouted boulders and high roughness that is sloped at 10(H) to 1(V) has been used successfully on several projects along the South Platte River and on Clear Creek, permitting safe passage of boaters as the move over them.

Drop structures or low-head dams in boatable channels should incorporate a boat chute designed in accordance with carefully planned components that are consistent with recreational requirements for boater safety. Often, physical model studies are used to verify the efficacy of the proposed design.

Hydraulic structures on boatable channels should not create obstructions that would pin a canoe, raft or kayak, and sharp edges should be avoided.

2.1.3 Grass and Wetland Bottom Channels

Structures for grass and wetland bottom (i.e., non boatable) channels are described in detail on the following pages and are represented by a variety of choices and shapes to suit the particular site and related hydraulics.

Based on experience, the sloped drop has been found to be more desirable than the vertical wall drop with a hardened energy dissipation basin. Vertical drops can create a reverse roller and backflow eddies that have been know to trap boaters. Because of boater and public safety concerns, vertical drops are less desirable than sloping drops in urban areas. Other disadvantages of a vertical drop include the turbulence and erosive effect of the falling water on the drop structure, necessitating high maintenance.

It is desirable to limit the height of most drops to 3 to 5 feet to avoid excessive kinetic energy and to avoid the appearance of a massive structure, keeping in mind that the velocity of falling water increases geometrically with the vertical fall distance. If vertical drops are use, it is best to limit their height to 3 feet.

2.1.4 Basic Approach to Drop Structure Design

The basic approach to design of drop structures includes the following steps:

1. Determine if the channel is, or will be, a boatable channel. If boatable, the drop or check structure should use a standard of care consistent with adequate public safety to provide for boater passage.

2. Define the representative maximum channel design discharge (often the 100-year) and other discharges appropriate for analysis, (e.g., low or trickle flows and other discharges expected to

occur on a more frequent basis) which may behave differently. All channels need to be designed for stability by limiting their erosion and degradation potential and for longevity by analyzing all the effects on channel stability at levels of flow, including the 100-year flood.

3. Approximate the channel dimensions and flow parameters including longitudinal slope. Identify the probable range of drop choices and heights with the aid of Figure HS-1.

4. Select drop structure alternatives to be considered for grass-lined or other channel types (see Section 2.2).

5. Decide if channel performance at maximum allowable criteria (i.e., velocity, depths, etc.) for grass-lined channels is practical or desirable. If not, or if the design flow is over 7,500 cfs, go to step 6; otherwise, the simplified design charts in Section 2.3.3 may be used to size the basic configuration of the crest. The designer should review the precautions given and the limits of application with respect to site conditions. Then the crest section and upstream channel transition will need to be refined for incorporation of the trickle or low-flow channel. This requires review of the upstream water surface profile and the supercritical flow downstream of the crest through the dissipation zone of the drop. Under conditions of a submerged jump due to a high tailwater elevation, steps to mitigate the reverse roller should be evaluated. If measures are taken to provide baffles or large boulders to break up the jet, then extensive analysis of the trickle zone hydraulics is not necessary. The steps involved are discussed in Section 2.3. Then go to step 7.

6. For refined analysis and optimal design of grass-lined channel drop structures, use the “detailed” hydraulic analysis in Section 2.3.2.

7. Perform soils and seepage analyses as necessary to obtain foundation design information.

8. In the case of drops for grass-lined channels, comply with the minimum specific criteria and follow the guidelines for the recommended types of drops (baffle chute, vertical hard basin, and grouted sloping boulder) presented in Section 2.3.4. Otherwise, provide a complete hydraulic analysis documenting the performance and design for the type of drop or other type of channel being considered. For channels with alluvial beds that present an erosion/degradation risk, a complete stability and scour analysis should be completed, accompanied by a geotechnical investigation and seepage analysis.

9. Use specific design criteria and guidelines to determine the final drop structure flow characteristics, dimensions, material requirements, and construction methods.

10. Obtain necessary environmental permits, such as a Section 404 permit.

2.2 Drop Selection

The primary concerns in selection of the type of drop structure should be functional hydraulic performance and public safety. Other considerations include land uses, cost, ecology, aesthetics, and maintenance, and environmental permitting.

Table HS-1 presents information to assist in the selection of appropriate drop structures applicable for various situations. Generally, the drops in any group are shown in order of preference. Comparative costs are often close. However, on-site conditions, such as public safety, and aesthetics may weight the selection of a drop structure type. Whenever public access is likely to occur, fencing not withstanding, the use of sloping drops is preferred for safety reasons over the use of vertical ones.

Table HS-1—Non-Boatable Drop Structure Selection for 3- to 5-Foot High Drops and Flows of 0 to 15,000 cfs

1. Easy or limited public access; downstream degradation likely. a) Grouted sloping boulder drop with toe imbedded in the stream bed b) Baffle chute drop
2. Limited public access; downstream degradation not likely. a) Grouted sloping boulder drop b) Vertical hard basin drop c) Baffle chute drop
3. Easy public access; downstream degradation not likely. a) Grouted sloping boulder drop b) Baffle chute drop

From an engineering design standpoint, there are two fundamental systems of a drop structure: the hydraulic surface-drop system and the foundation and seepage control system. The material components that can be used for the foundation and seepage control system are a function of on-site soils and groundwater conditions. The selection of the best components for design of the surface drop system is essentially independent of seepage considerations and is based on project objectives, channel stability, approach hydraulics, downstream tailwater conditions, height of drop, public safety, aesthetics, and maintenance considerations. Thus, foundation and seepage control system considerations are discussed separately. One factor that influences both systems is the extent of future downstream channel degradation that is anticipated. Such degradation can destroy a drop structure if adequate precautions are not provided.

2.3 Detailed Hydraulic Analysis

2.3.1 Introduction

Analysis guidelines are discussed in this section to assist the engineer in addressing critical hydraulic and seepage design factors. For a given discharge, there is a balance between the crest base width, upstream and downstream flow velocities, the Froude number in the drop basin, and the location of the jump. These parameters must be optimized for each specific application.

There are two levels of analysis possible. The first involves detailed analysis of all hydraulic conditions and leads to an optimal design for each structure. The concepts involved are described herein, and numerous references are available for more detailed information. The second level of analysis is a simpler approach that is based on configurations that will be adequate at the limits of permissible grass-lined channel criteria as described in Section 2.4.

There are two general categories of drops: sloping and vertical. For safety reasons, vertical drops should be avoided under urban conditions for public safety reasons. Performance of vertical or smooth sloping drops into a hard basin is relatively well documented. Their hydraulic analysis is briefly described herein. The design criteria for other drops such as vertical plunge pools and baffle chutes is based on empirical data and model studies.

2.3.2 Crest and Upstream Hydraulics

After preliminary channel layout has indicated probable drop location and heights (see the MAJOR DRAINAGE chapter for guidance, including the design spreadsheet UD-Channels), analysis and design begins with review of the crest section at the top of the drop. As flow passes through critical depth near the crest, upstream hydraulics are separated from downstream. Usually, the key task here is to determine critical depth at the crest based on the entire section. The critical flow state needs to be verified to ascertain that the downstream tailwater does not submerge the crest and effectively controls the hydraulics above the crest. If the downstream tailwater controls, then the structure must still be evaluated as a check for the peak discharge and as a drop at lower flows, if appropriate.

With control at the drop crest, water surface profile computations are used to establish the upstream abutment and bank heights. Computations should include a transition head loss, typically ranging from 0.3 (modest transitions in grass-lined channels) to 0.5 (channels approaching abrupt constrictions) times the change in velocity head across the transition (see Section 5.2), and allowance for the end contraction where the flow may effectively separate from the abutment end walls. Refer to Section 5.0 and standard hydraulic references for guidance (Chow 1959, Rouse 1949, and USACE 1994).

2.3.3 Water Surface Profile Downstream of the Crest

2.3.7.1 Critical Depth Along a Drop Structure.

Although this discussion concerns the hydraulics below the crest of a drop structure, the fundamental analysis of this hydraulics is established by the crest conditions. Main, low-flow and trickle channel regions are considered separately. Although the actual location of critical depth can vary according to the channel, transition, and drop geometry, the assumption is made that critical depth occurs at the crest, in a horizontal straight line across the crest section.

The assumption of critical flow conditions across the crest is illustrated conceptually by the diagrams in Figures HS-2 and the corresponding energy level across the section. At any point across the crest, the velocity is a function of the critical depth at that point. This causes a higher unit discharge applied to the trickle channel zone than across the main channel flow area. Figure HS-2 also illustrates that the water surface and energy grade line profiles will be different at the trickle (or low-flow) portions of the section than in the main channel flow zones and the forces exerted by flow on individual boulders on the sloping face of the drop.

2.3.7.2 Hydraulic Analysis.

After review of the crest and upstream hydraulics, the analysis proceeds to the supercritical flow and the hydraulic jump downstream. It is here that the designer should give special consideration to the potential of reverse rollers and avoid them in boatable channels and, where practicable, in grass-lined channels. Little flow dispersal from the trickle or low-flow zone to the main zone occurs through the supercritical portion of the drop. (Flow expansion is more likely downstream of the jump.) Therefore, unit discharge determined at the crest for either the trickle channel or the main portion of the drop is assumed to remain constant. The required basin length varies between these zones. Baffle chutes are the only type of drop where this distinction is not significant because the baffles break up the flow patterns and spread the flow more evenly over the width of the channel.

With the exception of baffle chute drops, separate analysis should be performed to evaluate the main drop and trickle or low-flow channel zones, as follows:

Critical depth, Y_c, is determined for the entire section area. The subscript ($_t$) or ($_m$) is added to refer to the trickle or low-flow zone or main channel zone, respectively. For example, in the main channel zone:

$$Y_{cm} = El_c - El_m \quad \text{(HS-1)}$$

Similarly, in the trickle or low-flow channel zone:

$$Y_{ct} = El_c - El_t \quad \text{(HS-2)}$$

in which:

El_c = critical water surface elevation

El_m = elevation of the main channel at the drop crest

El_t = elevation of the trickle or low-flow channel at the drop crest

The remaining hydraulic parameters, such as critical velocity, V_c (ft/sec), energy grade line, *EGL*, and unit discharge, q(cfs/ft), are determined separately for the main and trickle or low-flow channel zones by equations of the form:

$$V_c = (gY_c)^{1/2} \qquad \text{(HS-3)}$$

$$EGL = Y_c + \frac{V_c^2}{2g} + El_m(or\ \ El_t) \text{ at the drop crest} \qquad \text{(HS-4)}$$

$$q = Y_c^{3/2} g^{1/2} \qquad \text{(HS-5)}$$

where g is the acceleration of gravity, and each parameter would have the subscript ($_m$) or ($_t$) as appropriate for the main, trickle, or low-flow channel zone.

Water surface profiles for the drawdown along the slope of a sloping drop and through the basin may be calculated using the "Standard Step Method" (Chow 1959), or any equivalent method suitable for unit discharge computations. For baffle chutes and vertical drops, individual methods are given in later subsections. It is necessary to plot the energy grade line to assure calculations are reasonable.

2.3.7.3 Manning's *n* for Concrete, Boulders and Grouted Boulders.

Depending on the type of materials and the relative depth, the appropriate roughness parameters should be used in computations. Table HS-2 and Figure HS-3 it refers to for grouted boulders, give the recommended Manning's roughness values and are based on Chow (1959), Oliver (1967), Anderson et. al. (1973), Henderson (1966), Barnes (1967), Smith and Murray (1975), Stevens et. al. (1976), Bathurst, Li and Simons (1979) and Stevens (1984). Normal equations typically used for riprap do not apply to boulders and grouted boulders because of their near-uniform size and because the voids may be completely or only partially filled with grout. The roughness coefficient taken from Figure HS-3 varies with the depth of flow relative to the size of the boulders and the depth of grout used to lock them in place. Stepped grouted rock placement is another method that can be used to increase roughness and reduce velocities over the face of the drop.

Table HS-2—Suggested Approximate Manning's Roughness Parameter at Design Discharge for Sloping Drops

Smooth concrete	0.011 to 0.013
Stepped concrete where step heights equal 25% of nape depth	0.025*
Grouted Boulders	See Fig. HS-3

* This assumes an approach channel depth of at least 5 feet. Values would be higher at lesser flow depths

2.3.7.4 Avoid Low Froude Number Jumps in Grass-Lined Channels.

Low Froude number hydraulic jumps with longer areas of hydraulic instability are common in grass-lined channel applications. Baffles and rock placements that create turbulence and dissipate energy along the face of the drop are recommended to help counteract the adverse effects of low Froude number jumps and the associated tendency to carry residual energy and waves for extended distances downstream.

2.3.4 Hydraulic Jump Location

The water surface profile analysis starts at the crest and works downstream to analyze supercritical flow. Separate analysis for the low-flow, trickle, and main channels includes the review of hydraulic jumps. In the case of a baffle chute, no jump will occur because the baffles are constantly breaking up the flow, preventing supercritical flow. Examination of tailwater conditions is still important for a baffle chute to evaluate riprap and basin layout.

To determine the location of the hydraulic jump, a tailwater elevation has to be established by water surface profile analysis that starts from a downstream control point and works upstream to the drop basin. This backwater analysis is based upon entire cross sections for the downstream waterway. The hydraulic jump, in either the low-flow, trickle channel, or the main drop, will begin to form where the unit specific force of the downstream tailwater is greater than the specific force of the supercritical flow below the drop. Special consideration must be given to submerged hydraulic jumps because it is here that reverse rollers are most common. For submerged jumps, the resulting downstream hydraulics should be evaluated (Cotton 1995).

The determination of the jump location is usually accomplished through the comparison of specific force between supercritical inflow and the downstream subcritical flow (i.e., tailwater) conditions:

$$F = \left(\frac{q^2}{gy}\right) + \left(\frac{y^2}{2}\right) \tag{HS-6}$$

in which:

F = specific force (ft^2)

q = unit discharge (determined at crest, for low-flow, trickle, and main channel zones) (cfs/ft)

y = depth at analysis point (ft)

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

The depth, y, for downstream specific energy determination is the tailwater water surface elevation minus the ground elevation at the point of interest, which is typically the main basin elevation or the trickle channel invert (if the jump is to occur in the basin). The depth, for the upstream specific energy (supercritical flow), is the supercritical flow depth at the point in question.

Note that on low drops, the jump may routinely submerge the crest or may occur on the face of the drop. Refer to Little and Daniel (1981), Little and Murphey (1982), Chow (1959), USACE (1994), and Peterka (1984) for these cases.

The jump at sloping drops typically begins no further downstream than the drop toe. In vertical drops, the jump should begin where the jet hits the floor of the basin. This is generally accomplished in the main drop zone by depressing the basin to a depth nearly as low as the downstream trickle channel elevation. This will provide drainage for the basin.

2.3.5 Jump and Basin Length

The un-submerged jump length is typically between 3.6 and 6 times the tailwater depth, depending on the Froude number. For most cases, a basin length of 5 to 6 times the tailwater depth is the most advisable. A longer basin length is advisable for erosive soils or depending on the nature of the jump. Typically, at least 60% of the jump length is rock lined or otherwise reinforced. For baffle chute drops and vertical drops, basin dimensions are empirically derived.

In the trickle or low-flow channel alignment, the jump will tend to wash further downstream of the toe, and additional mitigation is recommended such as extending the basin length and/or providing baffles or large boulders that will break up the jet and dissipate energy.

2.3.6 Seepage Analysis

Subgrade erosion caused by seepage and structure failures caused by high seepage pressures or inadequate mass are of critical concern. These factors are important in the design and must be analyzed; otherwise, the structure might fail.

Seepage analysis can range from hand-drawn flow nets to computerized groundwater flow modeling. Advanced geotechnical field and laboratory testing techniques may be used to confirm the accepted permeability values where complicated seepage problems are anticipated. Several flow net analysis programs are currently available that are suitable for this purpose.

A minimal approach is Lane's Weighted Creep method. It can be used to determine dimensions or cutoff improvements that would provide an adequate seepage length. It should only be used as a guideline and, when marginal conditions or complicated geological conditions exist, a more precise analysis should be used. The involvement of a geotechnical engineer will often be necessary. Lane's method is given later in this section.

2.3.7 Force Analysis

Each component of a drop has forces acting upon it that require evaluation. This subsection describes the general forces, except forces on riprap for which the reader is referred to Isbash (1936), Oliver (1967), Smith (1975), Smith and Strung (1967), Stevens (1976), Taggart (1984), Abt (1986 and 1987), Wittler and Abt (1988), Maynord and Ruff (1987), Richardson (1988), and LSA (1986 and 1989). It is worth noting that the boulders are subject to all of the usual forces plus the hydrodynamic forces of interflow through voids and related pressure fluctuations. A complete presentation of forces acting on riprap and boulders is not presented herein. Forces are described here, as they would apply to sloping grouted boulder and reinforced concrete drops. Additional information on forces on baffle blocks is presented in the baffle chute subsection, and this information may also be useful to extrapolate for large boulders used as baffles in grouted boulder drops.

The various criteria for structural slab thicknesses given for each type of drop have generally taken these forces into consideration. It is the user's responsibility to determine the forces involved.

Figure HS-3 illustrates the forces involved for a grouted sloping boulder drop, which is similar to other sloping concrete drops or baffle chutes. Five location points are of concern. Point 1 is downstream of the toe, at a location far enough downstream to be beyond the point where the deflection (turning) force of the surface flow occurs. Point 2 is at the toe where the turning force is encountered. Point 3 is variable in location to reflect alternative drain locations. When a horizontal drain is used, Point 3 is at a location where the drain intercepts the subgrade of the structure. Point 4 is approximately 50% of the distance along the drop slope. Point 5 is at a point underneath the grout layer at the crest and downstream of the cutoff wall.

Point 3 is usually the critical pressure location, regardless of the drain orientation. In some cases, Point 1 may also experience a low safety factor when shallow supercritical flow occurs, such as when the jump washes downstream.

Seepage uplift is often an important force controlling structure stability. Weep drains, the weight of the structure, and the water on top of the structure counteract uplift. The weight of water is a function of the depth of flow. Thus, the greater the roughness, the deeper the flow condition and the greater the weight.

2.3.7.1 Shear Stress

The normal shear stress equation is transformed for unit width and the actual water surface profile by substituting S_e, the energy grade line slope for S_o, and the drop slope.

$$\tau = \gamma y S_e \qquad \text{(HS-7)}$$

in which:

τ = shear stress (lbs/ft^2)

γ = specific weight of water (lbs/ft^3)

y = depth of water at analysis point (ft)

2.3.7.2 Buoyant Weight of Structure

Each design should take into consideration the volume of grout and rock or reinforced concrete and the density of each. In the case of reinforced concrete, 150 pounds per cubic foot can be used as the specific weight (or 88 pounds per cubic foot net buoyant weight). Specific weight of rock is variable depending on the nature of the material.

2.3.7.3 Impact, Drag and Hydrodynamic Lift Forces

Water flowing down the drop will directly impact any abrupt rock faces or concrete structure projections into the flow. Technically, this is considered as a type of drag force, which can be estimated by equations found in various references. One should compare calculated drag force results with the forces shown later for baffle chute blocks (Section 2.5). Impact force caused by debris or rock is more difficult to estimate because of the unknown size, mass, and time elapsed while contact is made. Therefore, it is recommended that a conservative approach be taken with regard to calculating water impact (drag force), which generally will cover other types of impact force. Specialty situations, where impact force may be significant, must be considered on an individual basis. In addition, boulders and riprap are subject to hydrodynamic lift forces (Urbonas, 1968) that are caused by high velocities over the top of the stones and the zones of separation they create, resulting in significant reduction in pressure on the top while hydrostatic pressure remains unchanged at the stone's bottom.

2.3.7.4 Tur ning Force

A turning force impacts the basin as a function of slope change. Essentially, this is a positive force countering uplift and causes no great stress in the grouted rock or reinforced concrete. This force can be estimated as the momentum force of the projected jet area of water flowing down the slope onto the horizontal base and calculating the force required to turn the jet.

2.3.7.5 Friction

With net vertical weight, it follows that there would be a horizontal force resisting motion. If a friction coefficient of 0.5 is used and multiplied by the net weight, the friction force to resist sliding can be estimated.

2.3.7.6 Frost Heave

This value is not typically computed for the smaller drops anticipated herein. However, the designer should not allow frost heave to damage the structure, and, therefore, frost heave should be avoided and/or mitigated. In reinforced concrete, frost blankets, structural reinforcing, and anchors are sometimes utilized for cases where frost heave is a problem. If gravel blankets are used, then the seepage and transmission of pressure fluctuations from the hydraulic jump are critical.

2.3.7.7 Seepage Uplift Pressure

As explained previously, uplift pressure and seepage relief considerations are extremely important to structural stability and usually of greater concern than the forces described above. There can be troublesome pressure differentials from either the upstream or downstream direction when there is shallow supercritical flow on the drop slope or in the basin. One may consider an upstream cutoff to mitigate this problem. Weep locations with proper seepage control may be provided. For high drops (i.e., > 6 feet), more than one row of weep holes may be necessary.

A prudent approach is to use a flow net or other type of computerized seepage analysis to estimate seepage pressures and flows under a structure.

2.3.7.8 Dynamic Pressure Fluctuations

Laboratory testing (Toso 1986; Bowers and Toso 1988) has documented that the severe turbulence in a hydraulic jump can pose special problems often ignored in hydraulic structures. This turbulence can cause significant positive and negative pressure fluctuations along a structure.

A good example of the problem can be envisioned by a situation in which the entire sloping face of the drop is underlain by a gravel seepage blanket. The gravel could be drained to the bottom of the basin or other locations where the jump will occur. In such a case, the positive pressure fluctuations could be transmitted directly to the area under the sloping face, which then could destabilize the structure since there would not be sufficient weight of water over the structure in the area of shallow supercritical flow.

The key parameter is the coefficient of maximum pressure fluctuation, $C_{p\text{-}max}$, which is in terms of the velocity head of the supercritical flow just upstream of the jump:

$$C_{p-\max} = \frac{\Delta P}{\left(\dfrac{V_u^2}{2g}\right)} \qquad \text{(HS-8)}$$

in which:

ΔP = pressure deviation (fluctuation) from mean (ft)

V_u = incident velocity (just upstream of jump) (ft/sec)

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

Effectively, C_P is a function of the Froude number of the supercritical flow. The parameter varies as a function of X, which is the downstream distance from the beginning of the jump to the point of interest.

Table HS-3 presents recommended $C_{p\text{-}max}$ positive pressure values for various configurations. When the Froude number for the design case is lower than those indicated, the lowest value indicated should be used (do not reduce on a linear relationship) for any quick calculations. The values can be tempered by reviewing the C_p graphs, a few of which are given in Figures HS-4 through HS-6. Note that the graphs are not maximum values but are the mean fluctuation of pressure. The standard deviation of the fluctuations is also indicated, from which the recommended $C_{p\text{-}max}$ values were derived.

Figure HS-4 illustrates positive and negative pressure fluctuations in the coefficient, C_p, with respect to the location where the jump begins at the toe. Figure HS-5 presents the positive pressure fluctuation coefficient where the jump begins on the face. Figure HS-6 illustrates how the pressure fluctuations vary in a U.S. Bureau of Reclamation (USBR) Type II or III basin.

For the typical basin layouts given and where the drains are at the toe and connect directly to the supercritical flow, these pressure fluctuations should not be of great concern. However, when drains discharge to the jump zone and could transfer pressure fluctuations to areas under supercritical flow, pressure fluctuations are of concern.

Table HS-3—Nominal Limit of Maximum Pressure Fluctuations within the Hydraulic Jump (Toso, 1986)

Jump Condition	Froude Number	Suggested Maximum C_p
0° slope, developed inflow (boundary layer has reached surface)	3.0	1.0
30° slope, toe of jump at base of chute*	3.8	0.7
30° slope, toe of jump on chute*	3.3	0.8
30° slope, with Type II basin (USBR)	5.0	0.7
30° slope with Type III basin (USBR)	5.0	1.0

* Velocity head increased by elevation difference between toe of jump and basin floor, namely, depth at the drop toe.

2.3.7.9 Overall Analysis

All of the above forces can be resolved into vertical and horizontal components. The horizontal components are generally small (generally less than 1 psi) and capable of being resisted by the weight of the grout, rock, and reinforced concrete. When problems occur, they are generally the result of a net vertical instability.

The overall (detailed) analysis should include reviews of the specific points along the drop and the overall drop structure geotechnical and structural stability. All steps of this detailed analysis are not necessary for design of drops along modest capacity grass-lined channels, provided that the design is developed using the guidelines and configurations presented in the following simplified analysis approach section and that other District criteria are met. The critical design factors are seepage cutoff and relief and pressure fluctuations associated with the hydraulic jump that can create upward forces greater than the weight of water and structure over the point of interest. Underflow can easily lift a major slab of rock and grout and, depending upon the exposure, the surface flow could cause further weakening, undermining, or displacement. Generally, a 30-pound net downward safety allowance should be provided, and 60 pounds is preferred. An underdrain is generally needed as shown in detail 2 of Figure HS-7D to prevent hydrostatic uplift on the stones.

2.4 Simplified Drop Structure Designs for District's Grass-Lined Channels

2.4.1 Introduction and Cautions

As previously mentioned, there is a balance between the crest shape chosen, upstream channel stability, and the configuration of the drop downstream which will result in reasonable or optimal energy dissipation. Further, there is usually a single configuration of drop crest, upstream channel slope, and base width that will result in an acceptable drop structure performance for grass-lined channels designed using the District's criteria described in the MAJOR DRAINAGE chapter.

This subsection presents simplified relationships that provide basic configuration and drop-sizing parameters that may be used when the District's maximum allowable velocity and depth criteria for grass-lined channels are used.

Design guidance presented in this section is developed for channels that operate at the brink of maximum criteria (i.e., approximately having unit discharge of 25 cfs/ft for erosive soil and 35 cfs/ft for erosion resistant soils and Froude Number $\leq$ 0.8). They do not consider channel curvature, effects of other hydraulic structures, or unstable beds, all of which require detailed analysis. They do provide guidelines for initial sizing and reasonableness checking, but are not a substitute for comprehensive hydraulic analysis in the context of the entire waterway.

2.4.2 Applicability of Simplified Channel Drop Designs

This section presents guidelines and analysis steps and specific minimum design criteria for two types of drops. Grouted sloping boulder drops and vertical hard basin drops are the only two types of drops for which these simplified design procedures may be utilized when used in grass-lined channels. Other designs are available, but they are more limited in application and require an individual analysis. Regardless of the type of drop used, it should never be located within or immediately downstream of a curve in a channel. Namely, locate all drops on a tangent and not on a curve of a channel.

Photograph HS-5—Example of stepped downstream face for a sloping boulder drop structure. Note dissipation of energy at each step for low flow.

2.4.3 Simplified Grouted Sloping Boulder Drop Design

This type of structure has gained acceptance in the Rocky Mountain region due to close proximity to high-quality rock sources, design aesthetics, and successful applications. The quality of rock used and proper grouting procedure are very important to the structural integrity. There is no maximum height limit; however, the rock sizing procedure is more complex than the simplified procedures and details provided by Figures HS-7A, HS-7B and HS-7C for GSB drops 6-feet or less in height.

For typical channels the drop is designed with a hydraulic jump dissipator basin, although some energy loss is incurred due to the roughness of the grouted rock slope. In sandy soil channels the design provides for a scour at the toe and does not require an energy-dissipating basin. Structure integrity and containment of the erosive turbulence within the basin area are the main design objectives.

Photograph HS-6—Detail of the grouted sloping boulder drop with a trickle channel section creating the sight and sound of cascading water.

Construct boulder drops using uniform-height boulders with a minimum height specified in Table HS-4. Grout all boulders to a depth of 1/2 or 1/3 of their height through the approach, sloping face, and basin areas, except at the upstream crest where it needs to extend the full depth of the rock in order to provide stability of the approach channel. Figures HS-7A, HS-7B and HS-7C illustrate the general configuration of three types of GSB drops; one for a channel with a trickle channel (Figure HS-7A), one for one with a low-flow channel (Figure HS-7B) and one for channels in erosive soils or unstable conditions. (Figure HS-7C). Requirements for the grout, riprap and boulders are specified in the MAJOR DRAINAGE chapter of this *Manual*. Adequate seepage control with underdrains is important for a successful design whenever drop height exceeds 5-feet.

The following outlines the fundamental design steps and guidelines.

1. Hydraulics should be completed as described in Section 2.3 whenever the drop height exceeds 6 feet. Otherwise, use critical depth to size the boulders, using the boulder sizing procedure described below.
2. Grouted boulders must cover the crest and cutoff and extend downstream through the energy-dissipating basin when there is one, or through the imbedded toe of the drop when not present.
3. The vertical cutoff should be located at the upstream face of the crest, at a minimum depth of $0.8H_d$ or 4 feet, whichever is deeper. Evaluate specific site soils for use in seepage analysis and foundation suitability.

Photograph HS-7—An overall view of the drop structure from the previous page is illustrated here to emphasize the opportunities available for creating an attractive urban hydraulic setting for the riparian corridor.

4. The trickle or low-flow channel should extend through the drop crest section. Downstream, the trickle or low-flow channel protection should extend past the main channel protection, or large boulders and curves in the trickle or low-flow channel can be used in the basin area to help dissipate the energy.
5. Grout thickness, D_g, and rock thickness, D_r, should be determined based upon a minimum safety surplus net downward force of 30 pounds. The rocks must be carefully placed to create a stepped appearance, which helps to increase roughness. Minimum criteria for the simplified design process are referred to in step 8, below.
6. The main stilling basin should be depressed 1 to 2 feet deep in order to stabilize the jump. A row

of boulders should be located at the basin end to create a sill transition to the downstream invert elevation. It is advisable to bury riprap for a distance of 10 feet downstream of the sill to minimize any erosion that may occur due to secondary currents.

When the drop is located in sandy soils and in channels with lesser stability, the stilling basin is eliminated and the sloping face extended to where the top of the boulders are five feet (5') below the projected (i.e., after accounting for downstream degradation) downstream channel's invert.

7. Do not use longitudinal slopes steeper than 4:1. Longitudinal slopes flatter than 4:1 improve appearance and safety while steeper slopes reduce structural stability. With high public usage, very flat longitudinal slopes (i.e., flatter than 8H:1V) help to mitigate reverse roller formation at higher tailwater depths that can cause submerged hydraulic jump formation and create "keepers".
8. Simplified design criteria are provided in Table HS-4 for grouted sloping boulder drops. These criteria are valid only where the channel flow conditions meet the minimum criteria recommended in the MAJOR DRAINAGE chapter.

Table HS-4—Grouted Sloping Boulder Drops: Minimum Design Criteria for Grass-Lined Channels Meeting the District's Maximum Depth and Velocity Criteria

Design Parameter	Drop Height (H_d) 6 Feet or Less	Drop Height (H_d) Greater Than 6 Feet
Maximum longitudinal slope	4H to 1V	4H to 1V
Minimum boulder depth	Use V_c to size*	Use V_n to size***
Grout thickness—D_g	½ to $^1/_3$ D_r except at the upstream crest of the structure where full grout depth is needed	½ D_r to $^1/_3$ D_r except at the upstream crest of the structure where full grout depth is needed
Basin depression	1 to 2 feet (*see Step 6 above for sandy/unstable channels*)	Do sequential depth analysis
Grouted boulder approach—L_a	5 feet (min.)	8 feet
Basin length—L_b** Erosive (sandy channel) Non-erosive	20 feet (*see Step 6 above for sandy/unstable channels*) 15 feet	20 feet (*also see Step 6 above for sandy/unstable channels*) 15 feet
Basin width—B	Same as crest width (*see Step 6 above for sandy/unstable channels*)	
Trickle and low-flow zone provisions	Install large boulders in center basin zone to break up high flow stream (*see Step 6 above for sandy/unstable channels*)	
Trickle zone protection width below drop	$3b_1$ or b^2 (whichever is smaller; see Figure HS-7)	
Other provisions	A buried riprap zone should be installed for $2H_d$ (10 feet minimum) downstream of the basin (*see Step 6 above for sandy channels*) Do not locate a drop within a channel curve or immediately downstream of one.	

* Use critical velocity in low-flow and main channels to size boulders.

** Use drawdown velocity at H_d to size low-flow and main channel section boulders.

Sizing of boulders for the simplified grouted sloping boulder procedure is based on the following:

1. This procedure can be used only for channels designed using the specified maximum velocities and depths for grass-lined channels in this *Manual* (see the MAJOR DRAINAGE chapter).

2. For drops of 6-feet or less in height, one can use UD-Channels Spreadsheet to find the 100-year critical velocities in the low-flow and the main channels to size boulders for each section.

For drops greater than 6-feet in height, a detailed design procedure has to be used consisting of the following:

1. Determine the critical velocities using drawdown calculations to establish the 100-year flow depth at the toe of the drop.

   a. For a composite channel, find critical velocity, V_c, for the channel cross-section segment outside the low-flow section.

   b. For a composite channel, find critical velocity, V_{mc}, for the low-flow channel cross-section segment.

   c. For a simple trapezoidal or wetland bottom channel, find critical velocity, V_c, for the channel cross section.

2. Calculate rock-sizing parameter, R_p, for the channel cross-section segment outside the low-flow section or for a simple trapezoidal channel section using the critical velocity estimated for this segment of the cross section:

$$R_p = \frac{V_c S^{0.17}}{(S_s - 1)^{0.66}}$$

in which: S = longitudinal slope along direction of flow in ft/ft

S_s = Specific gravity of the rock. Assume 2.55 unless the quarry certifies higher specific gravity.

3. Calculate rock-sizing parameter, R_{pL}, for the channel cross-section segment within the low-flow section using the critical velocity for drops 6-feet in height (the draw-down velocity estimates at bottom of the drop for taller structures):

$$R_{pL} = \frac{V_{mc} S^{0.17}}{(S_s - 1)^{0.66}} \qquad \text{(HS-9)}$$

4. Select minimum boulder sizes for the cross-section segments within and outside the low-flow channel cross-section from Table HS-5. If the boulder sizes for the low-flow channel and the

overbank segments differ, decide to use only the larger sized boulders throughout the entire structure, or to specify two sizes, namely, one for the low-flow channel and the other for the overbank segments of the cross section. Consider the complexity of specifying two different sizes on the design drawings and in the construction of the structure before deciding.

Regardless of the design procedure used above, all boulders shall be grouted in accordance with the specifications Figure HS-8. All grouted boulders outside of the low-flow channel shall be buried with topsoil to a depth of no less than 4 inches (6 inches or more preferred for successful grass growth) above the top of the highest boulder and the surface vegetated with native grasses on the overbank bench and native grasses and dry-land shrubs on the overbank channel's side slopes.

Table HS-5—Boulder Sizes for Various Rock Sizing Parameters

	Ungrouted Boulders		Grouted Boulders *	
Rock Sizing Parameter, R_p	Minimum Dimensions of Boulder, D_r	Boulder Classification	Minimum Dimensions of Boulder, D_r	Boulder Classification
Less than 4.50	18 inches	B18	18 inches	B18
4.50 to 4.99	24 inches	B24	18 inches	B18
5.00 to 5.59	30 inches	B30	24 inches	B24
5.60 to 6.39	36 inches	B36	30 inches	B30
6.40 to 6.99	42 inches	B42	36 inches	B36
7.00 to 7.49	48 inches	B48	42 inches	B42
7.50 to 8.00	n/a	n/a	48 inches	B48

* Grouted to no less than ½ the height (+1"/- 0"), no more than ⅓ (+0"/- 1") of boulder height.

2.4.4 Vertical Hard Basin Drops

The vertical hard basin drops include a wide variety of structure designs, but they are not generally recommended for use in urban areas because of concerns for public safety, during wet and dry weather periods. In addition, vertical hard basin drops are to be avoided due to impingement energy, related maintenance and turbulent hydraulic potential (ASCE and WEF 1992). Whenever used, it is recommended their drop height, upstream invert to downstream channel invert, be limited to 3-feet.

The hydraulic phenomenon provided by this type of drop is a jet of water that overflows the crest wall into the basin below. The jet hits the hard basin and is redirected horizontally. With sufficient tailwater, a hydraulic jump is initiated. Otherwise, the flow continues horizontally in a supercritical mode until the specific force of the tailwater is sufficient to force the jump. Energy is dissipated through turbulence in the hydraulic jump. The basin is sized to contain the supercritical flow and the erosive turbulent zone. Figure HS-9 shows a vertical drop with a grouted boulder basin. The rock-lined approach length ends abruptly at a structural retaining crest wall that has trickle channel section.

Photograph HS-8—A vertical hard basin drop structure can be an effective tool for controlling grade, but its use in urban areas is not generally not recommended because of public safety concerns and aesthetics.

Basic design steps are as follows:

1. The design approach uses the unit discharge in the main and trickle channel to determine separately the water surface profile and jump location in these zones. The overall jump hydraulic problems are the same as previously described.

   Chow (1959) presents the hydraulic analysis for the "Straight Drop Spillway." Add subscript ($_t$) for the trickle channel area and subscript ($_m$) for the main channel area in the following equations. The drop number, D_n, is defined as:

   $$D_n = \frac{q^2}{\left(gY_f^3\right)} \qquad \text{(HS-10)}$$

   in which:

   q = unit discharge (cfs/ft)

   Y_f = effective fall height from the crest to the basin floor (ft)

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

   For hydraulic conditions at a point immediately downstream of where the nappe hits the basin floor, the following variables are defined as illustrated in Figure HS-10:

$$\frac{L_d}{Y_f} = 4.3 D_n^{0.27}$$

$$\frac{Y_p}{Y_f} = 1.0 D_n^{0.22}$$

$$\frac{Y_l}{Y_f} = 0.54 D_n^{0.425}$$

$$\frac{Y_2}{Y_f} = 1.66 D_n^{0.27}$$

in which:

Y_f = effective fall height from the crest to the basin floor (ft)

L_d = length from the crest wall to the point of impingement of the jet on the floor or the nappe length (ft)

Y_p = pool depth under the nappe just downstream of the crest (ft)

Y_1 = flow depth on the basin floor just below where the nappe contacts the basin (ft)

Y_2 = tailwater depth (sequent depth) required to cause the jump to form at the point evaluated (ft)

In the case where the tailwater does not provide a depth equivalent to or greater than Y_2, the jet will wash downstream as supercritical flow until its specific force is sufficiently reduced to allow the jump to occur. Determination of the distance to the hydraulic jump, D_j, requires a separate water surface profile analysis for the main and low-flow zones as described herein for sloping drops. Any change in tailwater affects the stability of the jump in both locations.

2. The hydraulic jump length, L_j, is approximated as 6 times the sequent depth, Y_2. The design basin length, L_b, includes nappe length, L_d, the distance to the jump, D_j, and 60% of the jump length, L_j. (The subscripts "m" and "t" in Equations HS-11 and HS-12 refer to the main and trickle channel zones, respectively.)

   At the main channel zone:

$$L_{bm} = L_{dm} + D_{jm} + 60\% \left(6Y_{2m}\right) \tag{HS-11}$$

   At the trickle channel flow zone, without baffles or boulders to break up the jet:

$$L_{bt} = L_{dt} + D_{jt} + 60\% \left(6Y_{2t}\right) \tag{HS-12}$$

3. Caution is advised regarding the higher unit flow condition in the low-flow zone. Large boulders and meanders in the trickle zone of the basin may help dissipate the jet and may reduce downstream if riprap extended downstream along the low-flow channel. When large boulders are used as baffles in the impingement area of the low-flow zone, the low-flow basin length L_{bt}, may be reduced, but not less than L_{bm}. Boulders should project into the flow 0.6 to 0.8 times the critical depth. They should be located between the point where the nape hits the basin and no closer than 10 feet from the basin end.

4. The basin floor elevation should be depressed in depth, and variable with drop height. Note that the basin depth adds to the effective tailwater depth for jump control. The basin can be constructed of concrete or grouted rock. Use of either material must be evaluated for hydraulic forces and seepage uplift.

   There should be a sill at the basin end to bring the invert elevation to that of the downstream channel and sidewalls extending from the crest wall to the sill. The sill is important in causing the hydraulic jump to form in the basin. Buried riprap should be used downstream of the sill to minimize any local scour caused by the lift over the sill.

5. Caution is advised to avoid flow impinging on the channel side slopes of the basin.

6. Crest wall and footer dimensions should be determined by conventional structural methods. Underdrain requirements should be determined from seepage analysis.

7. Seepage uplift conditions require evaluations for each use. Thus, seepage analysis should be completed to provide for control and weight/size of components (see Section 2.6).

8. Simplified design criteria are provided in Table HS-6 for vertical hard basin (grouted boulder) drops. These criteria are valid only where the channel flow conditions meet the criteria in the MAJOR DRAINAGE chapter of this *Manual* and the drop does not exceed 3-feet in height.

9. Drops with reinforced concrete basins will have slab thickness and drop lengths that vary somewhat from the simplified design in Item 8 above, depending upon hydraulic and seepage considerations.

Table HS-6—Vertical Drops With Grouted Boulder Basin: Simplified Design Criteria for Small Vertical Drops in Grass-Lined Channels Meeting District Criteria

Design Parameter	Criterion
Maximum Drop Height	3 feet, invert to invert
Boulder size—D_r*	18 inch minimum dimension
Grout thickness—D_g	10 inches**
Basin depression—B (see Figure HS-10)	1.5 ft
Basin length—L_b (see Figure HS-10)	25 ft
Approach length—L_a	10 ft buried riprap
Trickle flow zone provisions	Install large boulder or baffles in center zone to break up high flow stream, or apply separate water surface analysis
Other provisions	A buried riprap zone should be installed for 10 ft minimum downstream of the drop basin Consider the possible hazard to public when selecting this type of drop for use in urban areas.

* Boulder size refers to the minimum dimension of all boulders measured in any direction.

** Bury all grouted boulders on side slopes by filling all gaps and depressions to top of boulders with lightly compacted topsoil and capping with at least 4 inches of top soil; however, capping it with 6 to 12 inches of topsoil will insure a much more robust conditions the native grasses to be seeded on the soil cap.

2.5 Baffle Chute Drops

The USBR has developed design standards for a reinforced concrete chute with baffle blocks on the sloping face of the drop, commonly referred to as baffled apron or baffle chute drops. There are references such as *Hydraulic Design of Stilling Basins and Energy Dissipators* (Peterka 1984) and *Design of Small Canal Structures* (Aisenbrey, et al. 1978) that should be used for the design of these structures. A baffle chute drop was constructed on Harvard Gulch that can be inspected for long-term performance (Wright 1967).

The hydraulic concept involves flow repeatedly encountering obstructions (baffle blocks) that are of a nominal height equivalent to critical depth. The excess energy is dissipated through the drop by the momentum loss associated with reorientation of the flow. A minimum of four rows of baffle blocks is recommended to achieve control of the flow and maximum dissipation of energy. Guidelines are given for sizing and spacing the blocks. Designing for proper approach velocities is critical to structure performance. One advantage of this type of drop is that it does not require tailwater control. However, the designer does need to consider local flow and scour patterns in the transition back to the channel.

Optimal performance occurs for a unit discharge of 35 to 60 cfs/ft of chute width, which happens to be a well-matched design for the District's grass-lined channel criteria. Refer to Rhone (1977) for guidance on higher unit discharge and entrance modifications to address backwater effects.

Photograph HS-9—Close-up of the inside workings of a baffle chute drop after more than three decades of service.

The typical design consists of upstream transition walls, a rectangular approach chute, a sloping apron of 2:1 or flatter slope that has multiple rows of baffle blocks and downstream transition walls. The toe of the chute extends below grade and is backfilled with loose rock to prevent undermining of the structure by eddy currents or minor degradation of the downstream channel. This rock will rearrange to establish a stable bed condition and produce additional stilling action. The structure is effective without tailwater; however, tailwater reduces scour at the toe. Grouted and concrete basins have been used at the transition to the downstream trickle and main channels. The structure also lends itself to a variety of soils and foundation conditions.

There are fixed costs associated with the upstream transition walls, crest approach section, downstream transition walls and a minimum length of sloping apron (for four baffle rows). Consequently, the baffle chute becomes more economical with increasing drop height.

The potential for debris accumulation and subsequent maintenance must be considered. Caution is advised regarding streams with heavy debris flow because the baffles can become clogged, resulting in overflow, low energy dissipation, and direct impingement of the erosive stream jet on the downstream channel. Baffle chute drops are best suited for grass-lined channels and should not be used for boatable streams.

The basic design criteria and details are given in Figure HS-11 (adapted from Peterka 1984). Remaining structural design parameters must be determined for specific site conditions. Recommended design procedures are as follows:

1. Determine the maximum inflow rate and the design unit discharge, $q = \frac{Q}{W}$.

2. An upstream channel transition section with vertical wingwalls constructed 45 degrees to the flow direction causes flow approaching the rectangular chute section to contract. It is also feasible to use walls constructed at 90 degrees to the flow direction. In either configuration, it is important to analyze the approach hydraulics and water surface profile. Often, the effective flow width at the critical cross section is narrower than the width of the chute opening due to flow separation at the corners of the abutment (see Section 5.0).

3. The entrance transition should be followed by a rectangular flow alignment apron, typically 5 feet in length. The upstream approach channel velocity, V, should be as low as practical and less than critical velocity at the control section of the crest. Figure HS-11 gives the USBR-recommended chute entrance velocity. In a typical grass-lined channel, the entrance transition to the rectangular chute section will produce the desired upstream channel velocity reduction. The chute elevation (shown in Figure HS-11) should only be above the channel elevation when approach velocities cannot be controlled by the transition. Extra measures to prevent upstream aggradation are required with the raised crest configuration.

4. Normally, the baffles should be sized at height, H, equal to 0.8 times critical depth at peak flow. The chute face slope should be 2:1 for most cases but may be reduced for low drops or where a flatter slope is desirable. For unit discharge applications greater than 60 cfs/ft, the baffle height may be based on two-thirds of the peak flow; however, the chute sidewalls should be designed for peak flow (see Step 8 below).

   Baffle block widths and spaces should equal approximately $1.5H$ but not less than H. Other baffle block dimensions are not critical hydraulically. The spacing between the rows of baffle block should be H times the slope ratio. For example, a 2:1 slope makes the row spacing equal to $2H$ parallel to the chute floor. The baffle blocks should be constructed with the upstream face normal to the chute floor.

5. Four rows of baffle blocks are required to establish full control of the flow. At least 1½ rows of baffles should be buried in riprap where the chute extends below the downstream channel grade. Rock protection, assumed here as Type M riprap, should continue from the chute outlet to a minimum distance of approximately $4H$ at a riprap layer depth of 2.0 feet to prevent eddy currents from undermining the walls. Additional rows of baffles may need to be buried below grade to allow for downstream channel degradation. Determine if the downstream channel grade has been stabilized to determine how many rows of baffles may need to be buried.

6. The baffle chute wall height (measured normal to the floor slope) should be 2.4 times the critical depth based on peak discharge. The wall height will contain the main flow and most of the splash. The designer of the area behind the wall should consider that some splash may occur, but extensive protection measures are not required.

7. Determine upstream transition and apron sidewall height as required by backwater analysis. Lower basin wingwalls generally should be constructed normal to the chute sidewalls at the chute outlet to prevent eddy current erosion at the drop toe. These transition walls should be of a height equal to the channel normal depth in the downstream channel plus 1 foot and length sufficient to inhibit eddy current erosion.

8. The trickle flow channel should be maintained through the entrance transition apron, approach, and crest sections. It may be routed between the first row of baffle piers. The trickle channel should start again at the basin rock zone that should be slightly depressed and then graded up to transition into the downstream channel to focus the low flows into the trickle channel. Figure HS-12 illustrates one method of designing the trickle channel through the crest.

9. The conventional design shown in Figure HS-11 results in the top elevation of the baffles being higher than the crest, which causes a backwater effect upstream. Figure HS-12 may be used to estimate the extent of the effect and to determine corrective measures such as increasing the upstream freeboard or widening the chute. Note that blocks projecting above the crest will tend to produce upstream sediment aggradation. Channel aggradation can be minimized by the trickle channel treatment suggested in Step 8.

   Another means of alleviating these problems is by using the Fujimoto entrance developed by the USBR and illustrated in Figure HS-12. The upper rows of baffles are moved one row increment downstream. The important advantage of this entrance is that there is not a backwater effect of the baffles. The serrated treatment of the modified crest begins disrupting the flow entering the chute without increasing the headwater. More importantly, this configuration provides a level crest control. The designer may either bring the invert of the upstream trickle channel into this crest elevation, widening the trickle channel as it approaches the crest, or he or she may have a lower trickle channel and bring it through the serrated crest similar to Step 8.

10. Concrete walls and footer dimensions should be determined by conventional structural methods. Cutoffs and underdrain requirements should be determined by seepage analysis discussed earlier in this chapter.

11. The hydraulic impact forces on the baffles should be determined to allow the structural engineer to size adequate reinforcing steel. Figure HS-12 may be used as a guideline. The structural engineer should apply a conservative safety factor.

2.6 Seepage Control

2.6.1 Seepage Analysis Methods

The **preferred deterministic methodology** for seepage analysis is the use of manual and computerized flow net analyses. It is used to quantify groundwater flows, pressures, and critical gradients under hydraulic structures. Flow net analysis can quantify the effects of multiple strata of different soil media and complex geometries and situations. Full decryption of flow net analysis is beyond the scope of this *Manual* and the user is referred to Cedergren (1967), USBR (1987) and Taylor (1967) for more information and instruction in the use of flow net analysis techniques.

At an absolute minimum and as a first order of estimation, Lane's Weighted Creep Method (CWM) can be used to identify probable seepage problems, evaluate the need for control measures, and roughly estimate uplift forces. It is not as definitive as the above-mentioned flow net analysis. The CWM technique was originally proposed by E.W. Lane in 1935. This method has been deleted, however, in the 1987 revision of *Design of Small Dams* (USBR 1987), possibly indicating greater use of flow net and computer modeling methods or for other reasons that we do not know about. Although Lane's method is relatively well founded, it should be used as a guideline, and when marginal conditions or complicated geological conditions exist, the more sophisticated flow-net analysis should be used. The essential elements of Lane's method are as follows:

1. The weighted-creep distance through a cross section of a structure is the sum of the vertical creep distances, L_v (along contact surfaces steeper than 45 degrees), plus one-third of the horizontal creep distances, L_H (along contact surfaces less than 45 degrees).

2. The weighted-creep head ratio is defined as:

$$C_W = \frac{\left(\frac{L_H}{3} + L_V\right)}{H_S} \tag{HS-13}$$

   in which:

   C_W = creep ratio

   H_S = differential head between analysis points (ft)

3. Reverse filter drains, weep holes, and pipe drains help to reduce seepage problems, and recommended creep head ratios may be reduced as much as 10% if they are used.

4. In the case where two vertical cutoffs are used, then Equation HS-13 should be used along with Equation HS-14 to check the short path between the bottom of the vertical cutoffs.

$$C_{W2} = \frac{\left(L_{V-US} + \quad 2L_{H-C} + L_{V-DS}\right)}{H_S} \tag{HS-14}$$

in which:

C_{W2} = creep ratio where two vertical cutoffs are used

L_{V-US} = vertical distance on the upstream side of the upstream cutoff (ft)

L_{V-DS} = vertical distance on the downstream side of the downstream cutoff (ft)

L_{H-C} = horizontal distance between the two vertical cutoffs (ft)

5. If there are seepage lengths upstream or downstream of the cutoffs, they should be treated in the numerator of Equation HS-14 similar to Equation HS-13. Seepage is controlled by increasing the total seepage length such that C_W or C_{W2} is raised to the value listed in Table HS-7. Soils tests must be conducted during design and confirmed during construction.

6. The upward pressure to be used in design may be estimated by assuming that the drop in uplift pressure from headwater to tailwater along the contact line of the dam and foundation is proportional to the weighted-creep distance.

Table HS-7—Lane's Weighted Creep: Recommended Ratios

Material	Ratio
Very fine sand or silt	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0
Fine gravel	4.0
Medium gravel	3.0
Coarse gravel including cobbles	3.0
Boulders with some cobbles and gravel	3.0
Soft clay	3.0
Medium clay	2.0
Hard clay	1.8
Very hard clay or hardpan	1.6

2.6.2 Foundation/Seepage Control Systems

Table HS-8 presents some typical foundation conditions and systems that are often used for various drop heights. For each condition, cutoff types are listed in general order of preference for guidance purposes only. As a general rule, it is not recommended that groundwater flow cutoffs not be installed at the downstream ends of drop structures. Their presence can cause greater hydraulic uplift forces than would exist without a downstream cutoff. The design goal is to relieve the hydrostatic pressures along the structure and not to block the groundwater flow and cause higher pressures to build up.

The hydraulic engineer must calculate hydraulic loadings that can occur for a variety of conditions such as during construction, during dominant low flows, during flood flows, during design flows and other critical loading scenarios. The soils/foundation engineer combines this information with the on-site soils information to determine foundation requirements. Both engineers should work with a structural engineer to establish final loading diagrams and in selection and sizing of structural components.

Table HS-8—General Cutoff Technique Suitability

	Drop Height (ft)			
Soil Conditions	2	4	8	12
Sands and gravel over bedrock with sufficient depth of material to provide support—groundwater prevalent	SP[1] CTc CTf	SP[1] CTc/ST CTf/CTI	Sp/SwB[1] ST	Sp/SwB[1] ST
Sands and gravel with shallow depth to bedrock—groundwater prevalent	CTc CW SP[2]	CTc/ST CW SP[2]	ST CW SP[2]	ST CW SwB[2]
Sands and gravel with large depths to bedrock—groundwater prevalent	SP CTc	SP CTc/ST	SP ST	SP/SwB ST
Sands and gravel, no groundwater, or water table normally below requirement (for variation caused by depth to bedrock, see first case)	SP CTf/CTI CW	SP CTI CW	SP CTI	SP/SwB CTI
Clay (and silts)—medium to hard	CTc	CT	CT	CT
	CW	Reduce length for difficult backfill conditions		
	CTI/CTf	Only for local seepage zones/silts		
	ST	Expensive—for special problems		
Clays (and silts)—soft to medium with lenses of permeable material—groundwater present	CP CTc	SP CTc	SP CTc/ST	SP/SwB ST
Clay (and silts)—soft to medium with lenses of permeable material (may be moist but not significant groundwater source)	SP CTc CTf CW	SP CTc CTI CW	SP CTc/ST CTI CW	SP/SwB ST CTI CW

[1] Consider scour in sheet pile support.
[2] Excavate into bedrock and set into concrete.

Legend:

- SP Sheet pile
- SwB Sheet pile with bracing and extra measures
- CTc Cutoff trench backfilled with concrete
- ST Slurry trench; similar to CTc, but trench walls are supported with slurry and then later replaced with concrete or additives that provide cutoff
- CW Cutoff wall; conventional wall, possibly with footer, backfilled; note that the effective seepage length should generally be decreased because of backfill
- CTI Cutoff trench with synthetic liner and fill
- CTf Cutoff trench with clay fill

Photograph HS-10—Boatable channels of the District waterways provide enjoyment to a wide variety of citizens. The South Platte River example in this photograph provides an easily accessible boating experience.

2.7 Simplified Minimum Design Approach for Boatable Channels

Due to the fact that a special standard of care for the design of drops and low-head dams on boatable channels is required, the following design approach for boatable channels is limited to suggestions for the experienced hydraulic structure designer once the channel has been determined to be a boatable one.

1. Contact reliable whitewater boating experts to discuss general design objectives and boater safety concerns.

2. Select maximum height of individual drops—generally 4 feet. If they are more than 4 feet, a physical hydraulic model may be necessary.

3. Determine basic drop characteristics to be compatible with public safety and recreational boating. Suggestions are as follows:

    - Use a Froude number, F_r, less than 1.5 at the toe of the drop.

    - Avoid reverse rollers under all conditions of flow.

    - Assess stability of the structure taking into account expected downstream channel degradation.

    - Consider the slope of the downstream face of a sloping drop; 10(H) to 1(V) is common.

Photograph HS-11—Unprotected urban channels can experience bank erosion and degradation when established design criteria are not used. The invert of pipe used to be at invert of channel before degradation occurred.

- Provide boat chute with pilot rocks for routine boat passage of drop.
- Do not use an energy dissipating basin; instead, continue the sloping surface at least 5 feet below the downstream thalweg of the stream.
- Provide adequate warning signs and portage area.
- Use grouted sloping boulder or appropriately sized large ungraded sloping boulder structure.
- Consider vertical cutoff walls at the upstream end for seepage control.

4. Obtain peer review on the preliminary design.
5. Allow for follow-up rock adjustment after completion, especially for boat chutes.

2.8 Construction Concerns: Grass-Lined Channels

The selection of a drop or a grade control check and its foundation may be tempered by construction difficulty, access, material delivery, etc. Some of the important concerns are discussed below, although this is by no means an exhaustive list of the concerns possible for every site and situation.

2.8.1 Foundation/Seepage Control

Initial items that are especially important are site water control and foundation conditions. A common problem is destabilization of the foundation soils by rapid local dewatering of fine-grained, erosive soils, or soils with limited hydraulic conductivity. Often the preferred method is continuous pumping rates at perimeter locations (or well points) that allow the entire construction area to remain stable. Appropriate water control techniques for use during construction of a drop structure should be presented to the contractor. Diversion berms should be designed with planned berm failure points to avoid flooding of drop-structure sites during construction.

The actual subgrade condition with respect to seepage control assumptions must be inspected and field verified. The engineer who established the design assumptions and calculated the required cutoffs should inspect the cutoff for each drop and adjust the cutoff for the actual conditions encountered. For example, if the inspection of a cutoff trench reveals a sandy substrate rather than clay, then the cutoff trench may need to be deepened, or a different cutoff type may need to be implemented. Obviously, soil testing is an advisable precaution to minimize changes and avoid failures.

2.8.2 Baffle Chute Construction

There are numerous steps necessary in the construction of a baffle chute, but a contractor usually easily controls them. For quality control and inspection there are consistent, measurable, and repeatable standards to apply.

Baffle chutes are highly successful as far as hydraulic performance is concerned and are straightforward to construct. Steel, formwork, concrete placement and finish, and backfill generally require periodic inspection. Potential problems include foundation integrity, riprap quality control, water control, and the finish work with regard to architectural and landscape treatments. Formwork, form ties, and seal coatings can leave a poor appearance if not done properly.

2.8.3 Vertical Hard Basin Construction

Foundation and seepage concerns are critical with regard to the vertical wall. Poor construction and seepage control can result in sudden failure. The use of caissons or piles can mitigate this effect. Put in comparative terms with the baffle chute, seepage problems can result in displacement of the vertical wall with no warning, where the box-like structure of the baffle chute may experience some movement or cracking, but not total failure, and thus allow time for repairs.

The quality control concerns and measures for vertical basins are the same as for baffle chutes. The subsoil condition beneath the basin is important to insure that the stilling basin concrete or grouted rock bottom is stable against uplift pressures.

2.8.4 Sloping Grouted Boulder Construction

The sloping grouted boulder drops require significant construction control efforts in the field. Seepage analysis is required to determine a compatible combination of cutoff depth, location of weep and toe drains, and the thickness of grouted rock layer. The greatest danger lies with a “sugar-coated” grout job, where the grout does not penetrate the voids fully between the rock and the subgrade and leaves voids below the grout that act as a direct piping route for water, guaranteeing early failure.

Individual boulders should be larger in diameter than the grout layer so that the contractor and the inspector can verify the grout depth and have grout placed directly to the subgrade. The best balance appears to have the grout thickness set at 1/2 the boulder height, but no more than 2/3 boulder height, and to have an overall mass sufficient to offset uplift, plus a safety factor. Limiting grout thickness also improves the overall appearance of the grouted boulder structure.

The condition of the subgrade, adequate seepage control, and sub-drainage of the seepage flow are all critical. There is a tendency to disturb the subgrade during rock placement, leaving a potential piping route. This should be controlled by good subgrade preparation, careful rock placement, and removal of loose materials. Absolutely no granular bedding or subgrade fill using granular materials should be used to prevent conditions that will cause piping. Problems with rock density, durability and hardness are of concern and can vary widely for different locations. The rock should be inspected at regular intervals to meet minimum physical dimensions, strengths, durability and weights as defined in the specifications.

For aesthetic reasons, it is recommended that the grouted boulders above the low flow section and on the banks be covered with local soils, topsoil and revegetated.

2.9 Low-Flow Check and Wetland Structures

Urbanization causes more frequent and sustained flows, and therefore the trickle/low-flow channel and wetlands become more susceptible to erosion even though the overall floodplain may remain stable and able to resist major flood events. Erosion of the low-flow channel, if left uncontrolled, can cause degradation and destabilization of the entire channel. Low-flow grade-control check structures are designed to provide control points and establish stable bed slopes within the base flow channel. They should be used to limit longitudinal slope of the channel to about 0.3% to 0.5% and as described in the MAJOR DRAINAGE chapter. Low-flow check structures are not appropriate along incised floodplains and may not be economical for very steep channels, where higher drop structures may be needed.

Grouted sloping boulder and vertical hard basin designs can be adapted for use as check structures after considering (1) stable bed slopes for the unlined trickle or low-flow channel and (2) potential overflow erosion during submergence of the check structure and where flow converges back from the main channel sides or below the check structure.

The basic design steps for low-flow grade-control check structures include the following:

1. Determine a stable slope and configuration for the low-flow zone. For unlined channels, discharges from full floodplain flow to the dominant discharge should first be considered. The dominant discharge is more fully explained in sediment transport texts (Richardson 1988; Shen 1971; Simons 1977; Simons, Li and Associates 1982; and Muessetter 1983). It is generally defined as the flow that represents the average or equilibrium conditions controlling the channel bed. In the Denver region, the dominant discharge is typically the 2-year flood. Numerous references (Chow 1959; SCS 1977; and above references) cite information on permissible

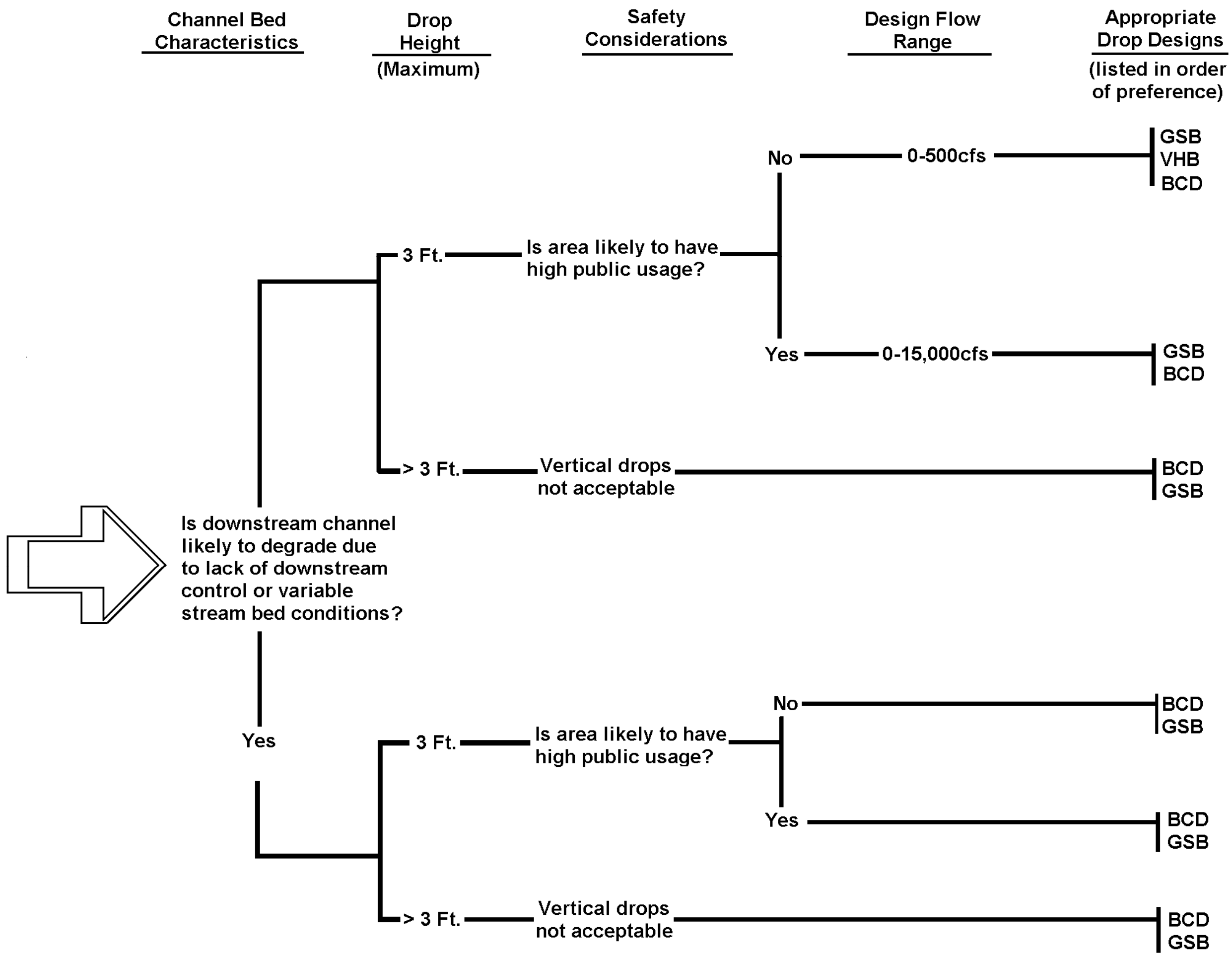
velocities. The range of stable longitudinal slopes for non-rock lined major drainageways in the Denver area is between 0.003 ft/t and 0.005 ft/ft. Two exceptions to this range exist, one is for larger streams and the South Platte River, where it can be much flatter, and the other is for steep waterways with small tributary catchments of relatively low imperviousness, where the final stable slopes can be steeper.

2. The configuration of the low-flow zone and number and placement of the check structures must be reviewed. A good rule is to have the check structures spaced so the drop does not exceed 3-feet after the downstream channel has degraded to the projected stable longitudinal slope.

One type of check structure that can be used to stabilize low-flow channels within relatively stable channels is the control check (see Figure HS-13a and Figure HS-13b). This type of a check structure can be constructed by filling an excavated narrow trench (12' minimum) with concrete if soil and groundwater conditions permit trenching to a depth of 6 feet, or by driving a concrete capped sheet piles to 10 foot depth when trenching is not possible.

Extend the cutoff walls into the main channel banks a minimum of 10 feet and make sure it rises sufficiently to contain the 5- to 10-year flow (depending on local criteria), but no less than 2-feet above the approach channel (outside the trickle flow section) to avoid side cutting.

Wetland channel check structures will typically do not have a trickle channel. When building check structures for wetland bottom channels, place riprap downstream of the cutoff wall to dissipate the kinetic energy when downstream backwater is low so as to avoid deep scour hole downstream.



WHERE:

VHB = VERTICAL HARD BASIN DROPS
GSB - GROUTED SLOPING BOULDER DROPS
BCD = BAFFLE CHUTE DROPS

Figure HS-1—Probable Range of Drop Choices and Heights

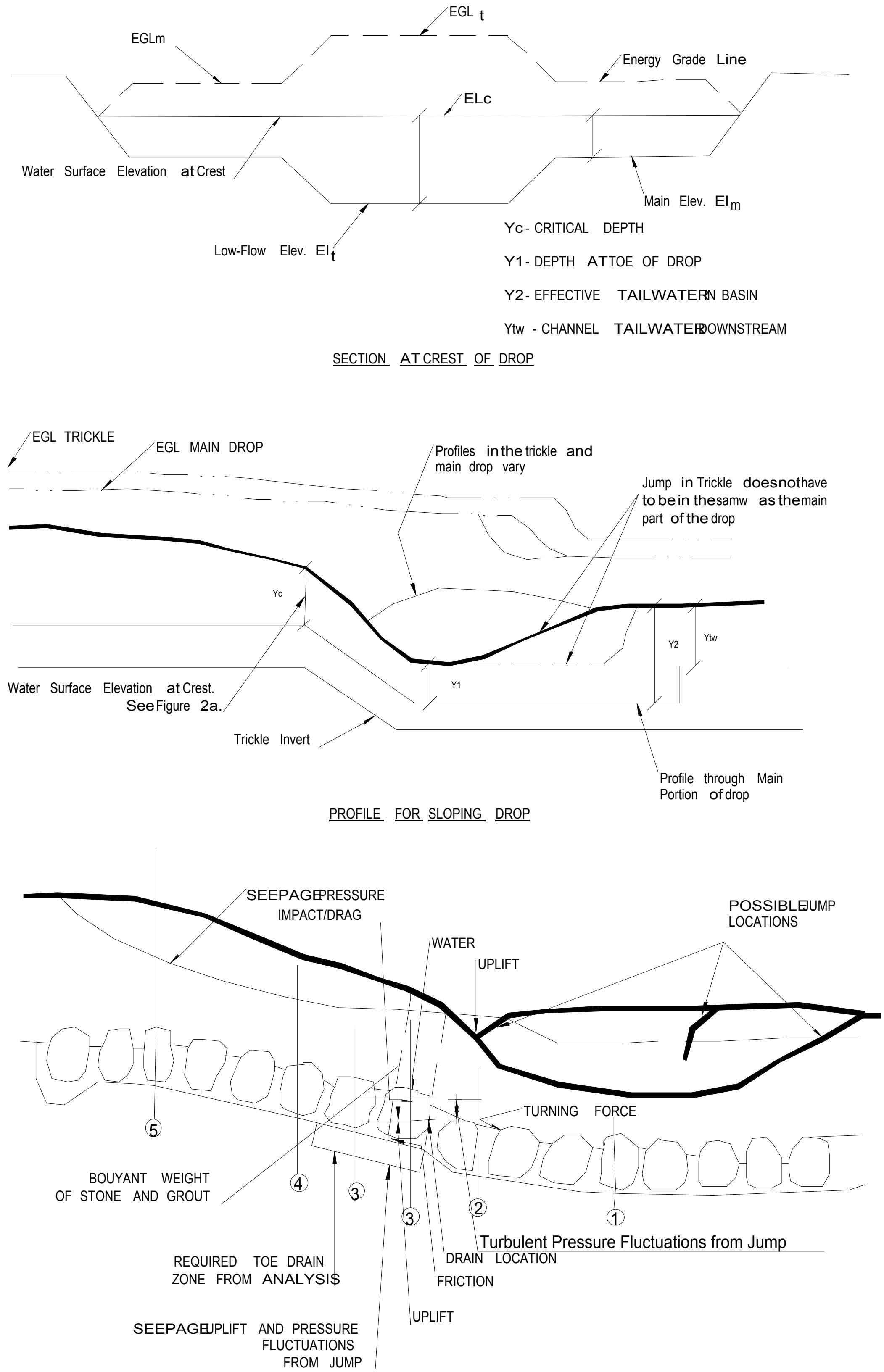
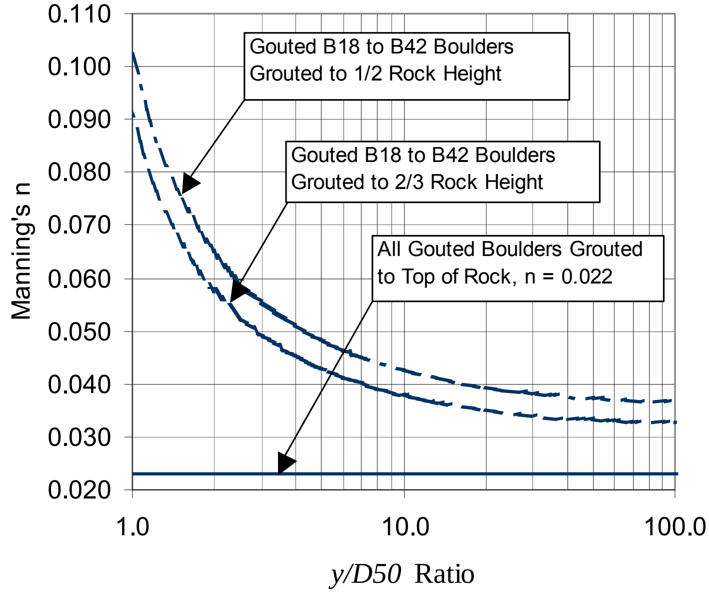


Figure HS-2—Hydraulic Analysis and Typical Forces at Sloping Boulder Drops



The following equations may be used to find the recommended Manning's *n* as a function of flow depth over height of the boulders, *y/D50* represented by the above two curves:

When the upper one-half (+/- 1") of the rock depth (height) is left ungrouted, the equation for *n* is:

$$n_{18''-42''(1/2)} = \frac{0.086 \cdot y^{0.17}}{\ln(1.64 \cdot y)}$$

Upper limit: $n \leq 0.15$ for above equation

When the upper one-third (+/- 1") of the rock depth (height) is left ungrouted, the equation for *n* is:

$$n_{18''-42''(2/3)} = \frac{0.086 \cdot y^{0.17}}{\ln(2.46 \cdot y)}$$

Upper limit: $n \leq 0.12$ for above equation

In both, y = depth of flow above top of rock, in feet

When rock is grouted to the top of the rock, Manning's is a constant n = 0.022.

Note that grouting only the lower ½ of the rock on the sloping face of the drop has a significantly higher Manning's *n* roughness coefficient and, as a result, greater flow depth and lower velocity, reducing the boulder size needed to have a stable structure.

Figure HS-3—Recommended Manning's *n* for Flow Over B18 to B42 Grouted Boulders

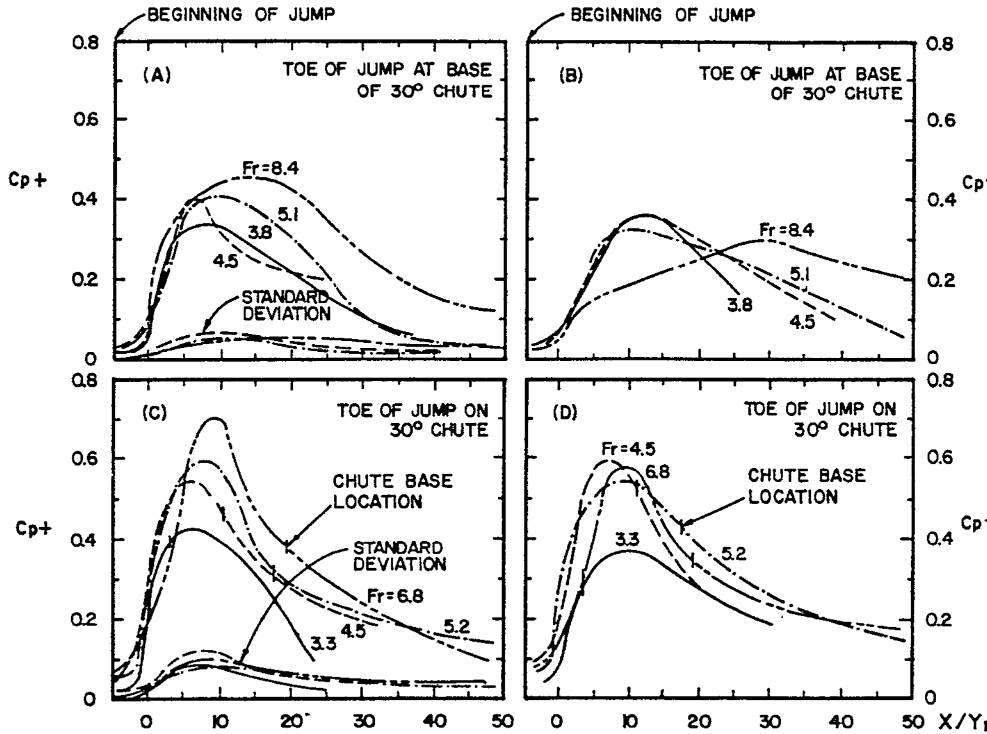


Figure HS-4—Coefficient of Pressure Fluctuation, C_p, at a Hydraulic Jump

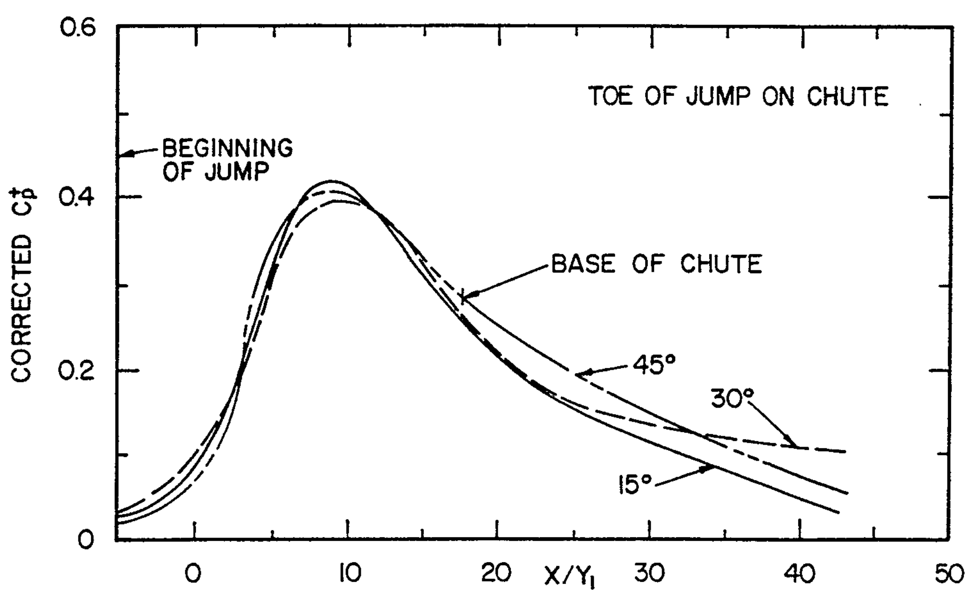


Figure HS-5—Pressure Fluctuation Coefficient, C_p, Normalized for Consideration of Slope and Jump Beginning on Slope

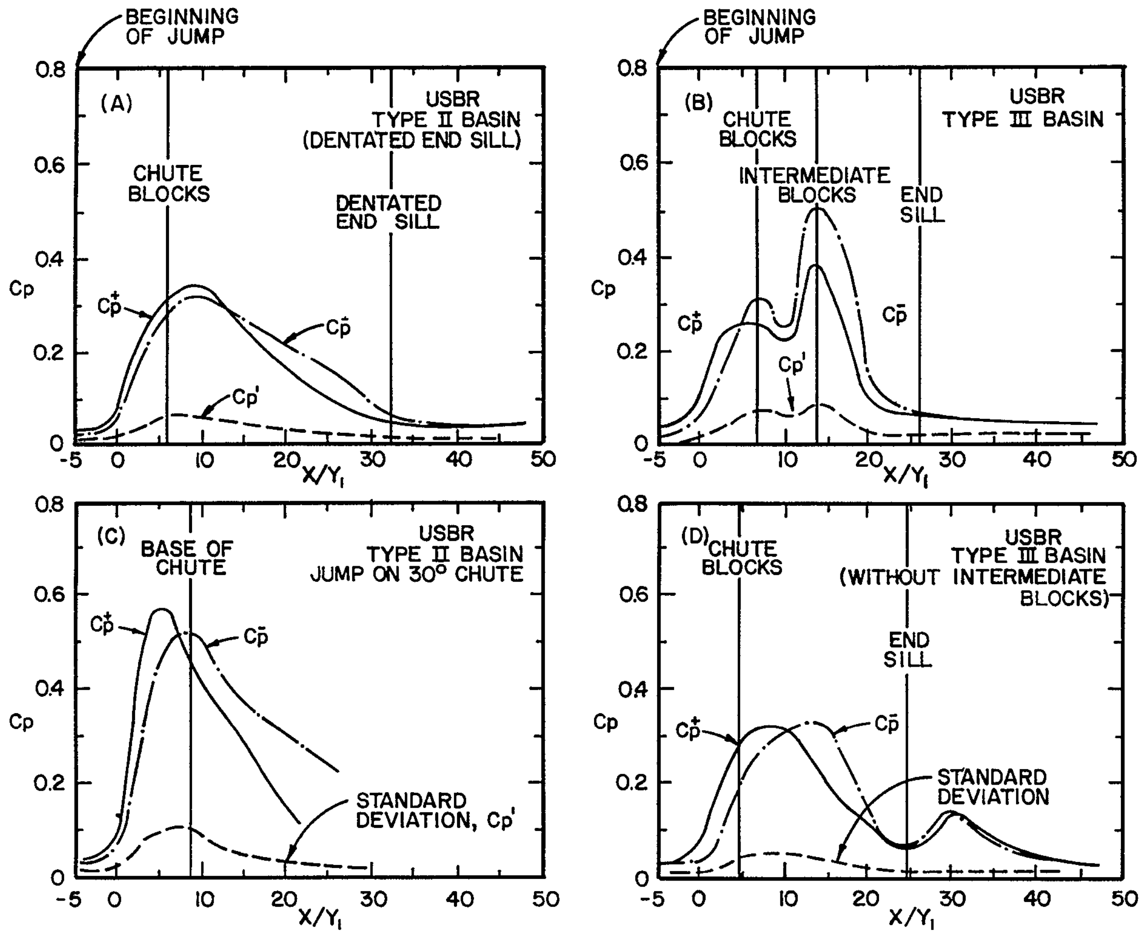
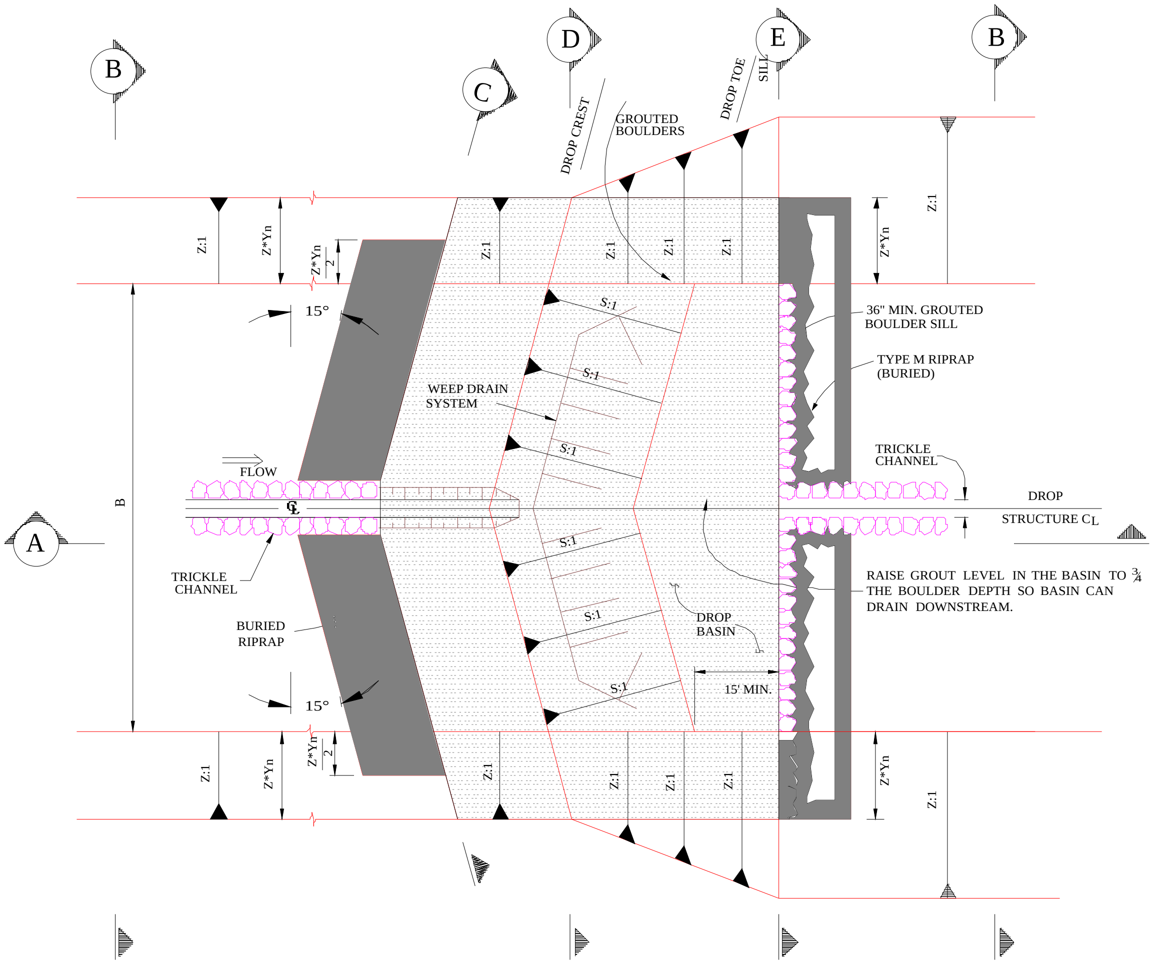
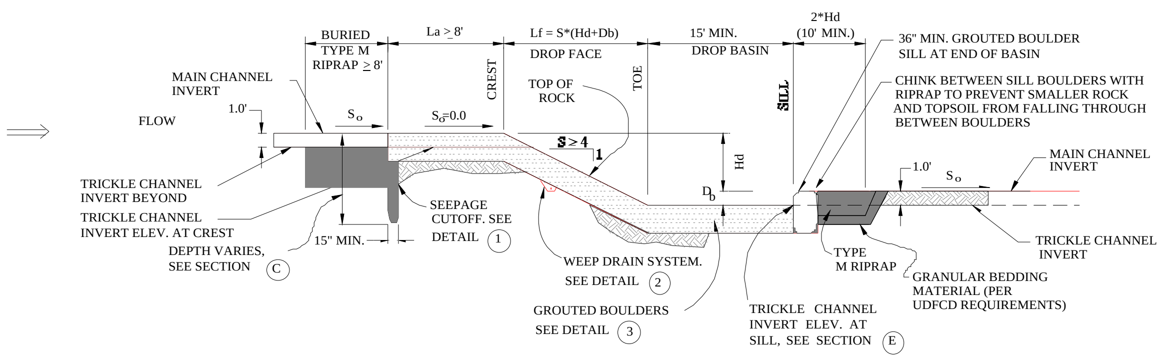


Figure HS-6—Coefficient of Pressure Fluctuation, C_p, in a Jump on a USBR II or III Basin



DROP STRUCTURE PLAN

NTS



DROP STRUCTURE PROFILE A

NTS

Figure HS-7A—Grouted Sloping Boulder Drop with Trickle Channel for Stabilized Channels in Erosion Resistant Soils (Figure 1 of 2)

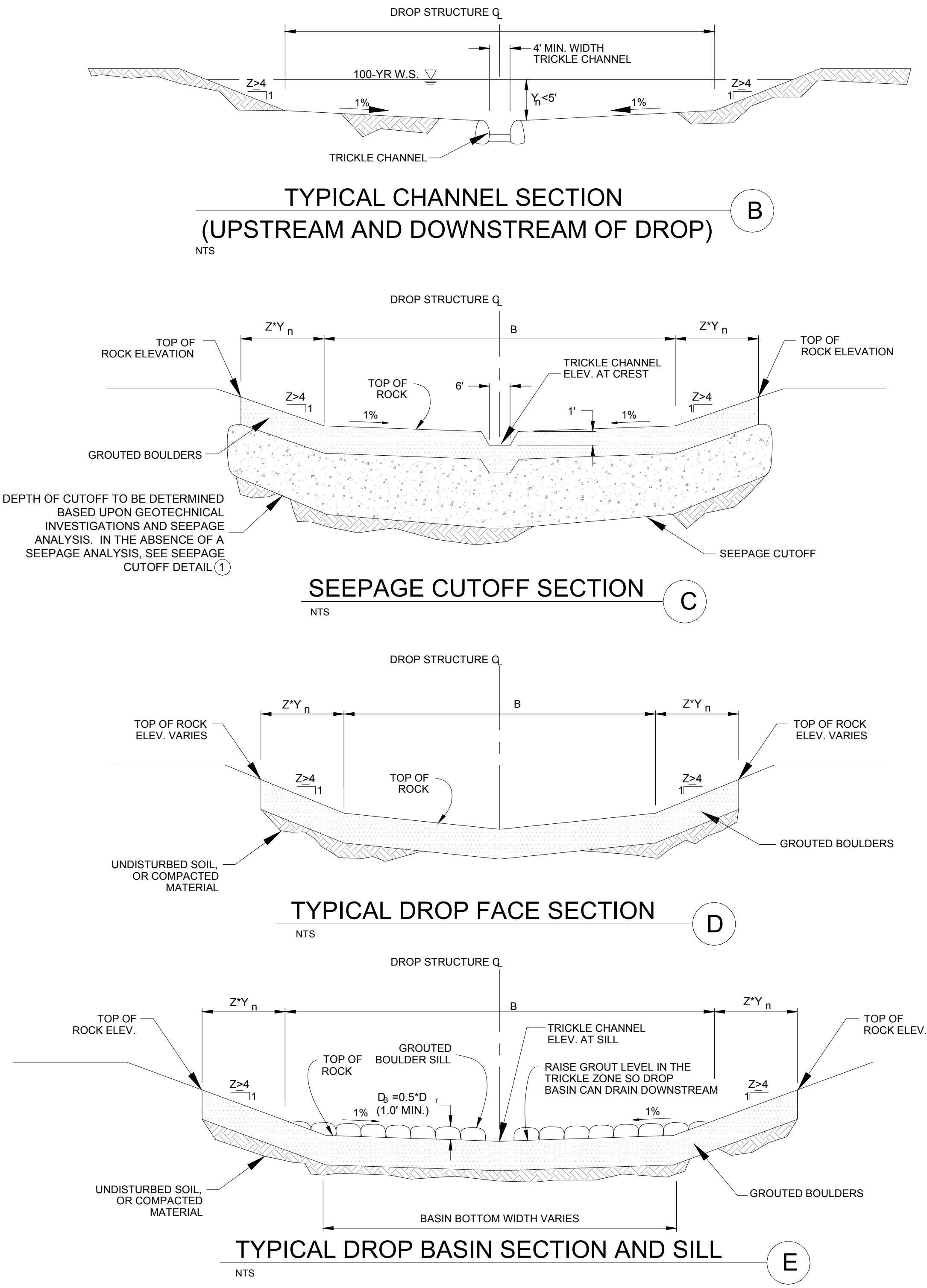


Figure HS-7A— Grouted Sloping Boulder Drop with Trickle Channel for Stabilized Channels and Erosion Resistant Soils (Figure 2 of 2)

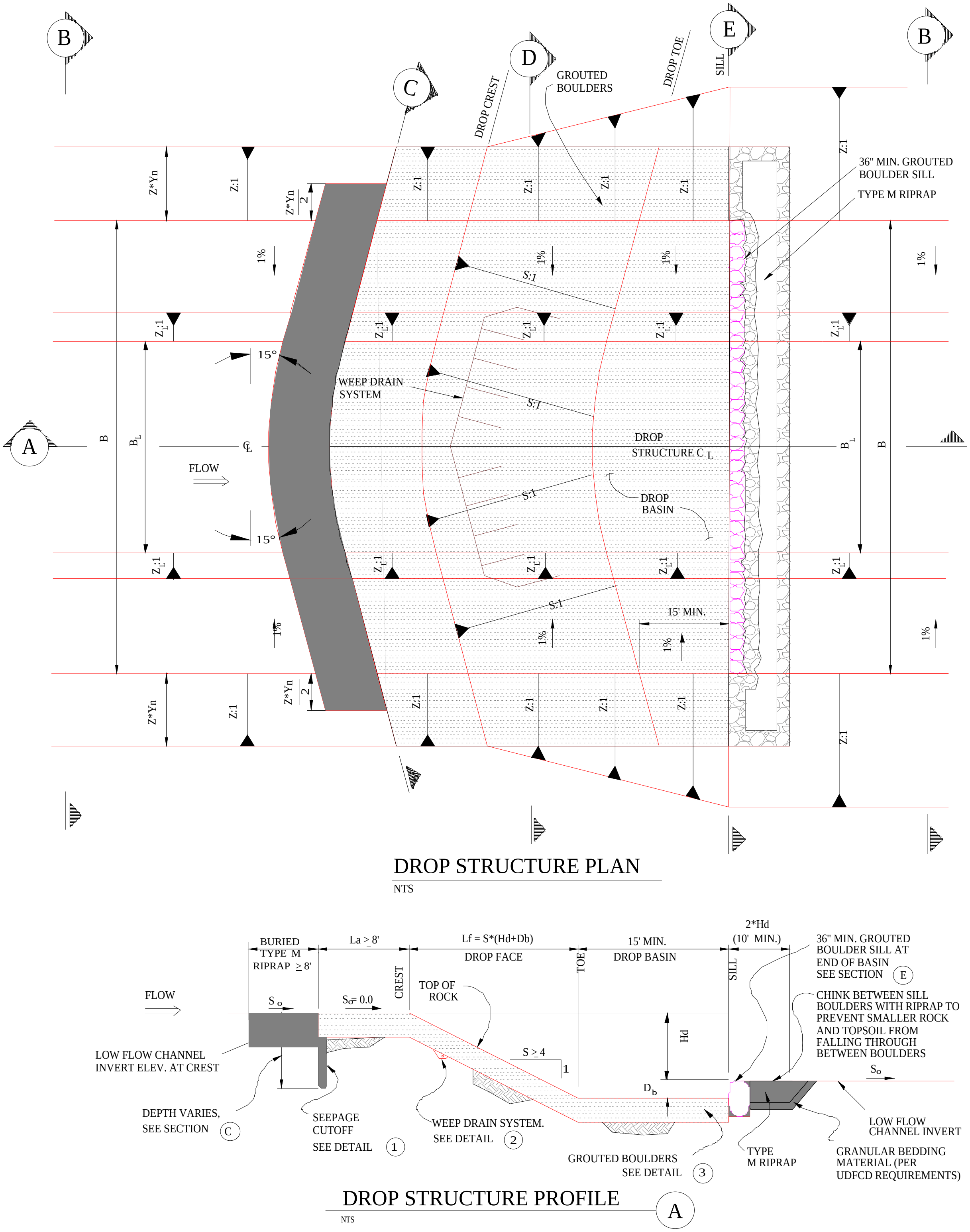
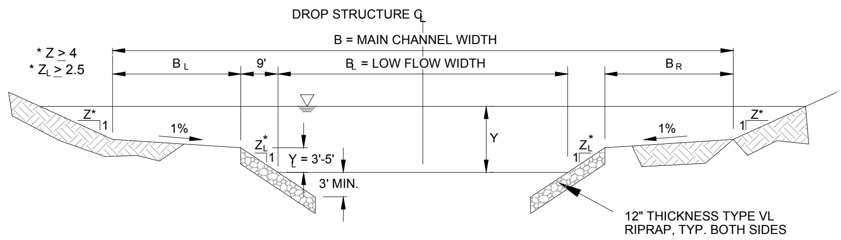


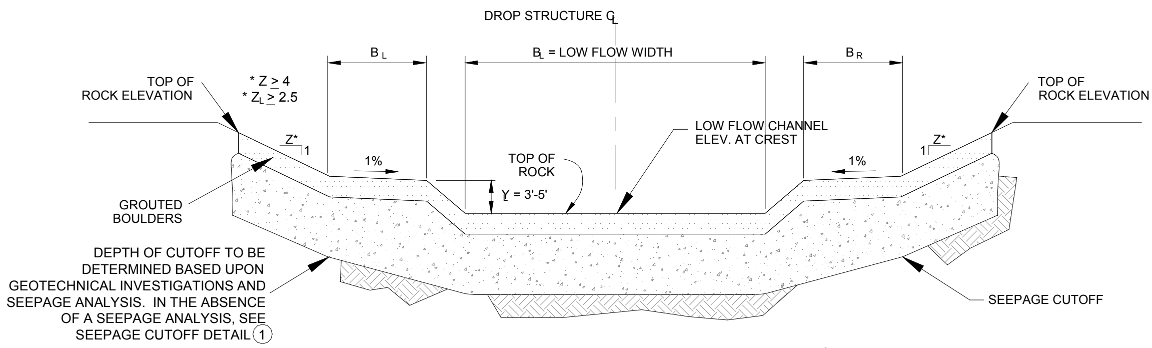
Figure HS-7B—Grouted Sloping Boulder Drop With Low-Flow Channel for Stabilized Channels in Erosion Resistant Soils (Figure 1 of 2)



TYPICAL CHANNEL SECTION
(UPSTREAM AND DOWNSTREAM OF DROP

NTS

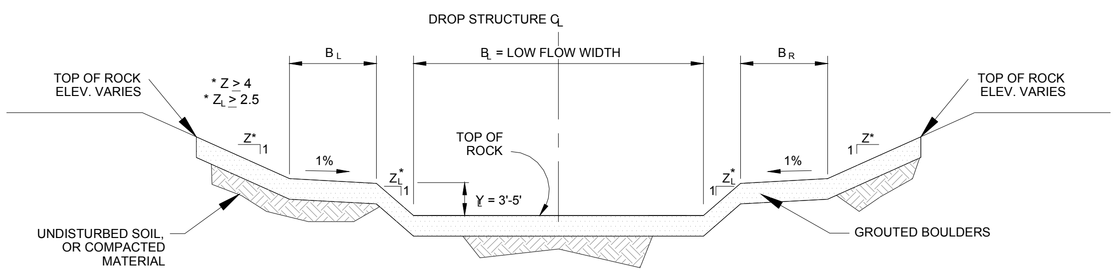
B



SEEPAGE CUTOFF SECTION

NTS

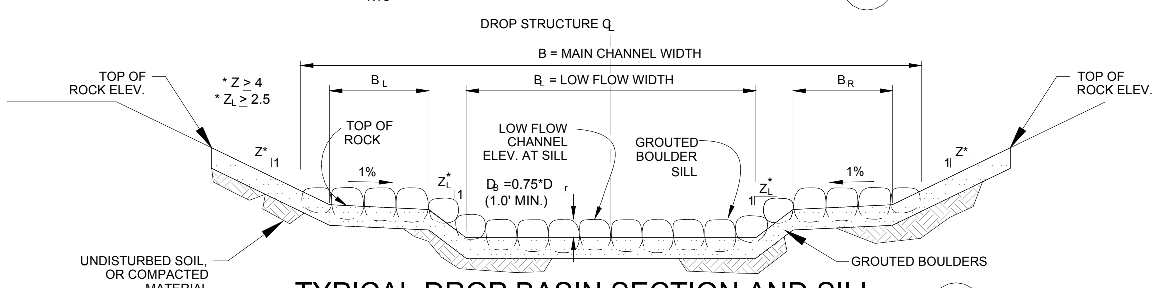
C



TYPICAL DROP FACE SECTION

NTS

D



TYPICAL DROP BASIN SECTION AND SILL

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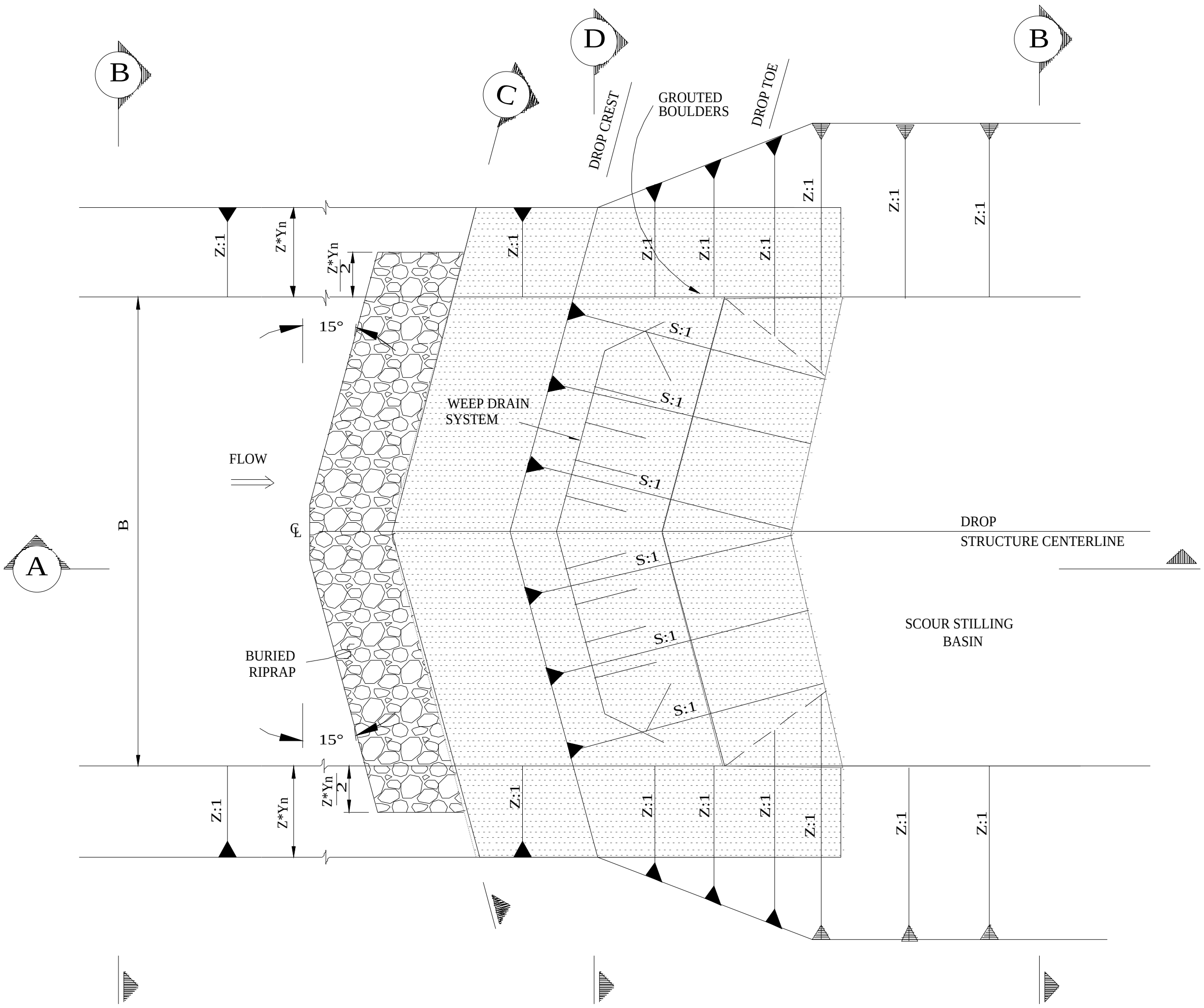
E

Figure HS-7B— Grouted Sloping Boulder Drop With Low-Flow Channel For Stabilized Channels and Erosion Resistant Soils (Figure 2 of 2)



DROP STRUCTURE PLAN

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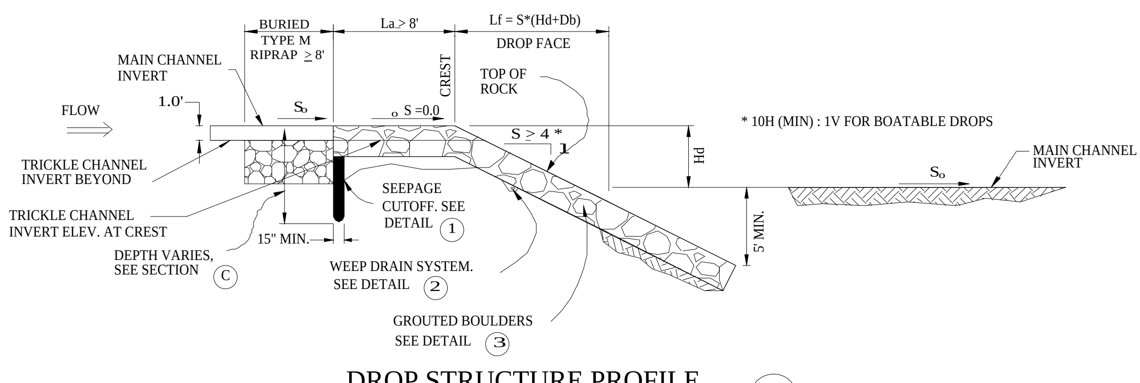
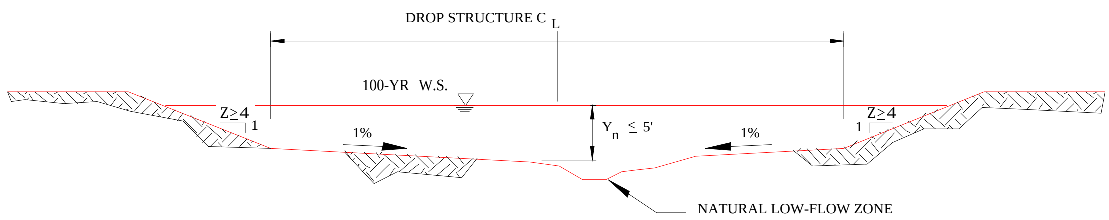


DROP STRUCTURE PROFILE A

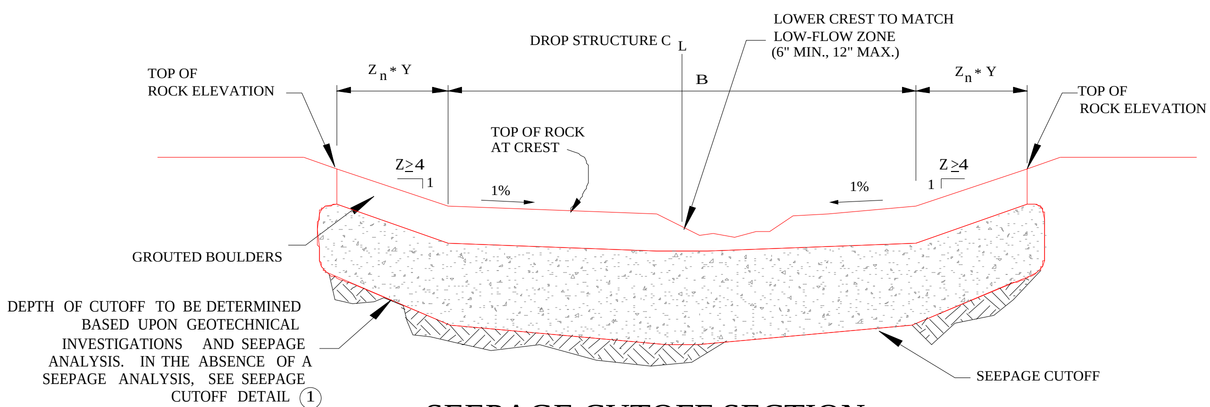
NTS

Figure HS-7C—Grouted Sloping Boulder Drop for Unstable Channels in Erosive Soils (Figure 1 of 2)



TYPICAL CHANNEL SECTION (UPSTREAM AND DOWNSTREAM OF DROP) B

NTS



SEEPAGE CUTOFF SECTION C

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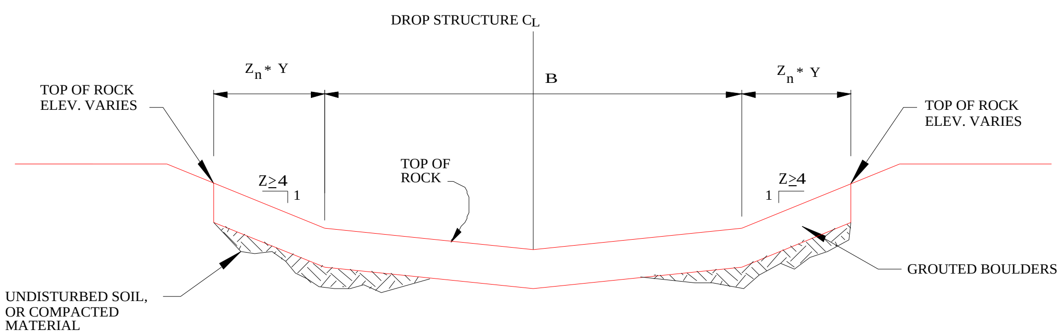
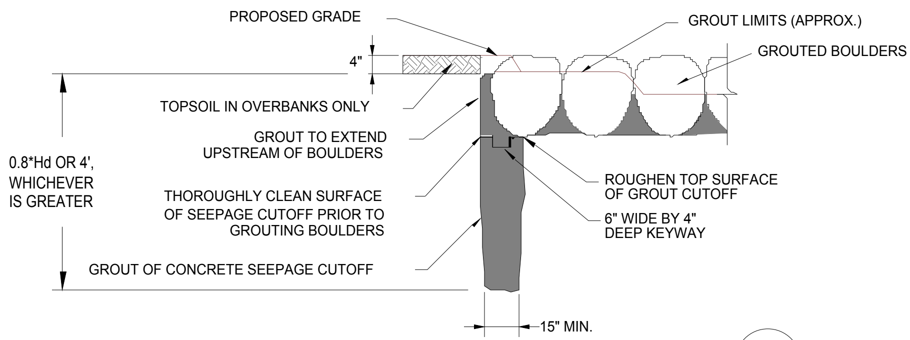


TYPICAL DROP FACE SECTION D

NTS

Figure HS-7C— Grouted Sloping Boulder Drop for Unstable Channels in Erosive Sandy Soils. (Figure 2 of 2)



SEEPAGE CUTOFF DETAIL 1

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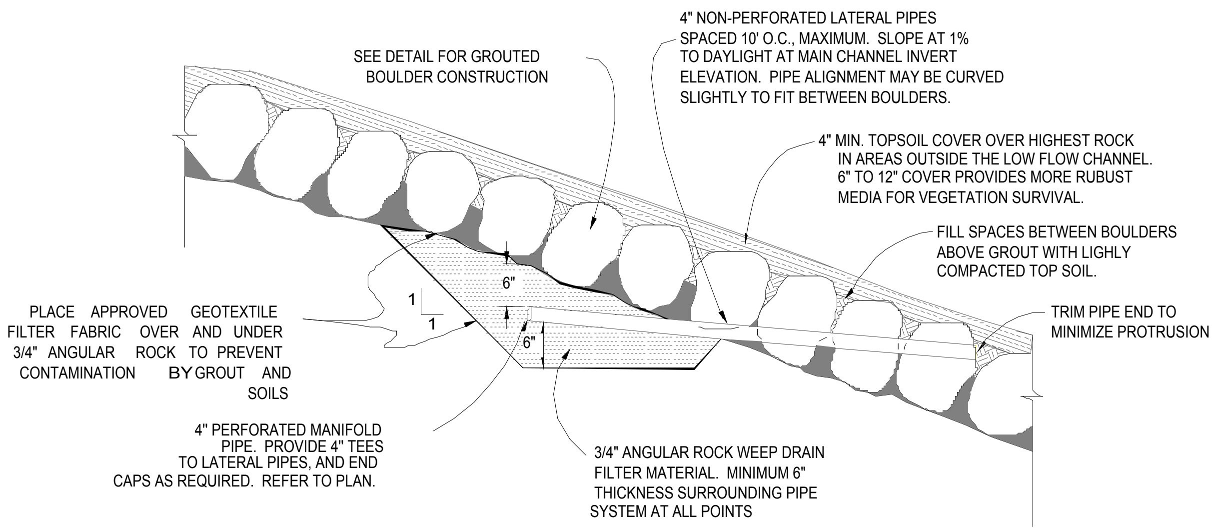

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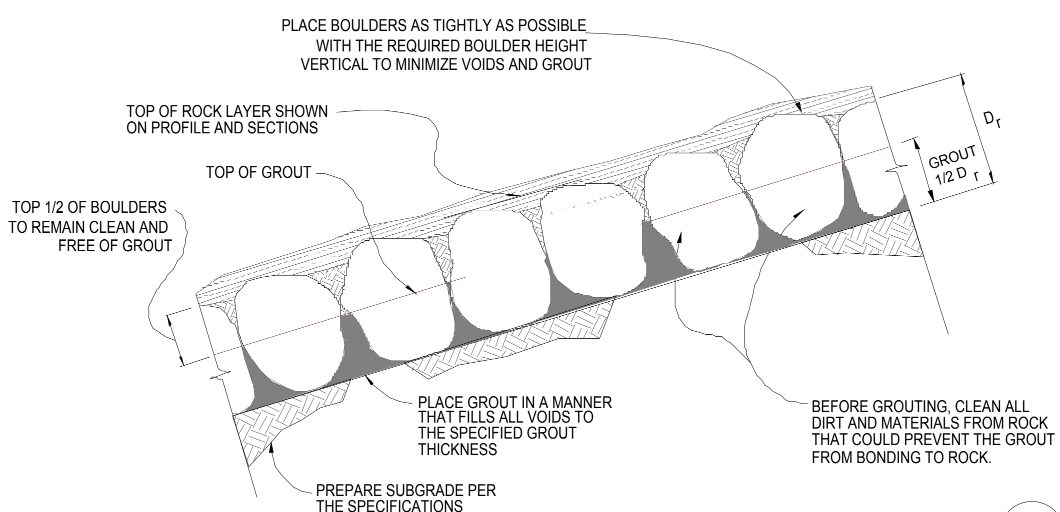
WEEP DRAIN SYSTEM DETAILS 2

USE ONLY IN DROPS HIGHER THAN 5 FEET



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GROUTED BOULDER PLACEMENT DETAIL 3

Figure HS-7D— Grouted Sloping Boulder Drop Details.

GROUT NOTES

Material Specifications

1. All grout shall have a minimum 28-day compressive strength equal to 3200 psi.
2. One cubic yard of grout shall have a minimum of six (6) sacks of Type II Portland cement.
3. A maximum of 25% Type F Fly Ash may be substituted for the Portland cement.
4. For Type A grout, the aggregate shall be comprised of 70% natural sand (fines) and 30% $\frac{3}{8}$ -inch rock (coarse).
5. For Type B grout, the aggregate shall be comprised of $\frac{3}{4}$ -inch maximum gravel, structural concrete aggregate.
6. Type B grout shall be used in streams with significant perennial flows.
7. The grout slump shall be 4-inches to 6-inches.
8. Air entrainment shall be 5.5%-7.5%.
9. To control shrinkage and cracking, 1.5 pounds of Fibermesh, or equivalent, shall be used per cubic yard of grout.
10. Color additive in required amounts shall be used when so specified by contract.

Placement Specifications

1. All Type A grout shall be delivered by means of a low pressure (less than 10 psi) grout pump using a 2-inch diameter nozzle.
2. All Type B grout shall be delivered by means of a low pressure (less than 10 psi) concrete pump using a 3-inch diameter nozzle
3. Full depth penetration of the grout into the boulder voids shall be achieved by injecting grout starting with the nozzle near the bottom and raising it as grout fills, while vibrating grout into place using a pencil vibrator.
4. After grout placement, exposed boulder faces shall be cleaned with a wet broom.
5. All grout between boulders shall be treated with a broom finish.
6. All finished grout surfaces shall be sprayed with a clear liquid membrane curing compound as specified in ASTM C-309.
7. Special procedures shall be required for grout placement when the air temperatures are less than 40°F or greater than 90°F. Contractor shall obtain prior approval from the design engineer of the procedures to be used for protecting the grout.
8. Clean Boulders by brushing and washing before grouting.

Figure HS-8—Specifications and Placement Instructions for Grout in Sloping Boulder Drops.

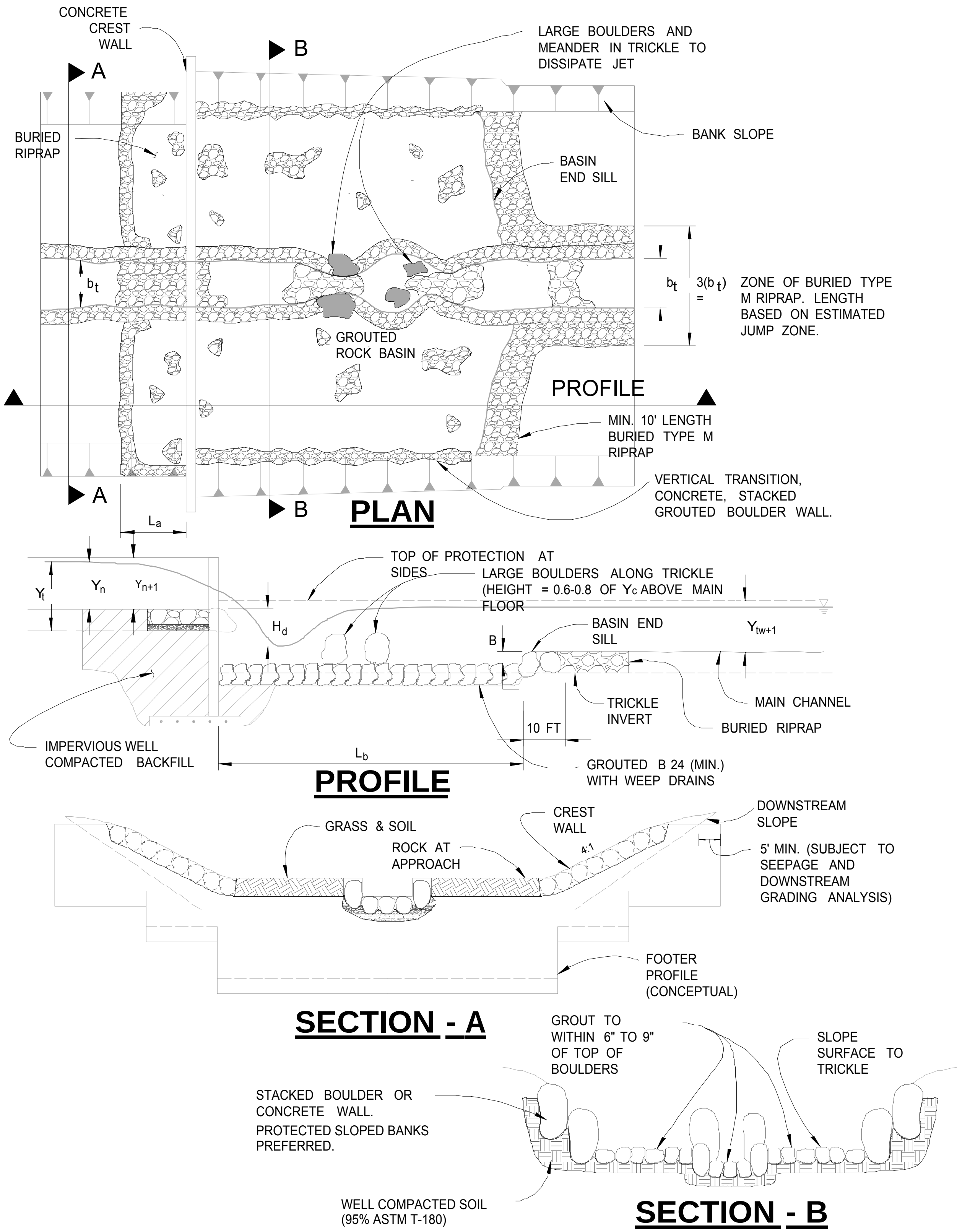


Figure HS-9—Vertical Hard Basin Drop

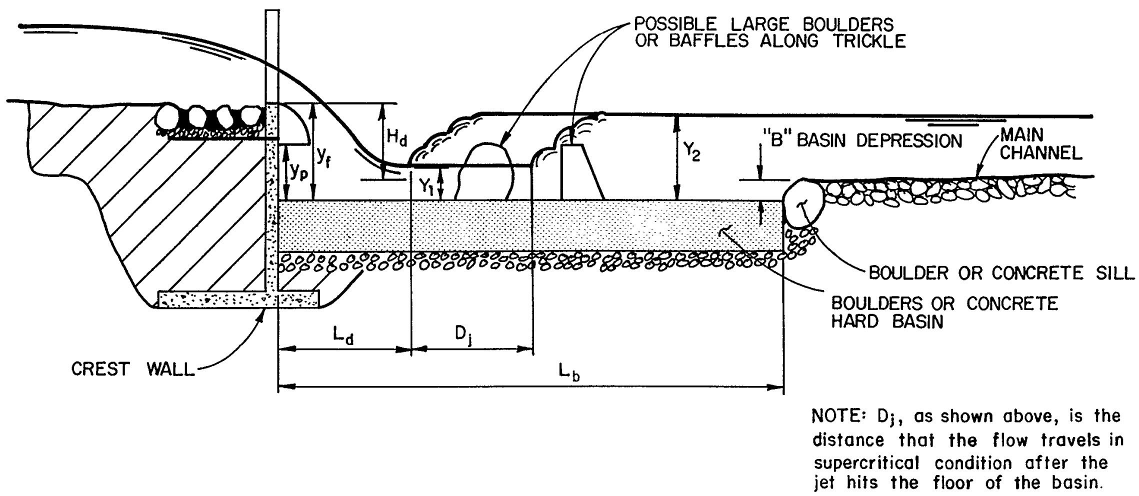
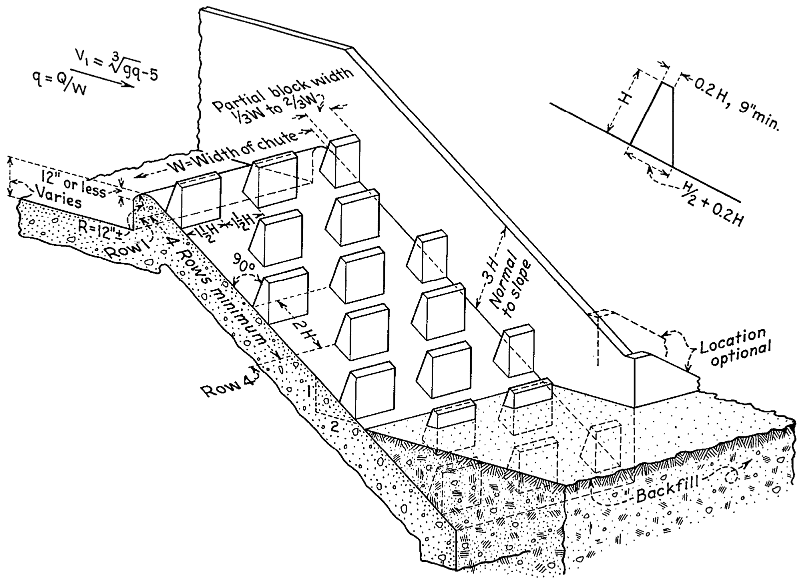
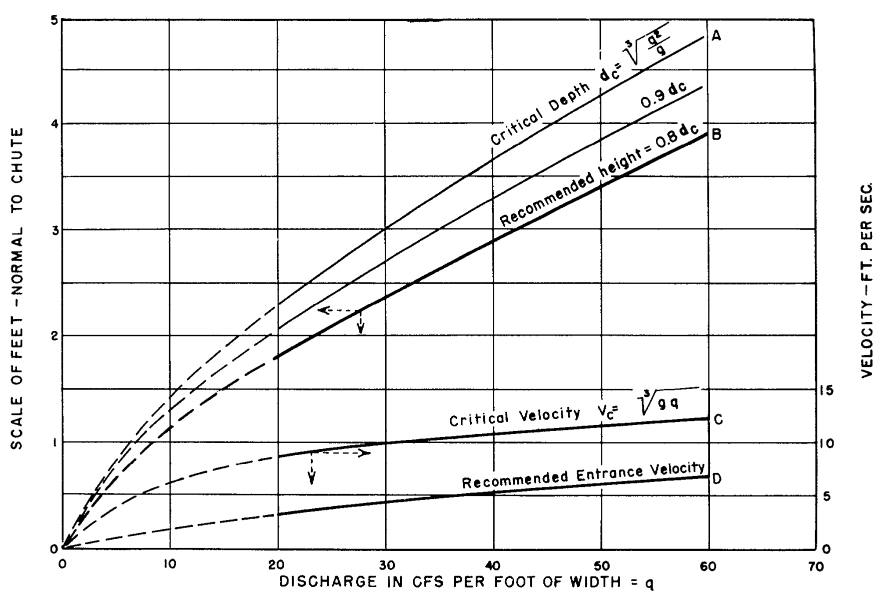


Figure HS-10—Vertical Drop Hydraulic System



(A) USBR ISOMETRIC



(B) DESIGN CRITERIA

Figure HS-11—Baffle Chute Drop Standard USBR Entrance

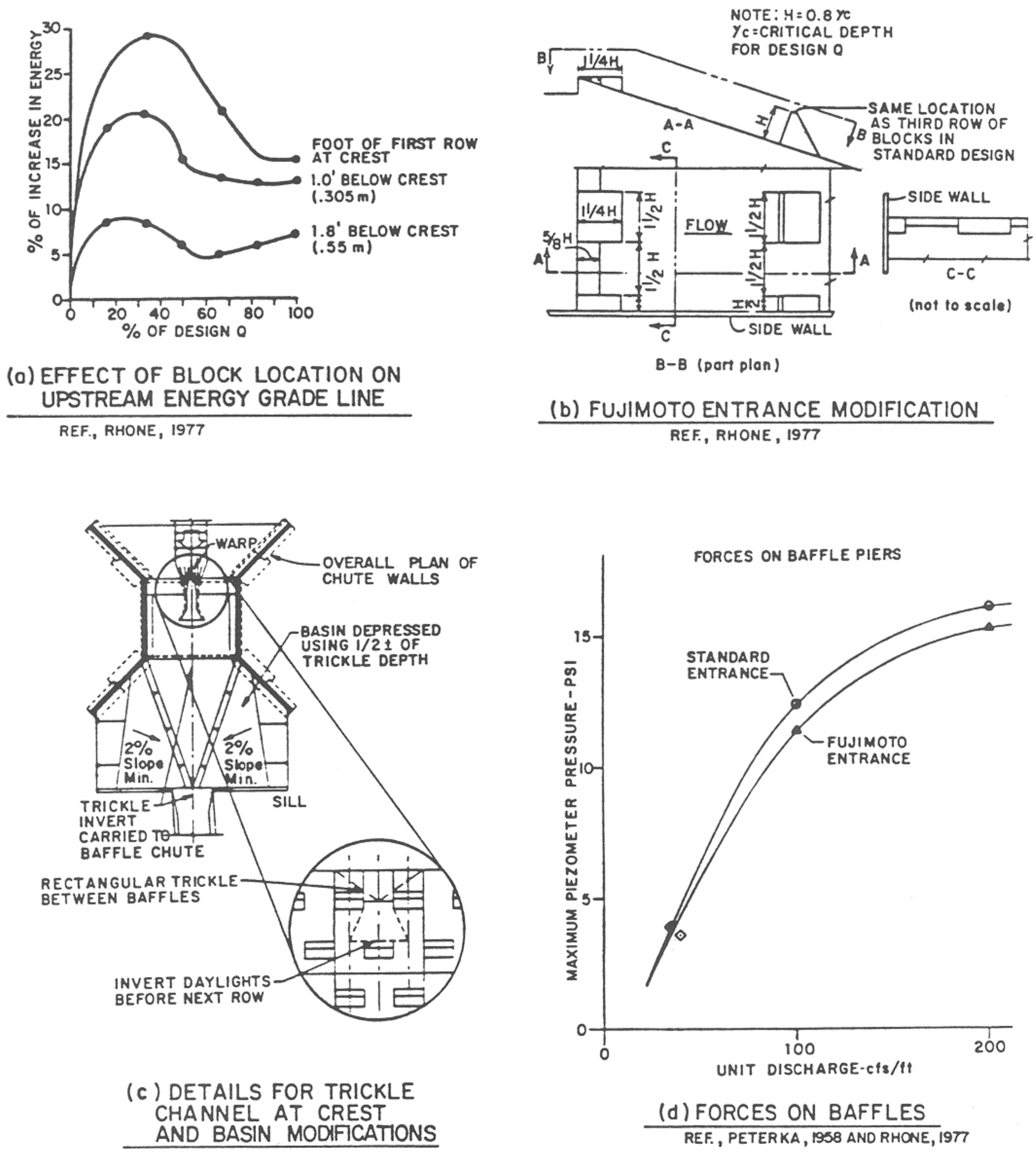
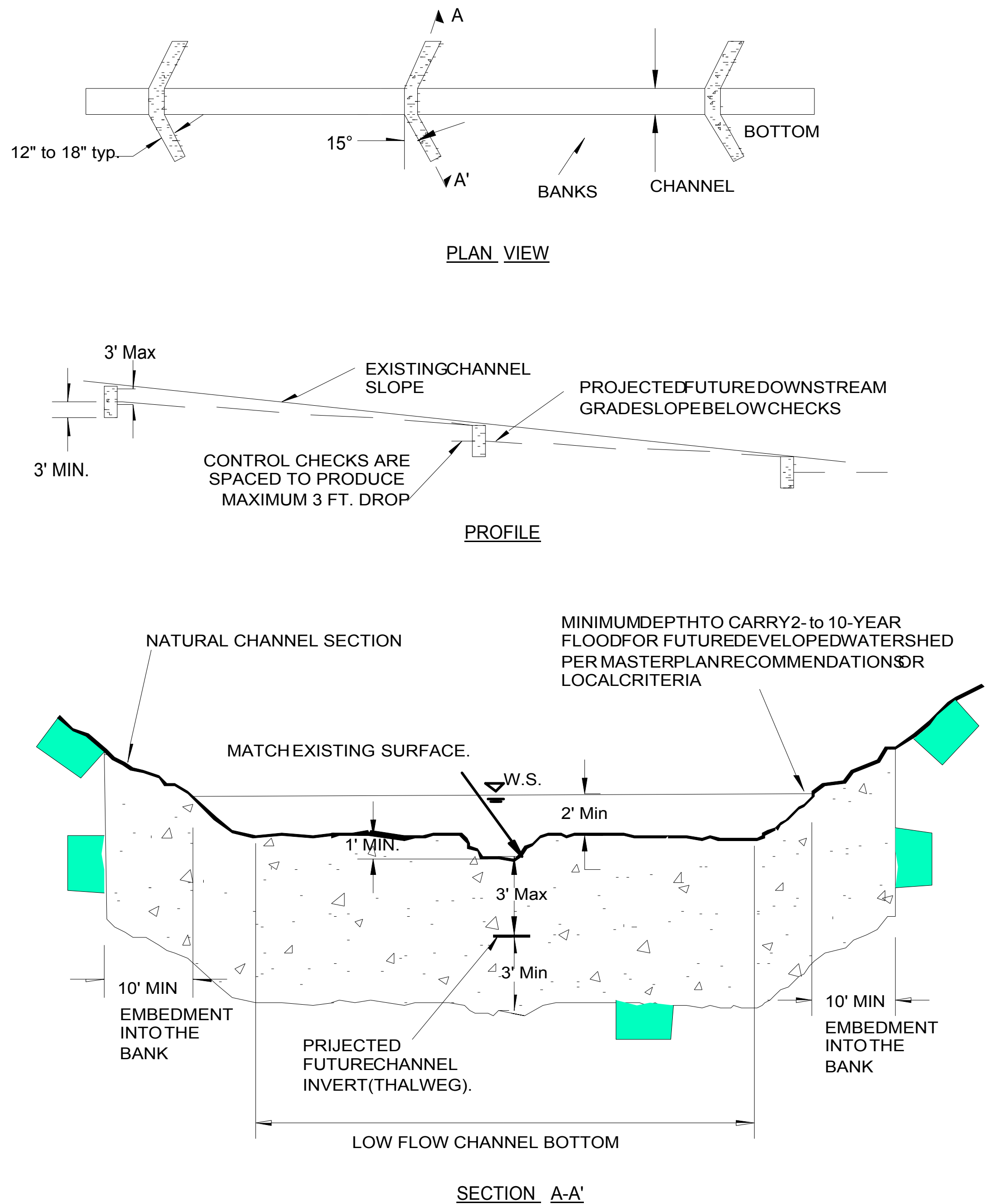


Figure HS-12—Baffle Chute Crest Modifications and Forces



NOTES:
1. TRENCH IN UNDISTURBED SOIL. DO NOT EXCAVATE TO FORM WALLS.
2. POUR AND VIBRATE CONCRETE INTO TRENCH.
3. IF SOIL CANNOT BE TRENCHED WITHOUT WALLS COLLAPSING, USE SHEET PILE CHECK SHOWN IN FIGURE HS-13b.

Figure HS-13a—Control Check for Stable Floodplain – Concrete Wall

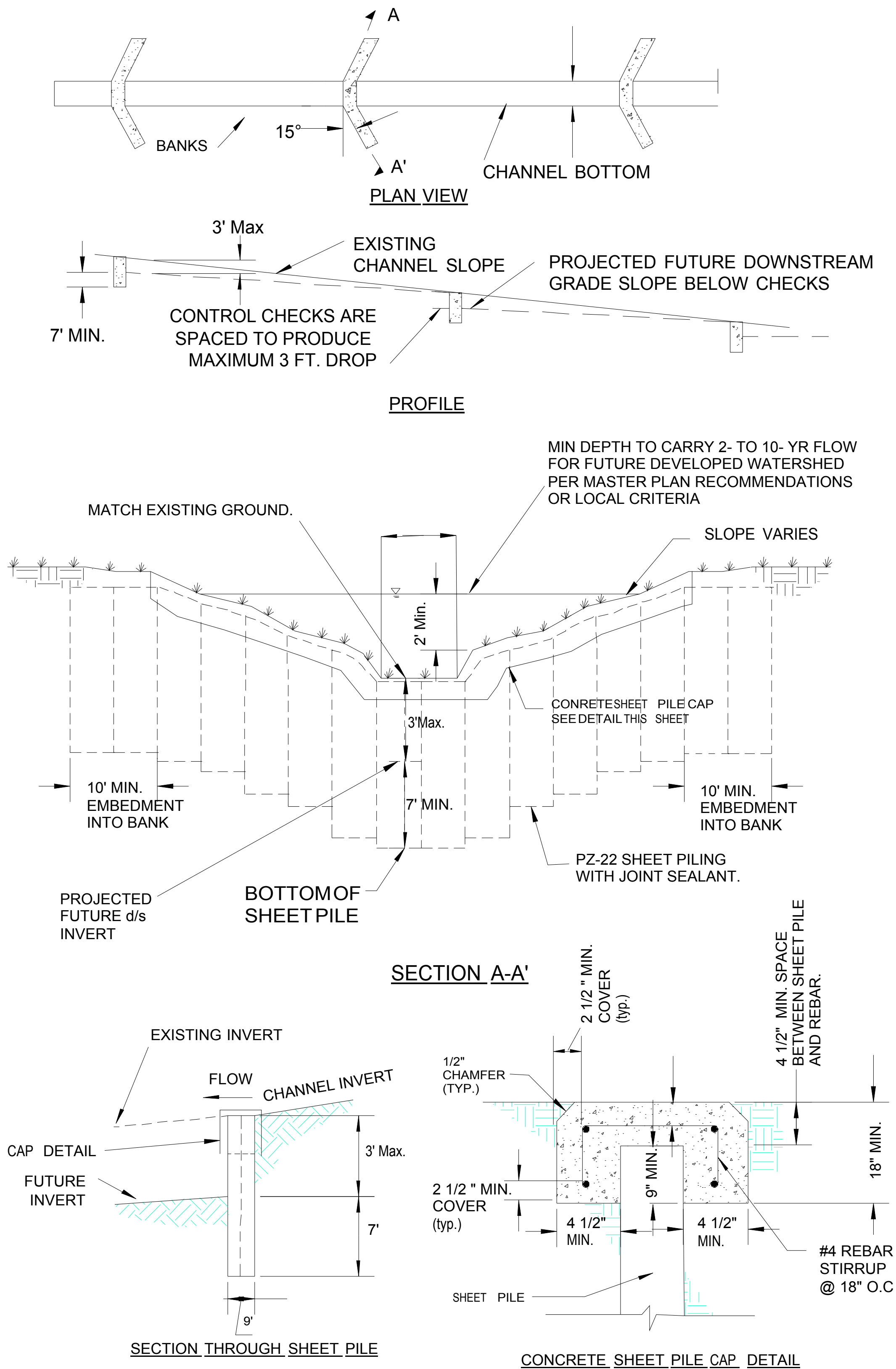


Figure HS-13b—Control Check for Stable Floodplain – Sheet Piling Type

3.0 CONDUIT OUTLET STRUCTURES

3.1 General

Energy dissipation or stilling basin structures are required to minimize scour damages caused by high exit velocities and turbulence at conduit outlets. Similarly, culverts nearly always require special consideration at their outlets. Outlet structures can provide a high degree of energy dissipation and are generally effective even with relatively low tailwater control. Rock protection at conduit outlets (see the MAJOR DRAINAGE chapter) is appropriate where moderate outlet conditions exist; however, there are many situations where rock basins are impractical. Reinforced concrete outlet structures are suitable for a wide variety of site conditions. In some cases, they are more economical than larger rock basins, particularly when long-term costs are considered.

Any outlet structure must be designed to match the receiving stream conditions. The following steps include an analysis of the probable range of tailwater and bed conditions that can be anticipated including degradation, aggradation, and local scour.

Hydraulic concepts and design criteria are provided in this section for an impact stilling basin and adaptation of a baffle chute to conduit outlets. Use of concrete is often more economical due to structure size or local availability of materials. Initial design selection should include consideration of a conduit outlet structure if any of the following situations exist: (1) high-energy dissipation efficiency is required, where hydraulic conditions approach or exceed the limits for alternate designs (see the MAJOR DRAINAGE chapter); (2) low tailwater control is anticipated; or (3) site conditions, such as public use areas, where plunge pools and standing water are unacceptable because of safety and appearance, or at locations where space limitations direct the use of a concrete structure.

Longer conduits with large cross-sectional areas are designed for significant discharges and often with high velocities requiring special hydraulic design at their outlets. Here, dam outlet and spillway terminal structure technology is appropriate (USBR 1987). Type II, III or IV stilling basins, submerged bucket with plunge basin energy dissipators and slotted-grating dissipators can be considered when appropriate to the site conditions. For instance, a plunge basin may have applicability where discharge is to a wet detention pond or a lake. Alternate designs of pipe exit energy dissipators are provided in this *Manual* that can be matched to a variety of pipe sizes and pipe outlet physical and hydraulic settings.

3.2 Impact Stilling Basin

Most design standards for an impact stilling basin are based on the USBR Type VI basin, often called “impact dissipator” or conduit “outlet stilling basin”. This basin is a relatively small structure that is very efficient energy in dissipating energy without the need of tailwater. The original hydraulic design reference by Biechley (1971) is based on model studies. Additional structural design details are provided by Aisenbrey, et al. (1974) and Peterka (1984).

The Type VI basin was originally designed to operate continuously at the design flow rate. However, it is applicable for use under the varied flow conditions of stormwater runoff. The use of this outlet basin is limited only by structural and economic considerations.

Energy dissipation is accomplished through the turbulence created by the loss of momentum as flow entering the basin impacts a large overhanging baffle. At high flow, further dissipation is produced as water builds up behind the baffle to form a highly turbulent backwater zone. Flow is then redirected under the baffle to the open basin and out to the receiving channel. A check at the basin end reduces exit velocities by breaking up the flow across the basin floor and improves the stilling action at low to moderate flow rates.

The generalized, slightly modified, USBR Type IV Impact Basin design configuration is shown in Figure HS-14, which consists of an open concrete box attached directly to the conduit outlet. The width, W, is a function of the Froude number and can be determined using Figure HS-15. The sidewalls are high enough to contain most of the splashing during high flows and slope down to form a transition to the receiving channel. The inlet pipe is vertically aligned with an overhanging L-shaped baffle such that the pipe invert is not lower than the bottom of the baffle. The end check height is equal to the height under the baffle to produce tailwater in the basin. The alternate end transition (at 45 degrees) is recommended for grass-lined channels to reduce the downstream scour potential.

The impact basin can also be adapted to multiple pipe installations. Such modifications are discussed later, but it should be noted that modifications to the design may affect the hydraulic performance of the structure. Model testing of designs that vary significantly from the standard is recommended.

3.2.1 Modified Impact Basins for Smaller Outlets

For smaller pipe outlets a modified version of the USBR Type IV Impact Basin is suggested in this *Manual*. Figure HS-16a provides a design layout for circular outlets ranging in size from 18-inches to 48-inches in diameter and Figure HS-16b for pipes 18-inches in diameter and smaller. The latter was added for primary use as an outlet energy dissipator upstream of forebays of small extended detention basins, sand filters and other structural best management practices requiring energy dissipation at the end of the pipe delivering water to the BMP facility.

Unlike the Type IV impact basin, the modified basins do not require sizing for flow under normal stormwater discharge velocities recommended for storm sewers in this *Manual*. However, their use is limited to exit velocities of 18 feet per second or less. For larger conduits and higher exit velocities, it is recommended that the standard Type IV impact basin be used instead.

3.2.2 Low-flow Modifications

The standard design will retain a standing pool of water in the basin bottom that is generally undesirable from an environmental and maintenance standpoint. As a result, the standard USBR design has been modified herein for urban applications to allow drainage of the basin bottom during dry periods. This situation should be alleviated where practical by matching the receiving channel low-flow invert to the basin invert. A low-flow gap is extended through the basin end check wall. The gap in the check should be as narrow as possible to minimize effects on the check hydraulics. This implies that a narrow and deeper (1½- to 2-foot) low-flow channel will work better than a shallow and wide gap section.

For the modified impact basin illustrated in Figure HS-16a, the downstream geometry recognizes the need for a trickle channel and also provides for a modification when this structure is used upstream of a forebay in an Extended Detention Basin or other BMP requiring energy dissipation at the entrance.

Low-flow modifications have not been fully tested to date. Caution is advised to avoid compromising the overall hydraulic performance of the structure. Other ideas are possible including locating the low-flow gap at one side (off center) to prevent a high velocity jet from flowing from the pipe straight down the low-flow channel. The optimal configuration results in continuous drainage of the basin area and helps to reduce the amount of siltation.

3.2.3 Multiple Conduit Installations

Where two or more conduits of different sizes outlet in proximity, a composite structure can be constructed to eliminate common walls. This can be somewhat awkward since each basin “cell” must be designed as an individual basin with different height, width, etc. Where possible, a more economical approach is to combine storm sewers underground, at a manhole or vault, and bring a single, combined pipe to the outlet structure.

When using a Type IV impact basin shown in Figure HS-14 for two side-by-side pipes of the same size, the two pipes may discharge into a single basin. If the basin’s design width for each pipe is W, the combined basin width for two pipes would be $1.5W$. When the flow is different for the two conduits, the design width W is based on the pipe carrying the higher flow. For the modified impact basin shown in Figure HS-16, add 1/2 D space between the pipes and to each outside pipe edge when two pipes discharge into the basin to determine the width of the headwall and extent the width of the impact wall to match the outside edges of the two pipes. The effect of mixing and turbulence of the combined flows in the basin has not been model tested to date.

Remaining structure dimensions are based on the design width of a separate basin W. If the two pipes have different flow, the combined structure is based on the higher Froude number when designing the Type IV basins. Use of a handrail is suggested around the open basin areas where safety is a concern. Access control screens or grating where necessary are a separate design consideration. A hinged rack has been used on a few projects in the District.

3.2.4 General Design Procedure for Type IV Impact Basin

1. Determine the design hydraulic cross-sectional area just inside the pipe, at the outlet. Determine

the effective flow velocity, V, at the same location in the pipe. Assume depth $D = \left(A_{sect}\right)^{1/2}$ and compute the Froude number = $\dfrac{V}{(gD)^{1/2}}$

2. The entrance pipe should be turned horizontally at least one pipe diameter equivalent length upstream from the outlet. For pipe slopes greater than 15 degrees, the horizontal length should be a minimum of two pipe diameters.

3. Determine the basin width, W, by entering the Froude number and effective flow depth into Figure HS-15. The remaining dimensions are proportional to the basin width according to Figure HS-14. The basin width should not be oversized since the basin is inherently oversized for less than design flows. Larger basins become less effective as the inflow can pass under the baffle.

4. Structure wall thickness, steel reinforcement, and anchor walls (underneath the floor) should be designed using accepted structural engineering methods. Note that the baffle thickness, t_b, is a suggested minimum. It is not a hydraulic parameter and is not a substitute for structural analysis. Hydraulic forces on the overhanging baffle may be approximated by determination of the hydraulic jet force at the outlet:

   F_j = 1.94 $V_{out} Q_{des}$ (force in pounds) (HS-15)

   Q_{des} = maximum design discharge (cfs)

   V_{out} = velocity of the outlet jet (ft/sec)

5. Type “M” rock riprap should be provided in the receiving channel from the end check to a minimum distance equal to the basin width. The depth of rock should be equal to the check height or at least 2.0 feet. Rock may be buried to finished grades and planted as desired.

6. The alternate end check and wingwall shown in Figure HS-14 are recommended for all grass-lined channel applications to reduce the scour potential below the check wall.

7. Ideally, the low-flow invert matches the floor invert at the basin end and the main channel elevation is equal to the top of the check. For large basins where the check height, d, becomes greater than the low-flow depth, dimension d in Figure HS-14 may be reduced by no more than one-third. It should not be reduced to less than 2 feet. This implies that a deeper low-flow channel (1.5 to 2.0 feet) will be advantageous for these installations. The alternate when d exceeds the trickle flow depth is that the basin area will not drain completely.

8. A check section should be constructed directly in front of the low-flow notch to break up bottom flow velocities. The length of this check section should overlap the width of the low flow and its dimension is shown in Figure HS-14.

3.3 Pipe Outlet Rundowns

3.3.1 Baffle Chute Rundown

The baffle chute developed by the USBR (1958) has also been adapted to use at pipe outlets. This structure is well suited to situations with large conduit outfalls and at outfalls to channels in which some future degradation is anticipated. As mentioned previously, the apron can be extended at a later time to account for channel degradation. This type of structure is only cost effective if a grade drop is necessary below the outfall elevation.

Figure HS-17 illustrates a general configuration for a baffled outlet application for a double box culvert outlet. In this case, an expansion zone occurs just upstream of the approach depression. The depression depth is designed as required to reduce the flow velocity at the chute entrance. The remaining hydraulic design is the same as for a standard baffle chute using conditions at the crest to establish the design. The same crest modifications are applicable to allow drainage of the approach depression, to reduce the upstream backwater effects of the baffles, and to reduce the problems of debris accumulation and standing water at the upstream row of baffles.

Flow entering the chute should be well distributed laterally across the width of the chute. The velocity should be below critical velocity at the crest of the chute. To insure low velocities at the upstream end, it may be necessary to provide a short energy dissipating pool. The sequent or conjugate depth in the approach basin should be sized to prevent jump sweep-out, but the basin length may be considerably less than a conventional hydraulic jump basin since its primary purpose is only to reduce the average entrance velocity. A basin length of twice the sequent depth will usually provide ample basin length. The end check of the pool may be used as the crest of the chute as shown in Figure HS-17.

3.3.2 Grouted Boulder Chute Rundown

Another option for rundowns at outlets of larger pipes is to use a grouted boulder rundown illustrated in Figure 18. This type of rundown has been used successfully for several large storm sewers entering the South Platte River. It is critical that the details shown in Figure 18 be strictly followed and the grout and the actual filling of spaces between the boulders with grout closely adhere to the recommendations for grouted boulders provided in the Major Drainage Chapter of this *Manual*.

If the exit velocities of the pipe exceeds 12 feet per second, an approach chute for the baffle chute rundown described above should be considered and provided. If this approach chute is lined with grouted boulders in a manner called for in the Major Drainage Chapter, the stilling basin sill can be eliminated.

3.4 Low Tailwater Riprap Basins at Pipe Outlets

3.4.1 General

The design of low tailwater riprap basins for storm sewer pipe outlets and at some culvert outlets is necessary when the receiving or downstream channel may have little or no flow or tailwater at time when the pipe or culvert is in operation. Design criteria are provided in Figures HS-19a through HS-20c.

3.4.2 Objective

By providing a low tailwater basin at the end of a storm sewer conduit or culvert, the kinetic energy of the discharge is dissipated under controlled conditions without causing scour at the channel bottom. Photograph HS-12 shows a fairly large low tailwater basin.

3.4.3 Low Tailwater Basin Design

Low tailwater is defined as being equal to or less than ⅓ of the height of the storm sewer, that is:

$$y_t \le \frac{D}{3} \quad \text{or} \quad y_t \le \frac{H}{3}$$

in which:

y_t = tailwater depth at design

D = diameter of circular pipe (ft)

H = height of rectangular pipe (ft)

3.4.3.1 Finding Flow Depth and Velocity of Storm Sewer Outlet Pipe

The first step in the design of a scour protection basin at the outlet of a storm sewer is to find the depth and velocity of flow at the outlet. Pipe-full flow can be found using Manning's equation.

$$Q_{full} = \frac{1.49}{n} A_{full} \left(R_{full}\right)^{2/3} S_o^{1/2} \qquad \text{(HS-16a)}$$

Then and the pipe-full velocity can be found using the continuity equation.

$$V_{full} = Q_{full} / A_{full} \qquad \text{(HS-16a)}$$

The normal depth of flow, d, and the velocity in a conduit can be found with the aid of Figure HS-20a and Figure HS-20b. Using the known design discharge, Q, and the calculated pipe-full discharge, Q_{full}, enter Figure HS-20a with the value of Q/Q_{full} and find d/D for a circular pipe of d/H for a rectangular pipe.

Compare the value of d/D (or d/H) with the one obtained from Figure HS-20b using the Froude parameter.

$$Q/D^{2.5} \quad \text{or} \quad Q/\left(wH^{1/5}\right) \qquad \text{(HS-16a)}$$

Choose the smaller of the two (d/D or d/H) ratios to calculate the flow depth at the end of the pipe.

$$d = D(d/D) \quad \text{or} \quad d = H(d/H) \qquad \text{(HS-16b)}$$

Again, enter Figure HS-19a using the smaller d/D (or d/H) ratio to find the A/A_{full} ratio. Then,

$$A = \left(A/A_{full}\right)A_{full} \qquad \text{(HS-16c)}$$

Finally,

$$V = Q/A \qquad \text{(HS-16d)}$$

In which for Equations 16a through 16d above:

A_{full} = cross-sectional area of the pipe (ft^2)

A = area of the design flow in the end of the pipe (ft^2)

n = Manning's n for the pipe full depth

Q_{full} = pipe full discharge at its slope (cfs)

R = hydraulic radius of the pipe flowing full, ft [R_{full} = D/4 for circular pipes, R_{full} = A_{full}/(2H + 2w) for rectangular pipes, where D = diameter of a circular conduit, H = height of a rectangular conduit, and w = width of a rectangular conduit (ft)]

S_o = longitudinal slope of the pipe (ft/ft)

V = design flow velocity at the pipe outlet (ft/sec)

V_{full} = flow velocity of the pipe flowing full (ft/sec)

3.4.3.2 Riprap Size

For the design velocity, use Figure HS-20c to find the size and type of the riprap to use in the scour protection basin downstream of the pipe outlet (i.e., B18, H, M or L). First, calculate the riprap sizing design parameter, P_d , namely,

$$P_d = \left(V^2 + gd\right)^{1/2} \qquad \text{(HS-16e)}$$

in which:

V = design flow velocity at pipe outlet (ft/sec)

g = acceleration due to gravity = 32.2 ft/sec^2

d = design depth of flow at pipe outlet (ft)

Photograph HS-12—Upstream and downstream views of a low tailwater basin in Douglas County protecting downstream wetland area. Burying and revegetation of the rock would blend the structure better with the adjacent terrain.

When the riprap sizing design parameter indicates conditions that place the design above the Type H riprap line in Figure HS-20, use B18, or larger, grouted boulders. An alternative to a grouted boulder or loose riprap basin is to use the standard USBR Impact Basin VI or one of its modified versions, described earlier in this Chapter of the *Manual*.

After the riprap size has been selected, the minimum thickness of the riprap layer, T, in feet, in the basin is set at:

$$T = 1.75 D_{50} \tag{HS-17}$$

in which:

D_{50} = the median size of the riprap (see Table HS-9.)

Table HS-9—Median (i.e., D_{50}) Size of District's Riprap/Boulder

Riprap Type	D_{50}—Median Rock Size (inches)
L	9
M	12
H	18
B18	18 (minimum dimension of grouted boulders)

3.4.3.3 Basin Length

The minimum length of the basin, L, in Figure HS-19, is defined as being the greater of the following:

for circular pipe: $L = 4D$ or $L = (D)^{1/2}\left(\frac{V}{2}\right)$ (HS-18) for rectangular pipe: $L = 4H$ or $L = (H)^{1/2}\left(\frac{V}{2}\right)$ (HS-19)

in which:

L = basin length

H = height of rectangular conduit

V = design flow velocity at outlet

D = diameter of circular conduit

3.4.3.4 Basin Width

The minimum width, W, of the basin downstream of the pipe's flared end section is set as follows:

for circular pipes: $W = 4D$ (HS-20)

for rectangular pipe: $W = w + 4H$ (HS-21)

in which,

W = basin width (Figure HS-19)

D = diameter of circular conduit

w = width of rectangular conduit

3.4.3.5 Other Design Requirements

All slopes in the pre-shaped riprapped basin are 2H to 1V.

Provide pipe joint fasteners and a structural concrete cutoff wall at the end of the flared end section for a circular pipe or a headwall with wingwalls and a paved bottom between the walls, both with a cutoff wall that extends down to a depth of:

$$B = \frac{D}{2} + T \quad \text{or} \quad B = \frac{H}{2} + T \qquad \text{(HS-22)}$$

in which,

B = cutoff wall depth

D = diameter of circular conduit

T = Equation HS-17

The riprap must be extended up the outlet embankment's slope to the mid-pipe level.

3.5 Culvert Outlets

Photograph HS-13—Culvert outlets when left unprotected cause downstream erosion. The designer's job is not complete until provisions are made to protect the outlet. Use of vegetated soil-riprap would blend this structure better into the natural landscape.

Culvert outlets represent a persistent problem because of concentrated discharges and turbulence that are not fully controlled prior to the flow reaching the standard downstream channel configuration described in the Major Drainage Chapter of this *Manual*. Too often the designer's efforts are focused on the culvert inlet and its sizing with outlet hydraulics receiving only passing attention. Culvert design is not complete until adequate attention is paid to the outlet hydraulics and proper stilling of the discharge flows.

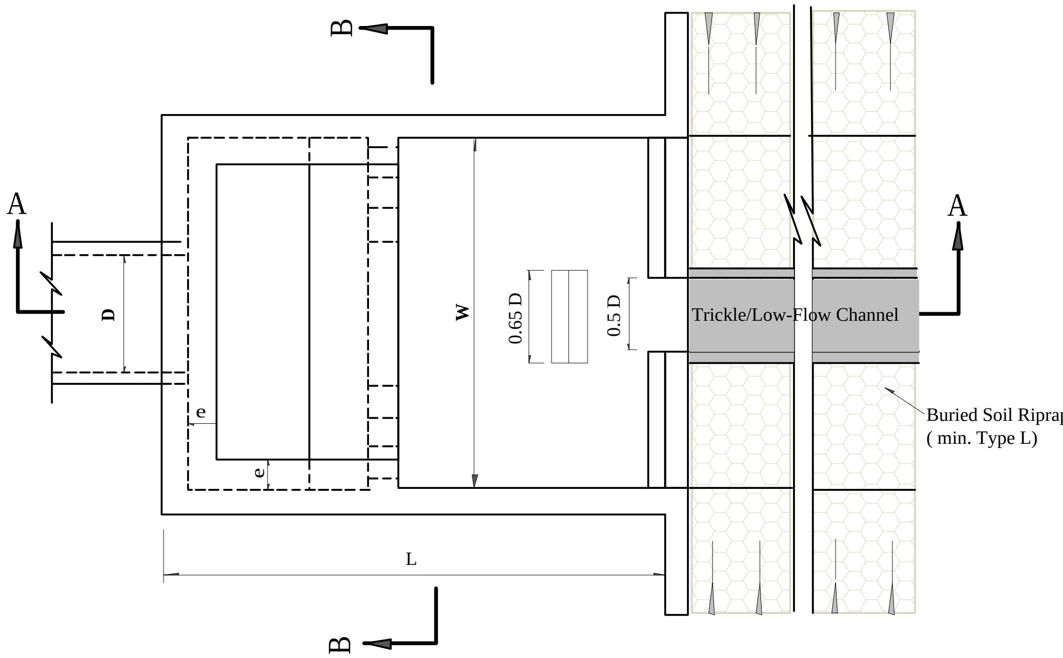
Culvert outlet energy dissipator and flow spreading may require special structures downstream of the culvert outlet to limit local scour, general stream degradation, and troublesome head cutting. Some of the techniques described in Sections 3.2, 3.3 and 3.4 may be applied at culver outlets as well if the downstream channel and/or tailwater conditions so indicate.

Local scour is typified by a scour hole at the pipe's outlet. High exit velocities cause this, and the effects extend only a limited distance downstream. Coarse material scoured from the hole is deposited immediately downstream, often forming a low bar. Finer material is transported further downstream. The dimensions of the scour hole change due to sedimentation during low flows and the varying erosive effects of storm events. The scour hole is generally deepest during passage of the flow when there is minimal tailwater depth at the outlet and not necessarily when the flow is highest. Methods for predicting scour hole dimensions are found in HEC No. 14 (Corry, et al. 1975) and need to be applied using a range of possible tailwater depth conditions during different design storms or floows.

General storm degradation, or head cutting, is a phenomenon independent of culvert performance. Natural causes produce a lowering of the streambed over time. The identification of a degrading stream is an essential part of the original site investigation. However, high-energy discharges from a culvert can often cause stream degradation for a limited in distance downstream. Both scour and steam degradation can occur simultaneously at a culvert outlet.

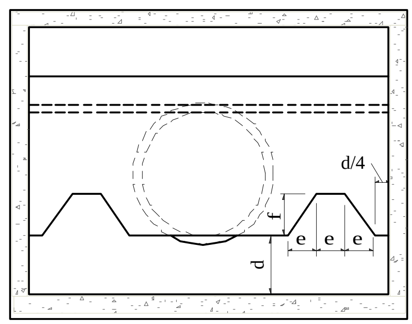
Various measures described in HEC No. *14* and in this *Manual* listed below need to be considered to protect the downstream channel or stream and control culvert outlet flow. It is beyond the scope of this *Manual* to provide detailed information about all available controls in HEC No. 14, but the District encourages the proper application and design as appropriate for the specific site.

1. Colorado State University rigid boundary basin
2. Tumbling flow rectangular section
3. Increased resistance—box culverts
4. Roughness elements—circular culverts
5. USBR Type II
6. USBR Type III
7. USBR Type IV
8. Contra Costa
9. Hook-type energy dissipator
10. Straight drop structure
11. Riprap basins
12. Channel check and drop structures and other energy dissipating and control structures described earlier in this Chapter
13. Use of properly anchored flared end sections – see Figure HS-19a

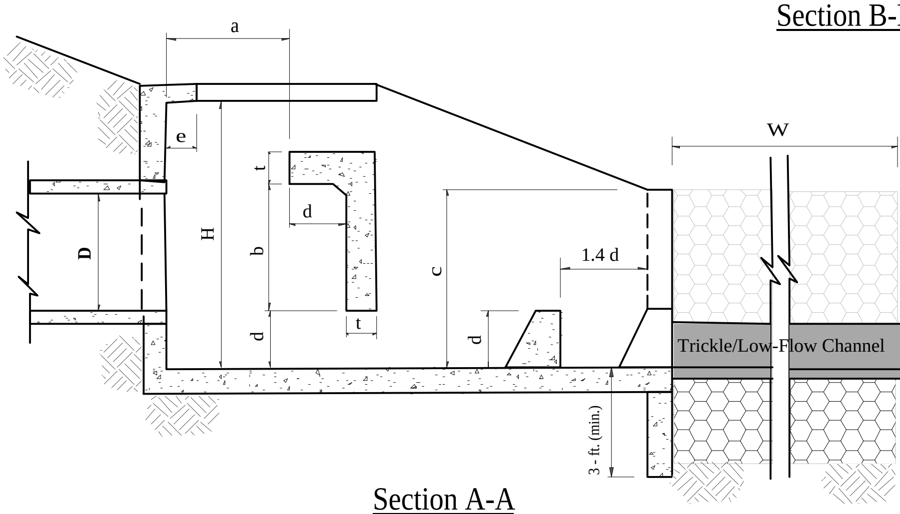


$$W = 2.94D\left[V/gD\right]^{0.556}$$
$$V = Velocity,\ ft/s$$
$$D = \sqrt{A}$$
$$A = Area\ of\ \ flow, sq.\ ft.$$

H = 3/4 W	*c = 1/2 W*
L = 4/3 W	*d = 1/6 W*
a = 1/2 W	*e = 1/12 W*
b = 3/8 W	*f = 1/8 W*
t = 1/12 W (suggested min.)	



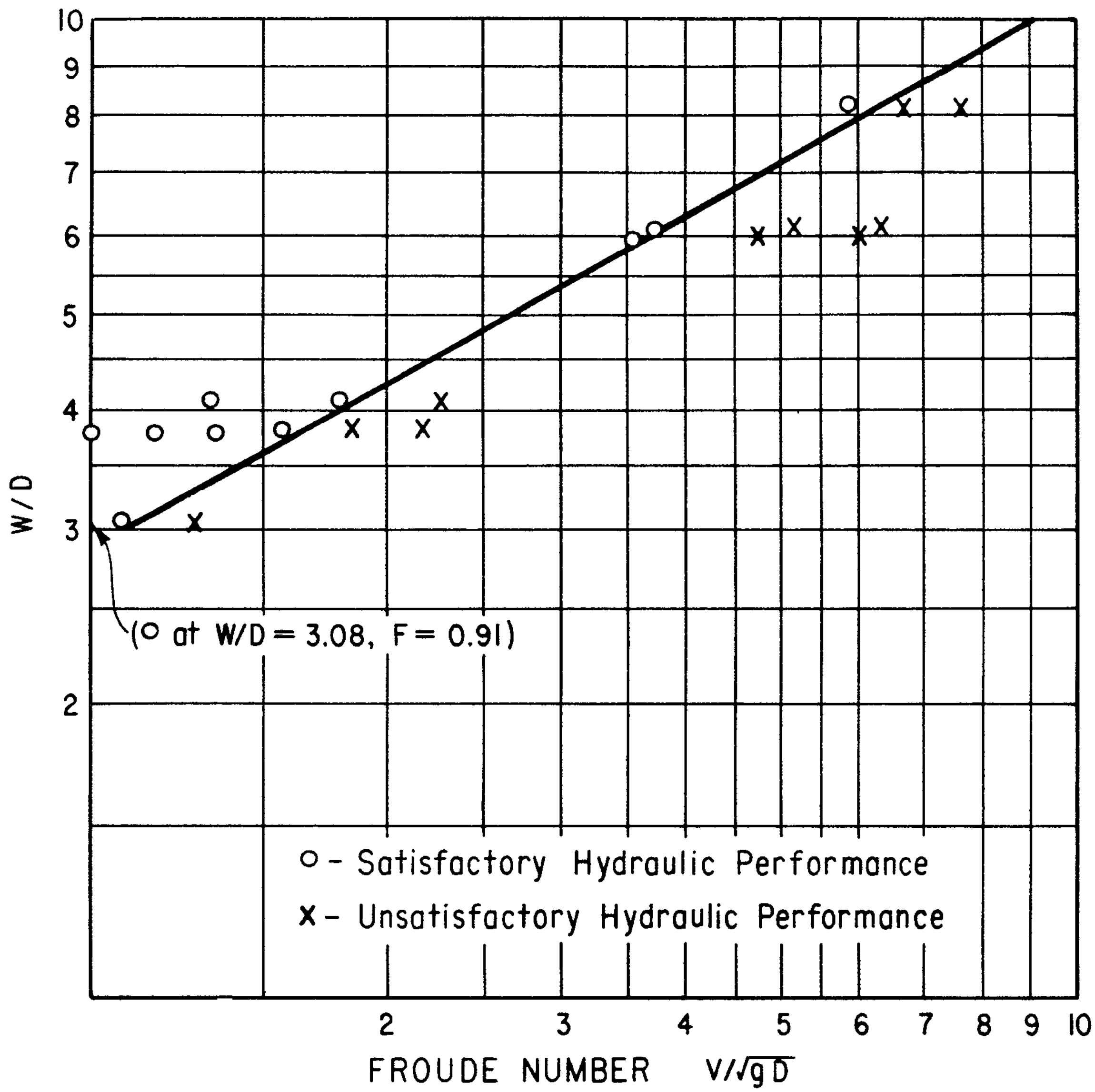
Section B-B



Section A-A

USGS Impact Stilling Basin Modified by UDFCD February 2004

Figure HS-14—General Design Dimensions for a USBR Type VI Impact Stilling Basin



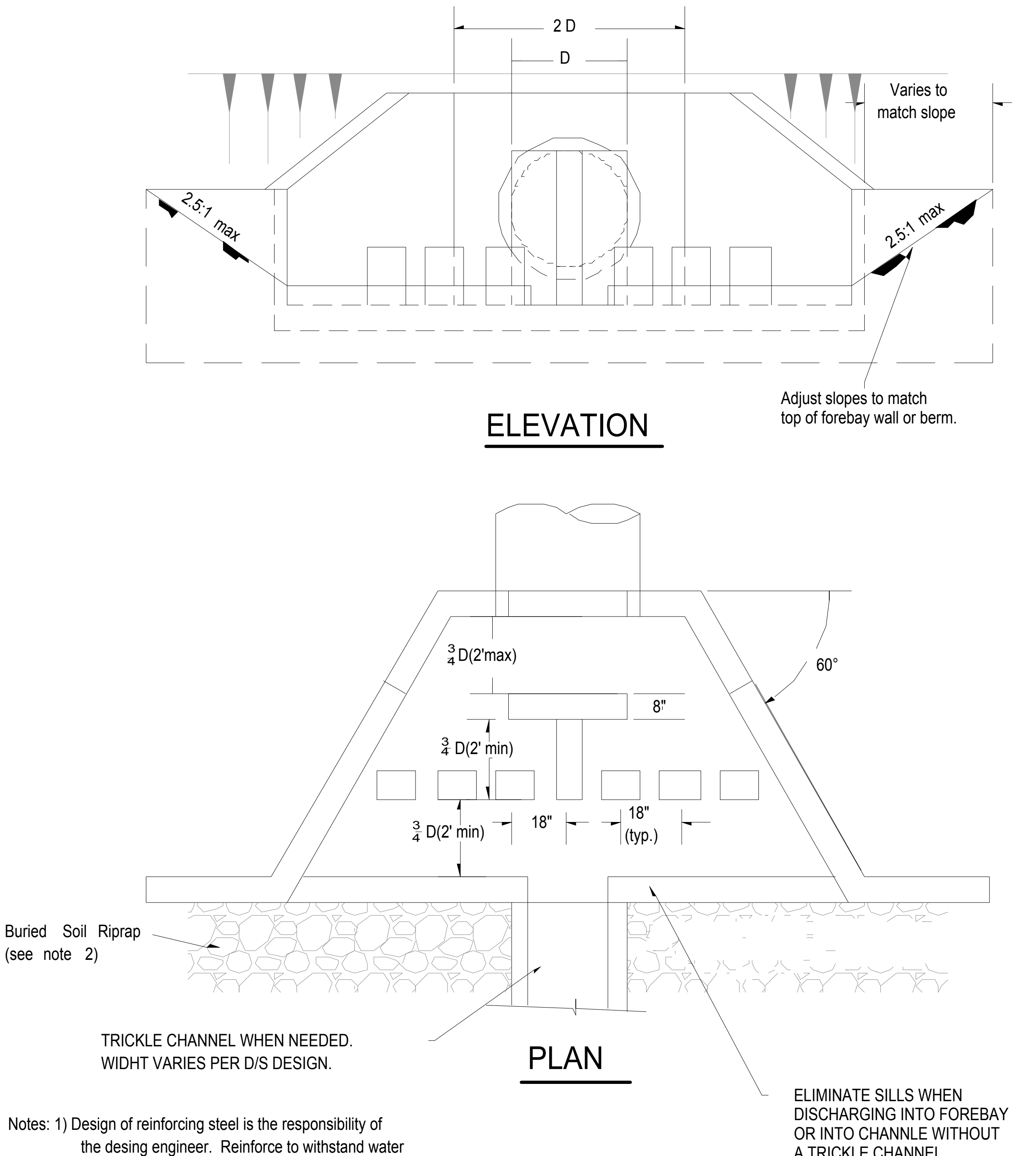
"W" is the inside width of the basin.

"D" represents the depth of flow entering the basin and is the square root of the flow area at the conduit outlet.

"V" is the velocity of the incoming flow.

The tailwater depth is uncontrolled.

Figure HS-15—Basin Width Diagram for the USBR Type VI Impact Stilling Basin)

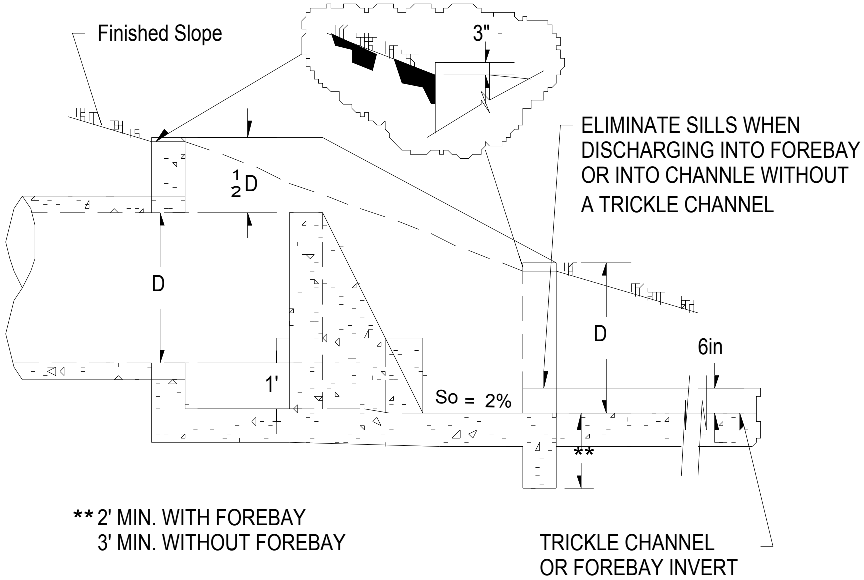


Notes: 1) Design of reinforcing steel is the responsibility of the desing engineer. Reinforce to withstand water earh pressueres.

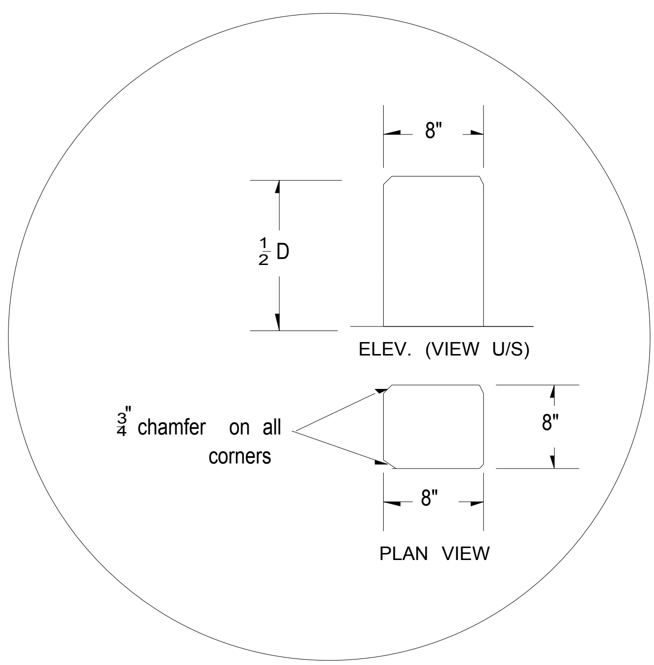
2) When dischargning into channel and not forebay install Type M buried soil riprap for a distance of 3*D downstream of structure.

Figure HS-16a Modified Impact Stilling Basin for Conduits 18” to 48” in Diameter

(Sheet 1 of 2)



PROFILE



BAFFLE BLOCK GEOMETRY

Figure HS-16a. Modified Impact Stilling Basin for Conduits 18” to 48” in Diameter (Sheet 2 of 2)

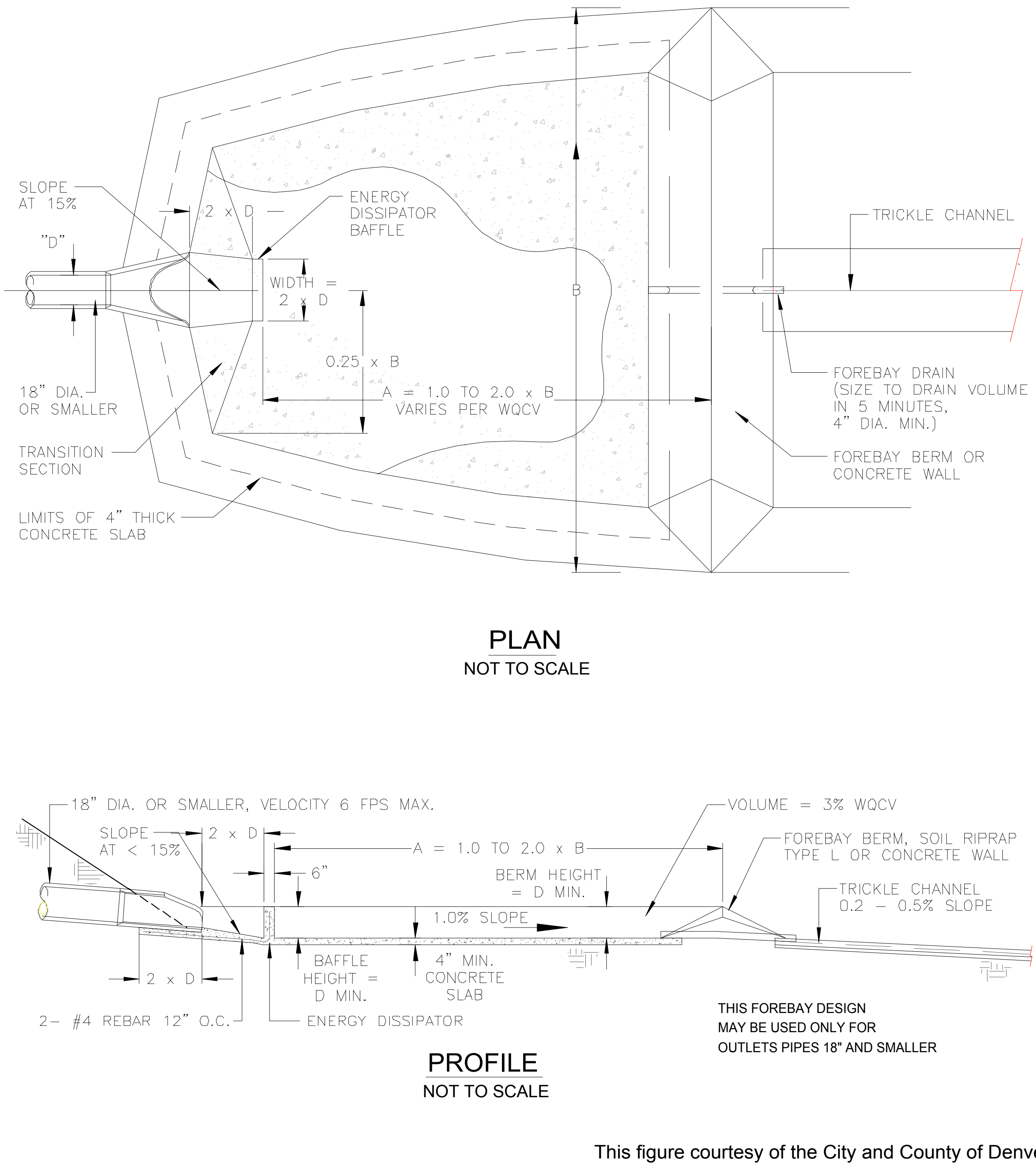


Figure HS-16b. Impact Stilling Basin for Pipes Smaller than 18" in Diameter Upstream of Forebays.

(Courtesy: Technical and Design Criteria, City and County of Denver, 2006)

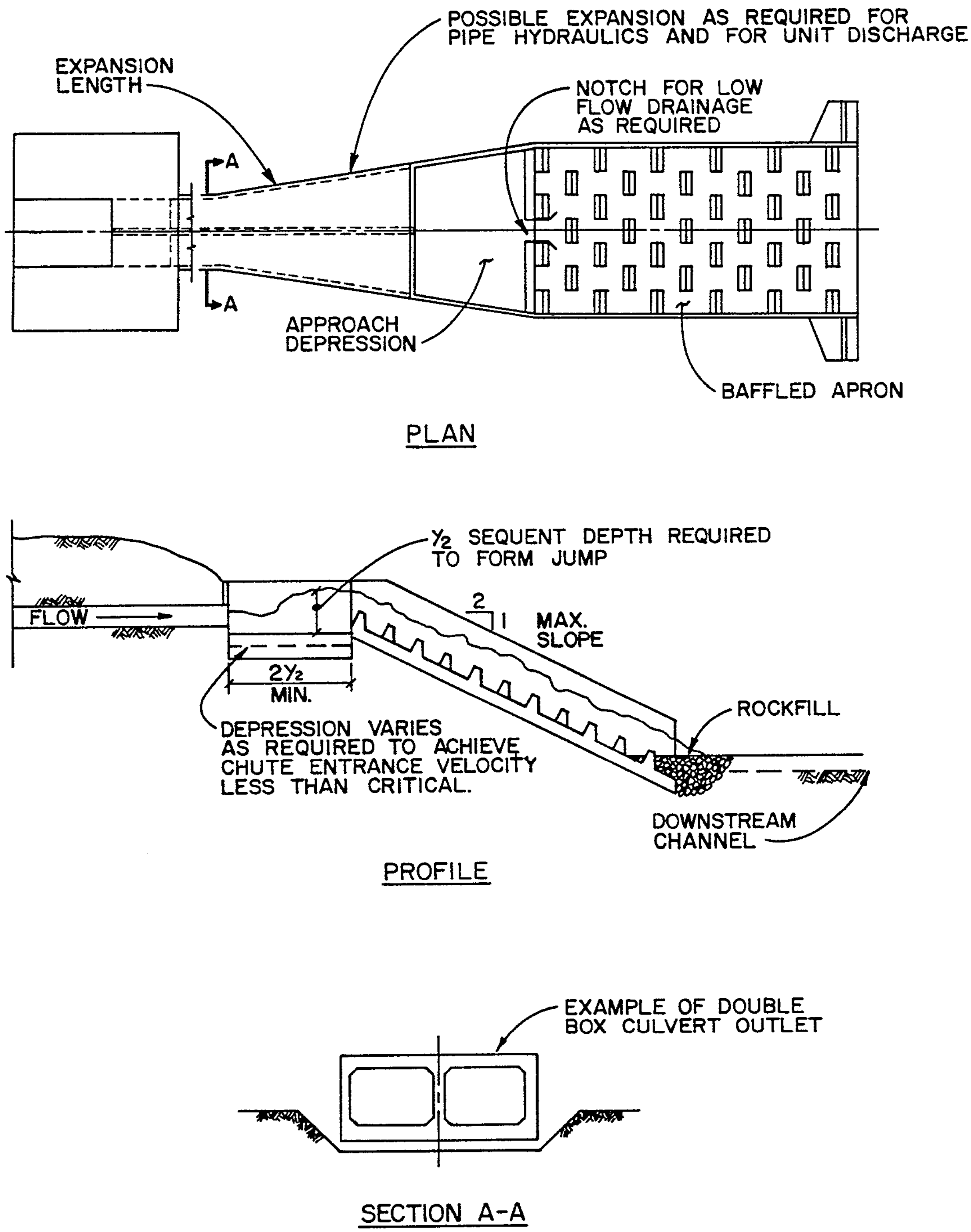
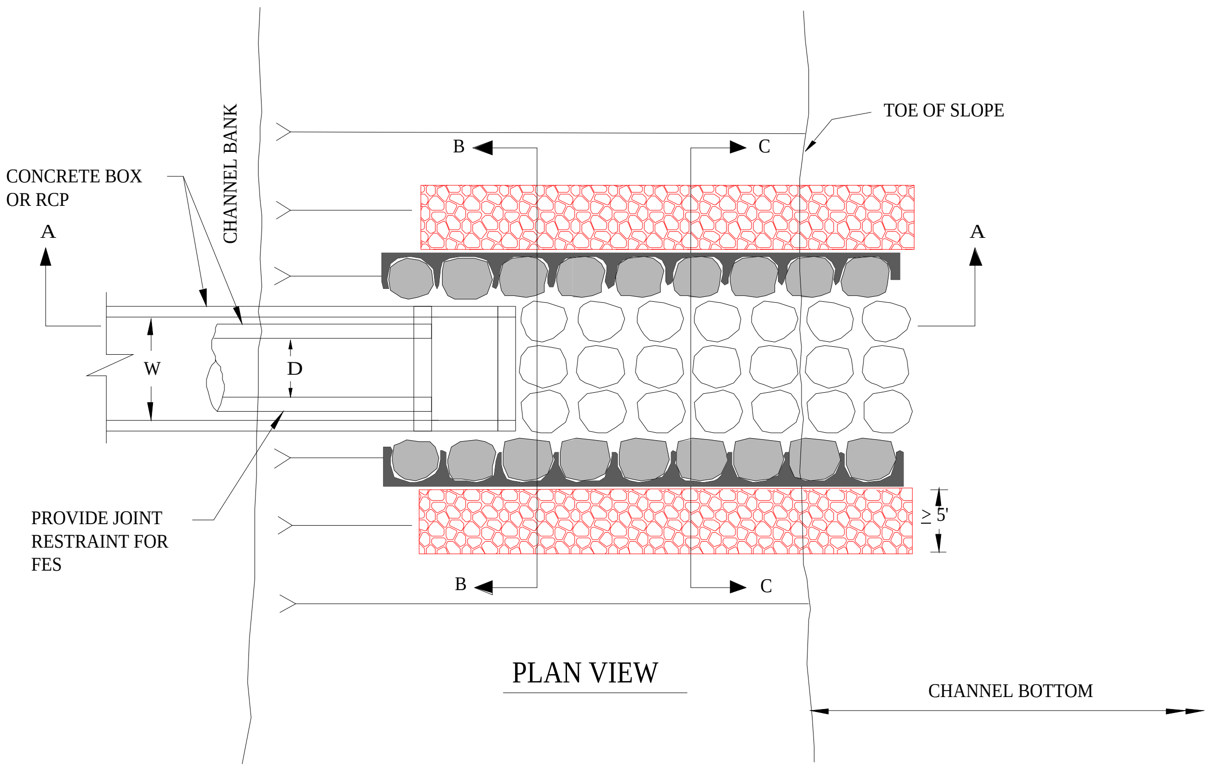
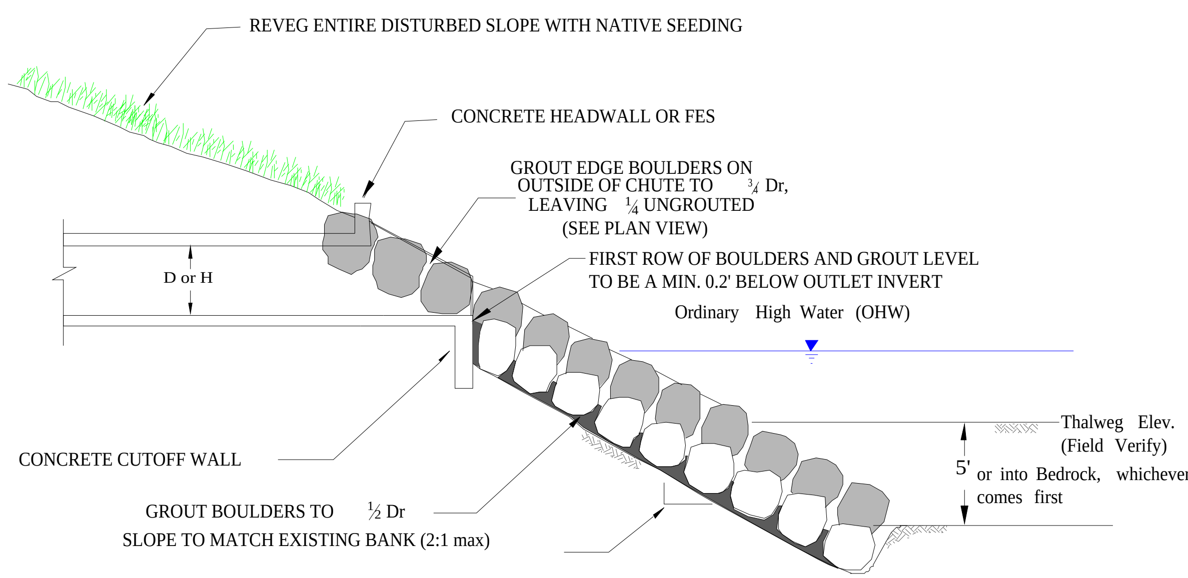


Figure HS-17—Baffle Chute Pipe Outlet



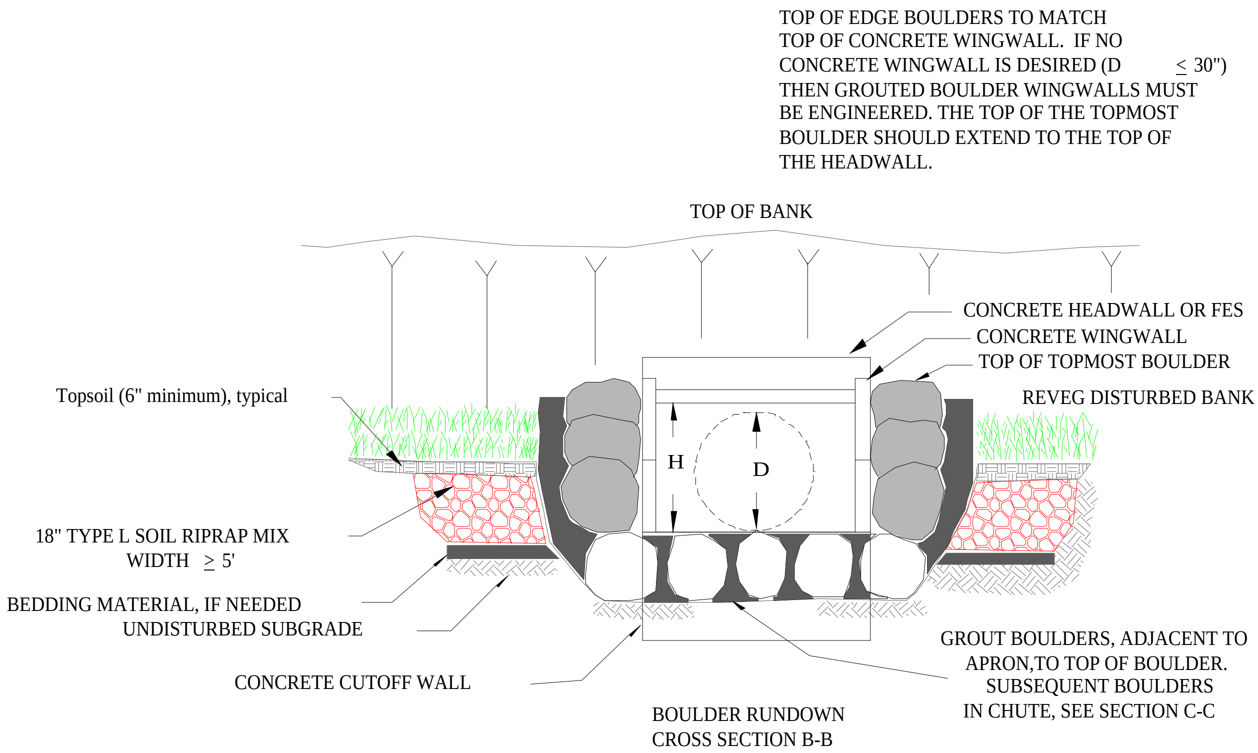


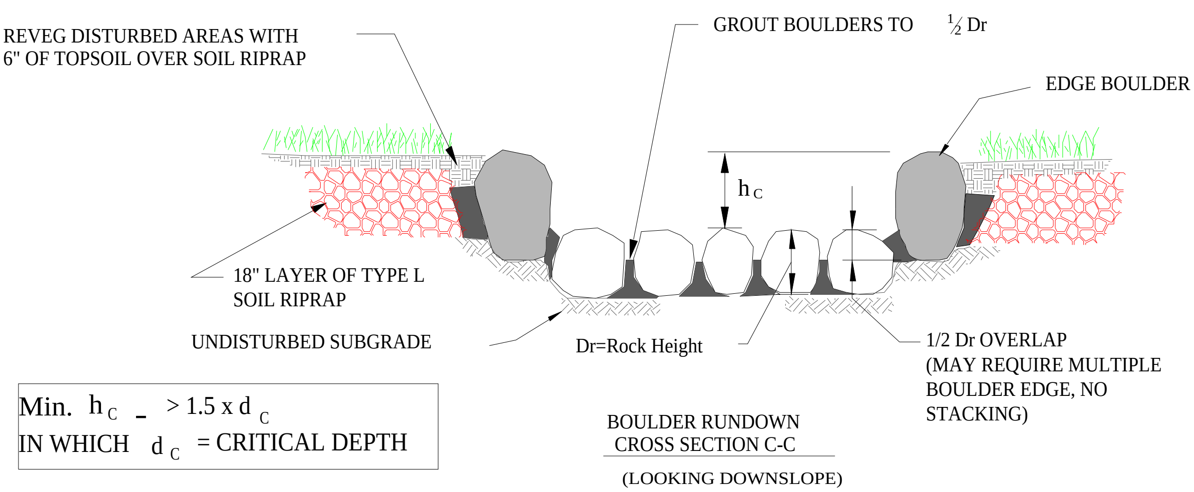
CROSS SECTION A-A

Sheet 1 of 2

Figure HS-18—Grouted Boulder Rundown

(Sheet 1 of 2)

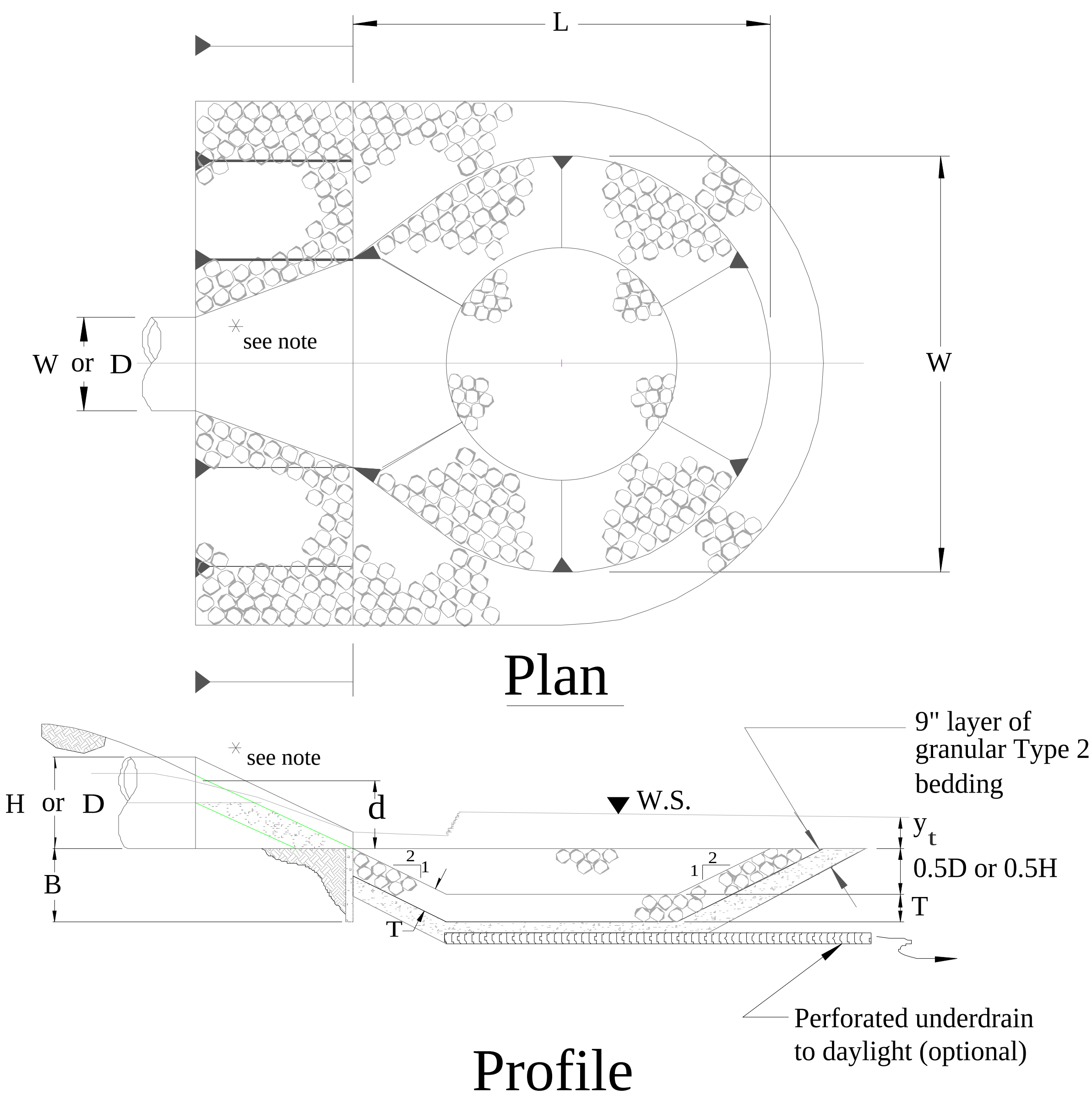




Sheet 2 of 2

Figure HS-18a—Grouted Boulder Rundown

(Sheet 2 of 2)



* Note: For rectangular conduits use a standard design for a headwall with wingwalls, paved bottom between the wingwalls, with an end cutoff wall extending to a minimum depth equal to B

Figure HS-19—Low Tailwater Riprap Basins for Storm Sewer Pipe Outlets—Low Tailwater Basin at Pipe Outlets
(Stevens and Urbonas 1996)

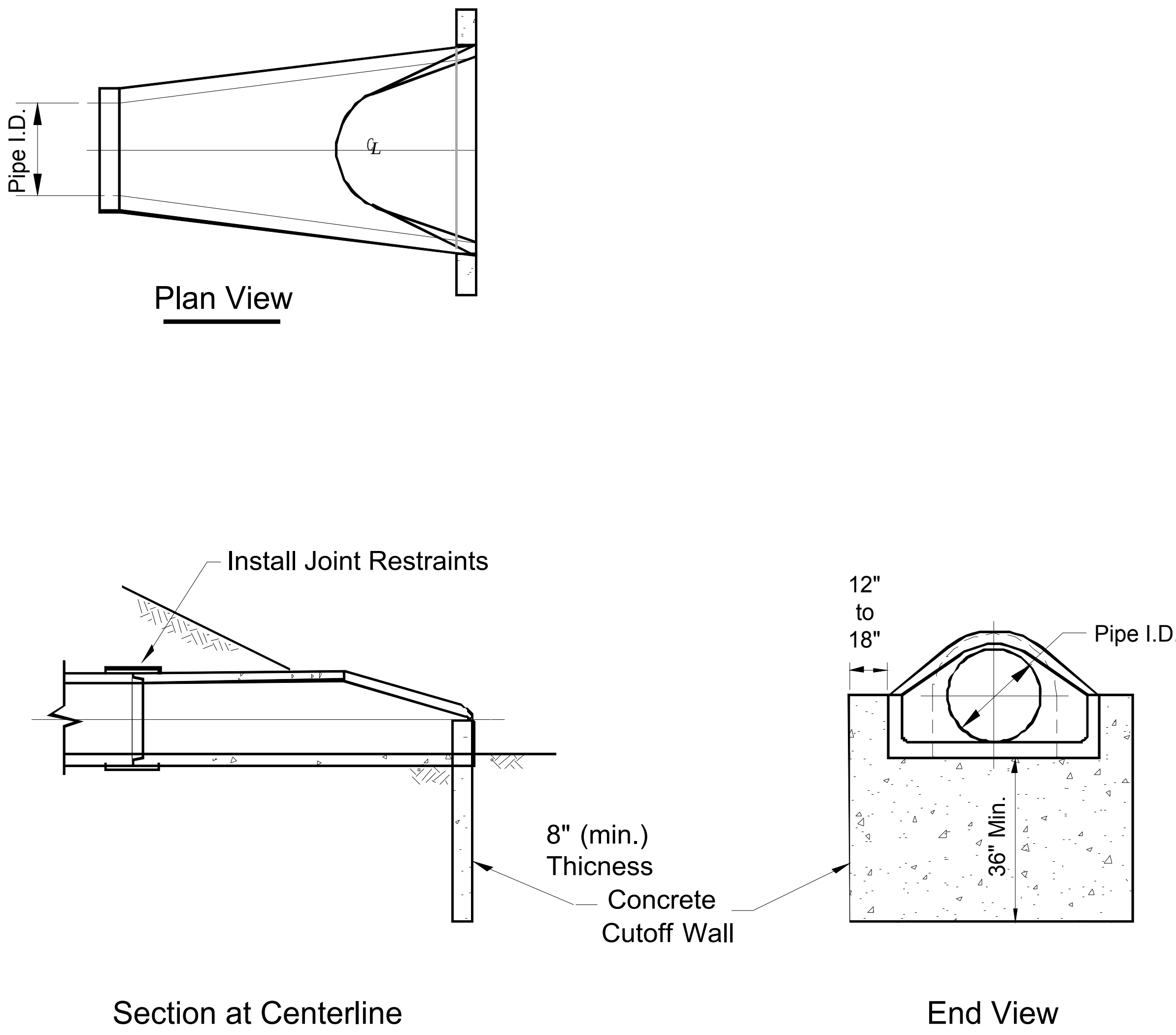


Figure HS-19a—Concrete Flared End Section with Cutoff Wall for all Pipe Outlets

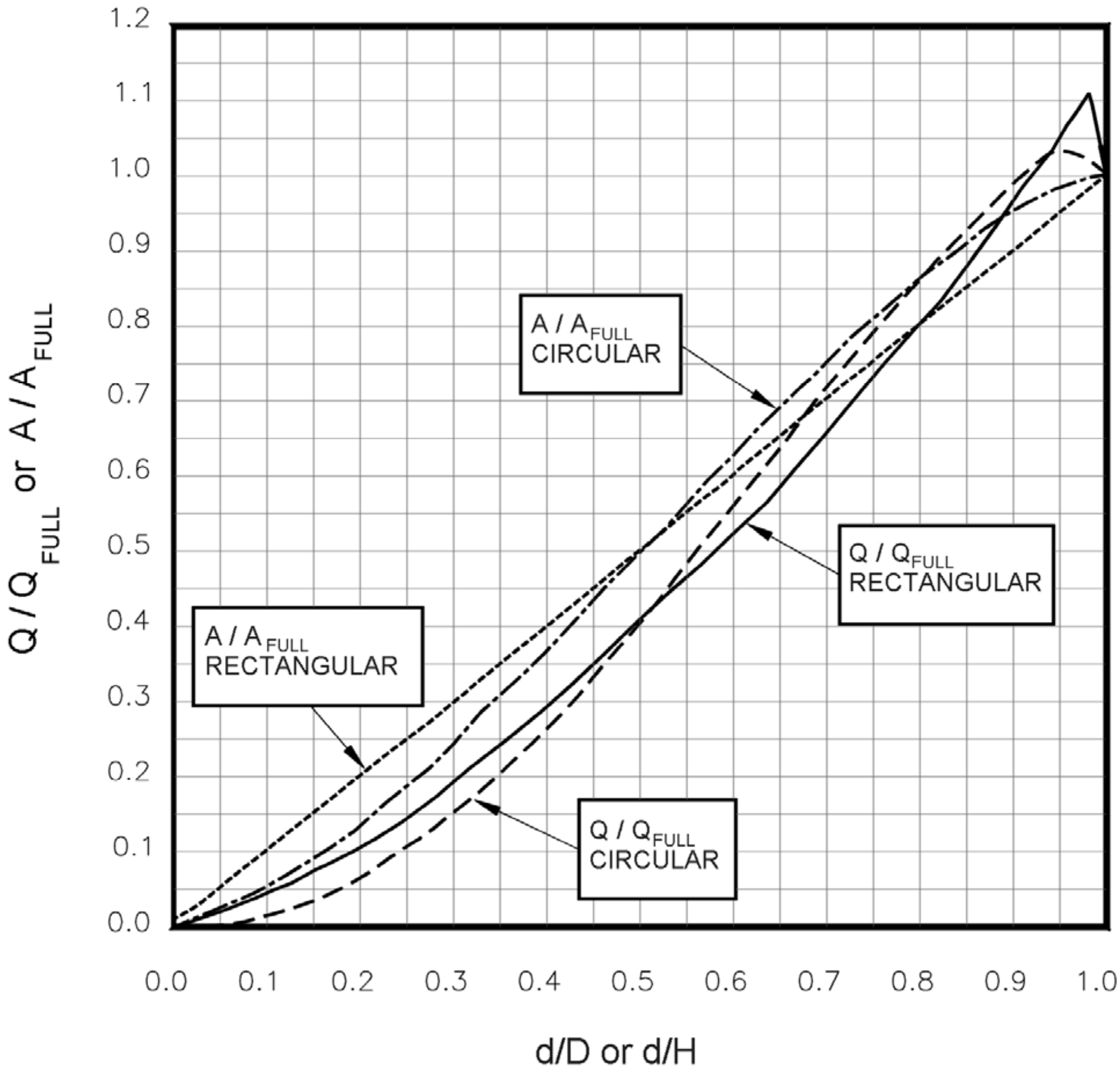


Figure HS-20a—Low Tailwater Riprap Basins for Storm Sewer Pipe Outlets—
Discharge and Flow Area Relationships for Circular and Rectangular Pipes
(Ratios for Flow Based on Manning's *n* Varying With Depth)
(Stevens and Urbonas 1996)

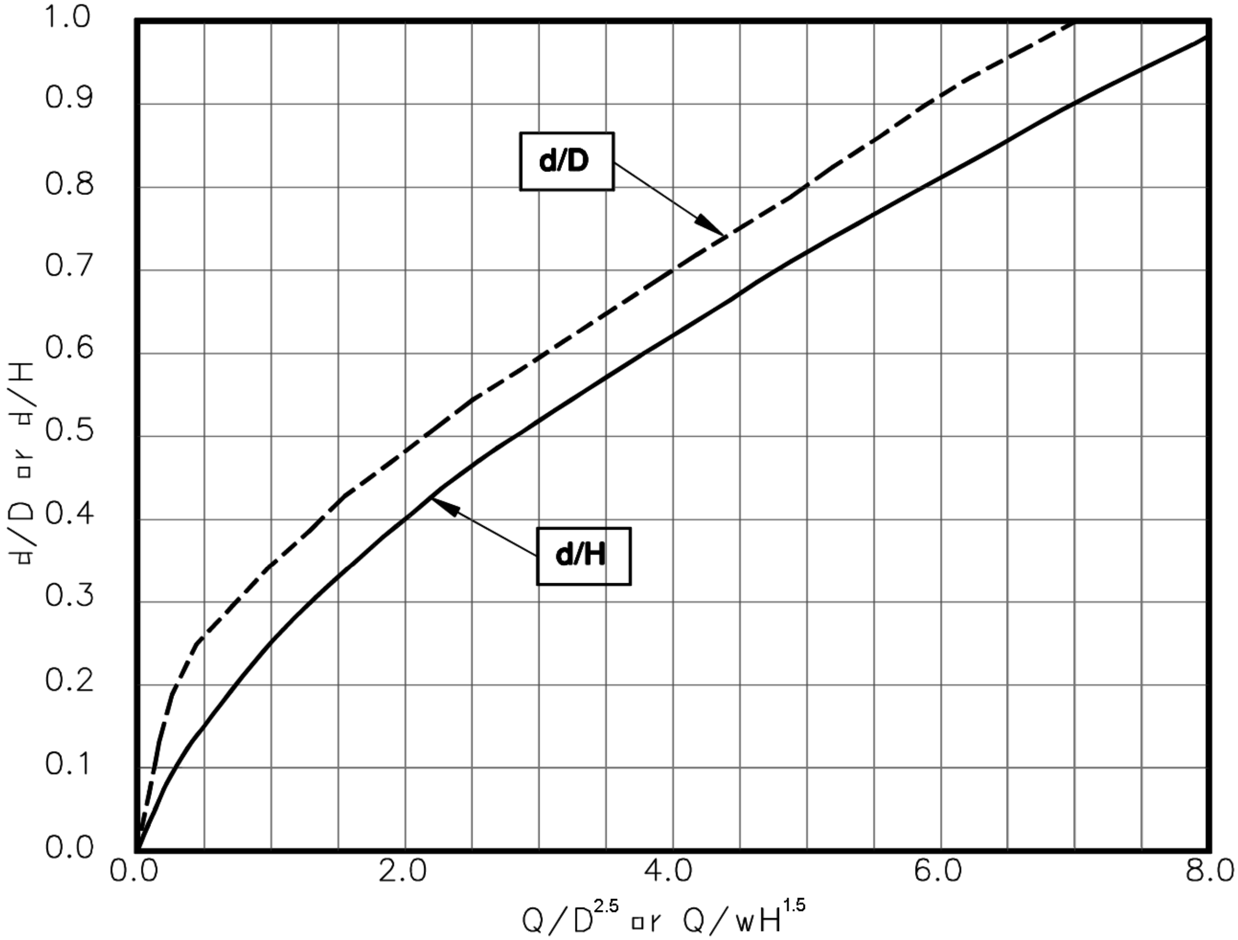


Figure HS-20b—Low Tailwater Riprap Basins for Storm Sewer Pipe Outlets—

Brink Depth for Horizontal Pipe Outlets

(Stevens and Urbonas 1996)

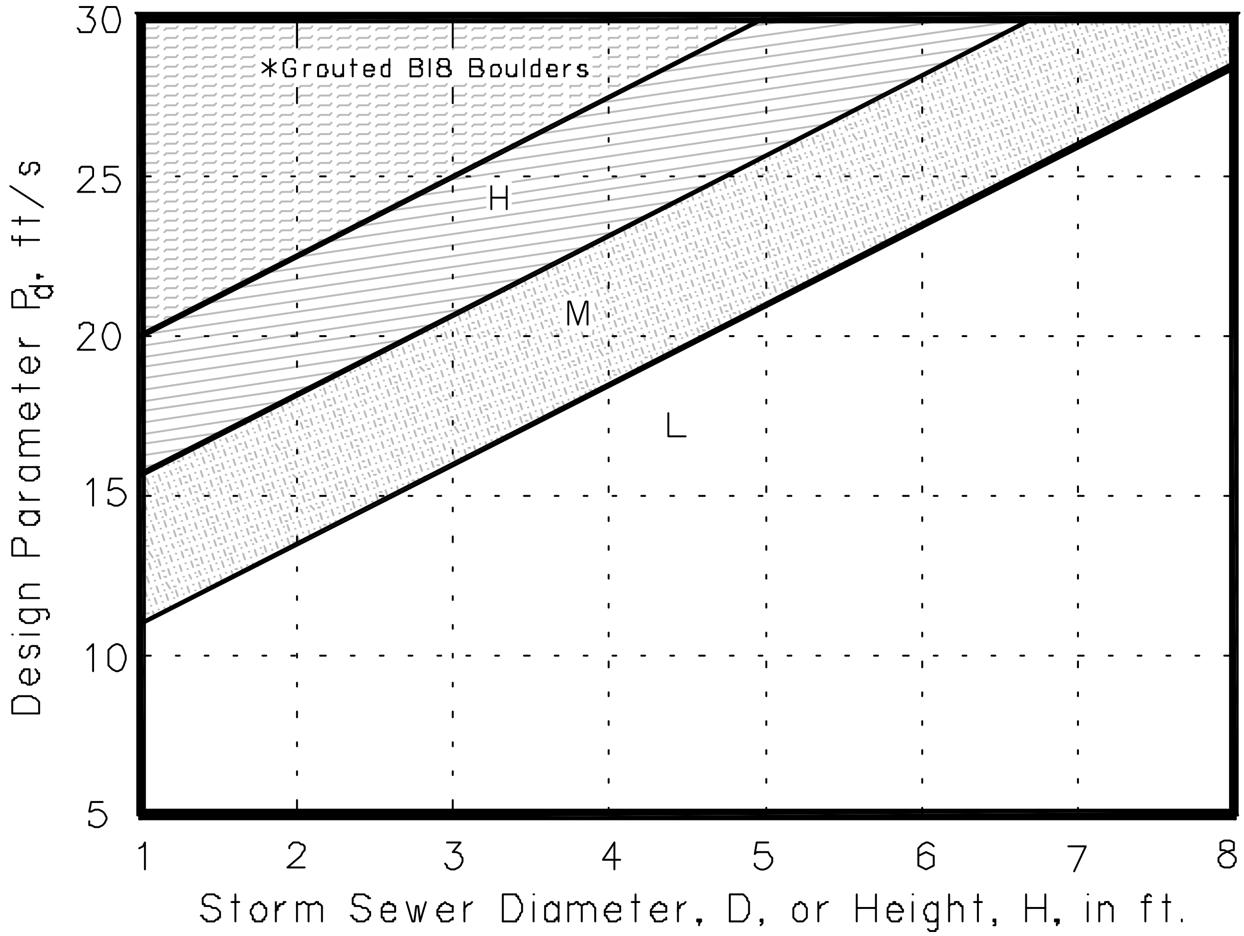


Figure HS-20c—Low Tailwater Riprap Basins for Storm Sewer Pipe Outlets—

Riprap Selection Chart for Low Tailwater Basin at Pipe Outlet

(Stevens and Urbonas 1996)

4.0 BRIDGES

There are extensive manuals on bridges that are available and should be used in bridge hydraulic studies and river stability analysis. Some of the best include:

1. *Hydraulics of Bridge Waterways* Hydraulic Design Series No. 1 (FHWA 1978). This is a good basic reference.
2. *Highway in the River Environment* (Richardson 1988 draft with appendices and 1974). This is particularly good for hydraulics, geomorphology, scour, and degradation.
3. *Design Manual for Engineering Analysis of Fluvial Systems for the Arizona Department of Water Resources* (LSA 1985). This is a prime reference on hydraulics and the three-level sediment transport analysis, with examples.

Photograph HS-14—A stable channel at bridges is important and includes caring for the stream downstream of the bridge as shown here on Cherry Creek.

4. *Hydraulic Analysis Location and Design of Bridges* Volume 7 (AASHTO 1987). This is a good overview document.
5. *Technical Advisory on Scour at Bridges* (FHWA 1988). This presents information similar to references 2, 3, and 4 above, but in a workbook format, and perhaps oversimplified.

Bridges are required across nearly all open urban channels sooner or later and, therefore, sizing the bridge openings is of paramount importance. Open channels with improperly designed bridges will either have excessive scour or deposition or not be able to carry the design flow.

4.1 Basic Criteria

Bridge openings should be designed to have as little effect on the flow characteristics as reasonable, consistent with good bridge design and economics. However, in regard to supercritical flow with a lined channel, the bridge should not affect the flow at all—that is, there should be no projections into the design water prism that could create a hydraulic jump or flow instability in form of reflecting and standing waves.

4.1.1 Design Approach

The method of planning for bridge openings must include water surface profiles and hydraulic gradient analyses of the channel for the major storm runoff. Once this hydraulic gradient is established without the bridge, the maximum reasonable effect on the channel flow by the bridge should be determined. In urban cases this should not exceed a backwater effect of more than 6 to 12 inches.

Velocities through the bridge and downstream of the bridge must receive consideration in choosing the bridge opening. Velocities exceeding those permissible will require special protection of the bottom and banks.

For supercritical flow, the clear bridge opening should permit the flow to pass under unimpeded and unchanged in cross section.

4.1.2 Bridge Opening Freeboard

The distance between the design flow water surface and the bottom of the bridge deck will vary from case to case. However, the debris that may be expected must receive full consideration in setting the freeboard. Freeboard may vary from several feet to minus several feet. There are no general rules. Each case must be studied separately. In larger waterways, streams and on rivers where large floating debris is likely, at least a 3-foot freeboard during a 100-year flood should be considered.

Bridges that are securely anchored to foundations and designed to withstand the dynamic forces of the flowing water might, in some cases, be designed without freeboard.

4.2 Hydraulic Analysis

The hydraulic analysis procedures described below are suitable, although alternative methods such as FHWA HY-4 or HEC-RAS are acceptable, as well.

The design of a bridge opening generally determines the overall length of the bridge. The length affects the final cost of the bridge. The hydraulic engineering in the design of bridges has more impact on the bridge cost than does the structural design. Good hydraulic engineering is necessary for good bridge design (FHWA 1978, Richardson 1974 and 1988).

The reader is referred to *Hydraulics of Bridge Waterways* (U.S. Bureau of Public Roads 1978) for more guidance on the preliminary assessment approach described below. In working with bridge openings, the designer may use the designation shown in Figure HS-21.

4.2.1 Expression for Backwater

A practical expression for backwater has been formulated by applying the principle of conservation of energy between the point of maximum backwater upstream from the bridge and a point downstream from the bridge at which normal stage has been reestablished, as shown in Sections 1 and 4, respectively, of Figure HS-21. The expression is reasonably valid if the channel in the vicinity of the bridge is reasonably uniform, the gradient of the bottom is approximately constant between Sections 1 and 4, there is no appreciable erosion of the bed in the constriction due to scour, and the flow is subcritical.

The expression for computation of backwater upstream from a bridge constricting the flow is as follows:

$$h_1^* = (K*)\left(\frac{(V_{n2})^2}{2g}\right) + \propto 1\left[\left(\frac{A_{n2}}{A_4}\right)^2 - \left(\frac{A_{n2}}{A_1}\right)^2\right]\frac{V_{n2}{}^2}{2g} \quad \text{(HS-23)}$$

in which:

h_1^* = total backwater (ft)

$K*$ = total backwater coefficient

$\propto 1 = \dfrac{qv^2}{QV_1^2}$ = kinetic energy coefficient

A_{n2} = gross water area in constriction measured below normal stage (ft^2)

V_{n2} = average velocity in constriction or Q/A_{n2} (ft/sec). The velocity V_{n2} is not an actual measurable velocity but represents a reference velocity readily computed for both model and field structures.

A_4 = water area at Section 4 where normal stage is reestablished (ft^2)

A_1 = total water area at Section 1 including that produced by the backwater (ft^2)

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

To compute backwater by Equation HS-23, it is necessary to obtain the approximate value of h_1^* by using the first part of the equation:

$$h_1^* = (K*)\left(\frac{V_{n2}^2}{2g}\right) \quad \text{(HS-24)}$$

The value of A_1 in the second part of Equation HS-23, which depends on h_1^*, can then be determined.

This part of the expression represents the difference in kinetic energy between Sections 4 and 1, expressed in terms of the velocity head $\frac{V_{n2}^2}{2g}$. Equation HS-24 may appear cumbersome, but it was set up as shown to permit omission of the second part when the difference in kinetic energy between Sections 4 and 1 is small enough to be insignificant in the final result.

To permit the designer to readily recognize cases in which the kinetic energy term may be ignored, the following guides are provided:

$M > 0.7$, where M = bridge opening ratio

$V_{n2} < 7$ ft/sec

$(K^*)\left(\frac{V_{n2}^2}{2g}\right) < 0.5$ ft

If values meet all three conditions, the backwater obtained from Equation HS-24 can be considered sufficiently accurate. Should one or more of the values not meet the conditions set forth, it is advisable to use Equation HS-23 in its entirety. The use of the guides is further demonstrated in the examples given in FHWA (1978) that should be used in all bridge design work.

4.2.2 Backwater Coefficient

The value of the overall backwater coefficient K^*, which was determined experimentally, varies with:

1. Stream constriction as measured by bridge opening ratio, M.
2. Type of bridge abutment: wingwall, spill through, etc.
3. Number, size, shape, and orientation of piers in the constriction.
4. Eccentricity, or asymmetric position of bridge with the floodplains.
5. Skew (bridge crosses floodplain at other than 90 degree angle).

The overall backwater coefficient K^* consists of a base curve coefficient, K_b, to which are added incremental coefficients to account for the effect of piers, eccentricity, and skew. The value of K^* is primarily dependent on the degree of constriction of the flow but also changes to a limited degree with the other factors.

4.2.3 Effect of M and Abutment Shape (Base Curves)

Figure HS-22 shows the base curve for backwater coefficient, K_b, plotted with respect to the opening ratio, M, for several wingwall abutments and a vertical wall type. Note how the coefficient K_b increases with channel constriction. The several curves represent different angles of wingwalls as can be identified by the accompanying sketches; the lower curves represent the better hydraulic shapes.

Figure HS-23 shows the relation between the backwater coefficient, K_b, and M for spill-through abutments for three embankment slopes. A comparison of the three curves indicates that the coefficient is little affected by embankment slope. Figures HS-22 and HS-23 are "base curves" and K_b is referred to as the "base curve coefficient." The base curve coefficients apply to normal crossings for specific abutment shapes but do not include the effect of piers, eccentricity, or skew.

4.2.4 Effect of Piers (Normal Crossings)

The effect on the backwater from introduction of piers in a bridge constriction has been treated as an incremental backwater coefficient designated ΔK_p, which is added to the base curve coefficient when piers are a factor. The value of the incremental backwater coefficient, ΔK_p, is dependent on the ratio that the area of the piers bears to the gross area of the bridge opening, the type of piers (or piling in the case of pile bents), the value of the bridge opening ratio, M, and the angularity of the piers with the direction of flood flow. The ratio of the water area occupied by piers, A_p, to the gross water area of the constriction, A_{n2}, both based on the normal water surface, has been assigned the letter J. In computing the gross water area, A_{n2}, the presence of piers in the constriction is ignored. The incremental backwater coefficient for the more common types of piers and pile bents can be obtained from Figure HS-24. The procedure is to enter Chart A, Figure HS-24, with the proper value of J and read ΔK and obtain the correction factor σ from Chart B, Figure HS-24, for opening ratios other than unity. The incremental backwater coefficient is then

$$\Delta K_p = \Delta K \sigma \tag{HS-25}$$

The incremental backwater coefficients for piers can, for all practical purposes, be considered independent of diameter, width, or spacing but should be increased if there are more than 5 piles in a bent. A bent with 10 piles should be given a value of ΔK_p about 20% higher than those shown for bents with 5 piles. If there is a good possibility of trash collecting on the piers, it is advisable to use a value greater than the pier width to include the trash. For a normal crossing with piers, the total backwater coefficient becomes:

$$K^* = K_b \text{ (Figures HS-22 or HS-23)} + \Delta K_p \text{ (Figure HS-24)} \tag{HS-26}$$

4.3 Design Procedure

The following is a brief step-by-step outline for determination of backwater produced by a bridge constriction:

1. Determine the magnitude and frequency of the discharge for which the bridge is to be designed.

2. Determine the stage of the stream at the bridge site for the design discharge.

3. Plot representative cross section of stream for design discharge at Section 1, if not already done under Step 2. If the stream channel is essentially straight and the cross section substantially uniform in the vicinity of the bridge, the natural cross section of the stream at the bridge site may be used for this purpose.

4. Subdivide the above cross section according to marked changes in depth of flow and roughness. Assign values of Manning's roughness coefficient, n, to each subsection. Careful judgment is necessary in selecting these values.

5. Compute conveyance and then discharge in each subsection.

6. Determine the value of the kinetic energy coefficient.

7. Plot the natural cross section under the proposed bridge based on normal water surface for design discharge and compute the gross water area (including area occupied by piers).

8. Compute the bridge opening ratio, M, observing modified procedure for skewed crossings.

9. Obtain the value of K_b from the appropriate base curve.

10. If piers are involved, compute the value of J and obtain the incremental coefficient, ΔK_p.

11. If eccentricity is severe, compute the value of eccentricity and obtain the incremental coefficient, ΔK_e (FHWA 1978).

12. If a skewed crossing is involved, observe proper procedure in previous steps, then obtain the incremental coefficient, ΔK_s, for proper abutment type.

13. Determine the total backwater coefficient, K^*, by adding incremental coefficients to the base curve coefficient, K_b.

14. Compute the backwater by Equation HS-23.

15. Determine the distance upstream to where the backwater effect is negligible.

Detailed steps illustrated by examples are presented in *Hydraulics of Bridge Waterways* (FHWA 1978).

4.4 Inadequate Openings

The engineer will often encounter existing bridges and culverts that have been designed for storms having return periods less than 100 years. In addition, bridges will be encountered which have been improperly designed. Often the use of the orifice formula will provide a quick determination of the adequacy or inadequacy of a bridge opening:

$$Q_m = C_b A_b \sqrt{2gH_{br}} \tag{HS-27}$$

or

$$H_{br} = 0.04\left(\frac{Q_m}{A_b}\right)^2 \tag{HS-28}$$

in which:

Q_m = the major storm discharge (cfs)

C_b = the bridge opening coefficient (0.6 assumed in Equation HS-27)

A_b = the area of the bridge opening (ft^2)

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

H_{br} = the head, that is the vertical distance from the bridge opening center point to the upstream water surface about $10H$ upstream from the bridge, where H is the height of the bridge, in feet. It is approximately the difference between the upstream and downstream water surfaces where the lower end of the bridge is submerged.

These expressions are valid when the water surface is above the top of the bridge opening.

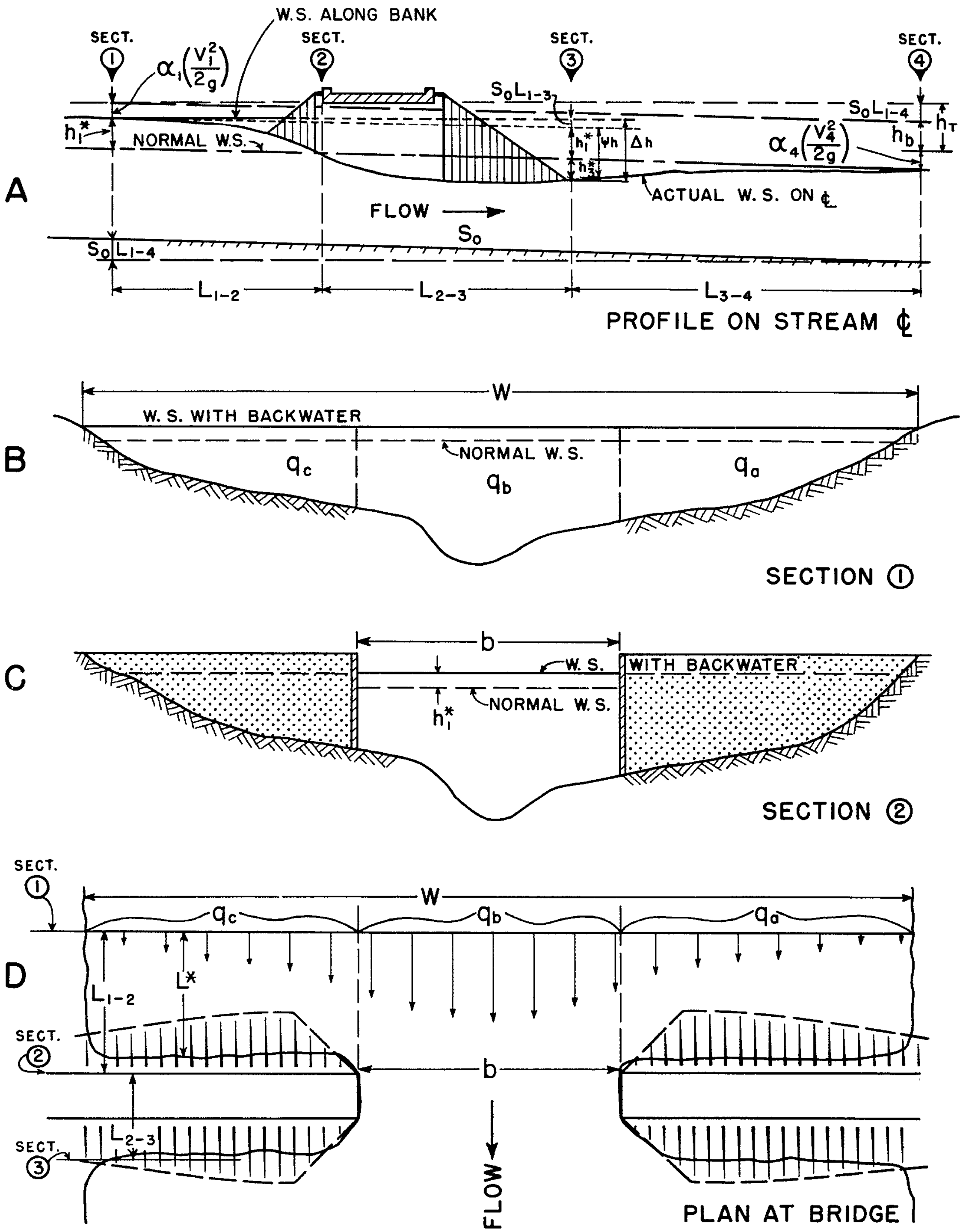


Figure HS-21—Normal Bridge Crossing Designation

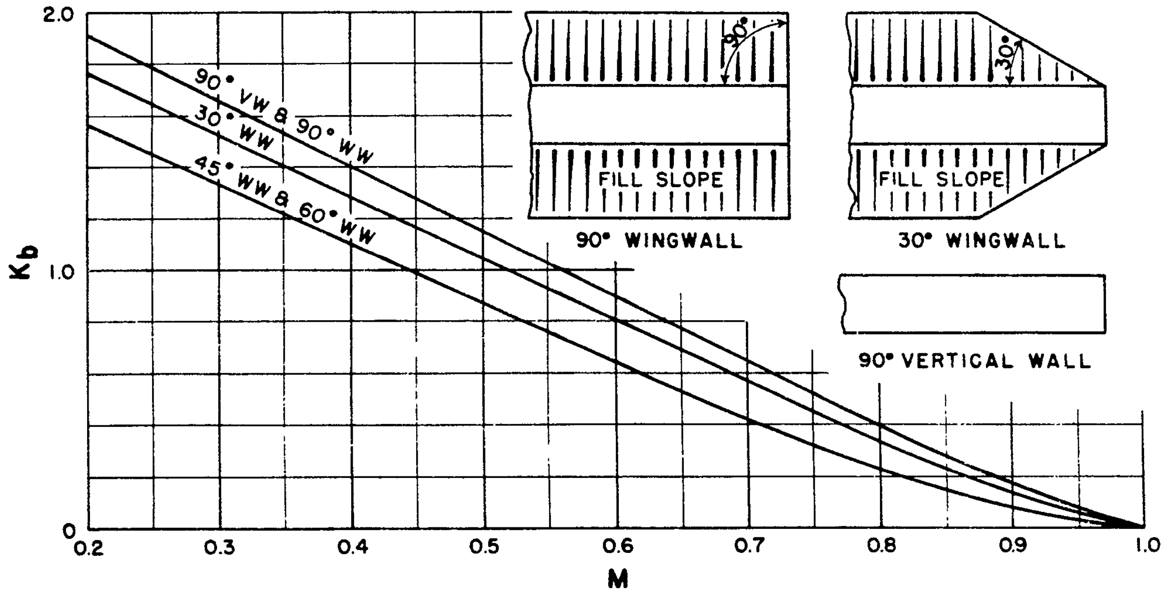


Figure HS-22—Base Curves for Wingwall Abutments

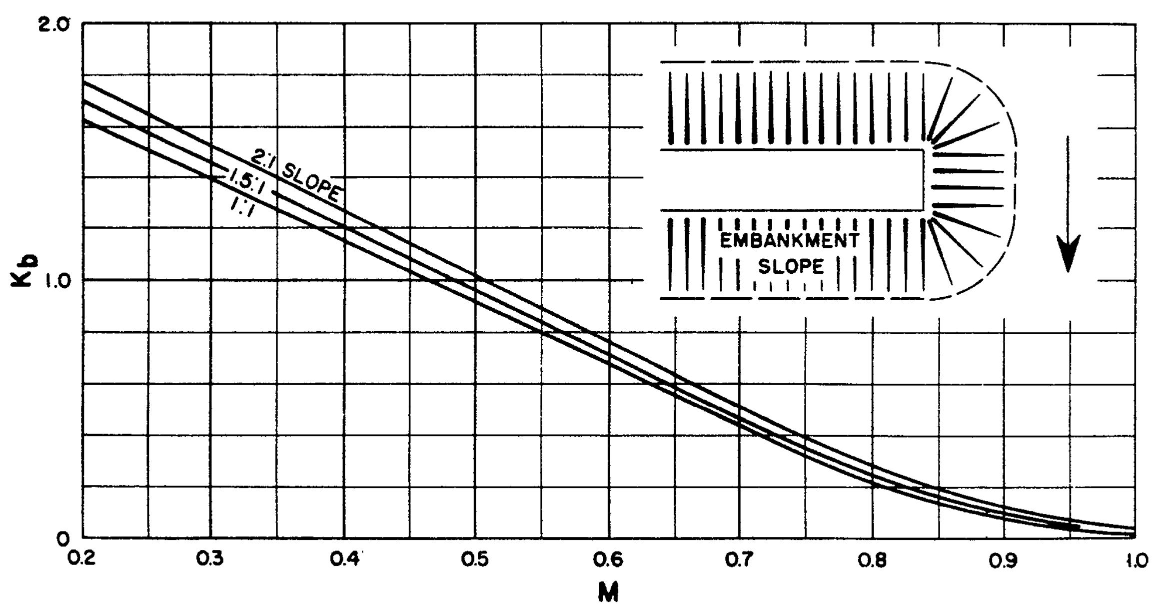


Figure HS-23—Base Curves for Spillthrough Abutments

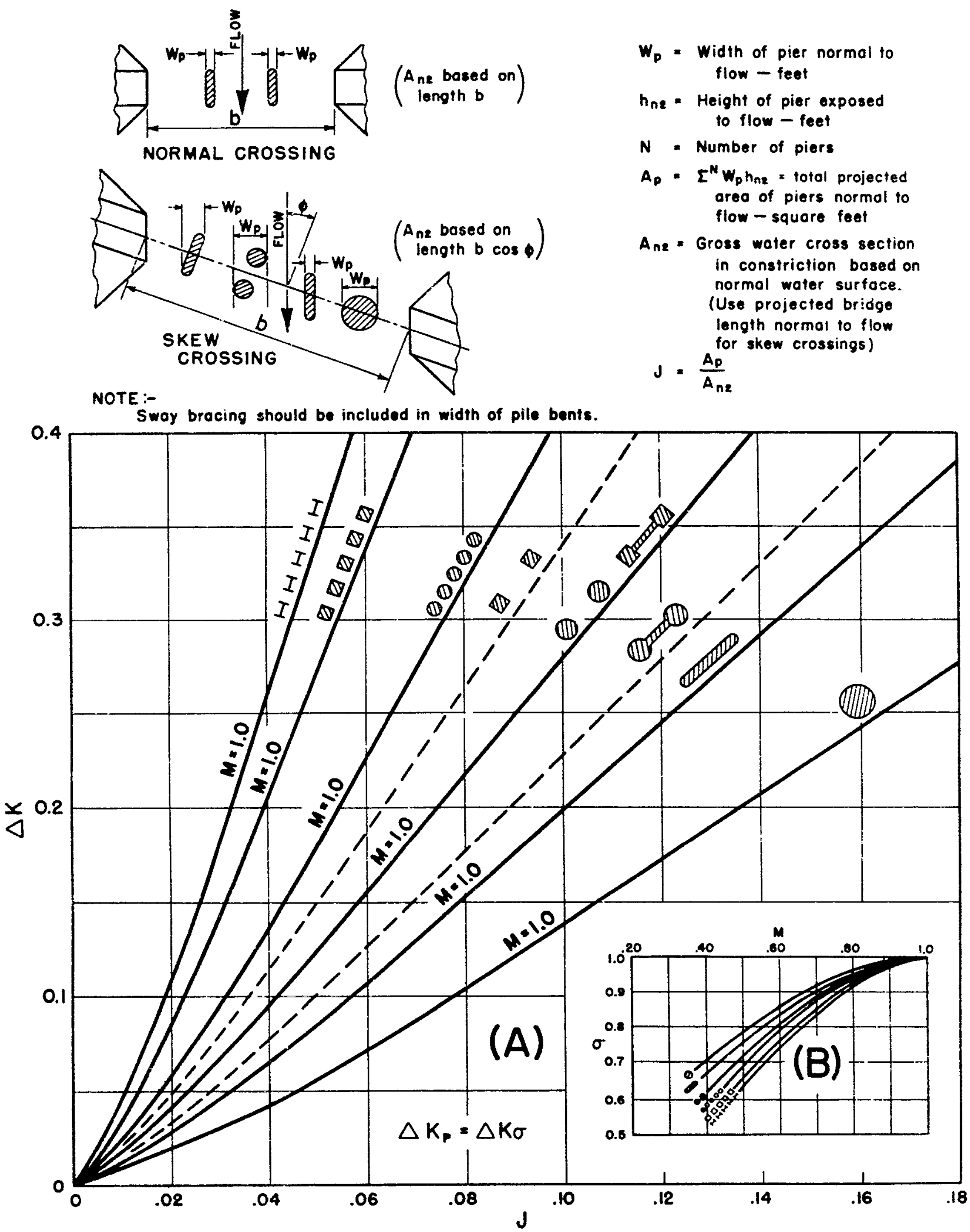


Figure HS-24—Incremental Backwater Coefficient for Pier

5.0 TRANSITIONS AND CONSTRICTIONS

5.1 Introduction

The purpose of this section is to outline typical design procedures for transition and constriction structures that are commonly encountered in the District's flood control and drainage projects. There are numerous references that can be useful for detailed analysis of different project objectives or site conditions (Rouse 1949, Chow 1959, USACE 1970 and 1982, FHWA 2000, SCS 1977). This topic is also addressed in MAJOR DRAINAGE, under riprap-lined channels.

5.2 Transition Analysis

5.2.1 Subcritical Transitions

Transitions for subcritical flow frequently involve localized structures or bank lining configurations that allow change in the cross section and produce a water surface profile based on gradually varied flow. The energy lost through a transition is a function of the friction, eddy currents and turbulence. The intent is often to minimize friction losses and/or erosional tendencies. Examples include transitions between trapezoidal and rectangular sections, modest transitions at bridges where little change takes place in the cross section, or slight encroachments into a channel to allow for utilities. Transitions can be handled with various structures, including concrete facilities (Figure HS-25) and riprap-lined channel reaches (see MAJOR DRAINAGE).

Standard water surface profile analysis is applied, with the addition of an energy loss at the transition. The loss is expressed as a function of the change in velocity head occurring across the contraction or expansion transition (from upstream to downstream locations). Figure HS-25 illustrates some of these transitions with basic design guidelines. Loss coefficients shown in Table HS-10 are applied to the difference in velocity head, as shown in Equation HS-29.

Analysis of transitions requires careful water surface profile analysis including verification of effective channel hydraulic controls. It is not uncommon to have a transition that is first thought to be performing in a subcritical mode, subsequently found to produce a supercritical profile with a hydraulic jump.

Energy Loss (ft) = Coefficient ($h_{v1} - h_{v2}$)

in which:

$$\left(h_{v1} - h_{v2}\right) = \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \qquad \text{(HS-29)}$$

V_1 = flow velocity upstream of transition

V_2 = flow velocity downstream of transition

Table HS-10—Subcritical Transition Energy Loss Coefficients

	Contraction	Expansion
Less than 4 inches between centerline and tangent lines	0.00	0.00
Less than 12.5 degrees between centerline and tangent lines	0 to 0.10	0 to 0.10
Warped type	0.10	0.20
Cylindrical quadrant type	0.15	0.25
Modest transitions	0.30	0.50
Straight line type	0.30	0.50
Square ended type	0.30+	0.75

5.2.2 Supercritical Transition Analysis

Supercritical transitions are beyond the scope of this *Manual* and require special analysis when used. The configuration of a supercritical transition is entirely different than subcritical transitions. Improperly designed and configured supercritical transitions can produce shock wave patterns which result in channel overtopping and other hydraulic and structural problems.

5.3 Constriction Analysis

5.3.1 Constrictions With Upstream Subcritical Flow

There are a variety of structures that are constrictions. They can include bridges, culverts, drop structures, and flow measurement devices. Constrictions of various types are used intentionally to control bed stability and upstream water surface profiles. For example, a constriction may be used to cause water to back up into or overflow into a flood storage pond.

The hydraulic distinction of constrictions is that they can cause rapidly varied flow. The upstream transition loss coefficients in Table HS-10 apply, but other factors come into play. Significant eddies can form upstream and downstream of the constriction depending upon the geometry. Flow separation will start at the upstream edge of the constriction, then the flow contracts to be narrower than the opening width. Typically, the width of contraction is 10% of the depth at the constriction for each side boundary. For example, at a typical drop with an abrupt crest contraction and assuming critical depth of 3.5 feet, the constriction on each side would be 0.35 feet or 0.7 feet total contraction from the opening width. Based on this contracted width and an assumption of critical conditions at that location, the upstream water surface profile may be computed.

In certain cases the flow regime will remain subcritical through the constriction. Chow (1959) presents guidelines developed by the U.S. Geological Survey for constrictions where the Froude number in the contracted section does not exceed 0.8. These cases are considered to be mild constrictions.

A consequence of abrupt contractions (and abrupt expansions) is that the velocities can be much higher in the center and change significantly across the constriction throat section. This results in a large energy coefficient and a further drop in water surface over what is first anticipated. This condition can produce strong eddy currents with high erosion potential. A constriction in an open channel needs to be carefully evaluated for velocity, scour, water surface, and related problems.

Constrictions used for flow depth control or flow measurement devices require a high degree of accuracy. The design information available that can be used for insuring a high degree of accuracy is limited. It is advisable to use model-tested or proven prototype layouts. As a secondary option, adjustable edge plates or other components can be provided to allow later changes at minimal cost if the constructed facilities should need refinement.

5.3.2 Constrictions With Upstream Supercritical Flow

This situation is highly complex and goes beyond the scope of this *Manual*. Possible shock waves or choked flow causing high upstream backwater or a hydraulic jump are distinct possibilities and are should be of major concern to designers. The situation is best avoided in urban channels and settings.

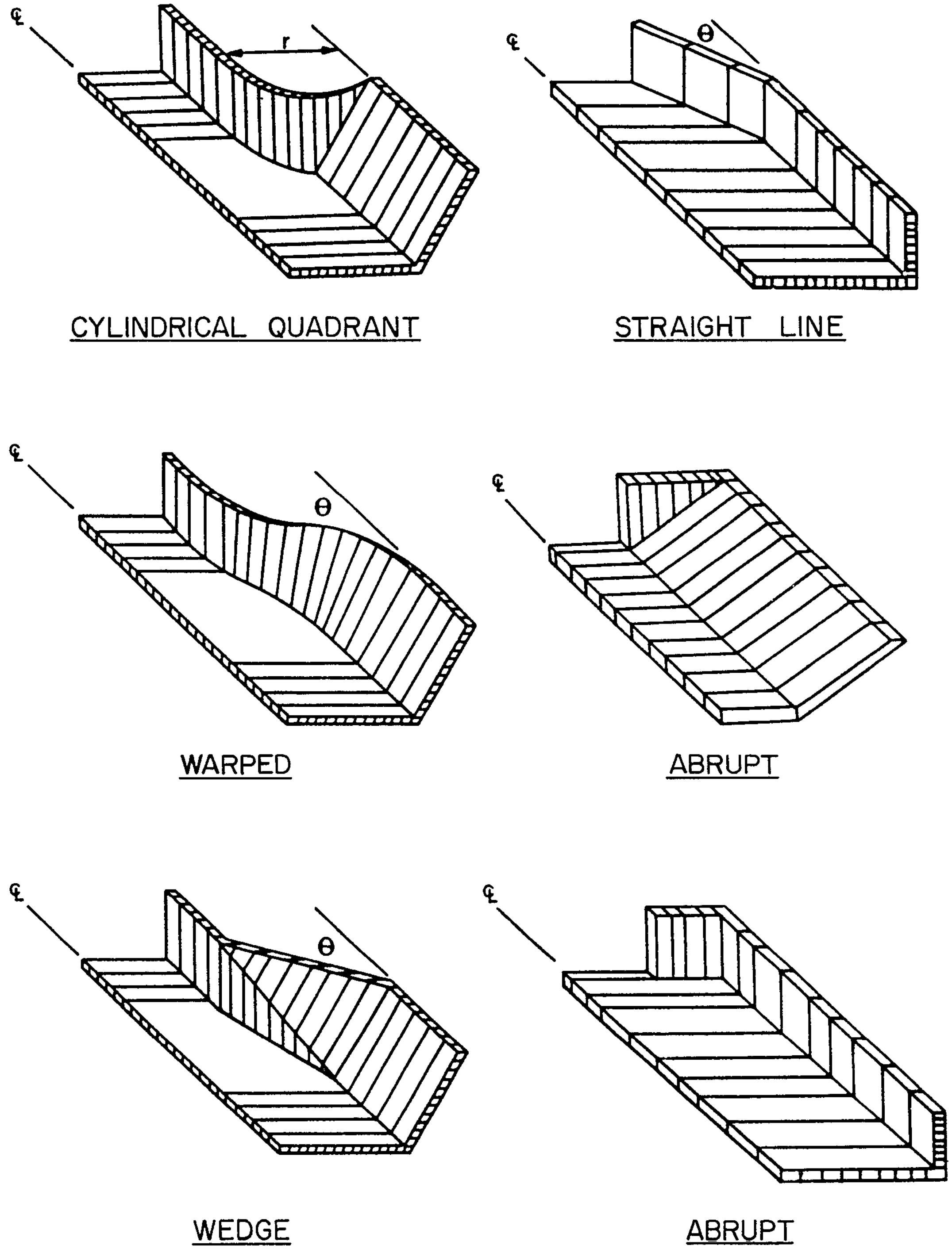


Figure HS-25—Transition Types

6.0 BENDS AND CONFLUENCES

6.1 Introduction

This section focuses on subcritical flow conditions. Because supercritical conditions can occur in various situations, a few supercritical conditions are also generally reviewed; however, supercritical flow analysis is not described in detail.

6.2 Bends

6.2.1 Subcritical Bends

Subcritical bends are required to have certain minimum curvatures described in the MAJOR DRAINAGE chapter. It is important that the engineer recognize the consequence of approaching and exceeding these criteria. Chow (1959), Rouse (1949) (see chapter by Ippen), and others illustrate flow patterns, superelevation, and backwater or flow resistance characteristics. Superelevation refers to a rise in water surface on the outer side of the bend. Effectively, the bend can behave like a contraction, causing backwater upstream and accelerated velocity zones, with high possibility of erosion on the outside of the bend and other locations. Significant eddy currents, scour, sedimentation, and loss of effective conveyance can occur on the inside of the bend.

Concrete-lined channels can be significantly affected by superelevation of the water surface. The designer should always add superelevation to the design freeboard of the channel. The equation for the amount of superelevation of the water surface, Δy that takes place is given as:

$$\Delta y = C_{se}\left(\frac{V^2 W_t}{g r_c}\right) \quad \text{(HS-30)}$$

in which:

C_{se} = coefficient, generally 0.5 for subcritical flow (see references for higher coefficients for supercritical)

V = mean channel velocity

W_t = channel top width of water surface

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

r_c = radius of the channel centerline curvature

6.2.2 Supercritical Bends

As with supercritical transitions, supercritical bend hydraulics are completely different than subcritical. Supercritical channels are not desirable in urban drainage; however, special situations occur where supercritical flows enter a curved channel. Some examples include at confluences where one channel is empty and the entering flow expands and becomes supercritical, at a sharp bend in a conduit with a slope that inherently leads to supercritical conditions, or at a channel drop that unavoidably ends up on a curve.

The main phenomenon to be aware of is shock waves, of which there are two types: positive and negative. On the outside of an angular bend, a positive shock wave will occur that results in a rise in water surface. The wave is stationary and crosses to the inside of the channel and then can continue to reflect back and forth. Where the flow passes the inside angular bend, a separation will occur, and a negative shock wave or drop in water surface will occur. This stationary negative shock wave will cross to the outside of the channel. Both shock waves will continue to reflect off the walls, resulting in a very disturbed flow pattern.

A basic control technique is to set up bend geometry to cause the positive shock wave to intersect the point where the negative wave is propagated. A bend usually requires two deflections on the outside and one bend on the inside. A beneficial aspect of the shock wave is that it turns the flow in a predictable pattern; thus, the channel walls have no more force imposed on them than that caused by the increased (or decreased) depths. This technique is described in Rouse (1949), USACE (1970), and Chow (1959).

Other control techniques include very gradual bends, super elevated floors and control sills, but these methods are generally less efficient. There is limited data on channels with sloping side banks, but it is clear there is a great tendency for shock waves to propagate up side slopes and divert flow out of the channel. Chow (1959) shows several good photographs of these problems. The SCS (1976) presents a documental report of a curved spillway on a modest flood control storage facility. During an overflow event, a shock wave pattern was produced that resulted in no flow on one side of a spillway, and great depths on the opposite.

Another problem observed at bends when channels operate under supercritical conditions is flow jumping out of the channel at the bend. When this happens, the downstream channel no longer carries the design flow and major damages to prosperities in line with the flows jumping out of the channel can and have occurred.

A special problem with long conduits used for flood control, particularly large box culverts, is that they will have an inherent tendency toward supercritical flow conditions at less than full capacity. When supercritical flow encounters bends or transitions, standing and reflective waves can occur which hit the ceiling of the culvert and can cause pressurized conditions or unstable conditions where the flow fluctuates between supercritical free surface flow and pressurized pipe flow conditions, often exacerbated by pressure variations in the pipe that can range from less than atmospheric to pressures approaching full velocity head. It is recommended that there be no bends or very gradual bends in conduits, along with air venting be provided when supercritical flows are expected in conduits, especially rectangular ones.

Use extreme caution in design anytime supercritical flow may occur and may encounter a bend or a transition.

6.3 Confluences

Some of the most difficult problems to deal with are confluences where the difference in flow characteristics may be great. When the flow enters the combined channel, the flow can diverge and drop in level if the flow capacity is suddenly increased. This can result in high velocity or unstable supercritical flow conditions with high erosion potential. When significant sediment flows exist, aggradation can occur at the confluence, resulting in loss of capacity in one or both upstream channels. The following material is adapted from *Hydraulic Design of Flood Control Channels* (USACE 1970).

6.3.1 Subcritical Flow Confluence Design

The design of channel junctions is complicated by variables such as the angle of intersection, shape and width of the channels, flow rates, and type of flow. The design of large complex junctions should be verified by model tests. The momentum equation design approach has been verified for small angles by Taylor (1944) and Webber and Greated (1966).

Figure HS-26 illustrates two types of junctions. The following assumptions are made for combining subcritical flows.

1. The side channel cross section is the same shape as the main channel cross section.
2. The bottom slopes are equal for the main channel and the side channel.
3. Flows are parallel to the channel walls immediately above and below the junction.
4. The depths are equal immediately above the junction in both the side and main channel.
5. The velocity is uniform over the cross sections immediately above and below the junction.

Assumption number 3 implies that hydrostatic pressure distributions can be assumed, and assumption number 5 suggests that the momentum correction factors are equal to each other at the reference sections.

The equation governing flow conditions for a vertical walled channel with the main channel width constant is shown in Figure HS-26(a) and the following equation:

$$\frac{Q_3^2}{gA_3}+\frac{by_3^2}{2}=\frac{Q_1^2}{gA_1}+\frac{(\cos\theta)Q_2^2}{gA_2}+\frac{by_1^2}{2} \tag{HS-31}$$

Or, for a vertical walled channel with the main channel width variable, Figure HS-26(b):

$$\frac{Q_3^2}{gA_3}+\frac{b_3y_3^2}{2}=\frac{Q_1^2}{gA_1}+\frac{(\cos\theta)Q_2^2}{gA_2}+\frac{b_3y_1^2}{2} \tag{HS-32}$$

Or, for a trapezoidal channel with the main channel width constant, Figure HS-26(a):

$$\frac{Q_3^2}{gA_3} + \left(\frac{b_1}{2} + \frac{Zy_3}{3}\right)y_3^2 = \frac{Q_1^2}{gA_1} + \frac{(\cos\theta)\,Q_2^2}{gA_2} + \left(\frac{b_1}{2} + \frac{Zy_1}{2}\right)y_1^2 \tag{HS-33}$$

Or, for trapezoidal channels with the main channel width variable, Figure HS-26(b):

$$\frac{Q_3^2}{gA_3} + \left(\frac{b_3}{2} + \frac{Zy_3}{3}\right)y_3^2 = \frac{Q_1^2}{gA_1} + \frac{(\cos\theta)\,Q_2^2}{gA_2} + \left(\frac{b_3}{2} + \frac{Zy_1}{3}\right)y_1^2 \tag{HS-34}$$

In which:

b = bottom width of the trapezoidal cross section

Z = side slope, horizontal to vertical

Momentum computations for a confluence involve a trial and error process. Starting with a known depth above or below the confluence, one iterates with an assumed depth on the unknown side of the confluence until the momentum has been balanced upstream to downstream.

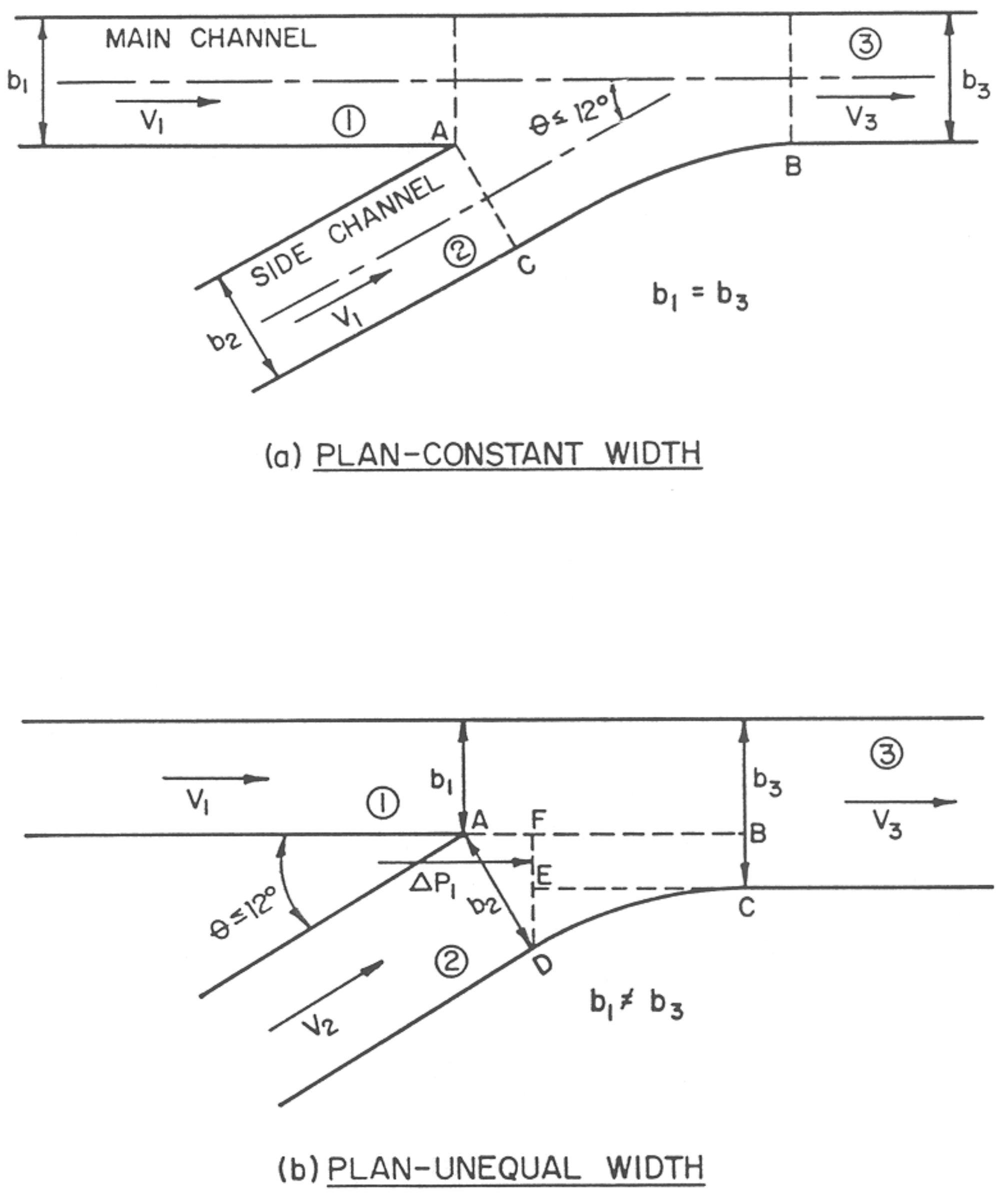


Figure HS-26—Channel Junction Definition Sketches

7.0 RUNDOWNS

A channel rundown is used to convey storm runoff from the bank of a channel to the invert of an open channel or drainageway. Rundowns can also convey runoff from streets and parking lots into channels or storage facilities. The purpose of these structures is to minimize channel bank erosion from concentrated overland flow. All too frequently, rundowns are treated as an afterthought, and receive little, if any, design attention. As a result, failure is common, resulting in unsightliness and a maintenance burden.

7.1 Cross Sections

Typical types of channel rundowns are presented in Figure HS-17, Figure HS-18 and Figure HS-27.

7.2 Design Flow

The channel rundown should be designed to carry the full design flow of the channel or storm sewer upstream of it (see the RUNOFF chapter) or 1 cfs, whichever is greater.

Photograph HS-15—A failed rundown that relied upon a geotextile membrane for stability.

7.3 Flow Depth

The maximum depth at the design flow should be equal to the calculated flow depth using drawdown calculations for the design flow plus 6 inches of freeboard. Due to the typical profile of a channel rundown beginning with a flat slope and then dropping steeply into the channel or storage facility, the design depth of flow should be the computed critical depth for the design flow.

7.4 Outlet Configuration for Trickle Channel

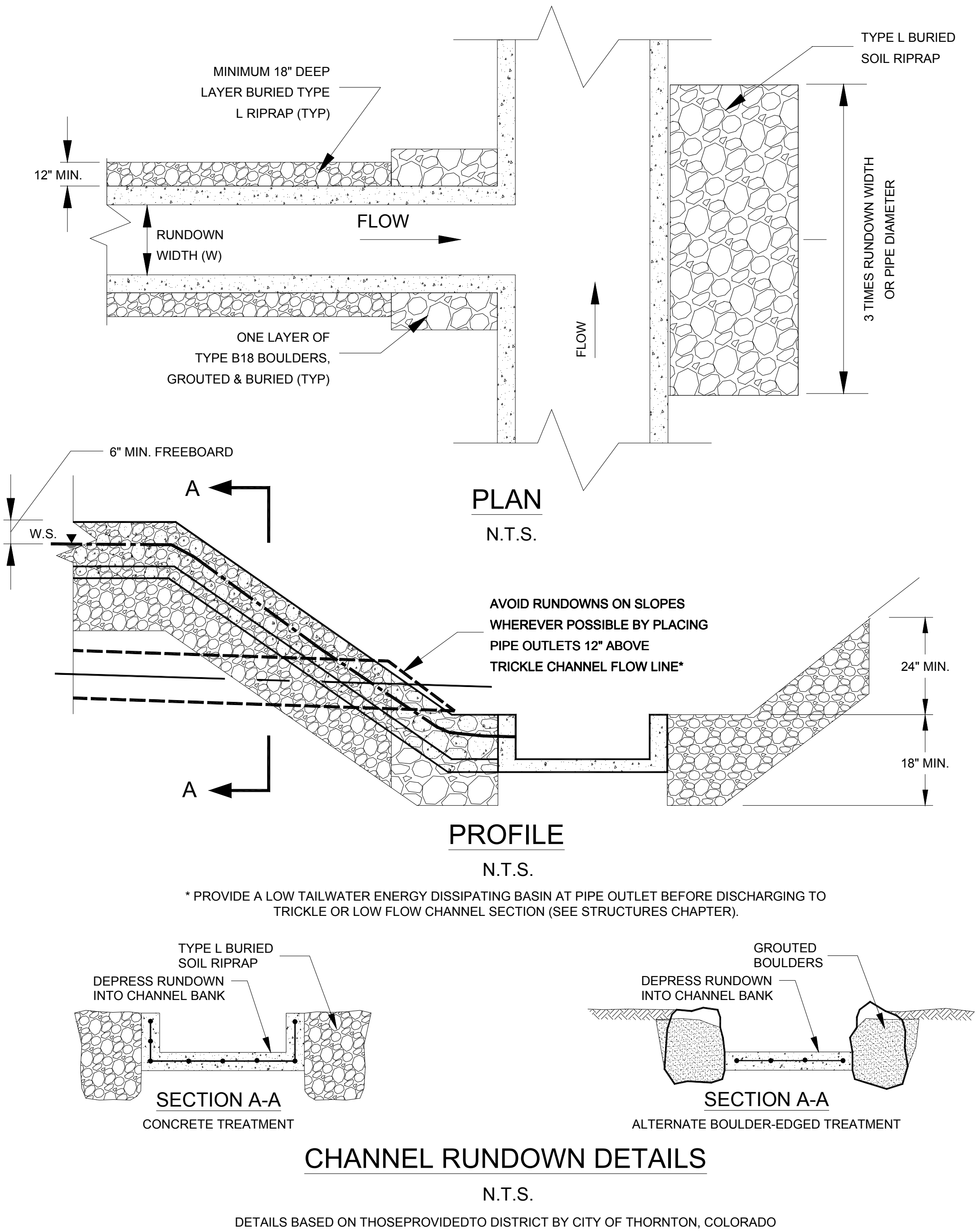
The channel rundown outlet should enter the drainageway at the trickle channel flow line. Erosion protection of the opposite channel bank should be provided by a layer of buried, grouted, Type B18 boulders. The width of this riprap erosion protection should be at least three times the channel rundown width or pipe diameter. Riprap protection should extend up the opposite bank to the minor storm flow depth in the drainageway or 2 feet, whichever is greater. Rundowns discharging into storage facilities should have comparable scour protection at the outlet, typically in the form of buried, grouted, Type B18 boulders. A forebay upstream of a trickle channel sized in accordance with Volume 3 recommendations of this *Manual* can provide this energy dissipation.

7.5 Outlet Configuration for Wetland Channel

For a wetland channel or low-flow channel, the rundown must be carried to the edge of the wetland where grouted rock is placed to dissipate the kinetic energy so that rundown discharge velocities do not cause erosion of the wetland. A low tailwater basin is also suitable for this purpose.

7.6 Grouted Boulder Rundowns

Instead of a concrete rundown, a grouted boulder rundown illustrated in Figure HS-18 can be provided. At a minimum, the width of a grouted boulder rundown should equal the width of the upstream storm sewer. The rundown depth should start at about ¾ of the height of the upstream pipe at the pipe and taper down to a depth equal to the calculated drawdown depth of water along the rundown plus 9 inches of freeboard. To find the depth of flow use Manning's n from Figure HS-3b. This will require iteration to find the value n that matches the depth of flow. Use boulders equal to at least 1/2 the height of the pipe (see boulder classifications in Table MD-8 of the MAJOR DRAINAGE chapter) grouted in accordance with the recommendations of Section 4.2.1.2 of the MAJOR DRAINAGE chapter.



* Provide a low tailwater energy dissipating basin at end of pipe before discharging to trickle or low-flow channel section.

Figure HS-27—Rundown

8.0 MAINTENANCE

8.1 General

Maintenance of structures includes removing debris, excessive vegetation and excessive sediment. Replacing or realigning stones, repairing grout and concrete, and replacing warning signs are also items of maintenance that cannot be avoided under normal conditions. Refer to the District's Maintenance Eligibility Guidelines as contained on the CD version of this *Manual* for specific guidance on maintenance provisions for many of the structures addressed in this chapter. See the District's Web site (www.udfcd.org) for the latest updates to these guidelines.

8.2 Access

During the design process, attention must be given to providing for adequate maintenance access from one or both banks in accordance with current District regulations and guidelines.

8.3 Maintenance Optimization

Structures should be designed in accordance with public works policies related to minimizing operation and maintenance requirements.

9.0 BOATABLE DROPS

9.1 Introduction

Low-head dams or drop structures on a stream that includes boating should not present undue hydraulic hazards to boaters, maintenance workers or to the public. This is why some low-head dams and drop structures are retrofitted. This section outlines the approach for use in improving recreational boater safety.

9.2 Retrofitting Existing Structures

Retrofitting low-head dams and drop structures generally includes installing a stepped or sloped downstream structure face and suitable boat chute with upstream pilot rocks; eliminating sharp edges; and providing appropriate barriers, signing and accessible portages with take-out and put-in landings. A structure that is too high for the site may be replaced with two or more structures to reduce the drop at a single location.

Retrofitting boatable low-head dams or drop structures requires specific care to insure that the retrofit meets the objective of enhancing public safety. Hydraulic model tests are common for retrofitting of low-head dams and drop structures.

9.2.1 Downstream Face

A vertical or steep downstream face of a structure to be retrofitted may be corrected with a rock face having a slope of 10(H) to 1(V). Large rock or derrick stone is often used. The engineer may select a stepped face of either concrete or stone.

9.2.2 Boat Chute

Installing a boat chute to provide passage around or over the low-head dam or drop is desirable for boatable streams, even where the total drop may be only 3 feet or less. The boat chute may be combined with a relatively flat, sloping downstream face in many instances. Pilot rocks planted upstream of the boat chute signal the entrance to the boat chute.

9.2.3 Sharp Edges

Exposed sheet piling edges, sharp concrete edges, sharp rock protuberances, and angle-iron ends should be avoided in boatable stream structures.

9.2.4 Barriers and Signing

A range of barriers may be considered for use at structures to help keep watercraft from crests, intakes, and areas of highly turbulent flow. Barriers often include buoy lines. Warning signs should be placed upstream of structures at easily visible locations.

Photograph HS-16—The unsightly and hazardous 8-foot-high Brown Ditch weir was replaced with three low-head drop structures having a 10:1 downstream slope and a boat chute. The resulting improvement by the USACE has provided for safe, enjoyable recreational boating.

9.2.5 Portages

At many hydraulic structures, portages are provided to permit beginning boaters to bypass a boat chute or to avoid a more challenging hydraulic structure. Portages have take-outs and put-ins at appropriate locations combined with suitable signing.

9.3 Safety

Retrofitting hydraulic structures on boatable streams should be undertaken with an adequate standard of care related to public safety for boating. A retrofit often includes installation of anchor points and suitable access for use by rescue personnel (Wright, et al. 1995).

10.0 STRUCTURE AESTHETICS, SAFETY AND ENVIRONMENTAL IMPACT

10.1 Introduction

Aesthetics, safety, overall integration with nearby land uses, and minimizing adverse environmental impacts are important aspects in the design of hydraulic structures. The planning, design, construction, and maintenance of hydraulic structures in an urban setting must include consideration of aesthetics, safety, and effects on the environment. Maximizing functional uses while improving visual quality and safety require good planning from the onset of the project and the coordinated efforts of owner/client, engineer, landscape architect, biologist, and planner.

10.2 Aesthetics and Environmental Impact

The combination and diversity of forms, lines, colors, and textures creates the visual experience. Material selection and placement of vegetation can provide visual character and create interesting spaces in and around hydraulic structures.

Photograph HS-17—Grouted sloping boulder drops can be built in series to create pleasing amenities and to provide stable and long-lived grade control structures.

Good planning may offer opportunities to minimize potentially adverse environmental impacts and maintain the natural habitat characteristics of the drainageway while fulfilling hydraulic, open space, and recreation requirements. As discussed in detail in the POLICY, PLANNING, and MAJOR DRAINAGE chapters, multiple uses of flood control structures, open space, and parks have proven to be an effective land use combination. Such structures as channels, overflow structures, grade controls, energy dissipators, maintenance roads, and others can blend in with the park environment.

In natural and urbanized areas, the use of vegetation for bank protection and landscape treatment is effective. Bioengineering strategies that incorporate vegetation and natural materials can improve habitat for fish and wildlife, and create a pleasant environment, as discussed in the MAJOR DRAINAGE chapter.

Plant selection and placement around structures and channel features and use of planting that reduces erosion, dissipates residual energy, and does not create debris or local scour problems are fundamental to good aesthetics and environmental quality, as well as hydraulic function. Inclusion of high-maintenance plantings and spaces with planting that are inaccessible or require extensive care are not advisable, since they may end up poorly maintained, become a nuisance, and be unattractive.

In highly developed streamside areas, concrete plazas and edge treatment can be combined to increase channel efficiencies while providing reasonable access to the waterway area. Geometric and architectural forms, hard edges, and formal arrangements of materials are generally associated with urban settings. However, all of these features require sound engineering and evaluation of the structure stability and the effects on the hydraulic characteristics of the channel. Such facilities are usually well received by the public.

A variety of materials and finishes are available for use in hydraulic structures. Concrete color additives, exposed aggregates and form liners can be used to create visual interest to otherwise stark walls. The location of expansion and control joints in combination with edges can be used to help create attractive design detailing of headwalls and abutments.

Natural materials, rock, and vegetation can be used for bank stability and erosion protection while providing unusual interest, spatial character, and diversity. The placement and type of the rock can provide poor or pleasing appearance. A stepped boulder arrangement for drops, where there is a larger top horizontal surface, is usually an appealing placement that also improves hydraulics.

10.3 Safety

Design and construction of urban drainage facilities must account for potential public safety hazards. When planning and providing for recreation within public parks and open space, safety must always be considered, and safety for the public and maintenance workers should be incorporated. The design engineer must consider the variations in hydraulic jumps as they relate to the tailwater elevation as illustrated in Figure HS-28. Some hydraulic structures and drainage features offer an invitation to play; therefore, what is constructed should be made safe and attractive. While safety, to a reasonable extent, becomes the responsibility of the user, appropriate warning signage must be used. In some instances, fencing and emergency access and egress should be provided.

Safety requirements are usually defined by local government agencies. However, case-made law may define the responsibilities of involved parties. Risk and liability are important with respect to including signs, handrails, or barriers at steep slopes or vertical drop-offs as well as other safety related features. Signage should be provided at locations where public use is intended near hydraulic structures and where hazards are not obvious to the average person. For boatable waterways the standard of care should include avoidance of hazardous hydraulics such as reverse rollers and reverse flow eddies associated with hydraulic structures. When bicycle paths are incorporated with the construction of structures, there should be adequate directional and warning signs, sight distance, and avoidance of unannounced sharp turns and dropoffs.

Photograph HS-18—Warning signs can be used to help achieve public boating safety, but signs cannot in themselves serve as a substitute for an appropriate standard of care in the design of a reasonable grade control structures on a boatable waterway.

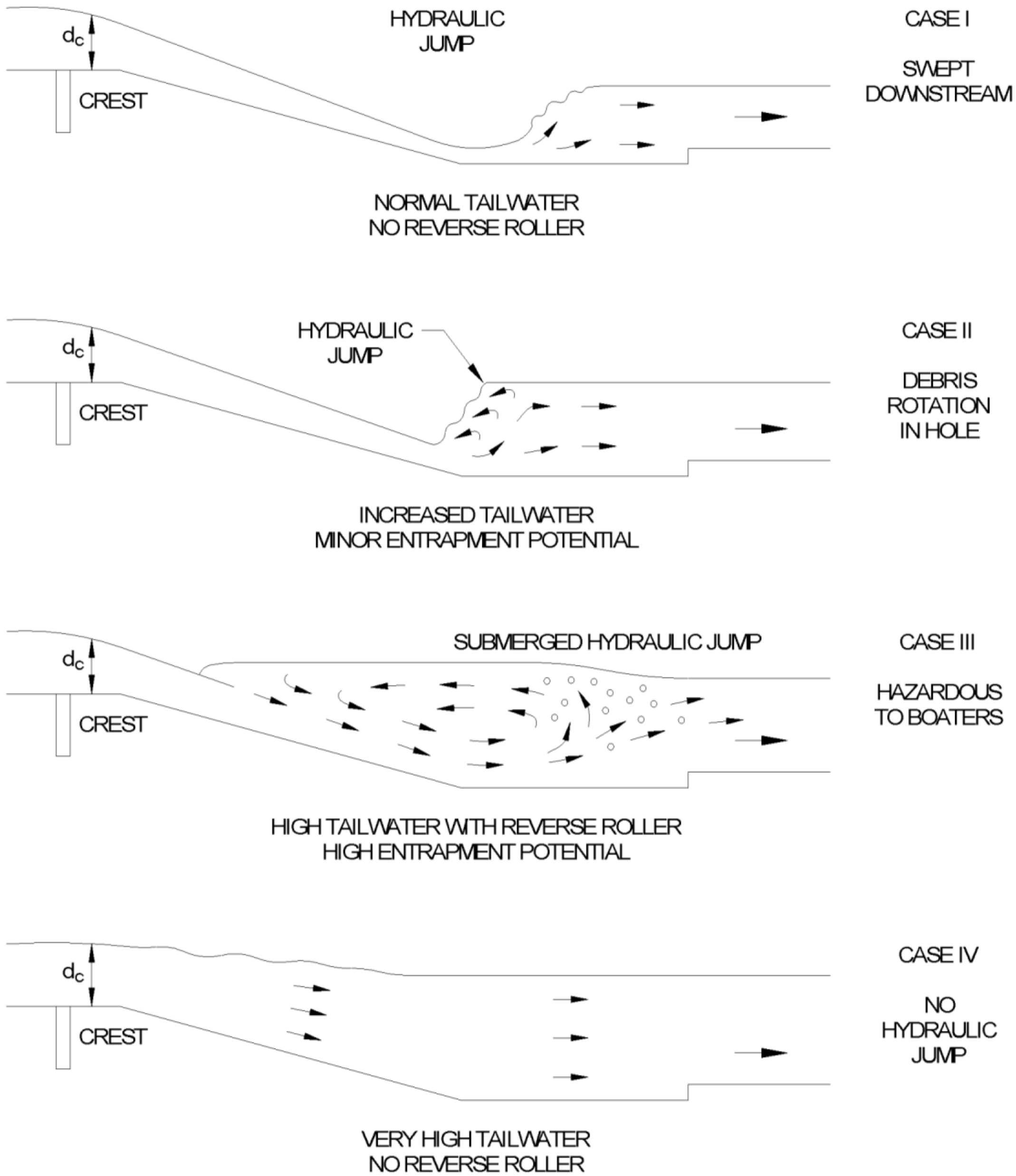


Figure HS-28—Hydraulic Jump Tailwater Stages as Related to Boating Hazards

11.0 CHECKLIST

Criterion/Requirement (Note: Before work begins in a floodplain, obtain a floodplain development permit form local jurisdiction)	✓
Drop Structures (All Types)	
Simplified design or detailed hydraulic analysis	
Soils and seepage analysis	
Environmental permits	
High public usage or low public usage	
Likely downstream degradation or no likely downstream degradation	
Critical depth at crest	
Transition head loss	
Hydraulic roughness	
Hydraulic jump length and location	
Basin length	
Seepage control (need detailed analysis or provisions for drops taller than 5 feet)	
Individual force analysis	
Trickle and low-flow zone provisions	
Sloping Drop Height > 6 feet, Use Special Design	
Sloping Drop Height $\leq$ 6 feet, Used Simplified Design	
Vertical Drop	
Rock sizing	
Boatable channel, or not	
Froude number at toe	
Reverse roller evaluation	
Portages and warning signs, with peer review	
Non-Boatable Grouted Sloping Boulder Drops	
Waterway is not boatable	
Maximum design discharge less than 7,500 cfs	
Uniform size boulders as per Table HS-4	
Drop height less than 5 feet	
Vertical cutoff minimum depth at crest of 0.8 H_d or 4 feet	
Trickle or low-flow channel through crest	
Net downward force of 30 PSF	
Stilling basin depressed 1 to 2 feet	
Drop face slope at 4:1, or flatter	
Grouted rock approach of 8 feet	
Basin length of 25 feet for erosive soils, and 20 feet for non-erosive soils	
Large boulders in center basin	
Buried downstream riprap zone 2 H_d or 10 feet	
If drop height exceeds 5 feet, detailed hydraulic analysis used (see Section 2.3)	
Vertical Hard Basin Drops	
Waterway is not boatable	
Maximum drop height of 3 feet	
Low probability for public access (public safety concern for vertical drops)	
Drop number D_n defined	
Hydraulic jump length is 6 times Y_2	
Basin floor depressed minimum 1.5 feet	
Minimum boulder size of 1.5 feet	
Grout thickness minimum 10 inches	
Basin length of 25 feet minimum	
Riprap approach length of 10 feet	

Criterion/Requirement	✓
Baffle Chute Drops	
Waterway is not debris-prone	
Waterway is not boatable	
Minimum of 4 baffle rows	
Unit discharge maximum of 60 cfs/ft	
Sloping apron of 2:1 or less	
Buried and protected toe with 1.5 baffle rows	
Baffle height of $0.8D_c$	
Wall height $2.4D_c$	
Boatable Channel Drops	
Maximum drop height of 4 feet	
Froude number at toe < 1.5	
Reverse rollers avoided	
Downstream face slope 10:1	
Pilot rocks and signing	
Suitable portage facilities	
Peer review	
Low-Flow Check and Wetland Structures	
Dominant discharge computed	
Trickle channel maximum depth of 3 feet, or 5 feet downstream of check	
Lateral overflow protection	
Trickle channel cutoff extension of 5 to 10 feet into bank	
Wetland checks extended 10 feet into bank	
Maintain upstream wetland water table	
Impact Stilling Basin Outlet Structures	
Horizontal entrance pipe	
Basin width as per Figure HS-15	
Calculate hydraulic force	
Type M riprap downstream	
Sill wall minimum of 2 feet	
Low Tailwater Basin Outlet Structures	
Riprap size as per Figure HS-20	
Minimum riprap thickness of 1.75 D_{50}	
Minimum basin length as per Equations HS-18 or HS-19	
Minimum basin width of $4D$ or $W + 4H$	
Riprap slopes of 2H to 1V	
Pipe fasteners and cutoff wall	
Culvert Outlet Energy Dissipator (Outlet Structures)	
Scour and degradation control	
Tailwater depth adequacy	
Bridges (Preliminary Assessment Only)	
Avoid scour and deposition	
Minimize hydraulic interferences	
Water surface profiles and hydraulic gradients determined	
Backwater effect less than 1 foot	
Banks and bottom protected from higher velocity flows	
Check for supercritical flow	
Adequate freeboard if debris prone	
Backwater coefficient K	
Procedure 4.3 followed for design	

Criterion/Requirement	✓
Boatable Drop Structures	
Downstream face at reasonable slope (e.g., 10H to 1V)	
Stepped face, or derrick stone	
Boat chute	
No sharp protrusions	
Pilot rocks	
Barriers if desirable	
Signing, informational and warning	
Portage with adequate signing	
Anchor points suitable for emergency rescue	
Peer review by whitewater expert	
General Items for Hydraulic Structures	
Visual quality	
Forms and lines	
Colors	
Vegetation	
Accessibility for maintenance; long-term maintenance assured	
Safety	
Public access	
Maintenance workers	
Hydraulic jump analysis with various tailwater elevations	
Signage	
Absence of reverse rollers and minimal reverse eddies	
Peer review	
Permitting	

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CULVERTS

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1.0 INTRODUCTION AND OVERVIEW

The function of a culvert is to convey surface water across a highway, railroad, or other embankment. In addition to the hydraulic function, the culvert must carry construction, highway, railroad, or other traffic and earth loads. Therefore, culvert design involves both hydraulic and structural design considerations. The hydraulic aspects of culvert design are set forth in this chapter.

Culverts are available in a variety of sizes, shapes, and materials. These factors, along with several others, affect their capacity and overall performance. Sizes and shapes may vary from small circular pipes to extremely large arch sections that are sometimes used in place of bridges.

The most commonly used culvert shape is circular, but arches, boxes, and elliptical shapes are used, as well. Pipe arch, elliptical, and rectangular shapes are generally used in lieu of circular pipe where there is limited cover. Arch culverts have application in locations where less obstruction to a waterway is a desirable feature, and where foundations are adequate for structural support. Box culverts can be designed to pass large flows and to fit nearly any site condition. A box or rectangular culvert lends itself more readily than other shapes to low allowable headwater situations since the height may be decreased and the span increased to satisfy the location requirements.

The material selected for a culvert is dependent upon various factors, such as durability, structural strength, roughness, bedding condition, abrasion and corrosion resistance, and water tightness. The more common culvert materials used are concrete and steel (smooth and corrugated).

Another factor that significantly affects the performance of a culvert is its inlet configuration. The culvert inlet may consist of a culvert barrel projecting from the roadway fill or mitered to the embankment slope. Other inlets have headwalls, wingwalls, and apron slabs or standard end sections of concrete or metal.

A careful approach to culvert design is essential, both in new land development and retrofit situations, because culverts often significantly influence upstream and downstream flood risks, floodplain management and public safety (Photograph CU-1). Culverts can be designed to provide beneficial upstream conditions (Photograph CU-2) and to avoid negative visual impact (Photograph CU-3).

Photograph CU-1—Public safety considerations for long culverts should be accounted for with culvert designs such as with this collapsible trash rack at a park-like location.

Photograph CU-2—Culverts can be designed to provide compatible upstream conditions for desirable wetland growth.

Photograph CU-3—Culverts can be integrated into the urban landscape without negative visual impact.

The information and references necessary to design culverts according to the procedure given in this chapter can be found in *Hydraulic Design of Highway Culverts*, Hydraulic Design Series No. 5 (FHWA 1985). Some of the charts and nomographs from that publication covering the more common requirements are given in this chapter. Nomographs and charts covering the range of applications commonly encountered in urban drainage are contained in Section 11.0. For special cases and larger sizes, the FHWA publication should be used.

1.1 Required Design Information

The hydraulic design of a culvert essentially consists of an analysis of the required performance of the culvert to convey flow from one side of the roadway (or other kind of embankment, such as a railroad) to the other. The designer must select a design flood frequency, estimate the design discharge for that frequency, and set an allowable headwater elevation based on the selected design flood and headwater considerations. These criteria are typically dictated by local requirements although state and federal standards will apply to relevant highway projects. The culvert size and type can be selected after the design discharge, controlling design headwater, slope, tailwater, and allowable outlet velocity have been determined.

The design of a culvert includes a determination of the following:

- Impacts of various culvert sizes and dimensions on upstream and downstream flood risks, including the implications of embankment overtopping.
- How will the proposed culvert/embankment fit into the relevant major drainageway master plan, and are there multipurpose objectives that should be satisfied?
- Alignment, grade, and length of culvert.
- Size, type, end treatment, headwater, and outlet velocity.
- Amount and type of cover.
- Public safety issues, including the key question of whether or not to include a safety/debris rack (Photograph CU-4).
- Pipe material.
- Type of coating (if required).
- Need for fish passage measures, in specialized cases.
- Need for protective measures against abrasion and corrosion.
- Need for specially designed inlets or outlets.
- Structural and geotechnical considerations, which are beyond the scope of this chapter.

Photograph CU-4—Public safety features such as the rack at the entrance to an irrigation ditch and the railing on the wingwalls must be considered.

1.1.1 Discharge

The discharge used in culvert design is usually estimated on the basis of a preselected storm recurrence interval, and the culvert is designed to operate within acceptable limits of risk at that flow rate. The design recurrence interval should be based on the criteria set forth in the POLICY chapter of this *Manual*. Specifically, refer to Tables DP-1 through DP-3 for street overtopping criteria.

1.1.2 Headwater

Culverts generally constrict the natural stream flow, which causes a rise in the upstream water surface. The elevation of this water surface is termed *headwater elevation*, and the total flow depth in the stream measured from the culvert inlet invert is termed *headwater depth*.

In selecting the design headwater elevation, the designer should consider the following:

- Anticipated upstream and downstream flood risks, for a range of return frequency events.
- Damage to the culvert and the roadway.
- Traffic interruption.
- Hazard to human life and safety.
- Headwater/Culvert Depth (HW/D) ratio.
- Low point in the roadway grade line.
- Roadway elevation above the structure.
- Elevation at which water will flow to the next cross drainage.

- Relationship to stability of embankment that culvert passes through.

The headwater elevation for the design discharge should be consistent with the freeboard and overtopping criteria in the POLICY chapter of this *Manual* (Tables DP1 through DP-3). The designer should verify that the watershed divides are higher than the design headwater elevations. In flat terrain, drainage divides are often undefined or nonexistent and culverts should be located and designed for the least disruption of the existing flow distribution.

1.1.3 Tailwater

Tailwater is the flow depth in the downstream channel measured from the invert at the culvert outlet. It can be an important factor in culvert hydraulic design because a submerged outlet may cause the culvert to flow full rather than partially full.

A field inspection of the downstream channel should be made to determine whether there are obstructions that will influence the tailwater depth. Tailwater depth may be controlled by the stage in a contributing stream, headwater from structures downstream of the culvert, reservoir water surface elevations, or other downstream features.

1.1.4 Outlet Velocity

The outlet velocity of a highway culvert is the velocity measured at the downstream end of the culvert, and it is usually higher than the maximum natural stream velocity. This higher velocity can cause streambed scour and bank erosion for a limited distance downstream from the culvert outlet. Permissible velocities at the outlet will depend upon streambed type, and the kind of energy dissipation (outlet protection) that is provided.

If the outlet velocity of a culvert is too high, it may be reduced by changing the barrel roughness. If this does not give a satisfactory reduction, it may be necessary to use some type of outlet protection or energy dissipation device. Most culverts require adequate outlet protection, and this is a frequently overlooked issue during design.

Variations in shape and size of a culvert seldom have a significant effect on the outlet velocity. Slope and roughness of the culvert barrel are the principal factors affecting the outlet velocity.

2.0 CULVERT HYDRAULICS

2.1 Key Hydraulic Principles

For purposes of the following review, it is assumed that the reader has a basic working knowledge of hydraulics and is familiar with the Manning's, continuity and energy equations, which were presented in the MAJOR DRAINAGE chapter (terms are defined in that chapter):

$$Q = \frac{1.49}{n} AR^{2/3}S^{1/2} \tag{CU-1}$$

$$Q = v_1 A_1 = v_2 A_2 \tag{CU-2}$$

$$\frac{v^2}{2g} + \frac{p}{\gamma} + z + \text{losses} = \text{constant} \tag{CU-3}$$

2.1.1 Energy and Hydraulic Grade Lines

Figures CU-1 and CU-2 illustrate the energy grade line (EGL) and hydraulic grade line (HGL) and related terms.

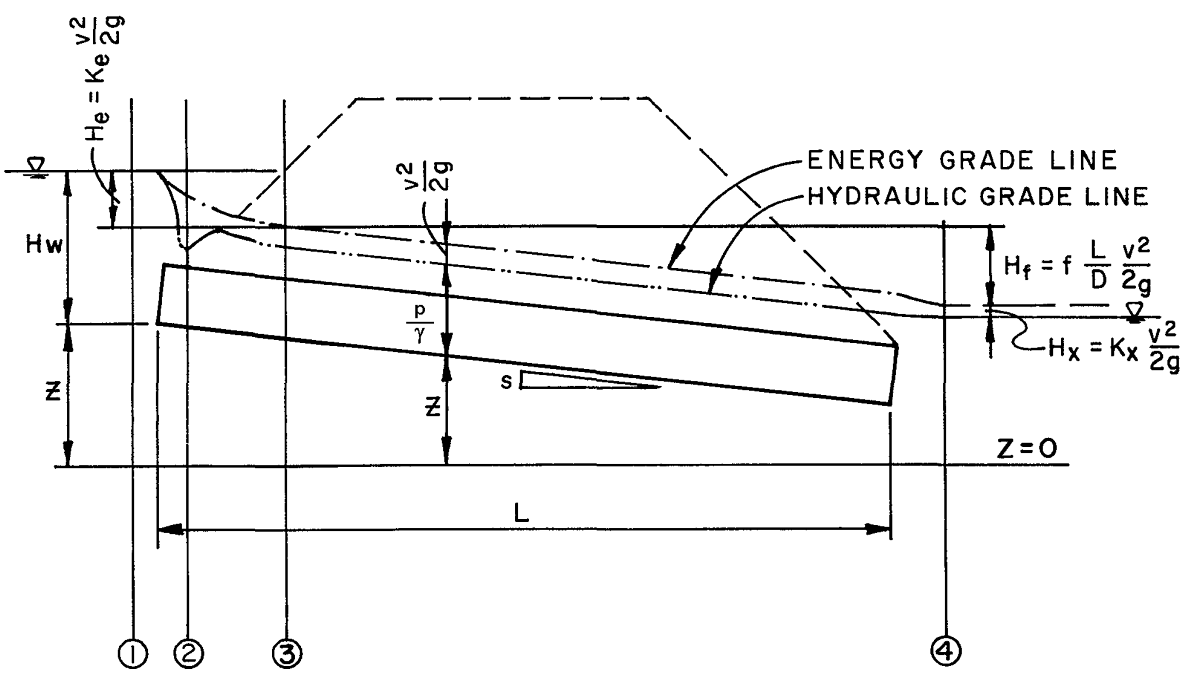


Figure CU-1—Definition of Terms for Closed Conduit Flow

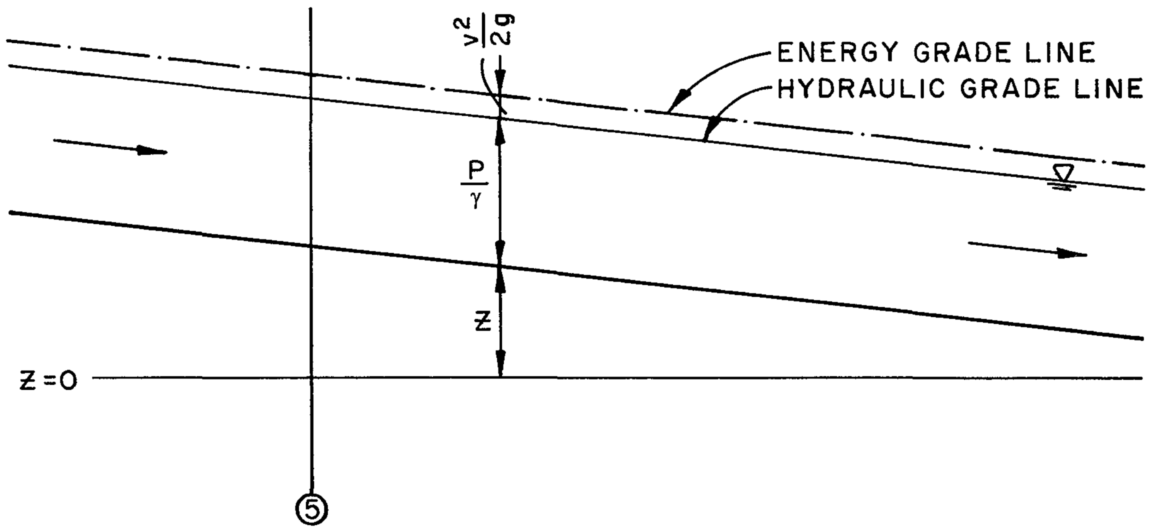


Figure CU-2—Definition of Terms for Open Channel Flow

The energy grade line, also known as the line of total head, is the sum of velocity head $v^2/2g$, the depth of flow or pressure head p/γ, and elevation above an arbitrary datum represented by the distance z. The energy grade line slopes downward in the direction of flow by an amount equal to the energy gradient H_L/L, where H_L equals the total energy loss over the distance L.

The hydraulic grade line, also known as the line of piezometric head, is the sum of the elevation z and the depth of flow or pressure head p/γ.

For open channel flow, the term p/γ is equivalent to the depth of flow and the hydraulic grade line is the same as the water surface. For pressure flow in conduits, p/γ is the pressure head and the hydraulic grade line falls above the top of the conduit as long as the pressure relative to atmospheric pressure is positive.

Approaching the entrance to a culvert as at Point 1 of Figure CU-1, the flow is essentially uniform and the hydraulic grade line and energy grade lines are almost the same. As water enters the culvert at the inlet, the flow is first contracted and then expanded by the inlet geometry causing a loss of energy at Point 2. As normal turbulent velocity distribution is reestablished downstream of the entrance at Point 3, a loss of energy is incurred through friction or form resistance. In short culverts, the entrance losses are likely to be high relative to the friction loss. At the exit, Point 4, an additional loss is incurred through turbulence as the flow expands and is retarded by the water in the downstream channel. At Point 5 of Figure CU-2 open channel flow is established and the hydraulic grade line is the same as the water surface.

There are two major types of flow conditions in culverts: (1) *inlet control* and (2) *outlet control*. For each type of control, a different combination of factors is used to determine the hydraulic capacity of a culvert. The determination of actual flow conditions can be difficult; therefore, the designer must check for both types of control and design for the most adverse condition.

2.1.2 Inlet Control

A culvert operates with inlet control when the flow capacity is controlled at the entrance by these factors:

- Depth of headwater
- Cross-sectional area
- Inlet edge configuration
- Barrel shape

When a culvert operates under inlet control, headwater depth and the inlet edge configuration determine the culvert capacity with the culvert barrel usually flowing only partially full.

Inlet control for culverts may occur in two ways. The least common occurs when the headwater depth is not sufficient to submerge the top of the culvert and the culvert invert slope is supercritical as shown in Figure CU-3.

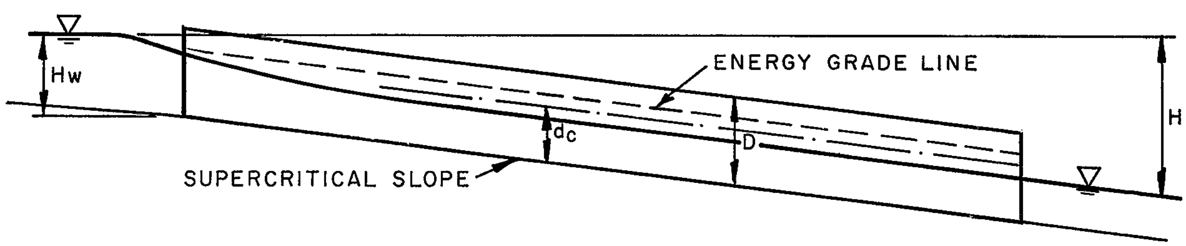


Figure CU-3—Inlet Control—Unsubmerged Inlet

The most common occurrence of inlet control is when the headwater submerges the top of the culvert (Figure CU-4), and the pipe does not flow full. A culvert flowing under inlet control is defined as a hydraulically short culvert.

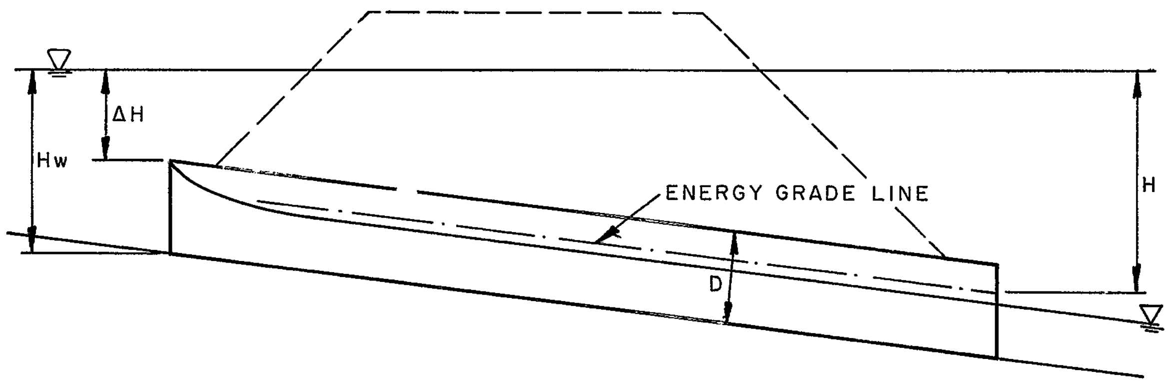


Figure CU-4—Inlet Control—Submerged Inlet

For a culvert operating with inlet control, the roughness, slope, and length of the culvert barrel and outlet conditions (including tailwater) are not factors in determining culvert hydraulic performance.

2.1.3 Outlet Control

If the headwater is high enough, the culvert slope sufficiently flat, and the culvert sufficiently long, the control will shift to the outlet. In outlet control, the discharge is a function of the inlet losses, the headwater depth, the culvert roughness, the culvert length, the barrel diameter, the culvert slope, and sometimes the tailwater elevation.

In outlet control, culvert hydraulic performance is determined by these factors:

- Depth of headwater
- Cross-sectional area
- Inlet edge configuration
- Culvert shape
- Barrel slope
- Barrel length
- Barrel roughness
- Depth of tailwater

Outlet control will exist under two conditions. The first and least common is that where the headwater is insufficient to submerge the top of the culvert, and the culvert slope is subcritical (Figure CU-5). The most common condition exists when the culvert is flowing full (Figure CU-6). A culvert flowing under outlet control is defined as a hydraulically long culvert.

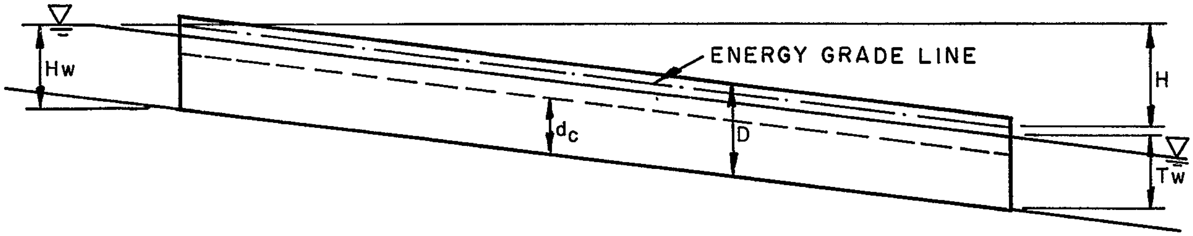


Figure CU-5—Outlet Control—Partially Full Conduit

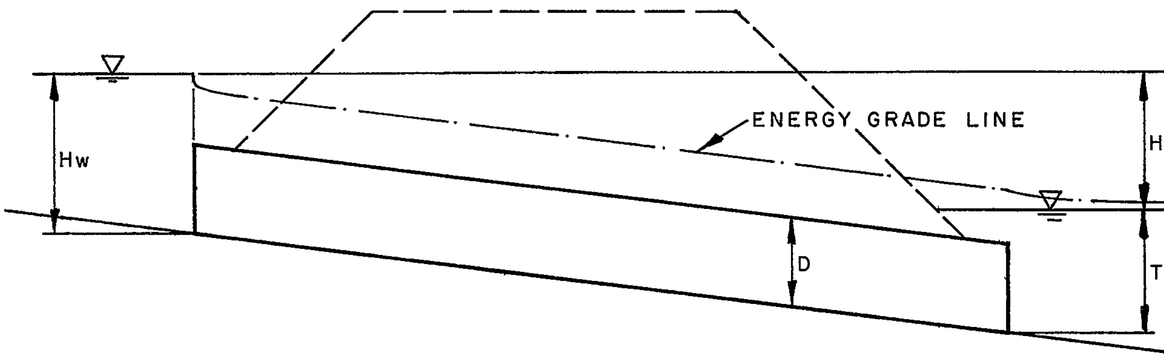


Figure CU-6—Outlet Control—Full Conduit

Culverts operating under outlet control may flow full or partly full depending on various combinations of the above factors. In outlet control, factors that may affect performance appreciably for a given culvert size and headwater are barrel length and roughness, and tailwater depth.

2.2 Energy Losses

In short conduits, such as culverts, the form losses due to the entrance can be as important as the friction losses through the conduit. The losses that must be evaluated to determine the carrying capacity of the culverts consist of inlet (or entrance) losses, friction losses and outlet (or exit) losses.

2.2.1 Inlet Losses

For inlet losses, the governing equations are:

$$Q = CA\sqrt{2gH} \qquad \text{(CU-4)}$$

$$H_e = K_e \frac{v^2}{2g} \qquad \text{(CU-5)}$$

where:

Q = flow rate or discharge (cfs)

C = contraction coefficient (dimensionless)

A = cross-sectional area (ft^2)

g = acceleration due to gravity, 32.2 (ft/sec^2)

H = total head (ft)

H_e = head loss at entrance (ft)

K_e = entrance loss coefficient

v = average velocity (ft/sec)

2.2.2 Outlet Losses

For outlet losses, the governing equations are related to the difference in velocity head between the pipe flow and that in the downstream channel at the end of the pipe.

2.2.3 Friction Losses

Friction head loss for turbulent flow in pipes flowing full can be determined from the Darcy-Weisbach equation.

$$H_f = f\left(\frac{L}{D}\right)\left(\frac{v^2}{2g}\right) \tag{CU-6}$$

where:

H_f = frictional head loss (ft)

f = friction factor (dimensionless)

L = length of culvert (ft)

D = Diameter of culvert (ft)

v = average velocity (ft/sec)

g = acceleration due to gravity, 32.2 (ft/sec^2)

The friction factor has been determined empirically and is dependent on relative roughness, velocity, and barrel diameter. Moody diagrams can be used to determine the friction factor. The friction losses for culverts are often expressed in terms of Manning's n, which is independent of the size of pipe and depth of flow. Another common formula for pipe flow is the Hazen-Williams formula. Standard hydraulic texts should be consulted for limitations of these formulas.

3.0 CULVERT SIZING AND DESIGN

FHWA (1985) Hydraulic Design Series No. 5, *Hydraulic Design of Highway Culverts*, provides valuable guidance for the design and selection of drainage culverts. This circular explains inlet and outlet control and the procedure for designing culverts. Culvert design basically involves the trial and error method:

1. Select a culvert shape, type, and size with a particular inlet end treatment.
2. Determine a headwater depth from the relevant charts for both inlet and outlet control for the design discharge, the grade and length of culvert, and the depth of water at the outlet (tailwater).
3. Compare the largest depth of headwater (as determined from either inlet or outlet control) to the design criteria. If the design criteria are not met, continue trying other culvert configurations until one or more configurations are found to satisfy the design parameters.
4. Estimate the culvert outlet velocity and determine if there is a need for any special features such as energy dissipators, riprap protection, fish passage, trash/safety rack, etc.

3.1 Description of Capacity Charts

Figure CU-7 is an example of a capacity chart used to determine culvert size. Refer to this figure in the following discussion.

Each chart contains a series of curves, which show the discharge capacity per barrel in cfs for each of several sizes of similar culvert types for various headwater depths in feet above the invert of the culvert at the inlet. The invert of the culvert is defined as the low point of its cross section.

Each size is described by two lines, one solid and one dashed. The numbers associated with each line are the ratio of the length, L, in feet, to 100 times the slope, s, in feet per foot (ft/ft) ($100s$). The dashed lines represent the maximum $L/(100s)$ ratio for which the curves may be used without modification. The solid line represents the division between outlet and inlet control. For values of $L/(100s)$ less than that shown on the solid line, the culvert is operating under inlet control and the headwater depth is determined from the $L/(100s)$ value given on the solid line. The solid-line inlet-control curves are plotted from model test data. The dashed-line outlet-control curves were computed for culverts of various lengths with relatively flat slopes. Free outfall at the outlet was assumed; therefore, tailwater depth is assumed not to influence the culvert performance.

For culverts flowing under outlet control, the head loss at the entrance was computed using the loss coefficients previously given, and the hydraulic roughness of the various materials used in culvert construction was taken into account in computing resistance loss for full or part-full flow. The Manning's n values used for each culvert type ranged from 0.012 to 0.032.

Except for large pipe sizes, headwater depths on the charts extend to 3 times the culvert height. Pipe arches and oval pipe show headwater up to 2.5 times their height since they are used in low fills. The dotted line, stepped across the charts, shows headwater depths of about twice the barrel height and indicates the upper limit of restricted use of the charts. Above this line the headwater elevation should be checked with the nomographs, which are described in Section 3.3.

The headwater depth given by the charts is actually the difference in elevation between the culvert invert at the entrance and the total head; that is, depth plus velocity head for flow in the approach channel. In most cases, the water surface upstream from the inlet is so close to this same level that the chart determination may be used as headwater depth for practical design purposes. Where the approach velocity is in excess of 3.0 ft/sec, the velocity head must be subtracted from the curve determination of headwater to obtain the actual headwater depth.

3.2 Use of Capacity Charts

1. The procedure for sizing the culvert is summarized below. Data can be tabulated in the Design Computation Form shown in Figure CU-8.

2. List design data: Q = flow or discharge rate (cfs), L = length of culvert (ft), allowable Hw = headwater depth (ft), s = slope of culvert (ft/ft), type of culvert barrel, and entrance.

3. Compute $L/(100s)$.

4. Enter the appropriate capacity chart in Section 11.0 with the design discharge, Q.

5. Find the $L/(100s)$ value for the smallest pipe that will pass the design discharge. If this value is above the dotted line in Figure CU-7, use the nomographs to check headwater conditions.

6. If $L/(100s)$ is less than the value of $L/(100s)$ given for the solid line, then the value of Hw is the value obtained from the solid line curve. If $L/(100s)$ is larger than the value for the dashed outlet control curve, then special measures must be taken, and the reader is referred to *Hydraulic Design of Highway Culverts* (FHWA 1985).

7. Check the Hw value obtained from the charts with the allowable Hw. If the indicated Hw is greater than the allowable Hw, then try the Hw elevation from the next largest pipe size.

3.3 Use of Nomographs

Examples of two nomographs for designing culverts are presented in Figures CU-9 and CU-10. The use of these nomographs is limited to cases where tailwater depth is higher than the critical depth in the culvert. The advantage of the capacity charts over the nomographs is that the capacity charts are direct where the nomographs are trial and error. The capacity charts can be used only when the flow passes through critical depth at the outlet. When the critical depth at the outlet is less than the tailwater depth, the nomographs must be used; however, both give the same results where either of the two methods may be used. The procedure for design requires the use of both nomographs and is as follows (refer to Figures CU-9 and CU-10):

1. List design data: Q (cfs), L (ft), invert elevations in and out (ft), allowable Hw (ft), mean and maximum flood velocities in natural stream (ft/sec), culvert type and entrance type for first selection.

2. Determine a trial size by assuming a maximum average velocity based on channel considerations to compute the area, $A = Q/V$.

3. Find Hw for trial size culvert for inlet control and outlet control. For inlet control, Figure CU-9, connect a straight line through D and Q to scale (1) of the Hw/D scales and project horizontally to the proper scale, compute Hw and, if too large or too small, try another size before computing Hw for outlet control.

4. Next, compute the Hw for outlet control, Figure CU-10. Enter the graph with the length, the entrance coefficient for the entrance type, and the trial size. Connect the length scale and the culvert size scale with a straight line, pivot on the turning line, and draw a straight line from the design discharge on the discharge scale through the turning point to the head scale (head loss, H). Compute Hw from the equation:

$$Hw = H + h_o - Ls \tag{CU-7}$$

where:

Hw = headwater depth (ft)

H = head loss (ft)

h_o = tailwater depth or elevation at the outlet of a depth equivalent to the location of the hydraulic grade line (ft)

L = length of culvert (ft)

s = slope of culvert (ft/ft)

For Tw greater than or equal to the top of the culvert, $h_o = Tw$, and for Tw less than the top of the culvert:

$$h_o = \frac{(d_c + D)}{2} \text{ or } T_w \text{ (whichever is greater)} \tag{CU-8}$$

where:

d_c = critical depth (ft)

Tw = tailwater depth (ft)

If Tw is less than d_c, the nomographs cannot be used, see *Hydraulic Design of Highway Culverts* (FHWA 1985) for critical depth charts.

5. Compare the computed headwaters and use the higher Hw to determine if the culvert is under inlet or outlet control. If outlet control governs and the Hw is unacceptable, select a larger trial size and find another Hw with the outlet control nomographs. Since the smaller size of culvert had been selected for allowable Hw by the inlet control nomographs, the inlet control for the larger pipe need not be checked.

3.4 Computer Applications, Including Design Spreadsheet

Although the nomographs discussed in this chapter are still used, engineers are increasingly designing culverts using computer applications. Among these applications are the FHWA's HY8 Culvert Analysis (Ginsberg 1987) and numerous proprietary applications. In addition, the District has developed spreadsheets to aid in the sizing and design of culverts. Both the UD-Culvert Spreadsheet application and FHWA's HY8 Culvert Analysis (Version 6.1) are located on the CD-ROM version of this *Manual*.

3.5 Design Considerations

Due to problems arising from topography and other considerations, the actual design of a culvert installation is more difficult than the simple process of sizing culverts. The information in the procedure for design that will be given is a guide to design since the problems encountered are too varied and too numerous to be generalized. However, the actual process presented should be followed to insure that some special problem is not overlooked. Several combinations of entrance types, invert elevations, and pipe diameters should be tried to determine the most economic design that will meet the conditions imposed by topography and engineering.

3.5.1 Design Computation Forms

The use of design computation forms is a convenient method to use to obtain consistent designs and promote cost-effectiveness. An example of such a form is Figure CU-8.

3.5.2 Invert Elevations

After determining the allowable headwater elevation, the tailwater elevation, and the approximate length, invert elevations must be assumed. Scour is not likely in an artificial channel such as a roadside ditch or a major drainage channel when the culvert has the same slope as the channel. To reduce the chance of failure due to scour, invert elevations corresponding to the natural grade should be used as a first trial.

For natural channels, the flow conditions in the channel upstream from the culvert should be investigated to determine if scour will occur.

3.5.3 Culvert Diameter

After the invert elevations have been assumed and using the design computation forms (e.g., Figure CU-8), the capacity charts (e.g., Figure CU-7), and the nomographs, the diameter of pipe that will meet the headwater requirements should be determined. Since small diameter pipes are often plugged by sediment and debris, it is recommended that pipe smaller than 18 inches not be used for any drainage where this *Manual* applies. Since the pipe roughness influences the culvert diameter, both concrete and corrugated metal pipe should be considered in design, if both will satisfy the headwater requirements.

3.5.4 Limited Headwater

If there is insufficient headwater elevation to obtain the required discharge, it is necessary to oversize the culvert barrel, lower the inlet invert, use an irregular cross section, or use any combination of the preceding.

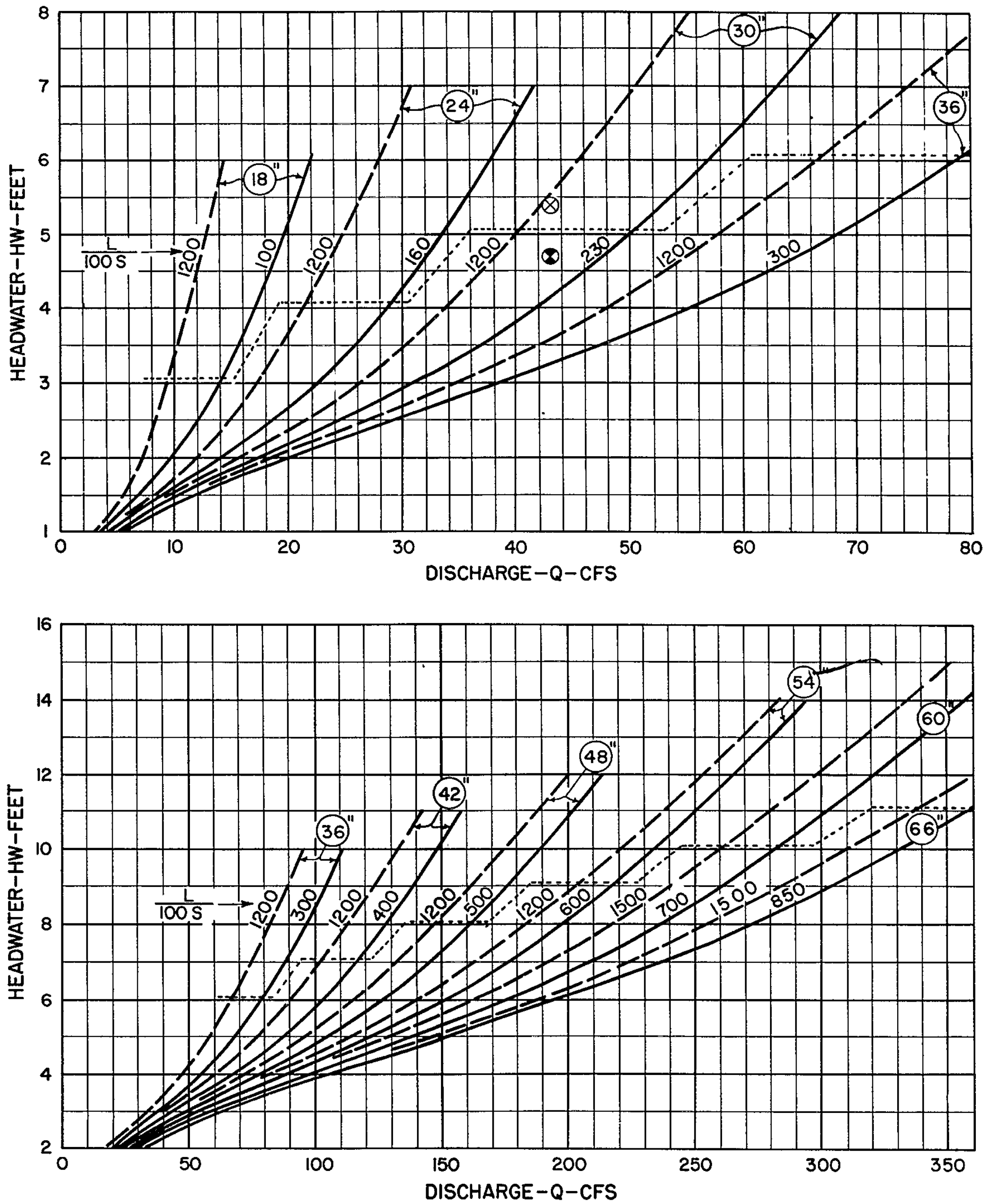
If the inlet invert is lowered, special consideration must be given to scour. The use of gabions or concrete drop structures, riprap, and headwalls with apron and toe walls should be investigated and compared to obtain the proper design.

3.6 Culvert Outlet

The outlet velocity must be checked to determine if significant scour will occur downstream during the major storm. If scour is indicated (and this will normally be the case), refer to Section 7.0 of the MAJOR DRAINAGE chapter ("Protection Downstream of Culverts") and to the HYDRAULIC STRUCTURES chapter for guidance on outfall protection. District maintenance staff have observed that inadequate culvert outlet protection is one of the more common problems within the District. Short-changing outlet protection is no place to economize during design and construction because downstream channel degradation can be significant and the culvert outlet can be undermined.

3.7 Minimum Slope

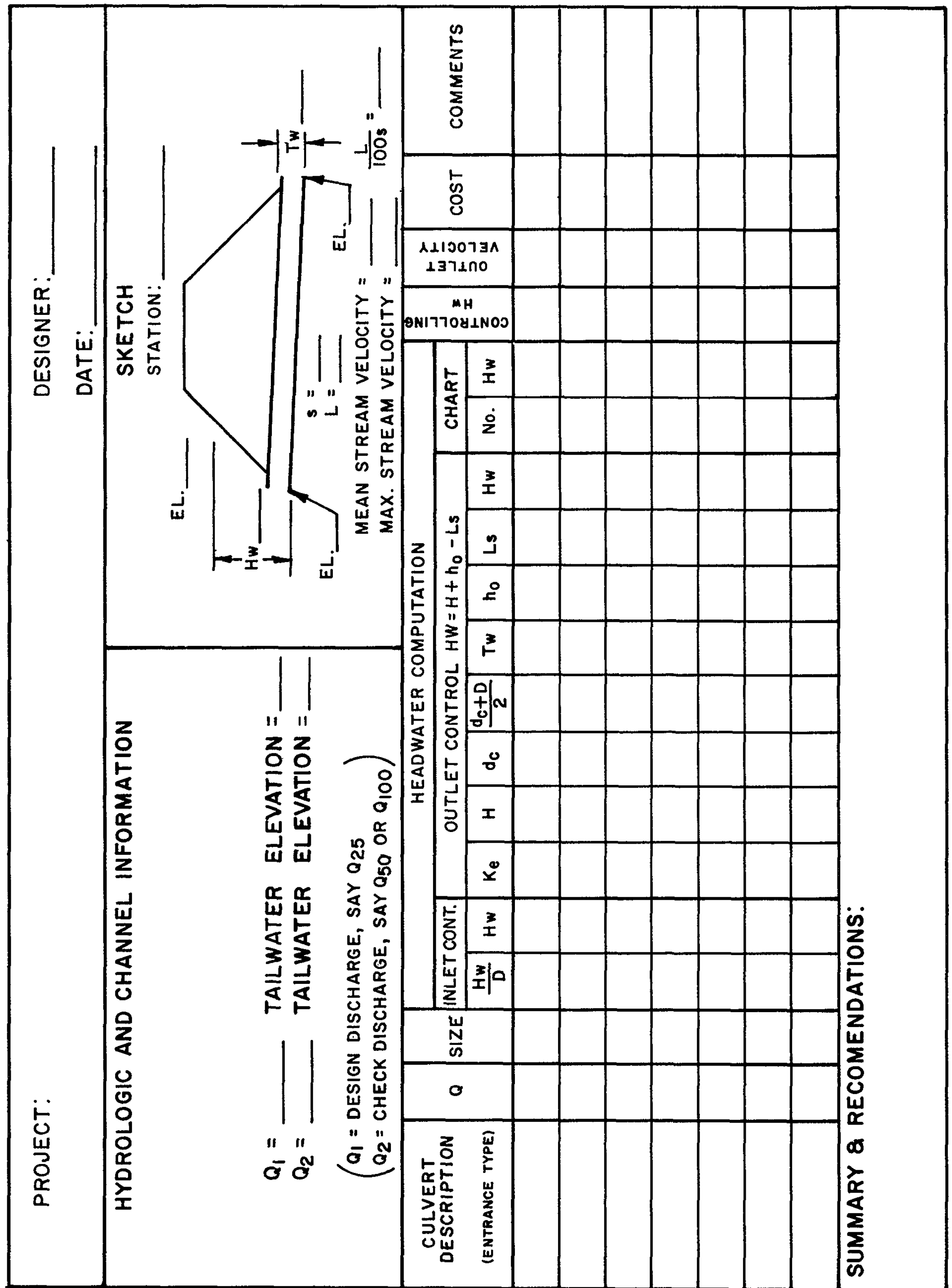
To minimize sediment deposition in the culvert, the culvert slope must be equal to or greater than the slope required to maintain a minimum velocity as described in the MAJOR DRAINAGE and STREETS/INLETS/STORM SEWERS chapters. The slope should be checked for each design, and if the proper minimum velocity is not obtained, the pipe diameter may be decreased, the slope steepened, a smoother pipe used, or a combination of these may be used.



From BPR
EXAMPLE
⊗ GIVEN:
43 CFS ; AHW = 5.4 FT.
L = 120 FT. ; S_0 = 0.002

SELECT 30"
HW = 4.7 FT.

Figure CU-7—Culvert Capacity Chart—Example



From BPR

Figure CU-8—Design Computation for Culverts—Blank Form

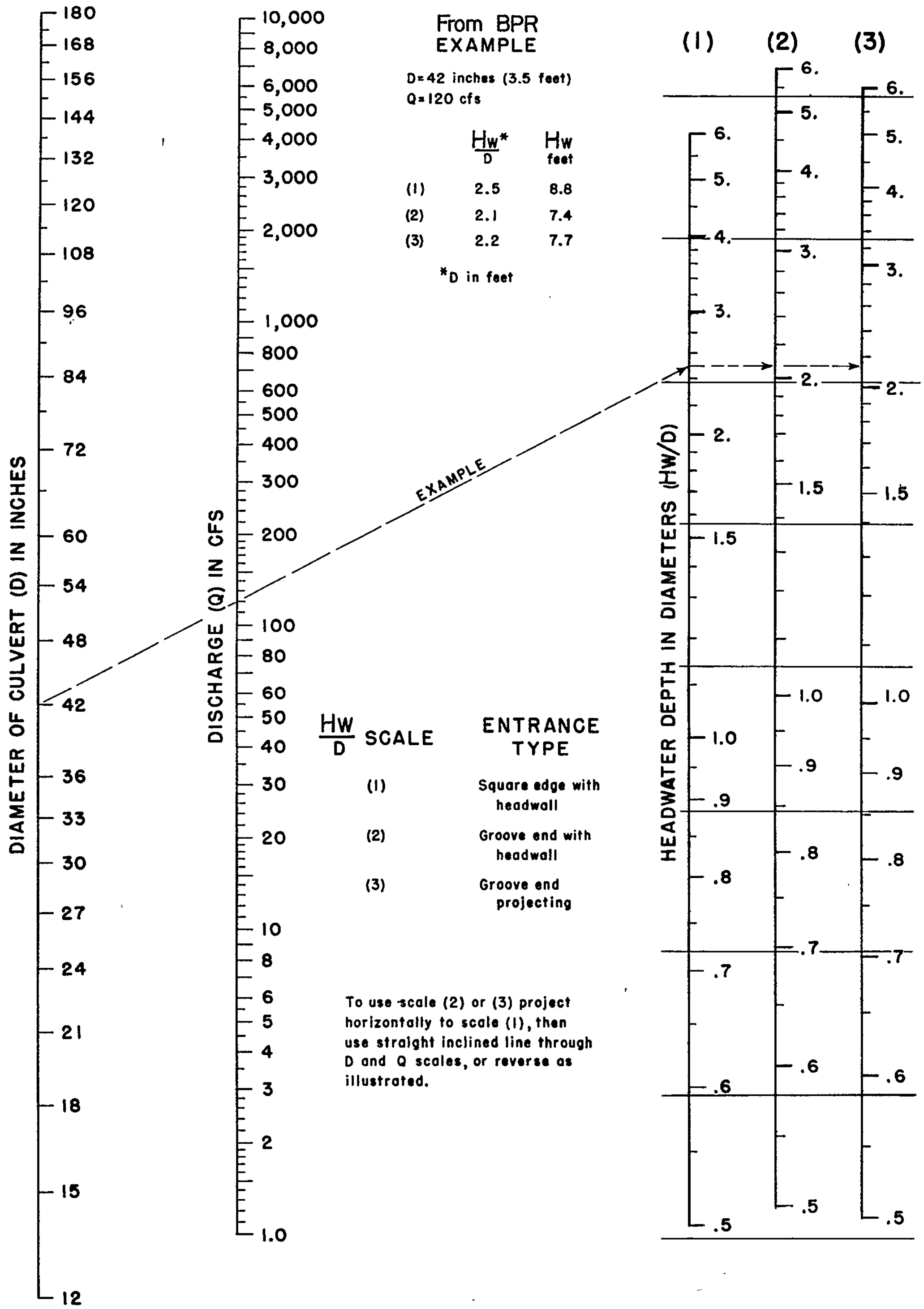


Figure CU-9—Inlet Control Nomograph—Example

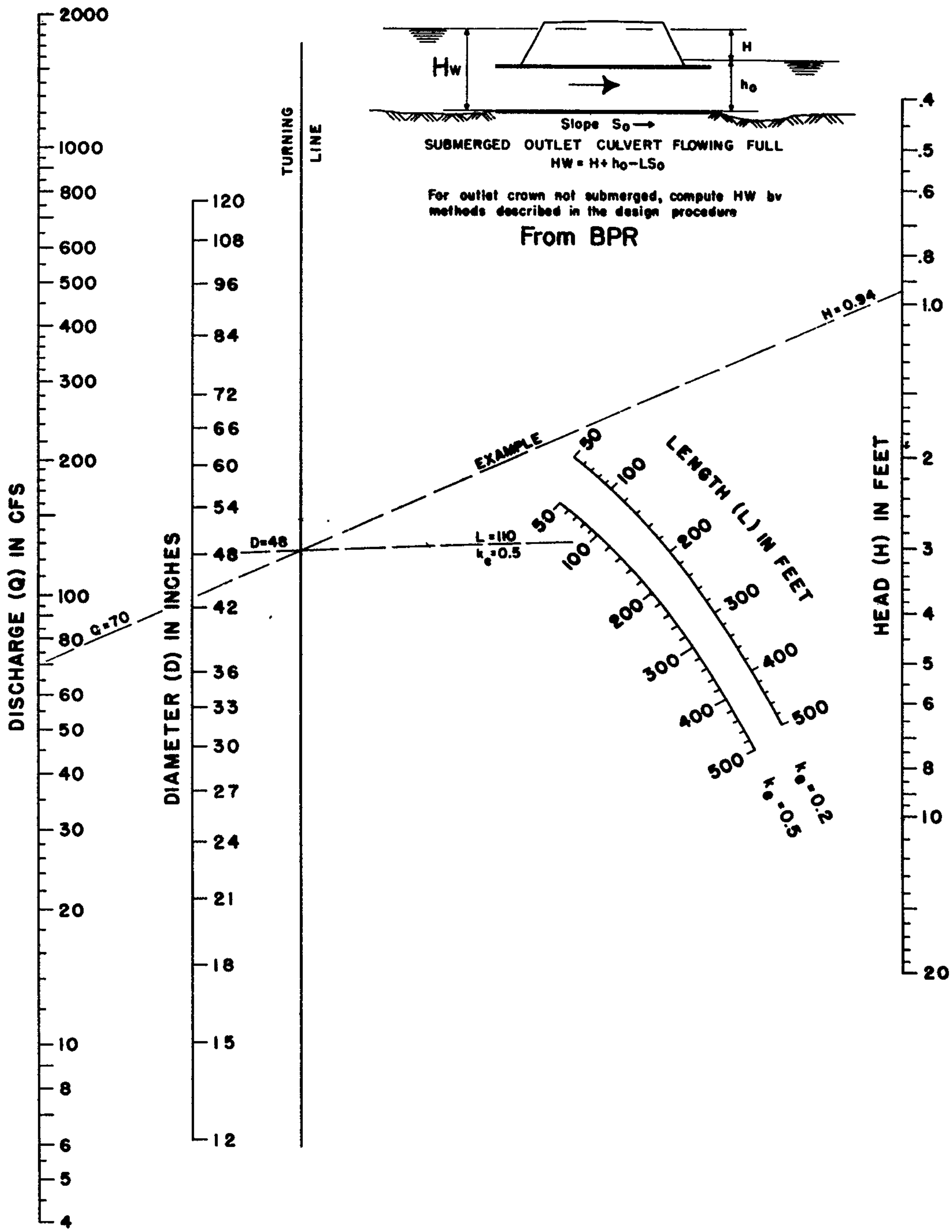


Figure CU-10—Outlet Control Nomograph—Example

4.0 CULVERT INLETS

A fact often overlooked is that a culvert cannot carry any more water than can enter the inlet. Frequently culverts and open channels are carefully designed with full consideration given to slope, cross section, and hydraulic roughness, but without regard to the inlet limitations. Culvert designs using uniform flow equations rarely carry their design capacity due to limitations imposed by the inlet.

The design of a culvert, including the inlet and the outlet, requires a balance between cost, hydraulic efficiency, purpose, and topography at the proposed culvert site. Where there is sufficient allowable headwater depth, a choice of inlets may not be critical, but where headwater depth is limited, where erosion is a problem, or where sedimentation is likely, a more efficient inlet may be required to obtain the necessary discharge for the culvert.

The primary purpose of a culvert is to convey flows. A culvert may also be used to restrict flow, that is, to discharge a controlled amount of water while the area upstream from the culvert is used for detention storage to reduce a storm runoff peak. For this case, an inefficient inlet may be the most desirable choice.

The inlet types described in this chapter may be selected to fulfill either of the above requirements depending on the topography or conditions imposed by the designer. The entrance coefficient, K_e, as defined by Equation CU-5, is a measure of the hydraulic efficiency at the inlet, with lower valves indicating greater efficiency. Inlet coefficients recommended for use are given in Table CU-1.

Table CU-1—Inlet Coefficients For Outlet Control

Type of Entrance	Entrance Coefficient, K_e
1. Pipe entrance with headwall	
Grooved edge	0.20
Rounded edge (0.15D radius)	0.15
Rounded edge (0.25D radius)	0.10
Square edge (cut concrete and CMP)	0.40
2. Pipe entrance with headwall & 45° wingwall	
Grooved edge	0.20
Square edge	0.35
3. Headwall with parallel wingwalls spaced 1.25D apart	
Grooved edge	0.30
Square edge	0.40
4. Special inlets—see Section 4.3	
5. Projecting Entrance	
Grooved edge	0.25
Square edge	0.50
Sharp edge, thin wall	0.90

4.1 Projecting Inlets

Projecting inlets vary greatly in hydraulic efficiency and adaptability to requirements with the type of pipe material used. Figure CU-11 illustrates this type of inlet.

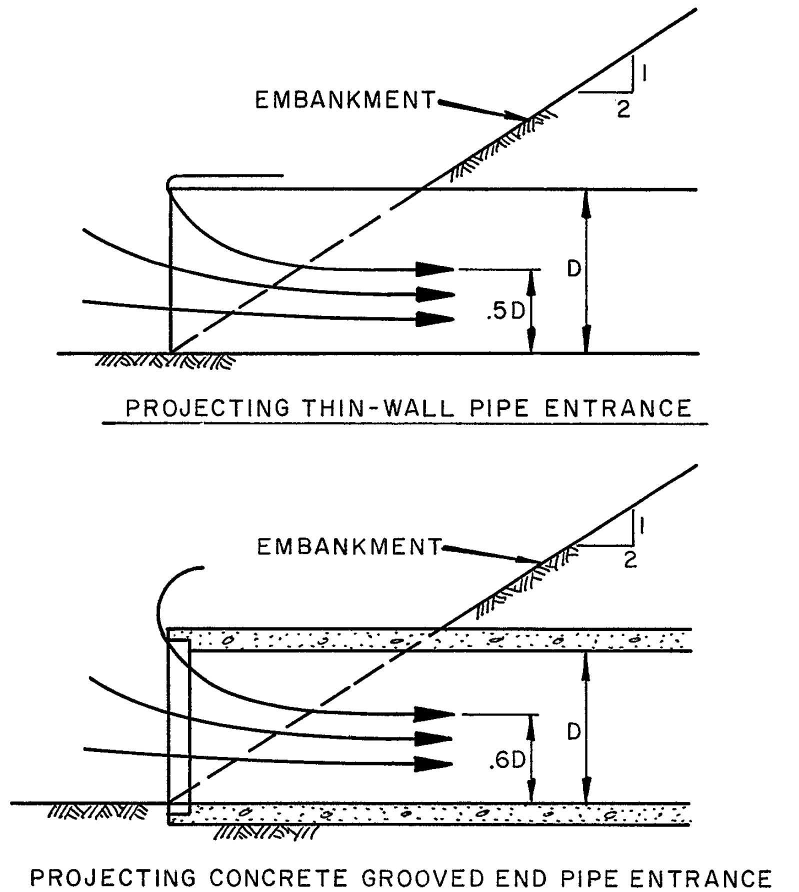


Figure CU-11—Common Projecting Culvert Inlets

The primary advantage of projecting inlets is relatively low cost. Because projecting inlets are susceptible to damage due to maintenance of embankment and roadways and due to accidents, the adaptability of this type of entrance to meet the engineering and topographical demands varies with the type of material used.

Corrugated metal pipe projecting inlets have limitations which include low efficiency, damage which may result from maintenance of the channel and the area adjacent to the inlet, and restrictions on the ability of maintenance crews to work around the inlet. The hydraulic efficiency of concrete-grooved or bell-end pipe is good and, therefore, the only restriction placed on the use of concrete pipe for projecting inlets is the requirement for maintenance of the channel and the embankment surrounding the inlet. Where equipment will be used to maintain the embankment around the inlet, it is not recommended that a projecting inlet of any type be used.

4.1.1 Corrugated Metal Pipe

A projecting entrance of corrugated metal pipe is equivalent to a sharp-edged entrance with a thin wall and has an entrance coefficient of about 0.9.

4.1.2 Concrete Pipe

Bell-and-spigot concrete pipe or tongue-and-groove concrete pipe with the bell end or grooved end used as the inlet section are quite efficient hydraulically, having an entrance coefficient of about 0.25. For concrete pipe that has been cut, the entrance is square edged, and the entrance coefficient is about 0.5.

4.2 Inlets with Headwalls

Headwalls may be used for a variety of reasons, including increasing the efficiency of the inlet, providing embankment stability, and providing embankment protection against erosion. The relative efficiency of the inlet varies with the pipe material used. Figure CU-12 illustrates a headwall with wingwalls.

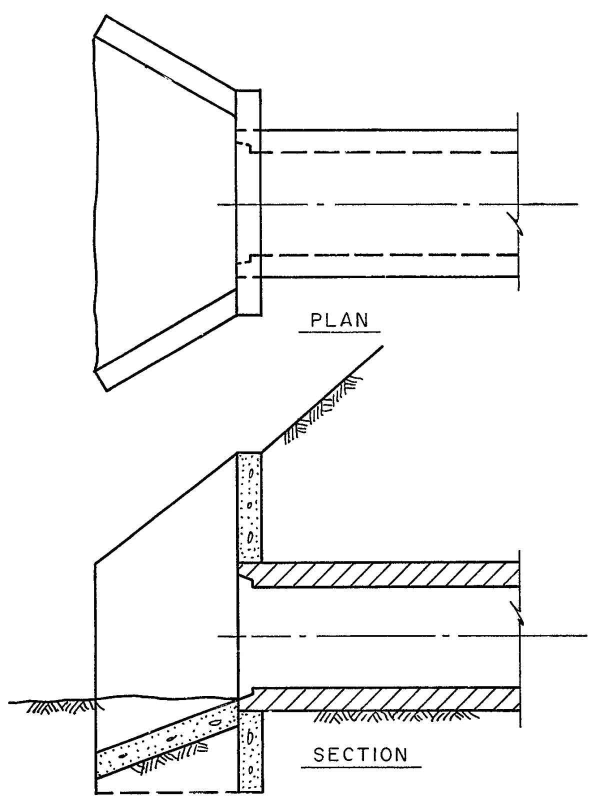


Figure CU-12—Inlet With Headwall and Wingwalls

4.2.1 Corrugated Metal Pipe

Corrugated metal pipe in a headwall is essentially a square-edged entrance with an entrance coefficient of about 0.4. The entrance losses may be reduced by rounding the entrance. The entrance coefficient may be reduced to 0.15 for a rounded edge with a radius equal to 0.15 times the culvert diameter, and to 0.10 for rounded edge with a radius equal to 0.25 times the diameter of the culvert.

4.2.2 Concrete Pipe

For tongue-and-groove or bell-end concrete pipe, little increase in hydraulic efficiency is realized by adding a headwall. The primary reason for using headwalls is for embankment protection and for ease of maintenance. The entrance coefficient is equal to about 0.2 for grooved and bell-end pipe, and equal to 0.4 for cut concrete pipe.

4.2.3 Wingwalls

Wingwalls are used where the side slopes of the channel adjacent to the entrance are unstable and where the culvert is skewed to the normal channel flow. Little increase in hydraulic efficiency is realized with the use of wingwalls, regardless of the pipe material used and, therefore, the use should be justified for reasons other than an increase in hydraulic efficiency. Figure CU-13 illustrates several cases where wingwalls are used. For parallel wingwalls, the minimum distance between wingwalls should be at least 1.25 times the diameter of the culvert pipe.

4.2.4 Aprons

If high headwater depths are to be encountered, or if the approach velocity of the channel will cause scour, a short channel apron should be provided at the toe of the headwall. This apron should extend at least one pipe diameter upstream from the entrance, and the top of the apron should not protrude above the normal streambed elevation.

Culverts with wingwalls should be designed with a concrete apron extending between the walls. Aprons must be re

inforced to control cracking. As illustrated in Figure CU-13, the actual configuration of the wingwalls varies according to the direction of flow and will also vary according to the topographical requirement placed upon them.

For conditions where scour may be a problem due to high approach velocities and special soil conditions, such as alluvial soils, a toe wall is often desirable for apron construction.

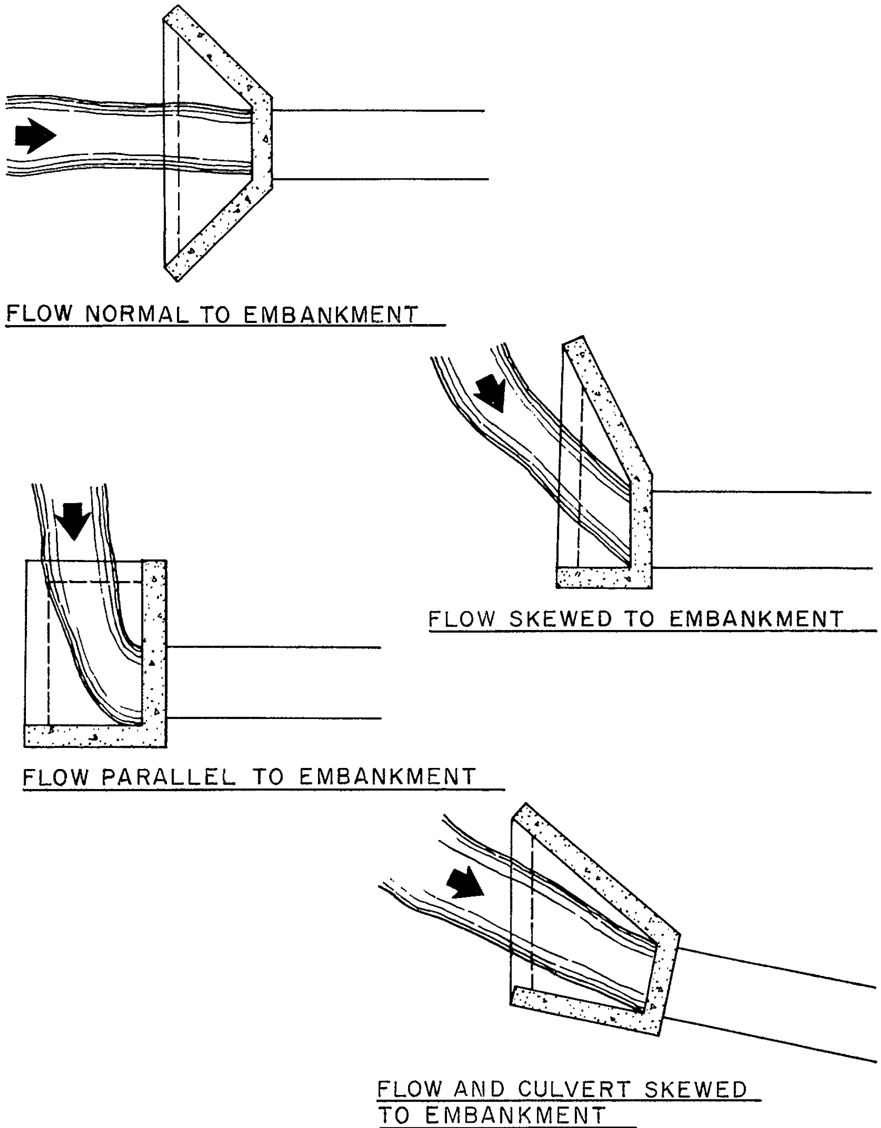


Figure CU-13—Typical Headwall-Wingwall Configurations

4.3 Special Inlets

There is a great variety of inlets other than the common ones described. Among these are special end-sections, which serve as both outlets and inlets and are available for both corrugated metal pipe and concrete pipe. Because of the difference in requirements due to pipe materials, the special end-sections will be discussed independently according to pipe material, and mitered inlets will also be considered.

4.3.1 Corrugated Metal Pipe

Special end-sections for corrugated metal pipe add little to the overall cost of the culvert and have the following advantages:

1. Less maintenance around the inlet.
2. Less damage from maintenance work and from accidents compared to a projecting entrance.
3. Increased hydraulic efficiency. When using design charts, as discussed in Section 3.0, charts for a square-edged opening for corrugated metal pipe with a headwall may be used.

4.3.2 Concrete Pipe

As in the case of corrugated metal pipe, these special end-sections may aid in increasing the embankment stability or in retarding erosion at the inlet. They should be used where maintenance equipment must be used near the inlet or where, for aesthetic reasons, a projecting entrance is considered too unsightly.

The hydraulic efficiency of this type of inlet is dependent on the geometry of the end-section to be used. Where the full contraction to the culvert diameter takes place at the first pipe section, the entrance coefficient, K_e, is equal to 0.5, and where the full contraction to the culvert diameter takes place in the throat of the end-section, the entrance coefficient, K_e, is equal to 0.25.

4.3.3 Mitered Inlets

The use of this entrance type is predominantly with corrugated metal pipe and its hydraulic efficiency is dependent on the construction procedure used. If the embankment is not paved, the entrance, in practice, usually does not conform to the side slopes, giving essentially a projecting entrance with K_e = 0.9. If the embankment is paved, a sloping headwall is obtained with K_e = 0.60 and, by beveling the edges, K_e = 0.50.

Uplift is an important factor for this type entrance. It is not good practice to use unpaved embankment slopes where a mitered entrance may be submerged to an elevation one-half the diameter of the culvert above the top of the pipe.

4.3.4 Long Conduit Inlets

Inlets are important in the design of culverts for road crossings and other short sections of conduit; however, they are even more significant in the economical design of long culverts and pipes. Unused capacity in a long conduit will result in wasted investment. Long conduits are costly and require detailed engineering, planning, and design work. The inlets to such conduits are extremely important to the functioning of the conduit and must receive special attention.

Most long conduits require special inlet considerations to meet the particular hydraulic characteristics of the conduit. Generally, on larger conduits, hydraulic model testing will result in better and less costly inlet construction.

4.4 Improved Inlets

Inlet edge configuration is one of the prime factors influencing the performance of a culvert operating under inlet control. Inlet edges can cause a severe contraction of the flow, as in the case of a thin edge, projecting inlet. In a flow contraction, the effective cross-sectional area of the barrel may be reduced to about one-half of the actual available barrel area. As the inlet configuration is improved, the flow contraction is reduced, thus improving the performance of the culvert.

A tapered inlet is a flared culvert inlet with an enlarged face section and a hydraulically efficient throat section. Tapered inlets improve culvert performance by providing a more efficient control section (the throat). However, tapered inlets are not recommended for use on culverts flowing under outlet control because the simple beveled edge is of equal benefit. The two most common improved inlets are the side-tapered inlet and the slope-tapered inlet (Figure CU-14). FHWA (1985) *Hydraulic Design of Highway Culverts* provides guidance on the design of improved inlets.

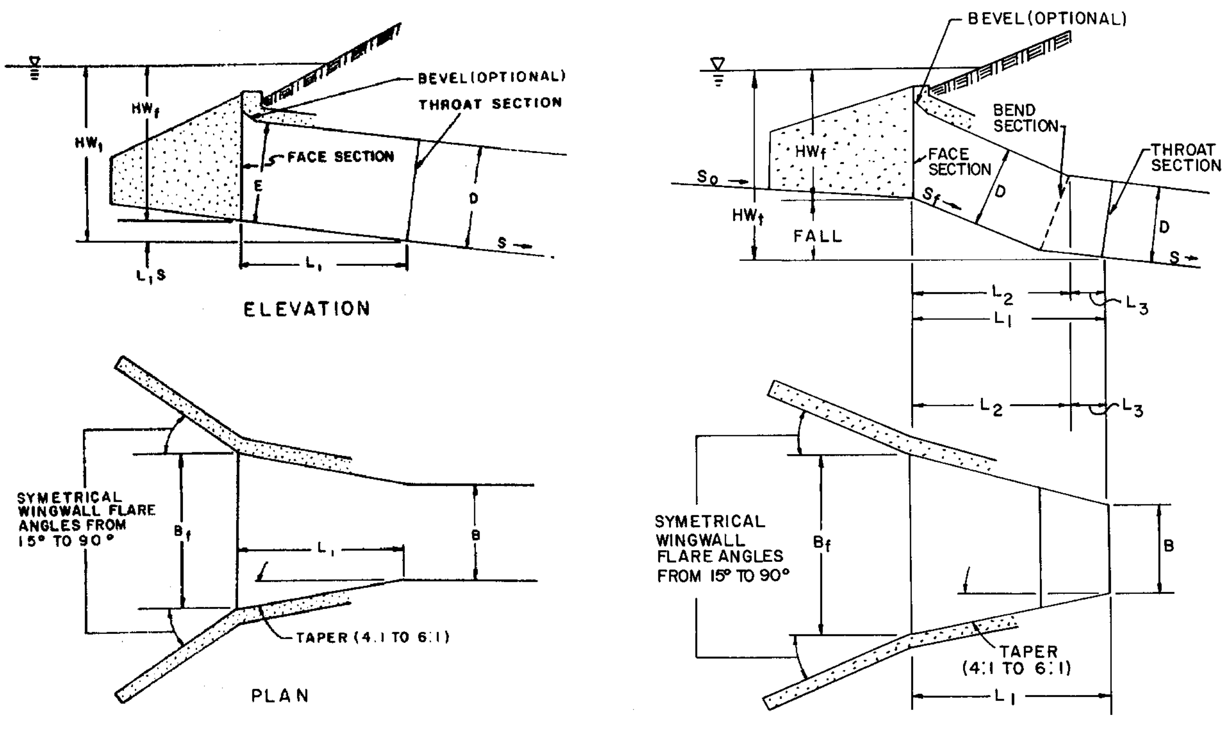


Figure CU-14—Side-Tapered and Slope-Tapered Improved Inlets

5.0 INLET PROTECTION

Inlets on culverts, especially on culverts to be installed in live streams, should be evaluated relative to debris control and buoyancy. These topics are addressed in this section, while broader discussion of the advantages and disadvantages of trash/safety racks is provided in Section 8.0.

5.1 Debris Control

Accumulation of debris at a culvert inlet can result in the culvert not performing as designed. The consequences may be damages from inundation of the road and upstream property. The designer has three options for coping with the debris problem:

1. Retain the debris upstream of the culvert.
2. Attempt to pass the debris through the culvert.
3. Install a bridge.

If the debris is to be retained by an upstream structure or at the culvert inlet, frequent maintenance may be required. The design of a debris control structure should include a thorough study of the debris problem.

The following are among the factors to be considered in a debris study:

- Type of debris
- Quantity of debris
- Expected changes in type and quantity of debris due to future land use
- Stream flow velocity in the vicinity of culvert entrance
- Maintenance access requirements
- Availability of storage
- Maintenance plan for debris removal
- Assessment of damage due to debris clogging, if protection is not provided

Hydraulic Engineering Circular No. 9, *Debris Control Structure*s (FHWA 1971), should be used when designing debris control structures.

5.2 Buoyancy

The forces acting on a culvert inlet during flows are variable and indeterminate. When a culvert is functioning with inlet control, an air pocket begins just inside the inlet that creates a buoyant effect when the inlet is submerged. The buoyancy forces increase with an increase in headwater depth under inlet control conditions. These forces, along with vortexes and eddy currents, can cause scour, undermine culvert inlets, and erode embankment slopes, thereby making the inlet vulnerable to failure, especially with deep headwater.

In general, installing a culvert in a natural stream channel constricts the normal flow. The constriction is accentuated when the capacity of the culvert is impaired by debris or damage.

The large unequal pressures resulting from inlet constriction are in effect buoyant forces that can cause entrance failures, particularly on corrugated metal pipe with mitered, skewed, or projecting ends. The failure potential will increase with steepness of the culvert slope, depth of the potential headwater, flatness of the fill slope over the upstream end of the culvert, and the depth of the fill over the pipe.

Anchorage at the culvert entrance helps to protect against these failures by increasing the deadload on the end of the culvert, protecting against bending damage, and by protecting the fill slope from the scouring action of the flow. Providing a standard concrete headwall or endwall helps to counteract the hydrostatic uplift and to prevent failure due to buoyancy.

Because of a combination of high head on the outside of the inlet and the large region of low pressure on the inside of the inlet due to separation, a large bending moment is exerted on the end of the culvert, which may result in failure. This problem has been noted in the case of culverts under high fills, on steep slopes, and with projecting inlets. Where upstream detention storage requires headwater depth in excess of 20 feet, reducing the culvert size is recommended rather than using the inefficient projecting inlet to reduce discharge.

6.0 OUTLET PROTECTION

Scour at culvert outlets is a common occurrence. It must be accounted for, as discussed below and in the HYDRAULIC STRUCTURES and MAJOR DRAINAGE chapters. The natural channel flow is usually confined to a lesser width and greater depth as it passes through a culvert barrel. An increased velocity results with potentially erosive capabilities as it exits the barrel. Turbulence and erosive eddies form as the flow expands to conform to the natural channel. However, the velocity and depth of flow at the culvert outlet and the velocity distribution upon reentering the natural channel are not the only factors that need consideration.

The characteristics of the channel bed and bank material, velocity, and depth of flow in the channel at the culvert outlet, and the amount of sediment and other debris in the flow are all contributing factors to scour potential. Due to the variation in expected flows and the difficulty in evaluating some of these factors, scour prediction is not a very exact science.

Scour in the vicinity of a culvert outlet can be classified into two separate types called local scour and general stream degradation.

6.1 Local Scour

Local scour is typified by a scour hole produced at the culvert outlet. This is the result of high exit velocities, and the effects extend only a limited distance downstream.

Coarse material scoured from the circular or elongated hole is deposited immediately downstream, often forming a low bar. Finer material is transported further downstream. The dimensions of the scour hole change due to sedimentation during low flows and the varying erosive effects of storm events. The scour hole is generally deepest during passage of the peak flow. Methods for predicting scour hole dimensions are found in *Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA 1983 and 2000).

6.2 General Stream Degradation

General stream degradation is a phenomenon that is independent of culvert performance. Natural causes produce a lowering of the streambed over time. The identification of a degrading stream is an essential part of the original site investigation. This subject is discussed in the MAJOR DRAINAGE chapter.

Both local and general scour can occur simultaneously at a culvert outlet. Protection against scour at culvert outlets varies from limited riprap placement to complex and expensive energy dissipation devices (Photograph CU-5). At some locations, use of a rougher culvert material may alleviate the need for a special outlet protection device. Pre-formed scour holes (approximating the configuration of naturally formed holes) dissipate energy while providing a protective lining to the streambed.

Photograph CU-5—Energy dissipation and outlet protection are essential to promote channel stability.

As discussed in the HYDRAULIC STRUCTURES chapter and in Section 7.0 of the MAJOR DRAINAGE chapter, riprapped channel expansions and concrete aprons protect the channel and redistribute or spread the flow. Barrel outlet expansions operate in a similar manner. Headwalls and cutoff walls protect the integrity of the fill. When outlet velocities are high enough to create excessive downstream problems, consideration should be given to more complex energy dissipation devices. These include hydraulic jump basins, impact basins, drop structures, and stilling wells. Design information for the general types of energy dissipators is provided in the MAJOR DRAINAGE and HYDRAULIC STRUCTURES chapters of this *Manual* and in *Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA 1983 and 2000).

7.0 GENERAL CONSIDERATIONS

7.1 Culvert Location

Culvert location is an integral part of the total design. The main purpose of a culvert is to convey drainage water across the roadway section expeditiously and effectively. The designer should identify all live stream crossings, springs, low areas, gullies, and impoundment areas created by the new roadway embankment for possible culvert locations. Note that environmental permitting constraints will often apply for new culverts or retrofits, such as a Section 404 permit that regulates construction activities in jurisdictional wetlands and "Waters of the United States."

Culverts should be located on existing stream alignments and aligned to give the stream a direct entrance and a direct exit. Abrupt changes in direction at either end may retard the flow and make a larger structure necessary. If necessary, a direct inlet and outlet may be obtained by means of a channel change, skewing the culvert, or a combination of these. The choice of alignment should be based on economics, environmental concerns, hydraulic performance, and/or maintenance considerations.

If possible, a culvert should have the same alignment as its channel. Often this is not practical and where the water must be turned into the culvert, headwalls, wingwalls, and aprons with configurations similar to those in Figure CU-13 should be used as protection against scour and to provide an efficient inlet.

7.2 Sedimentation

Deposits usually occur within the culvert barrels at flow rates smaller than the design flow. The deposits may be removed during larger floods dependent upon the relative transport capacity of flow in the stream and in the culvert, compaction and composition of the deposits, flow duration, ponding depth above the culvert, and other factors.

Culvert location in both plan and profile is of particular importance to the maintenance of sediment-free culvert barrels. Deposits occur in culverts because the sediment transport capacity of flow within the culvert is often less than in the stream.

Deposits in culverts may also occur due to the following conditions:

- At moderate flow rates the culvert cross section is larger than that of the stream, so the flow depth and sediment transport capacity is reduced.
- Point bars form on the inside of stream bends. Culvert inlets placed at bends in the stream will be subject to deposition in the same manner. This effect is most pronounced in multiple-barrel culverts with the barrel on the inside of the curve often becoming almost totally plugged with sediment deposits.

- Abrupt changes to a flatter grade in the culvert or in the channel adjacent to the culvert will induce sedimentation. Gravel and cobble deposits are common downstream from the break in grade because of the reduced transport capacity in the flatter section.

7.3 Fish Passage

At some culvert locations, the ability of the structure to accommodate migrating fish is an important design consideration. For such sites, state fish and wildlife agencies (such as the Colorado Division of Wildlife) should be consulted early in the roadway planning process. Some situations may require the construction of a bridge to span the natural stream. However, culvert modifications can often be constructed to meet the design criteria established by the fish and wildlife agencies.

7.4 Open Channel Inlets

Entrances to open channels often require the same careful planning and design as is needed for culverts and long conduits if the necessary hydraulic balance is to be achieved. The energy grade line should be analyzed by the designer to insure proper provision for balanced energy conversion, velocity control, energy loss, and other factors controlling the downstream flow. Channel confluences, in particular, require careful hydraulic design to eliminate scour, reduce oscillating waves, and minimize upstream backwater effects.

7.5 Transitions

Transitions from pipe flow to open channels, between different rigid channels, and from slow flow to supercritical flow must be designed using the concepts of conservation of energy and open channel hydraulics. Primarily, a transition is necessary to change the shape or cross section of flowing water.

Normally, the designer will have as an objective the avoidance of excessive energy losses, cross waves, and turbulence. It is also necessary to provide against scour and overtopping.

Supercritical flow transitions must receive more attention than is usually due subcritical flow transitions. Care must be taken to insure against unwanted hydraulic jumps or velocities causing critical depth. Froude numbers between 0.9 and 1.1 should be avoided.

In general, the rate at which the flow prism may be changed should not exceed perhaps 5 to 12½ degrees, depending upon velocity. Sharp angles should be avoided. The water surface hydraulic grade line should normally be smooth. Transition structure drawings are provided in the HYDRAULIC STRUCTURES chapter.

7.6 Large Stormwater Inlets

The functioning of large stormwater inlets, which collect major storm surface runoff at points of concentration, is dependent upon careful planning and design. Due regard must be given to debris, hail, and safety hazards.

7.6.1 Gratings

The design of gratings should focus on (1) public safety, (2) hydraulic function and (3) debris control. See Section 8.0 of this chapter for further discussion.

7.6.2 Openings

The hydraulic openings for large storm inlets need to be designed in general accordance with the guides given in Section 4.0. Inadequate openings are easily plugged when needed most, and give a false sense of security. Clear vertical openings should be at least 6 inches. For greater inlet flows, the length of opening needed may be so large that special design approaches are needed for shape and function.

7.6.3 Headwater

The required headwater over large inlets must be computed and kept within acceptable limits to avoid excessive ponding on streets and damage to adjacent property.

7.7 Culvert Replacements

When installing or replacing an existing culvert, careful consideration should be taken to ensure that upstream and downstream property owners are not adversely affected by the new hydraulic conditions. The potential upstream flooding impacts associated with the backwater from the calculated headwater depth must be considered and the determination of the available headwater should take into account the area inundated at the projected water surface elevation. If a culvert is replaced by one with more capacity, the downstream effects of the additional flow must be factored into the analysis. Assuring consistency with existing major drainageway master plans and/or outfall studies is important.

7.8 Fencing for Public Safety

Culverts are frequently located at the base of steep slopes. Large box culverts, in particular, can create conditions where there is a significant drop, which poses risk to the public. In such cases, fencing (guardrails) is recommended for public safety.

8.0 TRASH/SAFETY RACKS

The use of trash/safety racks at inlets to culverts and long underground pipes should be considered on a case-by-case basis. While there is a sound argument for the use of racks for safety reasons, field experience has clearly shown that when the culvert is needed the most, that is, during the heavy runoff, trash racks often become clogged and the culvert is rendered ineffective. A general rule of thumb is that a trash/safety rack will not be needed if one can clearly “see daylight” from one side of the culvert to the other, if the culvert is of sufficient size to pass a 48" diameter object and if the outlet is not likely to trap or injure a person. By contrast, at entrances to longer culverts and long underground pipes and for culverts not meeting the above-stated tests, a trash/safety rack is necessary.

The trash/safety rack design process is a matter of fully considering the safety hazard aspects of the problem, defining them clearly, and then taking reasonable steps to minimize these hazards while protecting the integrity of the water carrying capability of the culvert (see Photograph CU-6 for an example of how not to do it).

Photograph CU-6—Small trash racks at culvert entrance will increase the risk of entrance plugging.

In reviewing potential hazards to the public of a possibility of a person being swept into the culvert, it is also necessary to consider depth and velocity of upstream flow, the local currents in the vicinity of the culvert entrance, the general character of the neighborhood and whether it has residential population nearby, the length and size of the culvert, and other factors affecting safety and culvert capacity. Furthermore, in the event that someone was carried to the culvert with the storm runoff, the exposure hazard may in some cases be even greater if the person is pinned to the grating by the hydraulic pressures of the water rather than being carried through the culvert. Large, oversized racks positioned well in front of the culvert entrance can reduce the risk of pinning.

Where debris potential and/or public safety indicate that a rack is required, if the pipe diameter is more than 24 inches, the rack's open surface area must be, at the absolute minimum, at least four times larger. For smaller pipes, the factor increases significantly as suggested by Figure 7 in the "Typical Structural BMP Details" chapter of Volume 3 of this *Manual*. For culverts larger than 24 inches (i.e., in the smallest dimension), in addition to the trash rack having an open area larger than four times the culvert entrance, the average velocities at the rack's face shall be less than 2.0 feet per second at every stage of flow entering the culvert. The rack needs to be sloped no steeper than 3H:IV (the flatter the better) and have a clear opening at the bottom of 9 to 12 inches to permit debris at lower flows to go through. The bars on the face of the rack should be generally paralleling the flow and be spaced to provide 4½- to 5-inch clear openings between them. Transverse support bars need to be as few as possible, but sufficient to keep the rack from collapsing under full hydrostatic loads.

The District strongly recommends against the installation of trash racks at culvert outlets, because debris or a person carried into the culvert will impinge against the rack, thus leading to pressurized conditions within the culvert, virtually destroying its flow capacity and creating a greater hazard to the public or a person trapped in the culvert than not having one.

8.1 Collapsible Gratings

The District does not recommend the use of collapsible gratings. On larger culverts where a collapsible grating is deemed necessary by a local jurisdiction or an engineer, such gratings must be carefully designed from the structural standpoint so that collapse is achieved with a hydrostatic load of perhaps one-half of the maximum backwater head allowable. Collapse of the trash rack should be such that it clears the waterway opening adequately to permit the inlet to function properly without itself contributing to potential plugging of the culvert.

8.2 Upstream Trash Collectors

Where a safety hazard exists, a large trash rack situated diagonally across a stream a reasonable distance upstream from the culvert inlet offers an alternative. This type of rack may consist of a series of vertical pipes or posts embedded in the approach channel bottom with horizontal bars to deflect the debris to one side. If partial blocking of a properly designed rack occurs, it should be designed so that the backwater flow over the top of the rack is minimal. The rack must not cause the water to rise higher than the maximum allowable flood elevation. A trash rack at the culvert entrance can then provide a backup for safety.

9.0 DESIGN EXAMPLE

The following example problem illustrates the culvert design procedures using the FHWA nomographs and using UD-Culvert Spreadsheet application.

9.1 Culvert Under an Embankment

Given: $Q_{5\text{-yr}}$ = 20 cfs, $Q_{100\text{-yr}}$ = 35 cfs, L = 95 feet

The maximum allowable headwater elevation is 5288.5. The natural channel invert elevations are 5283.5 at the inlet and 5281.5 at the outlet. The tailwater depth is computed as 2.5 feet for the 5-year storm, and 3.0 feet for the 100-year storm.

Solution:

Step 1. Fill in basic data (Figure CU-15)

$Q_{5\text{-yr}}$ = discharge for 5-year storm

$Q_{100\text{-yr}}$ = discharge for 100-year storm

Headwater and tailwater elevations

Step 2. Set invert elevations at natural channel invert elevations to avoid scour. Compute s and $L/(100s)$.

Step 3. Start with an assumed culvert size for the 5-year storm by adopting a velocity of 6.5 ft/sec. In this case, first size is estimated by adopting a velocity of 6.5 ft/sec and computing A = 20/6.5 = 3.1 ft², giving a culvert diameter, D = 24 inches.

Step 4. For this example, two inlets are considered: square edge with headwall (K_e = 0.4) and groove end with headwall (K_e = 0.2). Also, assume concrete pipe will be used with a Manning's n of 0.012 (Note: the District recommends a minimum n of 0.013; however, 0.012 is used in this example to correspond to the FHWA nomograph.)

Step 5. Using the inlet control nomograph (Figure CU-16), the ratio of the headwater depth to the culvert diameter (Hw/D) is 1.47 for the square edge and 1.32 for the groove end. Thus, the inlet control headwater depths are 2.94 feet and 2.64 feet, respectively.

Step 6. The outlet control headwater depth is determined using the method described in Section 3.0. The head is determined from the nomograph (Figure CU-17). The resulting outlet control headwater depths are 2.13 feet for the square edge and 1.90 feet for the grove end inlet.

Step 7. Comparing the headwater depths for inlet control (2.94 feet and 2.64 feet) and outlet control (2.13 feet and 1.90 feet) shows that the culvert is inlet controlled with either inlet configuration. Furthermore, the calculated headwater depths are less than the allowable headwater depth. These results can also be determined using the UD-Culvert Spreadsheet.

Step 8. The next step is to evaluate the culvert for the 100-year flow of 35 cfs and tailwater depth of 3.0 feet. Using the same procedure, the culvert continues to be inlet controlled with the square-edge inlet and switches to outlet control with the more efficient groove-end inlet. However, both of the calculated headwater depths exceed the allowable headwater depth and, consequently, are not viable alternatives.

Step 9. Increase the pipe diameter to 27 inches and repeat the process. The resulting headwater depths are less than the allowable.

Step 10. Compute outlet velocities for each acceptable alternate.

Step 11. Compute cost for each alternate.

Step 12. Make recommendations.

PROJECT: Drainage Criteria Manual Culvert Design Example

DESIGNER: ____________

DATE: February 2001

HYDROLOGIC AND CHANNEL INFORMATION

Q_1 = 20 cfs TAILWATER ELEVATION = 84.0

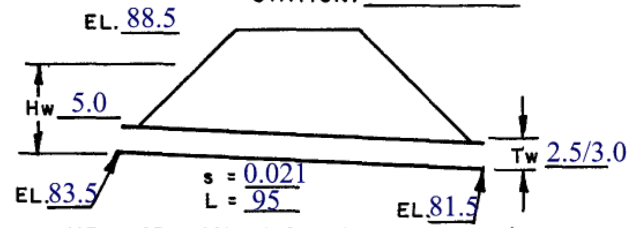
Q_2 = 35 cfs TAILWATER ELEVATION = 84.5

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}

Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: ____________



MEAN STREAM VELOCITY = ______

MAX. STREAM VELOCITY = ______

$\frac{L}{100s}$ = 45

CULVERT DESCRIPTION w/ Headwall (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION: INLET CONT. $\frac{Hw}{D}$	INLET CONT. Hw	OUTLET CONTROL $HW = H + h_o - L_s$: K_e	H	d_c	$\frac{d_c+D}{2}$	Tw	h_o	L_s	Hw	CHART No.	CHART Hw	CONTROLLING Hw	OUTLET VELOCITY	COST	COMMENTS
RCP w/ sq edge	20	24	1.47	2.94	0.4	1.63	0.80	1.40	2.5	2.5	2.0	2.13			2.94	6.4		Inlet Control
RCP w/ groove	20	24	1.32	2.64	0.2	1.40	0.80	1.40	2.5	2.5	2.0	1.90			2.64	6.4		Inlet Control
RCP w/ sq edge	35	24	3.08	6.16	0.4	4.90	0.96	1.48	3.0	3.0	2.0	5.90			6.16	11.1		Inlet Control HW>allowable
RCP w/ groove	35	24	2.50	5.00	0.2	4.40	0.96	1.48	3.0	3.0	2.0	5.40			5.40	11.1		Outlet Control HW>allowable
RCP w/ sq edge	35	27	2.03	4.57	0.4	2.85	1.01	1.63	3.0	3.0	2.0	3.85			4.57	8.8		Inlet Control
RCP w/ groove	35	27	1.71	3.85	0.2	2.50	1.01	1.63	3.0	3.0	2.0	3.50			3.85	8.8		Inlet Control

SUMMARY & RECOMENDATIONS: Choose 27-inch RCP to pass the 100-year flow within the allowable headwater. Either entrance type will work; allow cost to determine. Design outlet protection based on velocity of 8.8 ft/sec.

From BPR

Figure CU-15—Design Computation Form for Culverts—Example 9.1

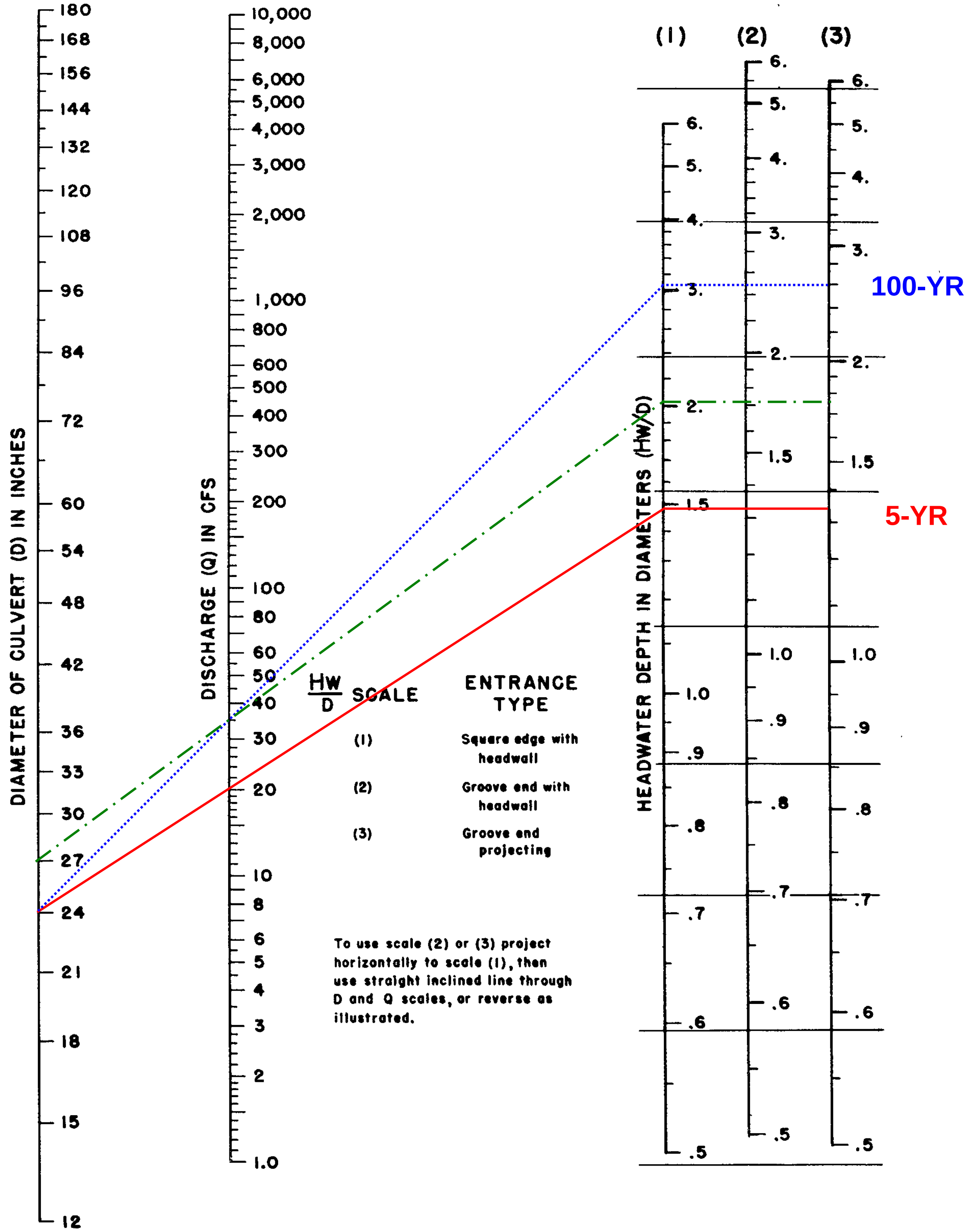


Figure CU-16—Headwater Depth for Concrete Pipe Culverts with Inlet Control—Example 9.1

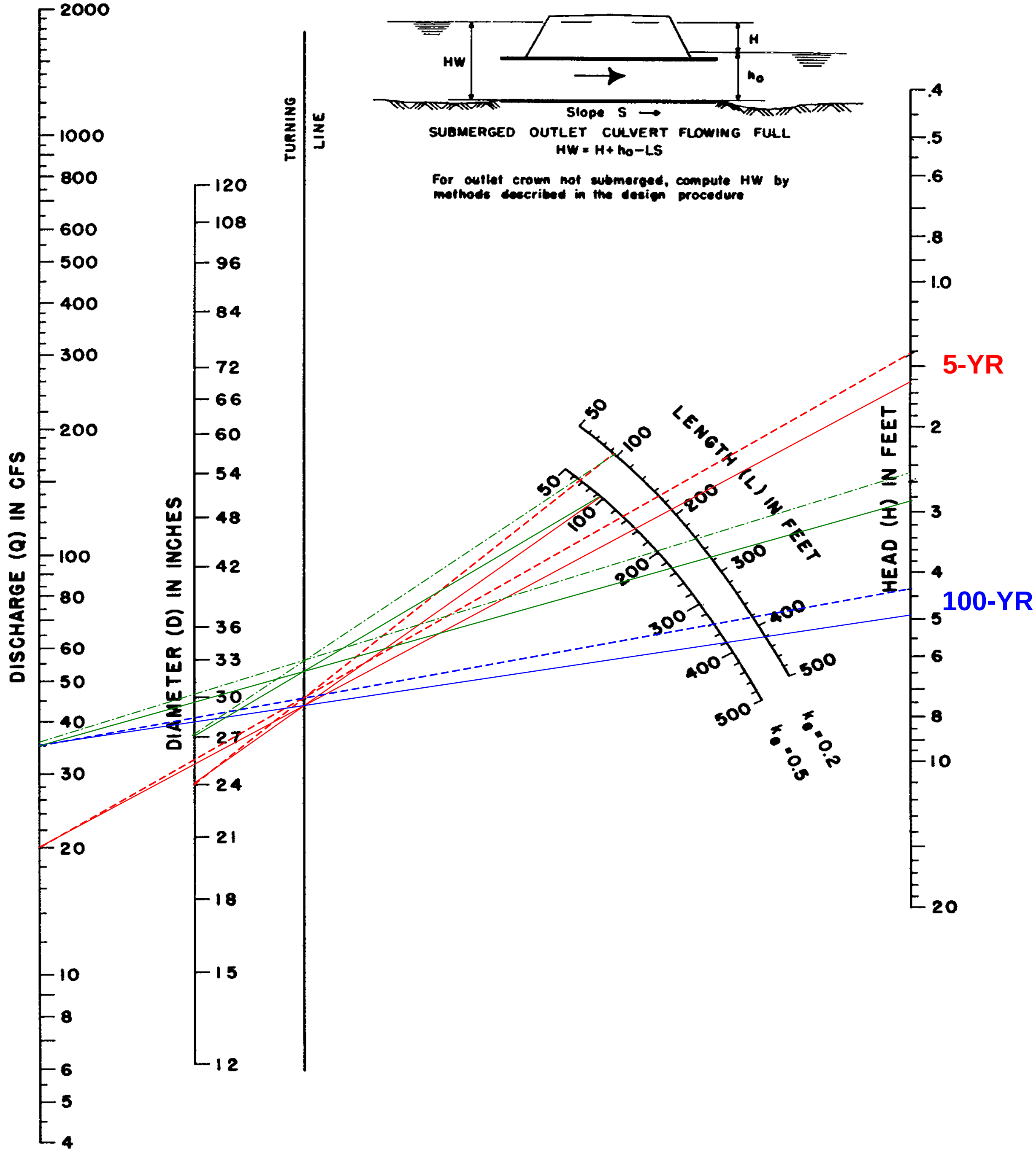
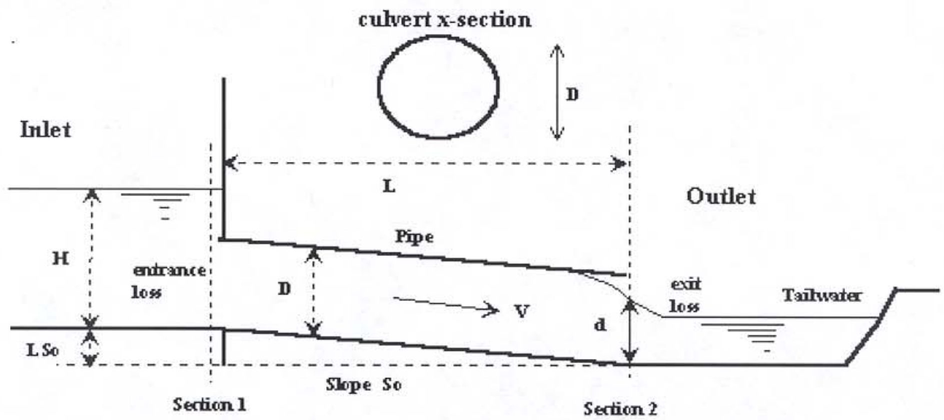


Figure CU-17—Head for Concrete Pipe Culverts Flowing Full (n = 0.012)—Example 9.1

Headwater Depth For Circular Culvert

Project: Drainage Criteria Manual
Pipe ID: Example 9.1



Design Information (input)

Design Discharge	**Q =**	**20.0**	**cfs**
Pipe Diameter	D =	2.00	ft
Inlet Edge Type (choose from pull-down list)	Inlet Type =	Square End with Headwall	
Inlet Invert Elevation	I_e =	83.50	ft
Outlet Invert Elevation	O_e =	81.50	ft
Pipe Length	L =	95.0	ft
Manning's Roughness N-Value	N =	0.012	*
Bend Loss Coefficient	K_b =	0.00	
Exit Loss Coefficient	K_x =	0.00	
Tailwater Water Surface Elevation	El. Y_t =	84.00	ft

Calculations (output)

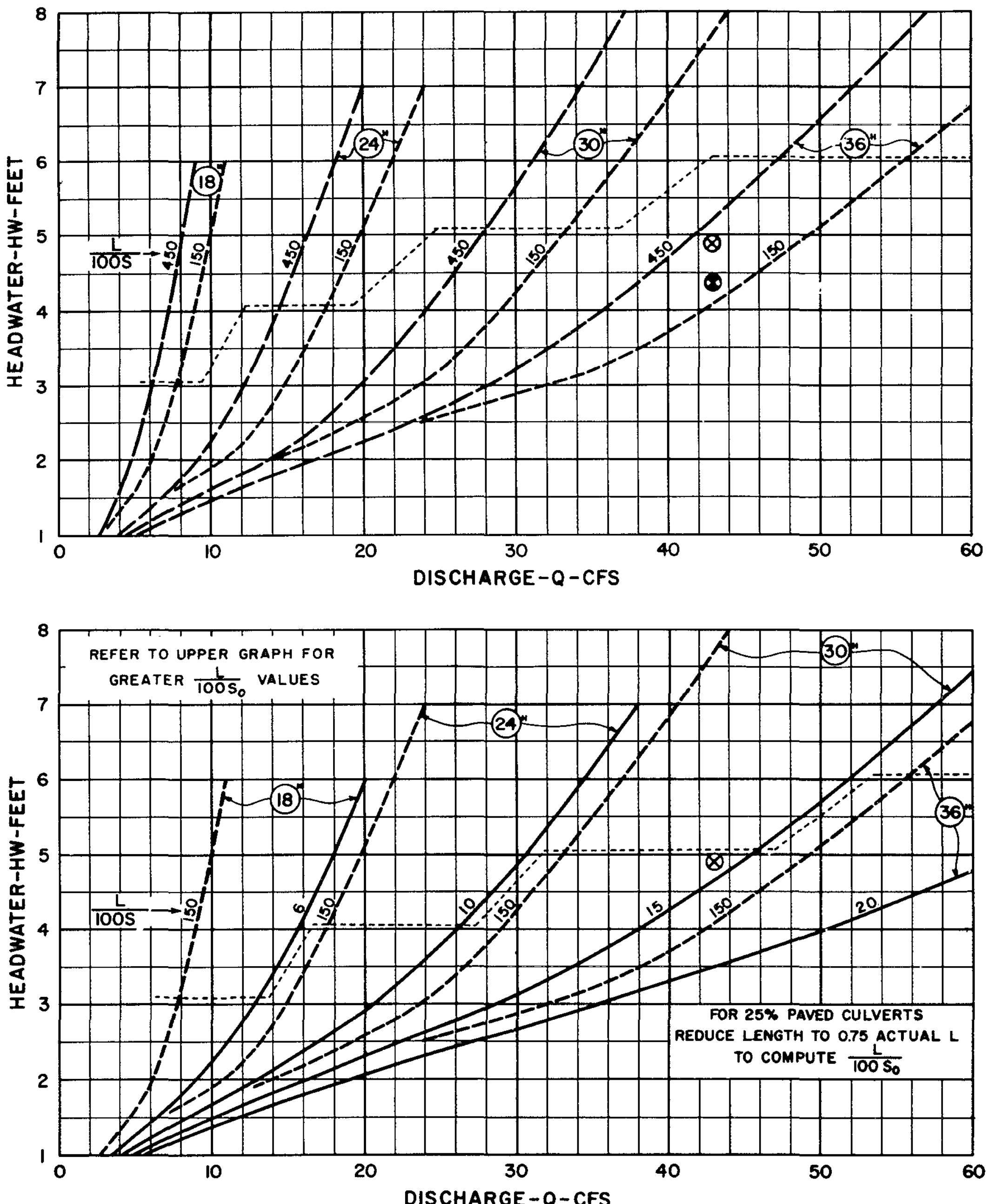
Pipe Cross Sectional Area	A_o =	3.14	sq ft
Culvert Slope	S_o =	0.0211	ft/ft
Normal Flow Depth	Y_n =	0.54	ft
Critical Flow Depth	Y_c =	0.80	ft
Headwater Depth by Inlet Control			
Headwater Depth by Inlet Control	**HW-inlet=**	**2.92**	**ft**
Headwater Depth by Outlet Control			
Tailwater Depth for Design	d =	2.50	ft
Friction Loss Coefficient over Culvert Length	K_f =	1.00	
Sum of All Loss Coefficients	K_s =	1.50	
Headwater Depth by Outlet Control	**HW-outlet=**	**2.07**	**ft**
Design Headwater Depth	**HW=**	**2.92**	**ft**
HW/D Ratio =	**HW/D=**	**1.46**	

10.0 CHECKLIST

Criterion/Requirement	✓
Culvert diameter should be at least 18 inches.	
Evaluate the effects of the proposed culvert on upstream and downstream water surface elevations.	
When retrofitting or replacing a culvert, evaluate the changes in the upstream and downstream flood hazard.	
Review any proposed changes with local, state, and federal regulators.	
When a culvert is sized such that the overlying roadway overtops during large storms, check the depth of cross flow with Table DP-3 in the POLICY chapter.	
Provide adequate outlet protection in accordance with the energy dissipator discussion in the MAJOR DRAINAGE and HYDRAULIC STRUCTURES chapters.	

11.0 CAPACITY CHARTS AND NOMOGRAPHS

Capacity charts and nomographs covering the range of applications commonly encountered in urban drainage are contained in this section. These charts are from the FHWA Hydraulic Design Series No. 5 (FHWA 1985), which also contains detailed instructions for their use. For situations beyond the range covered by these charts, reference should be made to the original publications.



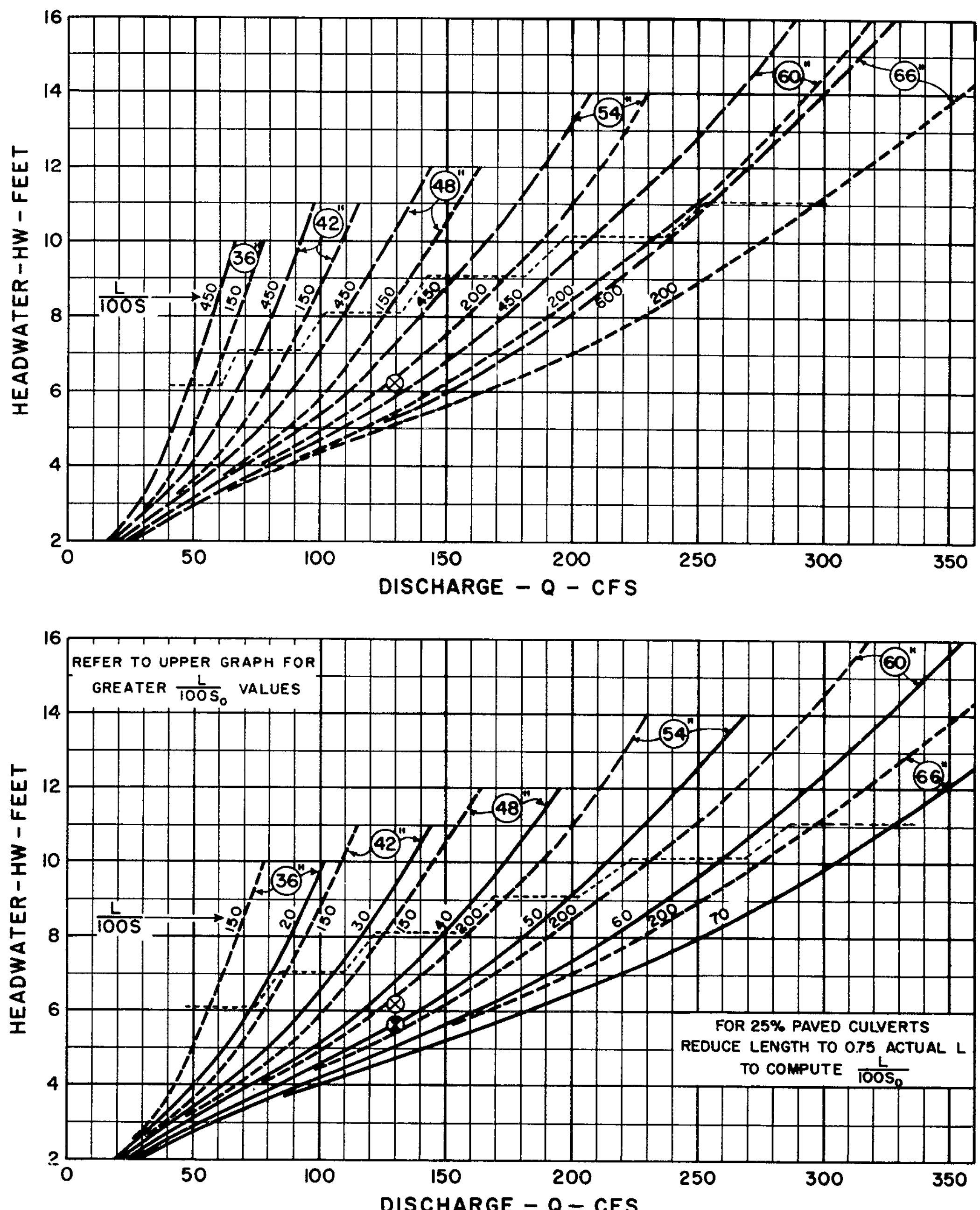
From BPR

EXAMPLE

⊗ GIVEN:
43 CFS; AHW = 4.9 FT.
L = 72 FT.; S_0 = 0.003

SELECT 36" UNPAVED
HW = 4.4 FT.

Figure CU-18—Culvert Capacity Standard Circular Corrugated Metal Pipe Headwall Entrance 18" to 36"



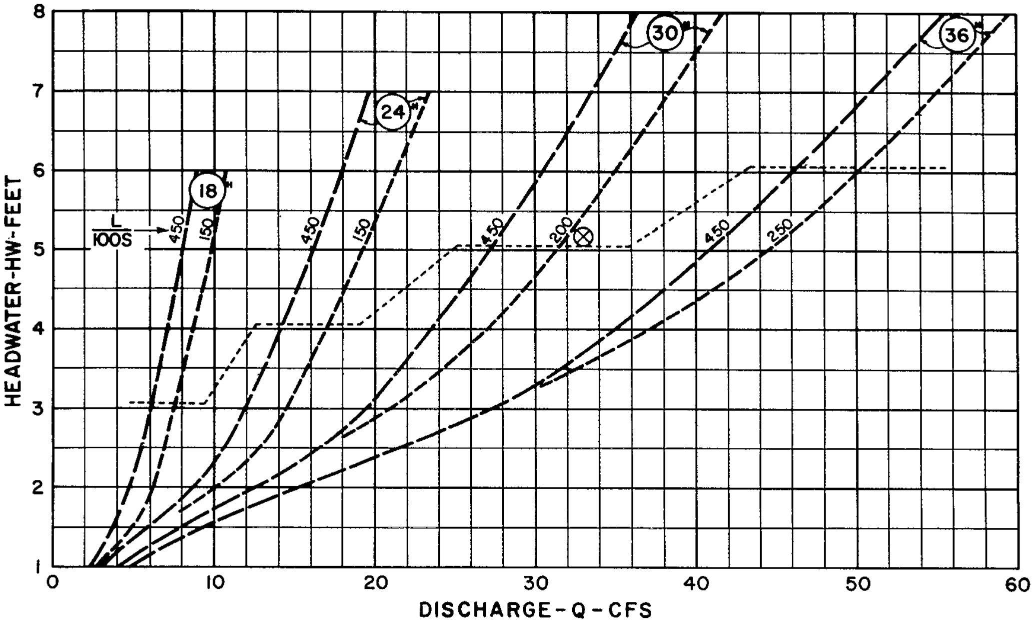
From BPR

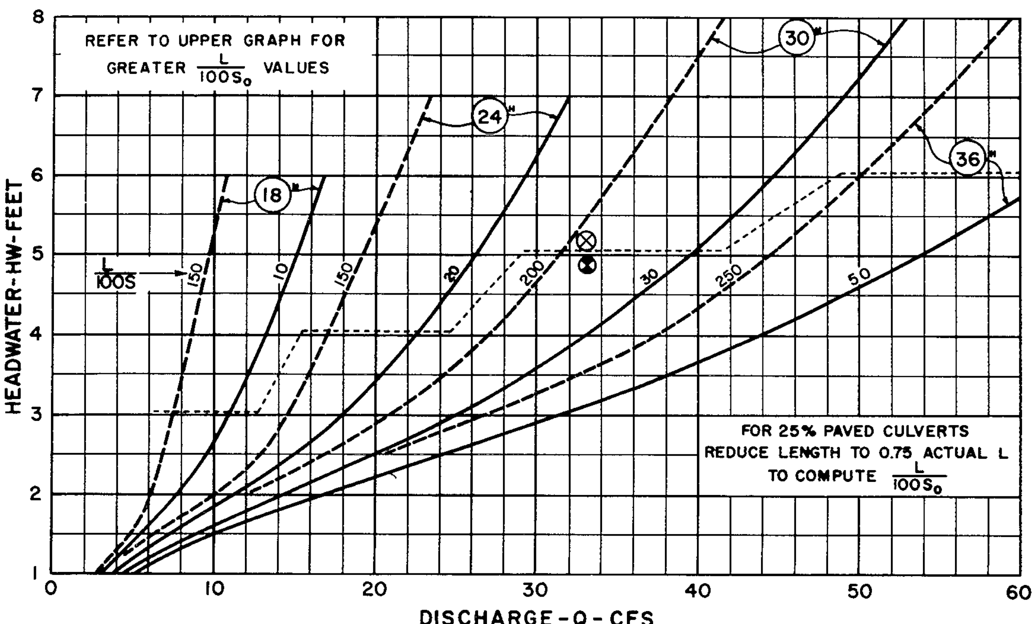
EXAMPLE

⊗ GIVEN:
130 CFS; AHW = 6.2 FT.
L = 120 FT.; S_0 = 0.025

SELECT 54" UNPAVED
HW = 5.6 FT.

Figure CU-19—Culvert Capacity Standard Circular Corrugated Metal Pipe Headwall Entrance 36" to 66"





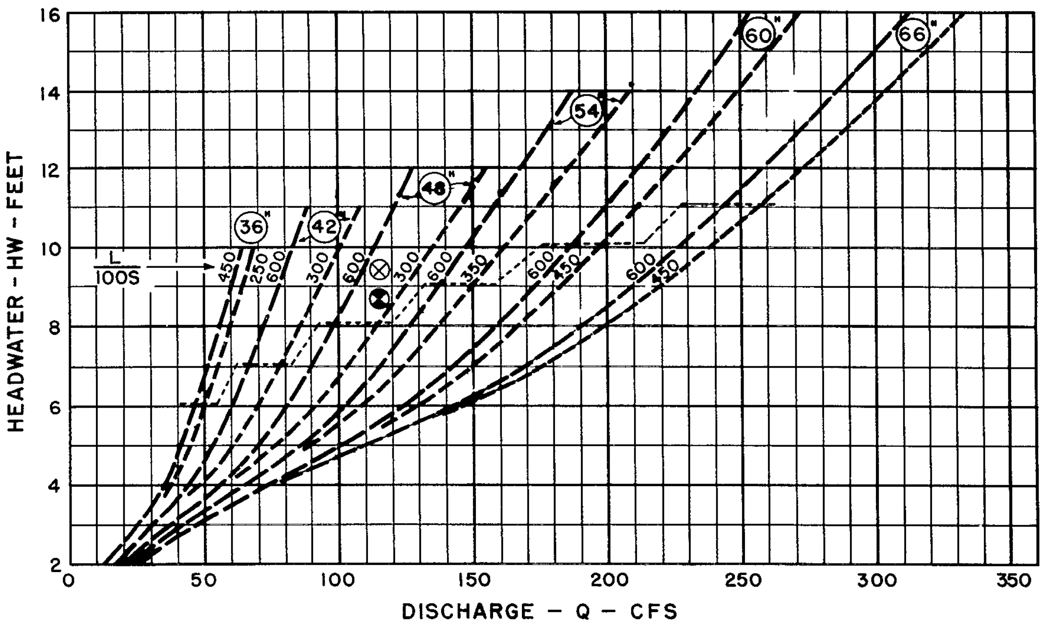
From BPR

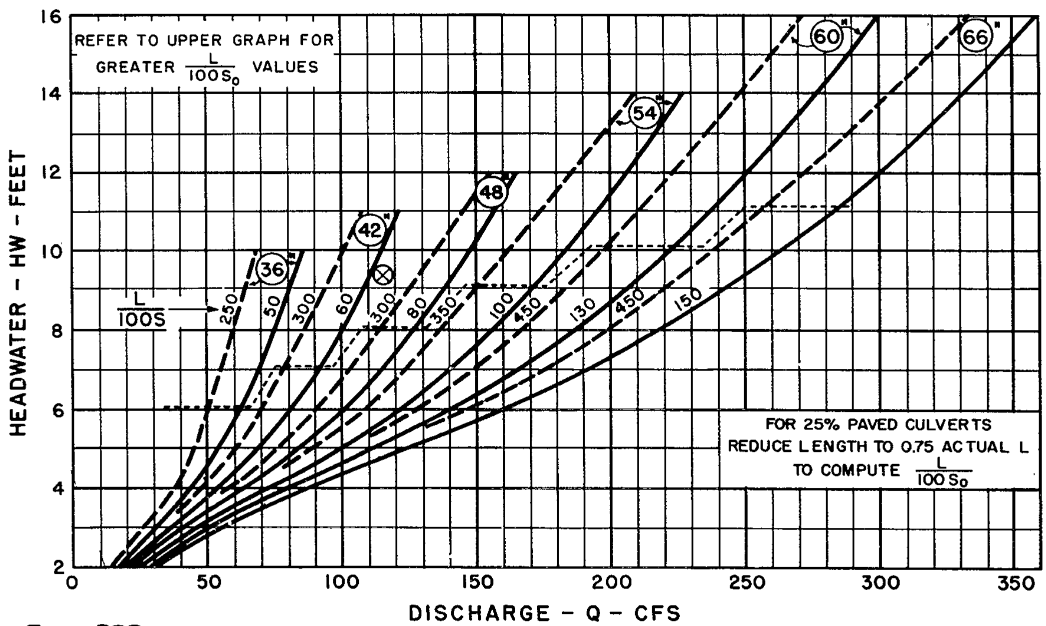
EXAMPLE

⊗ GIVEN:
33 CFS; AHW = 5.2 FT.
L = 70 FT.; S_0 = 0.005

⊗ SELECT 30" UNPAVED
HW = 4.9 FT.

Figure CU-20—Culvert Capacity Standard Circular Corrugated Metal Pipe Projecting Entrance 18" to 36"





From BPR

EXAMPLE

⊗ GIVEN:
115 CFS; AHW = 9.4 FT.
L = 135 FT.; S_0 = 0.0034

② SELECT 48" UNPAVED
HW = 8.6 FT.

Figure CU-21—Culvert Capacity Standard Circular Corrugated Metal Pipe Projecting Entrance 36” to 66”

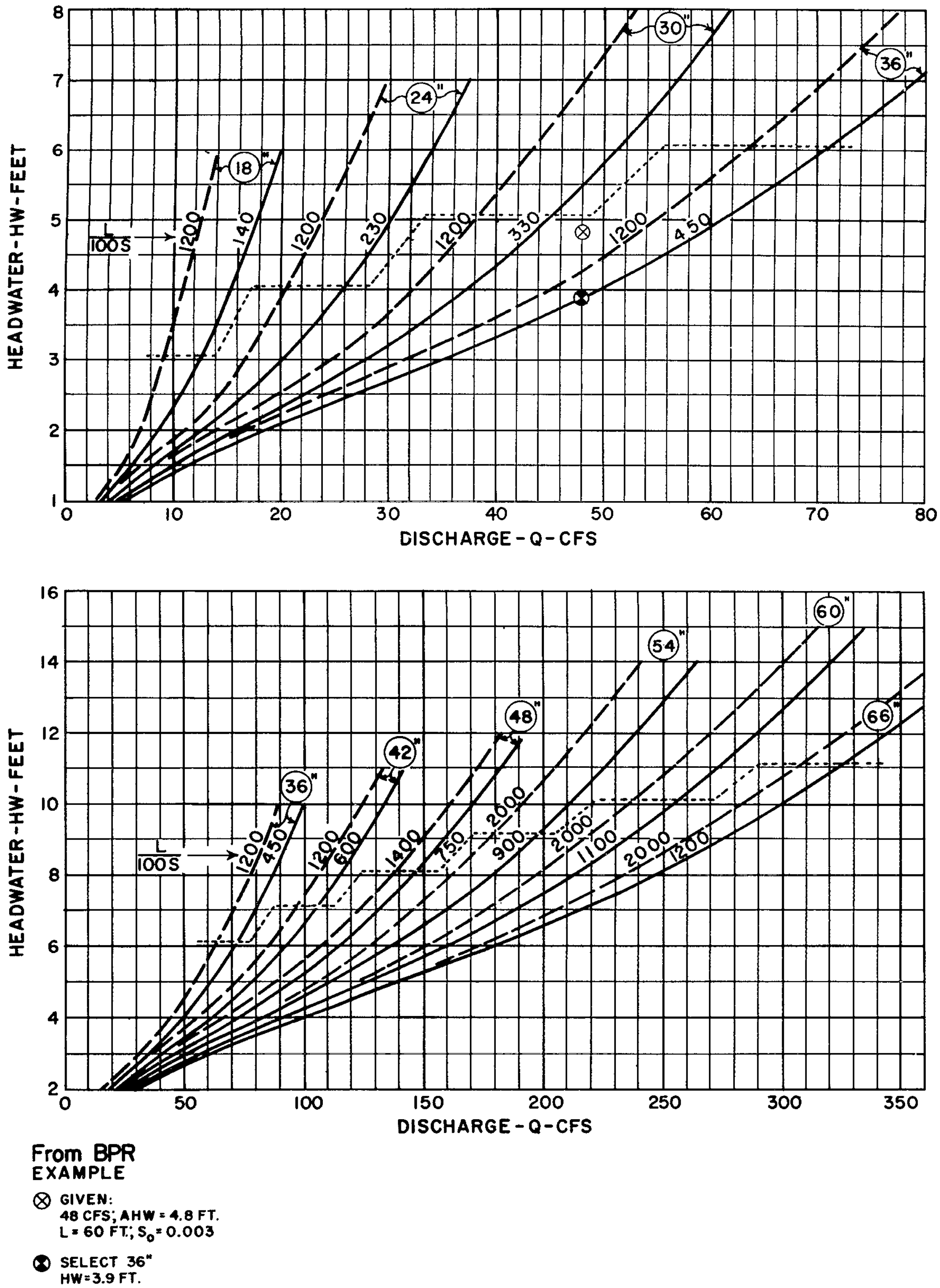
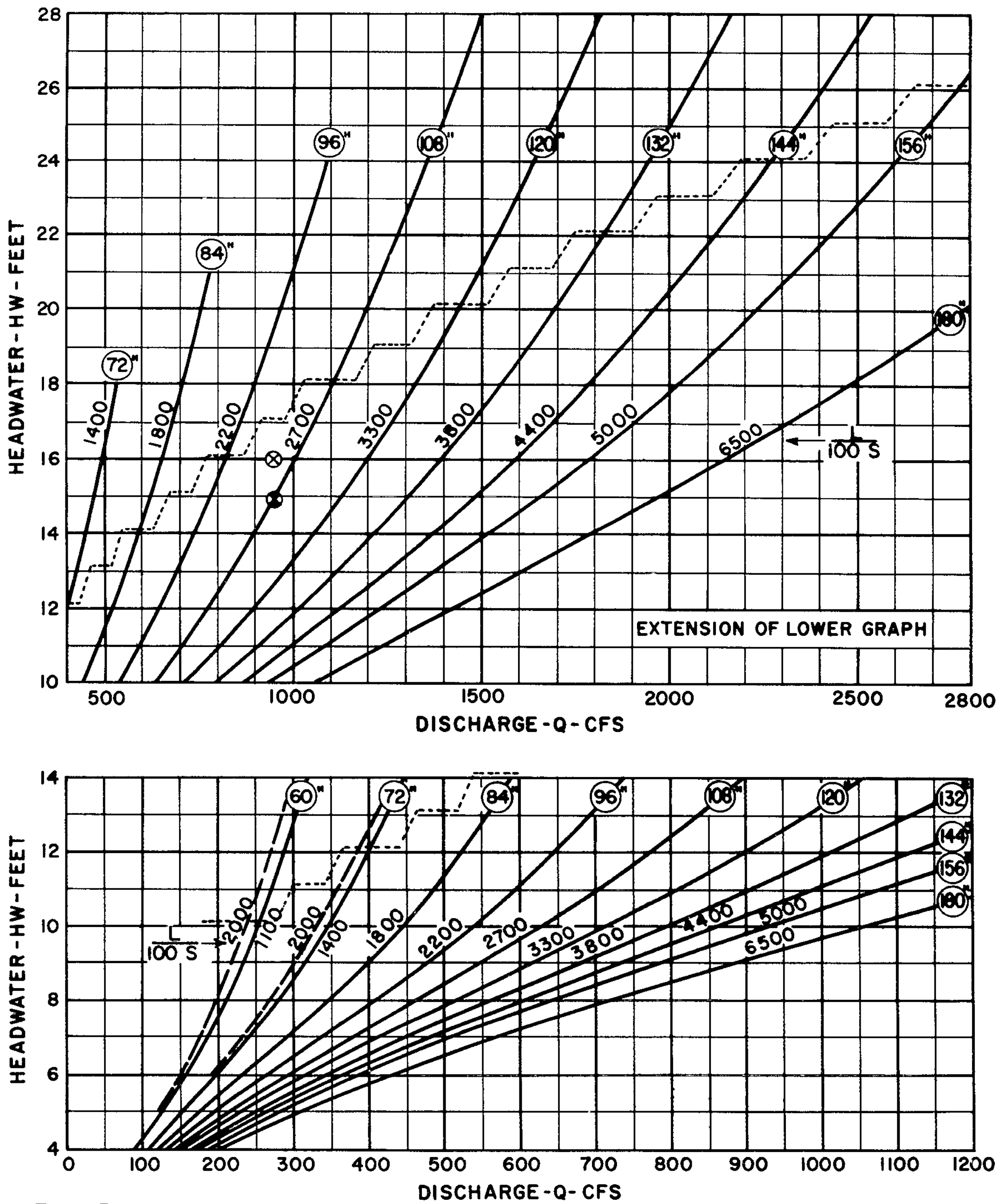


Figure CU-22—Culvert Capacity Circular Concrete Pipe Square-Edged Entrance 18” to 66”



From BPR
EXAMPLE

⊗ GIVEN:
950 CFS; AHW = 16 FT.
L = 480 FT.; S_0 = 0.040

● SELECT 108"
HW = 15.0 FT.

Figure CU-23—Culvert Capacity Circular Concrete Pipe Square-Edged Entrance 60" to 180"

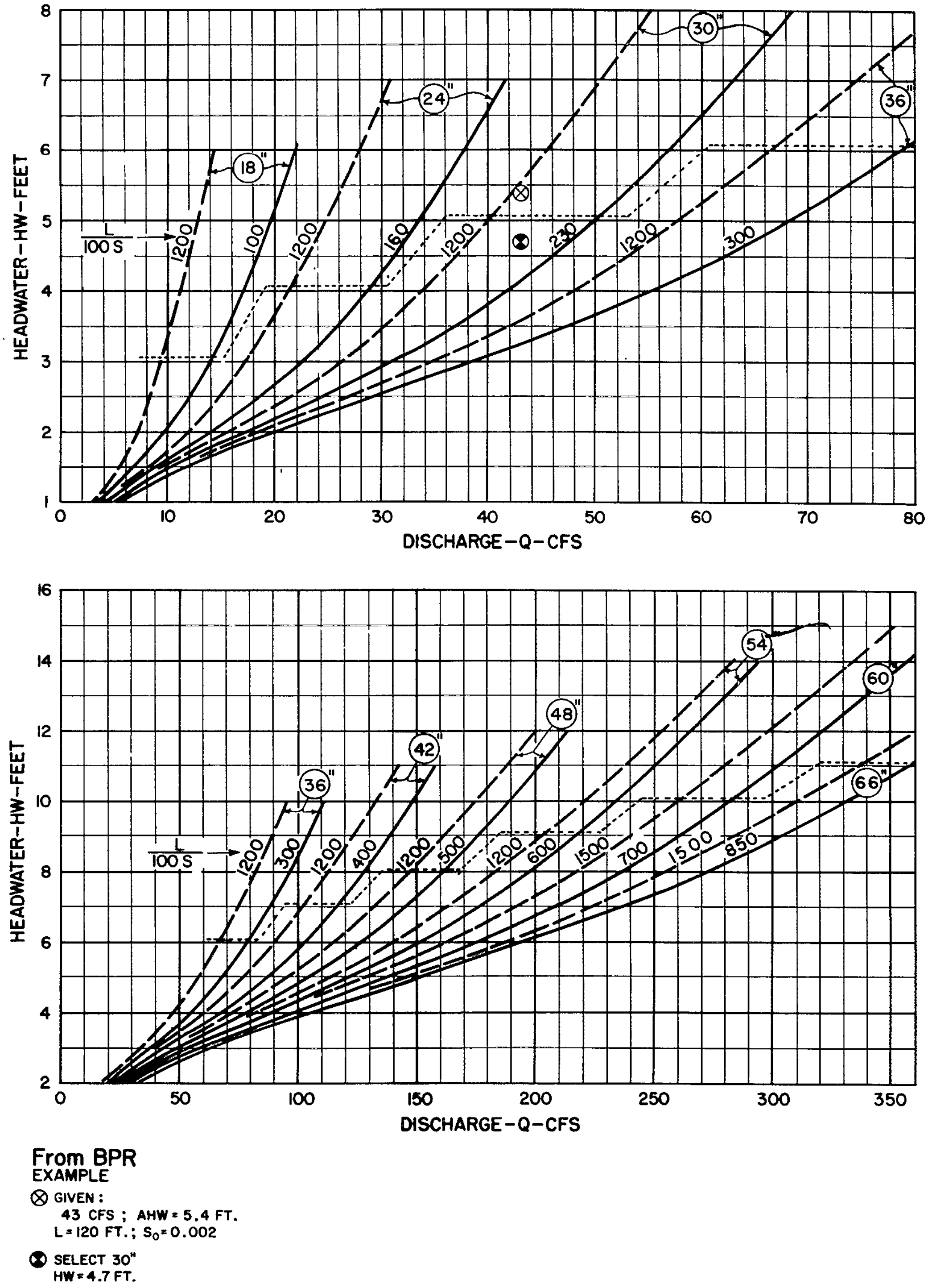
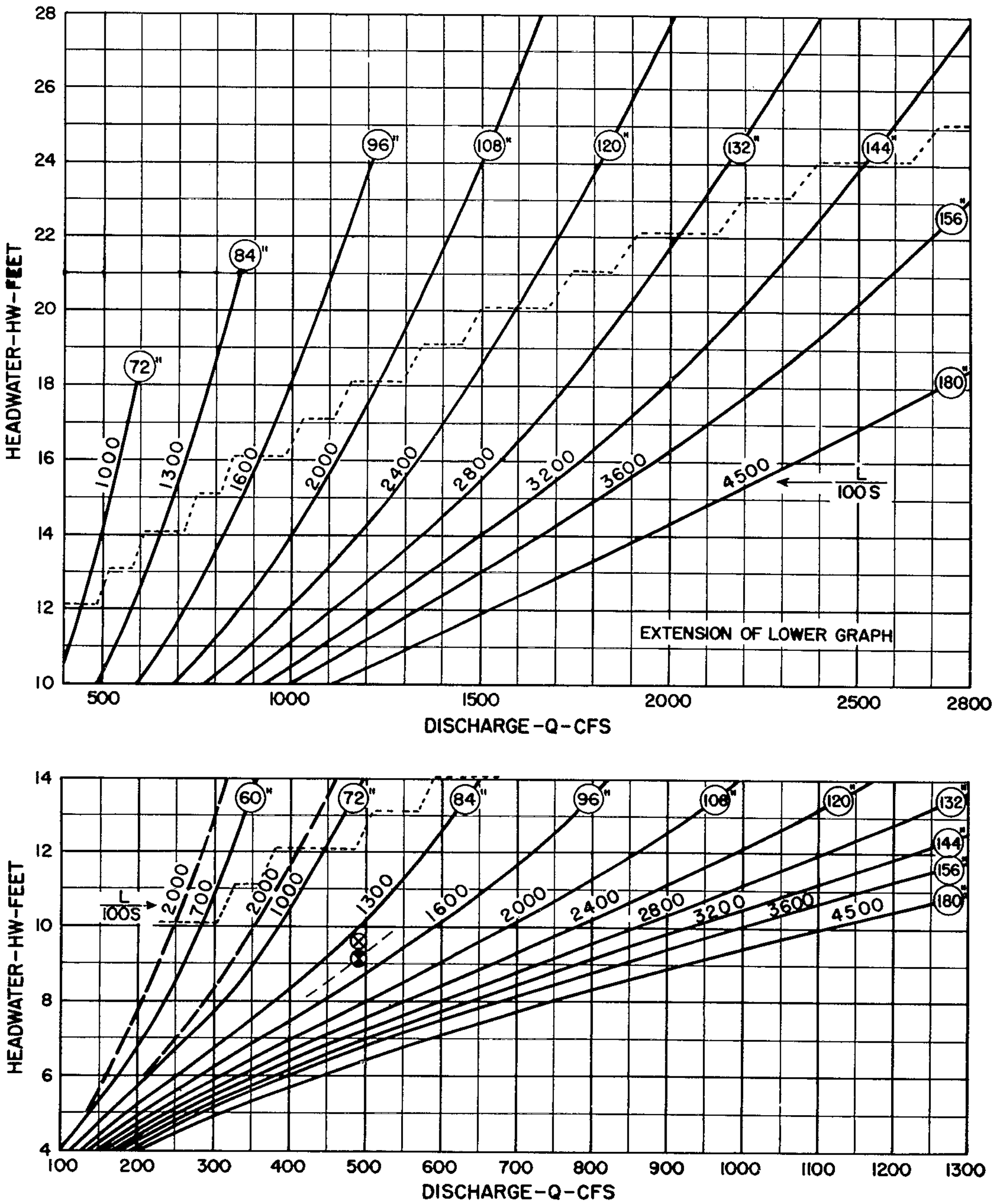


Figure CU-24—Culvert Capacity Circular Concrete Pipe Groove-Edged Entrance 18” to 66”



From BPR
EXAMPLE

⊗ GIVEN:
490 CFS ; AHW = 9.6 FT.
L = 60 FT. ; S_0 = 0.000

SELECT 90" ($\frac{L}{D}$ = 8)
HW = 9.2 FT.

Figure CU-25—Culvert Capacity Circular Concrete Pipe Groove-Edged Entrance 60" to 180"

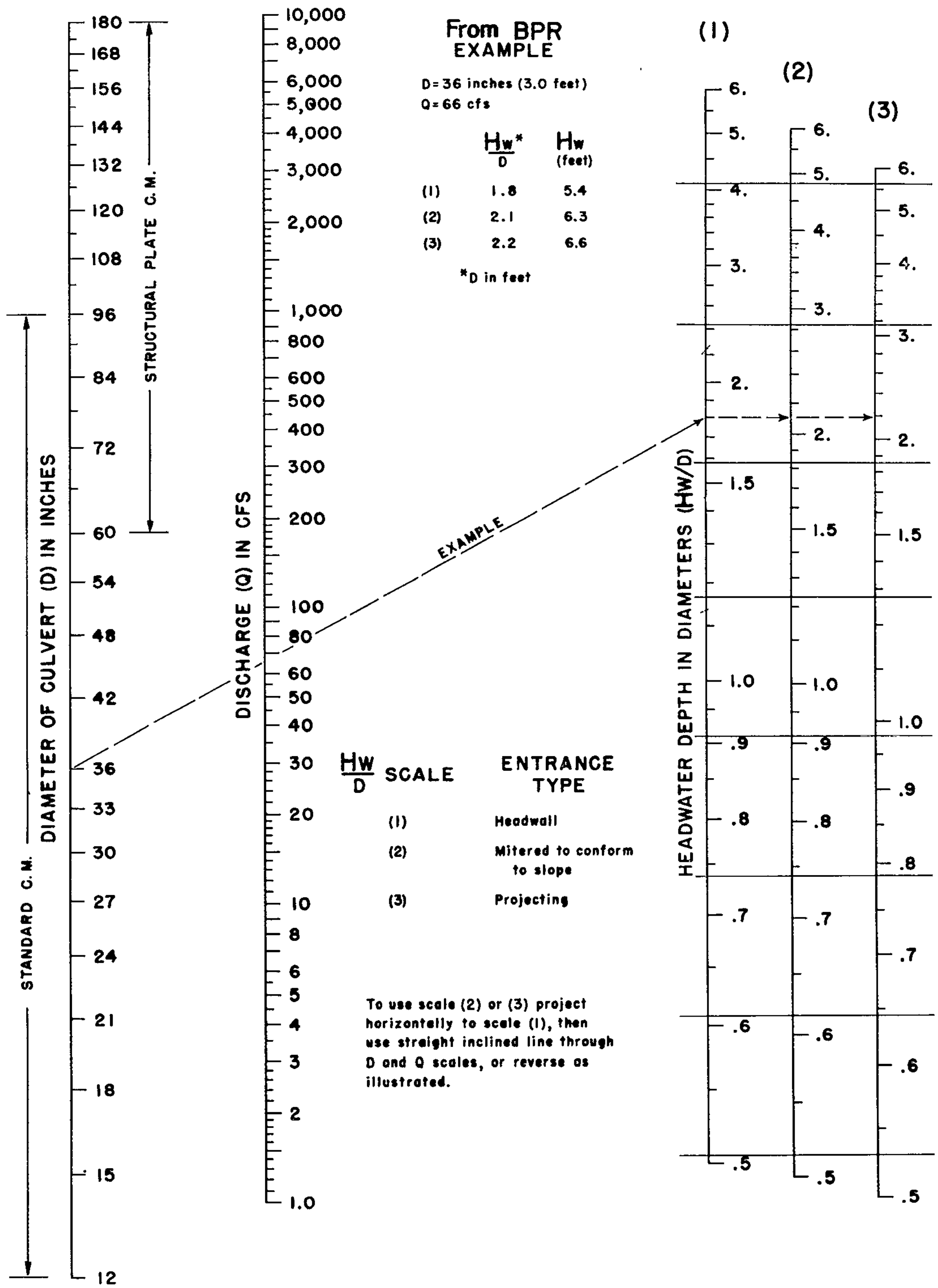


Figure CU-26—Headwater Depth for Corrugated Metal Pipe Culverts With Inlet Control

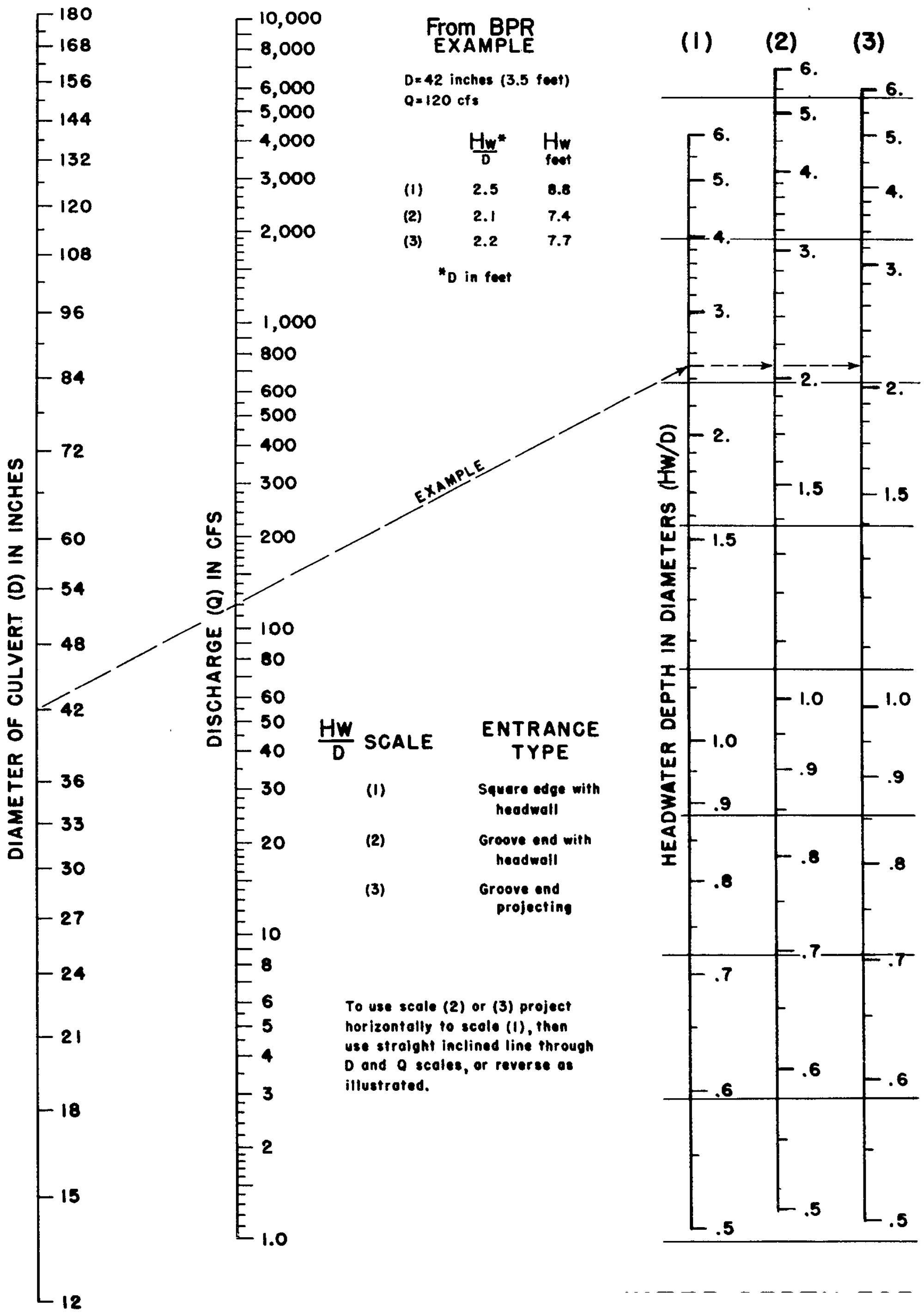


Figure CU-27—Headwater Depth for Concrete Pipe Culverts With Inlet Control

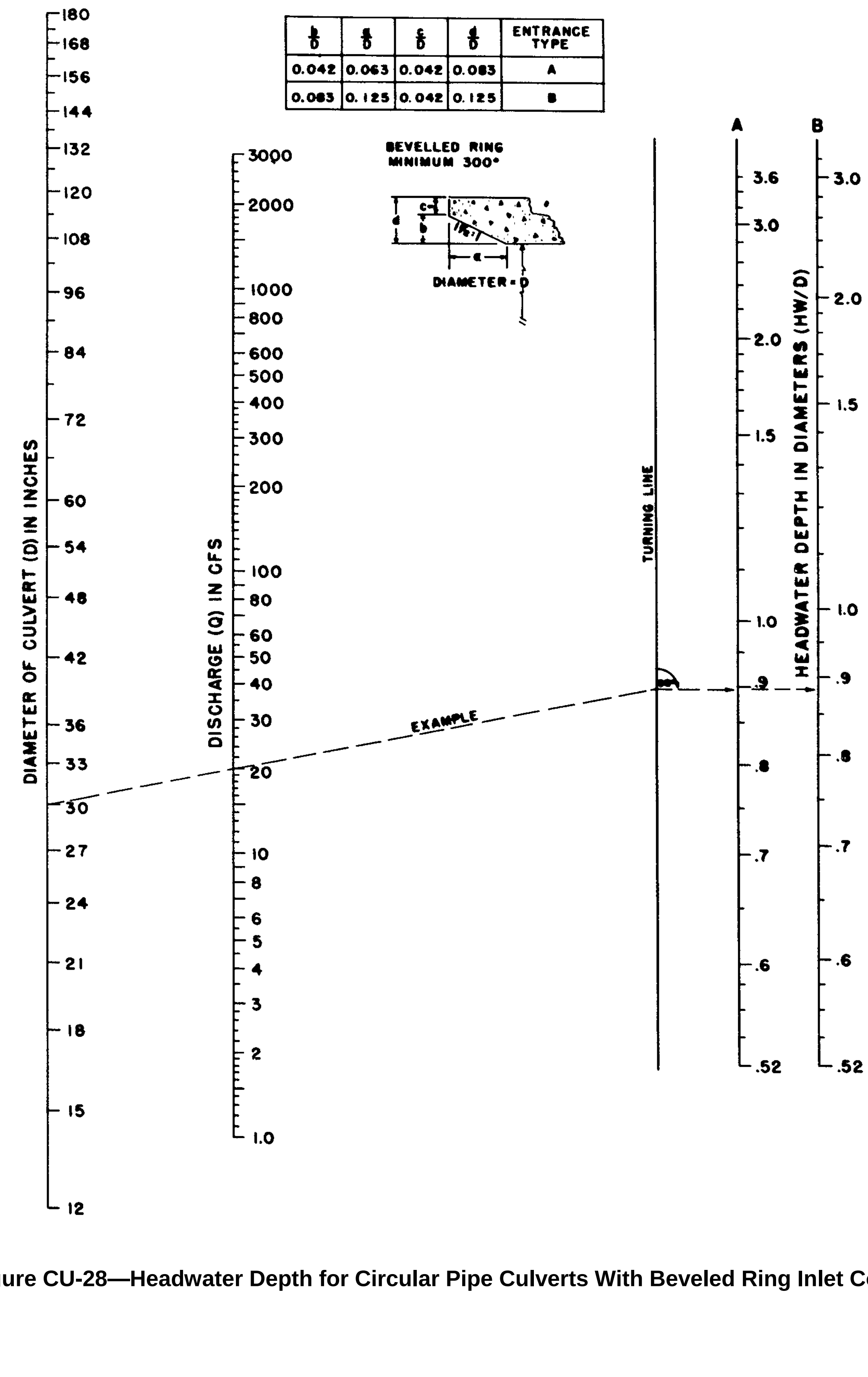


Figure CU-28—Headwater Depth for Circular Pipe Culverts With Beveled Ring Inlet Control

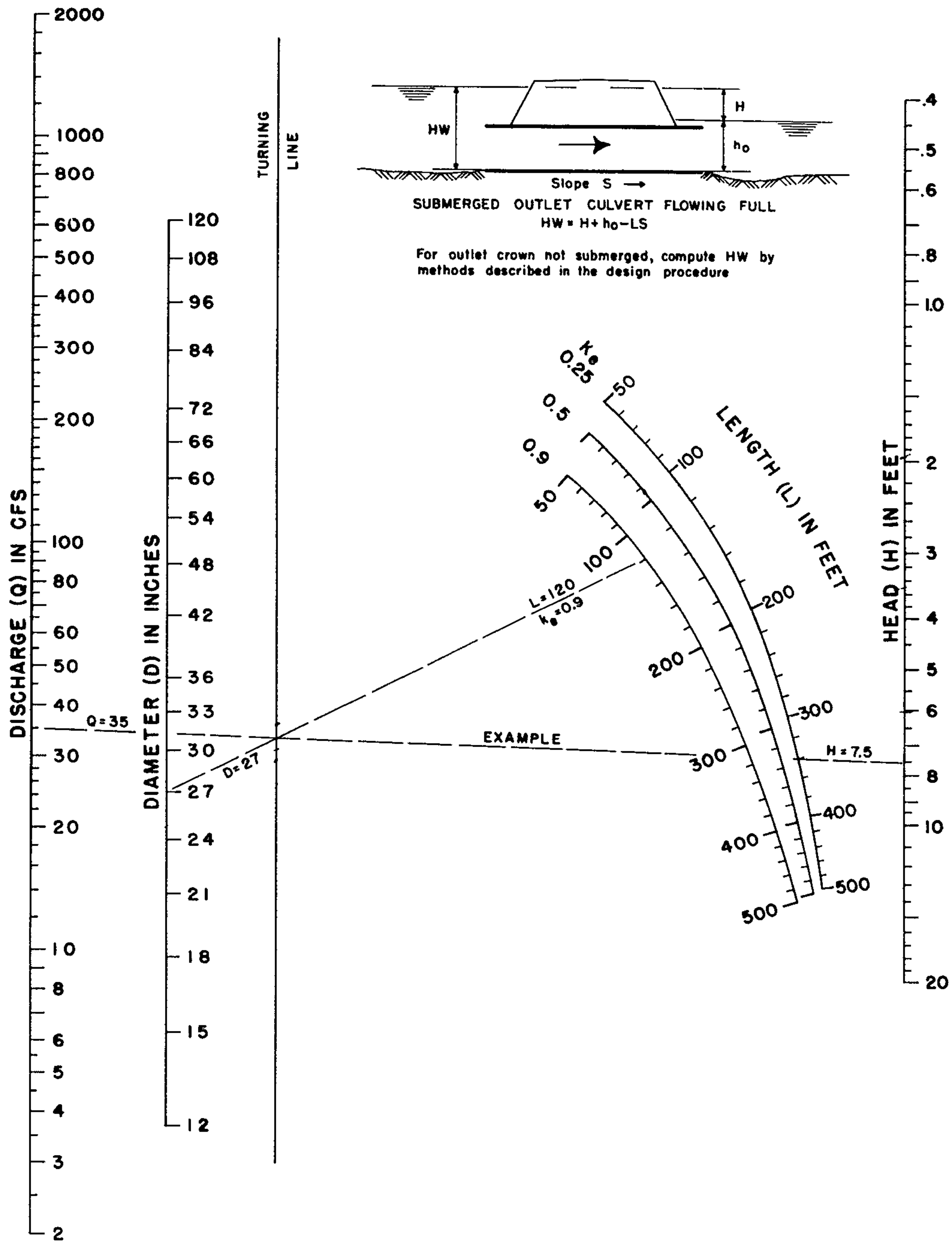


Figure CU-29—Head for Standard Corrugated Metal Pipe Culverts Flowing Full *n* = 0.024

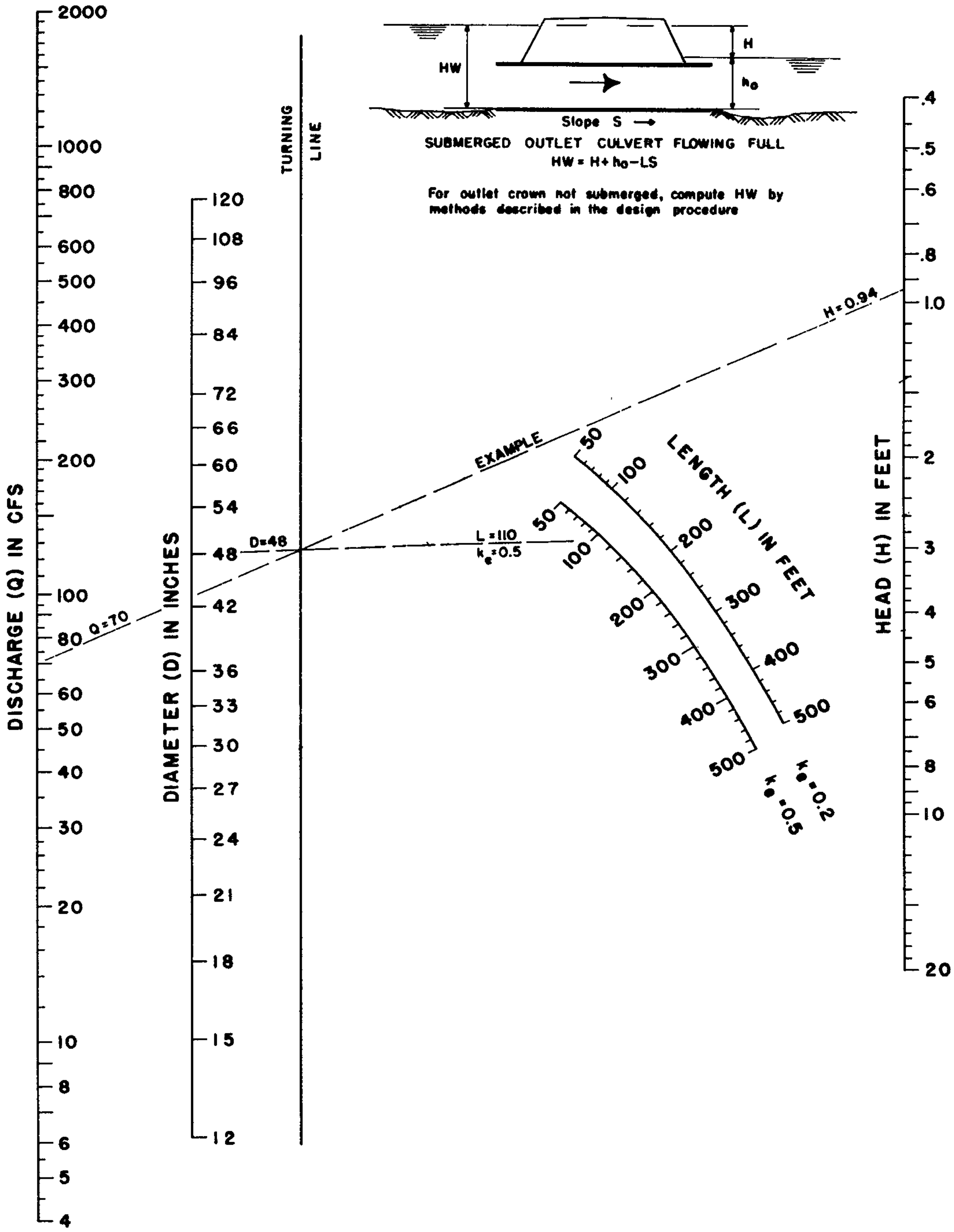


Figure CU-30—Head for Concrete Pipe Culverts Flowing Full n = 0.012

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STORAGE

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1.0 OVERVIEW

This chapter provides guidance for the analysis and design of stormwater *quantity* detention facilities. Detention facilities for the management of stormwater *quality* (i.e., extended detention basins, retention ponds, wetland basins, etc.) are described in Volume 3 of this *Manual*. Detention and retention basins are used for stormwater runoff quantity control to mitigate the effects of urbanization on runoff flood peaks. If there is a need to design a storage facility for both water quality and quantity control purposes, this chapter should be used in conjunction with Volume 3 of this *Manual*.

Topics discussed in this chapter include design storms used for detention, the application of different types of storage facilities, basis for hydrologic and hydraulic design, and various other design considerations. As is the case with major drainageways, the Urban Drainage and Flood Control District (District) strongly encourages the development of multipurpose, attractive detention facilities that are safe, maintainable and viewed as community assets rather than liabilities.

Photograph SO-1—Attractive wet and dry detention facilities in commercial settings have been shown to increase property value.

Photograph SO-2—Dry and extended dry detention facilities can blend into the landscape, especially with the assistance of experienced landscape architects.

2.0 APPLICATION OF DIFFERENT TYPES OF STORAGE

There are two basic approaches to designing storage facilities. When runoff storage facilities are planned on an individual-site basis, they are referred to as “on-site.” Larger facilities that have been identified and sized as a part of some overall regional plan are categorized as “regional” facilities. The regional definition can also be applied to detention facilities that encompass multiple land development projects.

On-site storage facilities usually are designed to control runoff from a specific land development site and are not typically located or designed with the idea of reducing downstream flood peaks along major drainageways for small and/or large storm runoff events. The total volume of runoff detained in the individual on-site facility is quite small, and the detention time for flood control purposes is relatively short. Therefore, unless design (i.e., sizing and flow release) criteria and implementation are applied uniformly throughout the urbanizing or redeveloping watershed, their effectiveness diminishes rapidly along the downstream reaches of waterways. The application of consistent design and implementation criteria and assurance of their continued maintenance and existence is of paramount importance if large numbers of on-site detention facilities are to be effective in controlling peak flow rates along major drainageways (Glidden 1981; Urbonas and Glidden 1983).

The principal advantage of on-site facilities is that developers can be required to build them as a condition of site approval. Major disadvantages include the need for a larger total land area for multiple smaller on-site facilities as compared to larger regional facility(ies) serving the same tributary catchment area. If the individual on-site facilities are not properly maintained, they can become a nuisance to the community and a basis for many complaints to municipal officials. It is also difficult to ensure adequate maintenance and long-term performance at levels they were design to provide. Prommesberger (1984) inspected approximately 100 on-site facilities built, or required by municipalities to be built, as a part of land developments over about a 10-year period. He concluded that a lack of adequate maintenance and implementation contributed to a loss of continued function or even presence of these facilities. He also concluded that a lack of institutional structures at the local government level was the major contributor to any of these facilities no longer being in existence or in original operational mode after their initial construction.

Photograph SO-3—On-site storage facility serving town home development (in background) coupled with park.

Facilities designed as part of a watershed planning process, in which the stormwater management needs for the watershed as a whole are developed in a staged regional plan, are called regional facilities. These are often planned and located as part of the District's master planning process. They are typically much larger than on-site facilities. The main disadvantage of the regional facilities is the lack of an institutional structure to fund their implementation early in the development process. Another disadvantage of regional facilities is that they can leave substantial portions of the stream network susceptible to increased flood peaks, and plans must be developed to take this condition into account. In addition, to promote water quality benefits, some form of on-site stormwater management is necessary upstream of the regional facilities. Examples include minimized directly connected impervious areas (MDCIA) that promote flow across vegetated surfaces utilizing "slow-flow" grassed swales and a number of other techniques described in Volume 3 of this *Manual* that reduce stormwater surface runoff volumes.

More economical and hydrologically reliable results can be achieved through stormwater management planning for an entire watershed that incorporates the use of regional facilities. Regional facilities also potentially offer greater opportunities in achieving multi-objective goals such as recreation, wildlife habitat, enhanced property value, open space, and others.

There are several types of stormwater storage facilities, whether they are classified as on-site or regional, namely:

1. Detention—Detention facilities provide temporary storage of stormwater that is released through an outlet that controls flows to pre-set levels. Detention facilities typically flatten and spread the

inflow hydrograph, lowering the peak to the desired (i.e., master plan, pre-development, existing, etc.) flow rate. Often these facilities also incorporate features designed to meet water quality goals.

2. Retention—Retention facilities, as defined this chapter, store stormwater runoff without a positive outlet, or with an outlet that releases water at very slow rates over a prolonged period. These differ in nature and design from “retention ponds” described in Volume 3 of this *Manual* that are used for water quality purposes.

3. Conveyance (Channel) Storage—Conveyance, or channel routing, is an often-neglected form of storage because it is dynamic and requires channel storage routing analysis. Slow-flow and shallow conveyance channels and broad floodplains can markedly retard the build up of flood peaks and alter the time response of the tributaries in a watershed.

4. Infiltration Facilities—Infiltration facilities resemble retention facilities in most respects. They retain stormwater runoff for a prolonged period of time to encourage infiltration into the groundwater. These facilities are difficult to design and implement because so many variables come into play.

5. Other Storage Facilities—Storage can occur at many locations in urban areas for which special considerations typically apply to the use and reliance upon such conditions. Stormwater detention may occur at:

   a) *Random depressions*. Depressions can be filled in during development and cannot be relied upon as permanent.

   b) *Upstream of railroad and highway embankments*. If the designer intends to utilize roadway, railroad, or other embankments for detention storage, some form of ownership of the flood storage pool and some form of control of the outlet must be acquired. An agreement with the roadway, railroad, or other agency that insures the continued existence of the facility over time has to be reached before relying on the facility. In addition, it is necessary to demonstrate that (1) roadway, railroad or other embankment stability will not be compromised, (2) embankment overtopping during larger storms will not impact upstream or downstream properties, and (3) the storage facility will remain in place as a detention facility in perpetuity. Storage behind road, railroad, and other embankments can also be lost due to site grading and fill changes and/or the installation of larger culverts or bridges.

   c) *Water storage reservoirs.* Colorado State law specifically exempts the reliance of water storage reservoirs for flood control by downstream properties. If the designer or project developers want to utilize them for detention storage, some form of ownership of the flood storage pool and outlet function must be acquired from the reservoir owner. An agreement with the reservoir owner that insures the continued existence of the facility or its detention function over time has to be reached before relying on such reservoirs. In addition, it is necessary to demonstrate that embankment and spillway are safe and stable to insure public safety.

It is beyond the scope of this *Manual* to address these kinds of specialized storage facilities in more detail, but readers are cautioned that the above-mentioned considerations must be taken into account before proposing their use as formal detention facilities.

Detention and retention facilities can be further subdivided into:

1. In-Line Storage—A facility that is located in-line with the drainageway and captures and routes the entire flood hydrograph. A major disadvantage with in-line storage is that it must be large enough to handle the total flood volume of the entire tributary catchment, including off-site runoff, if any.

2. Off-Line Storage—A facility that is located off-line from the drainageway and depends on the diversion of some portion of flood flows out of the waterway into the storage facility. These facilities can be smaller and potentially store water less frequently than in-line facilities.

Irrespective of which type of storage facility is utilized, the designer is encouraged to create an attractive, multipurpose facility that is readily maintainable and safe for the public, under both “dry” (i.e., dry weather) and “wet” (i.e., when runoff is occurring) conditions. Designers are also encouraged to consult with other specialists such as urban planners, landscape architects, and biologists during planning and design.

3.0 HYDROLOGIC AND HYDRAULIC DESIGN BASIS

3.1 Procedures for the Sizing of Storage Volumes

Three procedures for the sizing of detention storage volumes and one for the sizing of retention storage volumes are described in this *Manual*. For detention facilities, two of the procedures may be applied to on-site facilities and facilities serving relatively small tributary areas. For detention facilities serving larger catchments or ones classified as regional, the *Manual* recommends only one design protocol for use within the District, as described below.

3.1.1 Use of Simplified On-Site Detention Sizing Procedures

The three simplified procedures for the sizing of on-site detention volumes described here are "empirical equations" (Section 3.2.2), the modified "Rational Formula-based FAA Method" (Section 3.2.3) and "Full-Spectrum Detention" (Section 3.2.4). The uses of empirical equations by themselves are only applicable for small catchments not exceeding 90 acres. The Rational Formula-based FAA procedure may be applied to tributary catchments up to 160 acres in size, but the District suggests that it is best to limit ther use to tributary areas of 90 acres or less.

The Excess Urban Runoff Control, called *Full Spectrum Detention*, method may be applied to catchments of up to one-square mile in size; however, the simplified approach described in this chapter, including the use of the spreadsheet for this method, is best limited to areas of 160 acres or less.

3.1.2 Use of Hydrograph Routing Detention Sizing Procedure

Whenever the area limits described above in Section 3.1.1. are exceeded (for tributary catchments larger than 90 acres for empirical equations and FAA Method and 160 acres for the *Full Spectrum Detention* method), the District recommends the use of hydrograph flood routing procedures (e.g., using CUHP-generated hydrographs and reservoir routing calculations). In addition, if there are upstream detention facilities in the watershed that catch and route runoff for portions of the upstream tributary area, hydrograph routing methods should be employed.

To be considered as a sub-regional or regional facility by the District, namely part of the major drainageway system, the detention basin has to have a tributary area of 130 acres or more.

If off-site tributary areas contribute runoff to an on-site detention facility, the total tributary area, assuming fully developed off-site land uses, must be included in the sizing of the on-site storage volumes in order to account for the total runoff volume in the watershed.

3.1.3 Water Quality Capture Volume in Sizing Detention Storage

When detention storage volume is sized for a site that also incorporates a water quality capture volume (WQCV) defined in Volume 3 of this *Manual*, check with the local jurisdiction to determine how to account for this volume. Some municipalities within the District will permit partial or full use of the WQCV within the calculated 100-year volume. Others require that the 100-year volume be added to the WQCV. All jurisdictions require the WQCV be added to the 5- or 10-year volume. When clear written local criteria on this matter are absent, the District recommends that no less than 50% of the WQCV be added to the calculated 100-year volume for 100-year volumes obtained using empirical equations and the FAA Method. However, unless the local jurisdiction requires adding all or part of the WQCV to the 100-year volume obtained using the simplified *Full Spectrum Detention* design; District does not recommend adding any part of the WQCV to the 100-year volume. When the analysis is done using hydrograph routing methods, each level of controls needs to be accounted for and the resultant 100-year control volume used in final design.

3.2 Sizing of On-Site Detention Facilities

3.2.1 Maximum Allowable Unit Release Rates for On-Site Facilities

The maximum allowable unit release rates in the Denver area per acre of tributary catchment for on-site detention facilities for various design return periods are listed in Table SO-1. These maximum releases rates will apply for all on-site detention facilities unless other rates are recommended in a District-approved master plan. For regional facilities see Section 3.2.5.

Allowable unit release rates in Table SO-1 for each a soil group in the tributary catchment shall be area-weighted to composite the allowable unit release rate for the total catchment. Multiply this rate by the total tributary catchment's area to obtain the design release rates in cubic feet per second (cfs). Whenever Natural Resources Conservation Service (NRCS) soil surveys are not available, approximate their equivalent types using results of detailed soil investigations at the site.

Table SO-1—Maximum Unit Flow Release Rates (cfs/acre) from On-Site Detention Facilities

Design Return Period (Years)	NRCS Hydrologic Soil Group		
	A	B	C & D
2	0.02	0.03	0.04
5	0.07	0.13	0.17
10	0.13	0.23	0.30
25	0.24	0.41	0.52
50	0.33	0.56	0.68
100	0.50	0.85	1.00

3.2.2 Empirical Equations for the Sizing of On-Site Detention Storage Volumes

Urbonas and Glidden (1983), as part of the District's ongoing hydrologic research, conducted studies that evaluated peak storm runoff flows along major drainageways. The following set of empirical equations provided preliminary estimates of on-site detention facility sizing for areas within the District. They are intended for single return period control and not for use when off-site inflows are present or when multi-stage controls are to be used (e.g., 10- and 100-year peak control). In addition, these equations are not intended to replace detailed hydrologic and flood routing analysis, or even the analysis using the Rational Formula-based FAA method for the sizing of detention storage volumes. The District does not promote the use of these empirical equations. It does not object, however, to their use by local governments who have adopted them or want to adopt them as minimum requirements for the sizing of on-site detention for small catchments within their jurisdiction. If the District has a master plan that contains specific guidance for detention storage or sizing of on-site detention facilities, those guidelines should be followed instead. The empirical equations for NRCS Soil types B, C and D are as follows:

$$V_i = K_i A \tag{SO-1}$$

for the 100-year:

$$K_{100} = \frac{(1.78\,I - 0.002\,I^2 - 3.56)}{900} \tag{SO-2}$$

for the 10-year:

$$K_{10} = \frac{(0.95\,I - 1.90)}{1{,}000} \tag{SO-3}$$

for the 5-year:

$$K_5 = \frac{(0.77I - 2.65)}{1{,}000} \tag{SO-4}$$

For Soil Type A, Equations SO-1 and SO-2 tend to underestimate the needed 100-year detention volume. Instead, Equation SO-5 needs to be used to estimate the 100-year detention volume for Type A Soils (i.e., V_{100A}):

$$V_{100A} = \left(-0.00005501 \cdot I^2 + 0.030148 \cdot I - 0.12\right) \cdot \frac{A}{12} \tag{SO-5}$$

in which:

V_i = required volume where subscript i = 100-, 10- or 5-year storm, as appropriate (acre-feet)

K_i = empirical volume coefficient where subscript i = 100-, 10- or 5-year storm, as appropriate

I = fully developed tributary catchment imperviousness (%)

A = tributary catchment area (acres)

Design Example 6.1 shows calculations of allowable release rate and storage requirement using empirical equations.

3.2.3 Rational Formula-Based Modified FAA Procedure

The Rational Formula-based Federal Aviation Administration (FAA) (1966) detention sizing method (sometimes referred to as the "FAA Procedure"), as modified by Guo (1999a), provides a reasonable estimate of storage volume requirements for on-site detention facilities. Again, this method provides sizing for one level of peak control only and not for multi-stage control facilities.

The input required for this Rational Formula-based FAA volume calculation procedure includes:

A = the area of the catchment tributary to the storage facility (acres)

C = the runoff coefficient

Q_{po} = the allowable maximum release rate from the detention facility based on Table SO-1 (cfs)

T_c = the time of concentration for the tributary catchment (see the RUNOFF chapter) (minutes)

P_1 = the 1-hour design rainfall depth (inches) at the site taken from the RAINFALL chapter for the relevant return frequency storms

The calculations are best set up in a tabular (spreadsheet) form with each 5-minute increment in duration being entered in rows and the following variables being entered, or calculated, in each column:

1. Storm Duration Time, T (minutes), up to 180 minutes.
2. Rainfall Intensity, I (inches per hour), calculated using Equation RA-3 from the RAINFALL chapter.
3. Inflow volume, V_i (cubic feet), calculated as the cumulative volume at the given storm duration using the equation:

$$V_i = CIA\left(60T\right) \tag{SO-6}$$

4. Outflow adjustment factor m (Guo 1999a):

$$m = \frac{1}{2}\left(1 + \frac{T_c}{T}\right) \qquad 0.5 \le m \le 1 \text{ and } T \ge T_c \tag{SO-7}$$

5. The calculated average outflow rate, Q_{av} (cfs), over the duration T:

$$Q_{av} = mQ_{po} \tag{SO-8}$$

6. The calculated outflow volume, V_o, (cubic feet), during the given duration and the adjustment factor at that duration calculated using the equation:

$$V_o = Q_{av}\left(60T\right) \tag{SO-9}$$

7. The required storage volume, V_s (cubic feet), calculated using the equation:

$$V_s = V_i - V_o \tag{SO-10}$$

The value of V_s increases with time, reaches a maximum value, and then starts to decrease. The maximum value of V_s is the required storage volume for the detention facility. Sample calculations using this procedure are presented in Design Example 6.2. The modified *FAA Worksheet* of the UD-Detention Spreadsheet performs these calculations.

3.2.4 Simplified Full-Spectrum Detention Sizing (Excess Urban Runoff Flow Control)

With urbanization, the runoff volume increases. Percentage-wise, this increase is much more noticeable for the smaller storm events than for the very big ones, such as the 100-year storm. Wulliman and Urbonas (2005) suggested a concept they termed *Full Spectrum Detention*. This concept was studied using extensive modeling, including continuous simulations of a calibrated watershed. Based on this modeling the original set of equations was slightly modified to increase the EURV by 10%. The protocol that resulted and that is described below reduced runoff peak flows from urbanized areas to more closely approximate the runoff peaks along major drainageways before urbanization occurred.

This concept captures a volume of runoff defined as the *Excess Urban Runoff Volume*" (EURV) and then releases it over approximately 72-hours. EURV is larger than the Water Quality Capture Volume (WQCV) defined in Volume 3 of this *Manual* and varies with the type of NRCS soil group upon which urbanization occurs. EURV includes within its volume the WQCV, which then makes it unnecessary to deal with it separately when the *Full Spectrum Detention* design is used. *Full Spectrum Detention* Equations SO-11, -12 and -13 may by used to find the EURV depths in watershed inches. They were developed using the hydrologic methods described in this *Manual*.

NRCS Soil Group A: $$EURV_A = 1.1 \cdot \left(2.0491 \cdot i - 0.1113\right) \tag{SO-11}$$

NRCS Soil Group B: $$EURV_B = 1.1 \cdot \left(1.2846 \cdot i - 0.0461\right) \tag{SO-12}$$

NRCS Soil Group C/D: $$EURV_{CD} = 1.1 \cdot (1.1381 \cdot i - 0.0339) \tag{SO-13}$$

in which, $EURV_K$ = Excess Urban Runoff Volume in watershed inches ($K = A, B$ *or* CD),

i = Imperviousness ratio (I/100)

By combining the capture and slow release of the EURV with the 100-year control volumes for Soil Types B, C and D recommended by Equations SO-1 and SO-2 or for Soil Type A recommended by Equation SO-5 with the 100-year release rates based on recommendations in Table SO-1, this concept was found to be more effective in controlling peak flow along major drainageways for almost all levels of storms than provided by the simplified equations or the FAA Method, even for relatively large urban catchments.

The EURV is found using volumes obtained for each soil type, which are then area weighted in proportion to the total catchment's area. The watershed inches of EURV are then converted to cubic feet or acre-feet. The total 100-year detention basin volume is found using Equations SO-1 and SO-2 for Type B, C and D soils or Equation SO-5 for Type A soils, which are also area-weighted by soil types and converted to cubic feet or acre feet. The outlet is designed to empty the EURV in approximately 72 hours. Volumes exceeding EURV are controlled by an outlet designed for a composite maximum 100-year release rate based on unit rates recommended in Table SO-1.

Equation 13a was developed to assist in the sizing of the openings of the perforated plate outlet to drain the EURV, provided the outlet follows the standardized design described in Volume 3, namely the perforations are spaced vertically on 4" centers. The equation is only applicable for water depths in the basin between one and eight feet. Designers should not extrapolate beyond this range. Outlets needing greater or lesser depths need to be designed individually using ether EPA SWMM, UD-Detention spreadsheet or other appropriate software. The *Full-Spectrum Worksheet* of the UD-Detention Spreadsheet performs all of these calculations for the standardized designs, including adjustments for imperviousness due to Level 1 and 2 of MDCIA, accounts for the effects of various soil type distributions in the tributary catchment and has a provision for selecting the local government's policy in how the WQCV is treated as part of the 100-year volume, although the District does not recommend adding any portion of the WQCV to the 100-year volume calculated using this spreadsheet.

$$A_O = \frac{88V^{\left(0.95/H^{0.085}\right)}}{T_D\, S^{0.09}\, H^{\left(2.6S^{0.3}\right)}} \qquad \text{SO-13a}$$

Where:

A_O = area per row of orifices spaced on 4" centers (in^2)

V = design volume (WQCV or EURV, acre ft)

T_D = time to drain the prescribed volume (hrs) (Typically 72 hours for EURV)

H = depth of volume (ft)

S = slope (ft/ft)

Whenever possible, it is suggested that circular orifice openings be used, beveled on the downstream side. The goal is to find a commonly available drill-bit size that will match the needed area with as few columns of perforations as possible. To achieve this, the designer should seek a drill bit size that will deliver an area within +5% and -10% of the one calculated using Equation SO-13a.

3.2.5 Excess Urban Runoff Flow Control at Regional Facilities

The simplified full-spectrum detention concept described above is appropriate for volume and outlet sizing of detention facilities serving on-site watersheds of up to 160 acres. For full-spectrum basins serving larger watersheds, the EURV portion of the basin still needs to be sized using Equations SO-10 through SO-12 and the outlet designed to empty this volume in approximately 72-hours. The 100-year peak flow control volume above the EURV has to be sized, and its outlet designed, using full hydrograph routing protocols. The hydrograph routing option is also available for smaller sub-watersheds as well. Regardless of which 100-year sizing and outlet design option is used for regional facilities, the maximum 100-year release rates cannot exceed the release rates based on unit discharges recommended in Table SO-1 or pre-developed peak 100-year flow rates for the tributary watershed, whichever are less, or those recommended in a District accepted master plan.

3.2.6 Multi-Level Control

The District recommends that no more than two levels of controls, in addition to the WQCV controls, be used for on-site detention facilities. These levels can be the 10- or 100-year storm, in combination with the 2-, 5- or the 10-year storm, as appropriate. More levels of control may appear to provide increased protection, but the added complexity of design and the questionable accuracy of results rarely justifies it. As an alternative to this three-level control recommended above, one can chose the two-level control offered by Sections 3.2.4 and 3.2.5 above to achieve broader levels of peak runoff control and possibly less expensive outlet design.

3.2.7 On-Site Detention and UDFCD 100-year Floodplain Management Policy

While UDFCD has confidence in the ability of many on-site detention basins to control peak flow rates to predevelopment level for small urban catchments, this is not the case for larger watersheds. The complexities of predicting where each on-site detention basin is going to be installed as areas urbanize, how each is going to be designed and built, and then applying the detention routing technology on an evolving and diffuse system of control facilities is beyond anyone's ability to assess or predict. In addition, the UDFCD has no ability or power to insure that all on-site detention facilities will continue to be maintained and their function will not deteriorate over time. In fact, evidence suggests to the contrary (Prommersberger, 1984) that many on-site detention facilities do not receive needed maintenance and do not provide the original design function over time. Prommersberger (1984) found that many, in fact, have never been built as designed. In response to these complexities of implementation and future maintenance uncertaities, the UDFCD adheres to the following policies when developing hydrology for the delineation and regulation of the 100-year flood hazard zones within its boundaries:

1. Hydrology has to be based on fully developed watershed condition as estimated to occur, at a minimum, over the next 50 years.
2. No on-site detention basin will be recognized in the development of hydrology unless:

a. It serves a watershed that is larger than 130-acres, and
b. It provides a regional function, and
c. It is owned and maintained by a public agency, and
d. The public agency has committed itself to maintain the detention facility so that it continues to operate in perpetuity as designed and built.

Photograph SO-4—This on-site dry detention facility (note short concrete dam) promotes pollutant removal in smaller runoff events.

These policies are for the definition and administration of the 100-year floodplain and floodway zones and the design of facilities along major drainageways. They are not intended to discourage communities from using on-site detention, including the EURV control (i.e., Full-Spectrum Detention) discussed above. On-site detention can be very beneficial for stormwater quality and quantity management, reducing the sizes of local storm sewers and other conveyances, and providing a liability shield (defense) when needing to address the issue of keeping stormwater-related damages from increasing to downstream properties as lands are developed. However, unless detention is regional in nature with a government having property rights to operate and maintain it in perpetuity, and is designed in accordance with an approved master plan, it will not be considered eligible for District's maintenance assistance program (see Chapter 5 for maintenance eligibility discussion). Furthermore, Colorado law requires detention be provided to control the 100-year peak flow for all new development in the unincorporated portions of all counties.

3.3 Design Storms for Sizing Storage Volumes

Typically, more than one design storm usually is controlled when designing detention or retention facilities. Water quality storage and release is based on the recommendations in Volume 3 of this *Manual*. For drainage and flood control design, the 2-, 5-, 10-, 25- and 100-year design storms are often considered and used, as required by local municipality. Sizing may sometimes be driven by downstream conveyance system capacities and public safety concerns in addition to standard local detention sizing requirements. Sizing of emergency spillways may also require the use of design storms larger than the 100-year storm. What follows is a thumbnail description of the factors to consider for each.

Photograph SO-5—Multipurpose detention facilities are strongly encouraged, as they often become community focal points.

3.3.1 Water Quality Capture Volume

This was discussed in detail under Sections 3.1.3 and 3.2.4 for facilities that include quantity and quality storage, and the reader is referred to them. The specific recommendations for the sizing of the WQCV are given in Volume 3 of this *Manual*.

3.3.2 Drainage and Flood Control

Sizing of storage facilities and outlet works for flood control purposes is generally based on whether the facility is on-site or regional. For an individual development sites, local municipalities will dictate which design storms need to be addressed. On a watershed level, full system master planning studies are needed to identify the appropriate release rates for various design storms. Whenever a District-approved master plan recommends detention sites and release rates, or on-site detention/retention storage and release rates, this sizing and rates should be used in final design of detention/retention facilities. Other considerations that have to be taken into account include downstream system stability, the drainageway's capacity to convey discharges from the detention/retention facility in combination with the downstream runoff contributions, potential for flood damages to downstream properties, and other factors that may be specific to each situation.

3.3.3 Spillway Sizing

The overflow spillway of a storage facility should be designed to pass flows in excess of the design flow of the outlet works. When the storage facility falls under the jurisdiction of the Colorado State Engineer's Office (SEO), the spillway's design storm is prescribed by the SEO (SEO 1988). If the storage facility is not a jurisdictional structure, the size of the spillway design storm should be based upon the risk and consequences of a facility failure. Generally, embankments should be fortified against and/or have spillways that, at a minimum, are capable of conveying the total not-routed peak 100-year storm discharge from a fully developed total tributary catchment, including all off-site areas, if any. Detailed analysis, however, of downstream hazards should be performed and may indicate that the embankment protection and/or spillway design needs to be for events much larger than the 100-year design storm.

3.3.4 Retention Facilities

A retention facility (a basin with a zero release rate or a very slow release rate) is used when there is no available formal downstream drainageway, or one that is grossly inadequate. When designing a retention facility, the hydrologic basis of design is difficult to describe because of the stochastic nature of rainfall events. Thus, sizing for a given set of assumptions does not ensure that another scenario produced by nature (e.g., a series of small storms that add up to large volumes over a week or two) will not overwhelm the intended design. For this reason, retention basins are not recommended as a permanent solution for drainage problems. They have been used in some instances as temporary measures until a formal system is developed downstream. When used, they can become a major nuisance to the community duo to problems that may include mosquito breeding, safety concerns, odors, etc.

When a retention basin is proposed as a temporary solution, the District recommends that it be sized to capture, as a minimum, the runoff equal to 1.5 times the 24-hour, 100-year storm plus 1-foot of freeboard. The facility also has to be situated and designed so that when it overtops, no human-occupied or critical structures (e.g., electrical vaults, homes, etc.) will be flooded, and no catastrophic failure at the facility (e.g., loss of dam embankment) will occur. It is also recommended that retention facilities be as shallow as possible to encourage infiltration and other losses of the captured urban runoff. When a trickle outflow can be accepted downstream or a small conduit can be built, provided and sized it in accordance with the locally approved release rates, preferably capable of emptying the full volume in 14 days or less.

3.4 Reservoir Routing of Storm Hydrographs for Sizing of Storage Volumes

The reservoir routing procedure for the sizing of detention storage volumes is more complex and time consuming than the use of empirical equations, FAA procedure or the simplified *Full Spectrum Detention* protocol. Its use requires the designer to develop an inflow hydrograph for the facility. This is generally accomplished using CUHP and UDSWM computer models as described in the RUNOFF chapter of this *Manual*. The hydrograph routing sizing method is an iterative procedure that follows the steps detailed below (Guo 1999b).

1. Select Location: The detention facility's location should be based upon criteria developed for the specific project. Regional storage facilities are normally placed where they provide the greatest overall benefit. Multi-use objectives such as the use of the detention facility as a park or for open space, preserving or providing wetlands and/or wildlife habitat, or others uses and community

needs influence the location, geometry, and nature of these facilities.

Photograph SO-6—Public safety is an important design consideration for detention facilities, including the potential need for safety/debris racks on outfall structures, as shown in this dry pond.

2. Determine Hydrology: Determine the inflow hydrograph to the storage basin and the allowable peak discharge from the basin for the design storm events. The hydrograph may be available in published district outfall system planning or a major drainageway master plan report. The allowable peak discharge is limited by the local criteria or by the requirements spelled out in a District-approved master plan.

3. Initial Storage Volume Sizing: It is recommended that the initial size of the detention storage volume be estimated using the modified FAA method described in Section 3.2.3, the Full Spectrum Detention protocols in Section 3.2.4 or the hydrograph volumetric method detailed in Section 3.4.1.

4. Initial Shaping of the Facility: The initial shape of the facility should be based upon site constraints and other goals for its use discussed under item 1, above. This initial shaping is needed to develop a stage-storage-discharge relationship for the facility. The design spreadsheets provided on the CD version of this *Manual* are useful for initial sizing.

5. Outlet Works Preliminary Design: The initial design of the outlet works entails balancing the initial geometry of the facility against the allowable release rates and available volumes for each stage of hydrologic control. This step requires the sizing of outlet elements such as a perforated plate for controlling the releases of the WQCV, orifices, weirs, outlet pipe, spillways, etc.

6. Preliminary Design: A preliminary design of the overall detention storage facility should be completed using the results of steps 3, 4 and 5, above. The preliminary design phase is an

iterative procedure where the size and shape of the basin and the outlet works are checked using a reservoir routing procedure and then modified as needed to meet the design goals. The modified design is then checked again using the reservoir routing and further modified if needed. Though termed “preliminary design,” the storage volume and nature and sizes of the outlet works are essentially in final form after completing this stage of the design. They may be modified, if necessary, during the final design phase.

7. Final Design: The final design phase of the storage facility is completed after the hydraulic design has been finalized. This phase includes structural design of the outlet structure, embankment design, site grading, a vegetation plan, accounting for public safety, spillway sizing and assessment of dam safety issues, etc.

Photograph SO-7—This retention pond has an embankment with upstream and downstream gentle sideslopes, which promotes dam safety and multipurpose use.

3.4.1 Initial Sizing

The intent of initial sizing of the facility is only to determine a starting point for the reservoir routing procedure that will be used to prepare the preliminary design for the facility. The initial sizing methods are not adequate for final design of the facility. Two methods for initial sizing are discussed below.

The Rational Formula-based modified FAA method may be used to find an initial storage volume for any size catchment. This technique for initial sizing yields best results when the tributary catchment area is less than 320 acres. The designer needs to understand that the design volumes may need to be adjusted significantly regardless of the tributary area once full hydrograph routing is performed.

It is also possible to use the inflow hydrograph, along with desired maximum release rates, to make an initial estimate of the required storage volume. This technique assumes that the required detention volume is equal to the difference in volume between the inflow hydrograph and the simplified outflow hydrograph. It is represented by the area between these two hydrographs from the beginning of a runoff event until the time that the allowable release occurs on the recession limb of the inflow hydrograph (Guo 1999b) (see Figure SO-1). The inflow hydrograph is generally obtained using CUHP/SWMM computer model computations. The outflow hydrograph can be approximated using a straight line between zero at the start of the runoff to a point where the allowable discharge is on the descending limb of the inflow hydrograph, T_p. The volume are calculated by setting up tabular calculations with the following columns:

1. The time T (in minutes) from 0 to T_p in 5-minute increments. T_p is the time (in minutes) where the descending limb of the inflow hydrograph is equal to the allowable release rate.

2. The inflow rate Q_i (cfs) to the detention basin corresponding to the time T. The inflow rate is an input value that is generally obtained from a CUHP/SWMM hydrologic analysis.

3. The outflow rate Q_o (cfs) calculated as:

$$Q_O = \frac{T}{T_p} Q_{po} \tag{SO-14}$$

   in which, Q_{po} is the peak outflow rate, where allowable peak outflow rate is determined from a District master plan, local ordinance, or other considerations described in Section 3.3.2.

4. The incremental storage volume = (column 2 – column 3) · 300 seconds.

5. The total storage volume calculated as the sum of the values in column 4.

Design Example 6.3 illustrates this procedure.

The **Hydrograph Worksheet** of the UD-Detention Spreadsheet performs these computations.

3.4.2 Initial Shaping

The initial shaping of the storage basin provides a starting point for defining the stage-storage relationship. The stage-storage relationship has to be refined during preliminary design phase of the project. The initial shaping is easiest when regular geometry (such as a triangle, rectangle, or elliptical) is used for approximation. The detention volume needed for any specific design storm is combined with site constraints (e.g., size or depth limitations, number of control stages, etc.) and the simplified formulas describing the basin geometry in order to develop an initial depth, length, and width for the basin. Design spreadsheets can be used to assist in preliminary shaping of the basin. This does not mean that the District encourages the use of storage facilities with uniform geometric properties. To the contrary, the District encourages designers to collaborate with landscape architects to develop storage facilities that are visually attractive, fit into the fabric of the landscape, and enhance the overall character of an area. However, using regular geometries can speed up initial sizing of a non-uniformly shaped facility.

3.4.3 Outlet Works Design

Outlet works are structures that control the release rates from storage facilities. Figure SO-2 illustrates three concepts for detention basin outlets. Two are from Volume 3 of the *Manual* that provides for a three-level flow control including the control of the Water Quality Capture Volume (WQCV). The other is for a two-level control designed for release of the *Excess Urban Runoff Volume* (EURV) over 72-hours and control of the 100-year peak flow to a specified maximum rate. Both include an orifice plate for release of the WQCV or the EURV. The first concept also provides for the 2- to 10-year (or other return period) storm controls through drop boxes and orifices at the bottom of the boxes. The other provides and orifice at the bottom of one drop box to control the 100-year (or other return period) release rate. The weir length of the drop box is best oversized after reducing its length by the trash rack bars so as not to become the primary control when the trash rack has some clogging. The goal is to have the orifice at the bottom of the box and in front of the outlet pipe exercise the desired flow control at the maximum stage in the basin.

The hydraulic capacity of the various components of the outlet works (orifices, weirs, pipes) can be determined using standard hydraulic equations. The discharge pipe of the outlet works functions as a culvert. See the CULVERTS chapter of the *Manual* for guidance regarding the calculation of the hydraulic capacity of outlet pipes. The following discussion regarding weirs and orifices is adapted from *Urban Drainage Design Manual*, Hydraulic Engineering Circular No. 22 (Brown, Stein, and Warner 1996). A rating curve for the entire outlet can be developed by combining the rating curves developed for each of the components of the outlet and then selecting the most restrictive element that controls a given stage for determining the composite total outlet rating curve.

Design aids for the design of basins and outlet works are provided on several of the worksheets in the UD-Detention Spreadsheet available for downloading from the District's web site.

3.4.3.1 Orifices

Multiple orifices may be used in a detention facility, and the hydraulics of each can be superimposed to develop the outlet-rating curve. For a single orifice or a group of orifices, as illustrated in Figure SO-3a, orifice flow can be determined using Equation SO-15.

$$Q = C_o A_o (2gH_o)^{0.5} \tag{SO-15}$$

in which:

Q = the orifice flow rate through a given orifice (cfs)

C_o = discharge coefficient (0.40 – 0.65)

A_o = area of orifice (ft^2)

H_o = effective head on each orifice opening (ft)

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

If the orifice discharges as a free outfall, the effective head is measured from the centroid of the orifice to the upstream water surface elevation. If the downstream jet of the orifice is submerged, then the effective head is the difference in elevation between the upstream and downstream water surfaces.

For square-edged, uniform orifice entrance conditions, a discharge coefficient of 0.61 should be used. For ragged edged orifices, such as those resulting from the use of an acetylene torch to cut orifice openings in corrugated pipe, a value of 0.4 should be used.

3.4.3.2 Weirs

Relationships for sharp-crested, broad-crested, V-notch, and proportional weirs are provided below:

Sharp-Crested Weirs: Typical sharp-crested weirs are illustrated in Figures SO-4a through SO-4d. Equation SO-16 provides the discharge relationship for sharp-crested weirs with no end contractions (illustrated in Figure SO-4a):

$$Q = C_{scw} L H^{1.5} \qquad \text{(SO-16)}$$

in which:

Q = discharge (cfs)

L = horizontal weir length (ft)

H = head above weir crest excluding velocity head (ft)

H_c = height of weir crest above the approach channel bottom (ft)

C_{scw} = 3.27 + 0.4 (H/H_c)

The value of the coefficient C_{SCW} varies with the ratio H/H_c (see Figure SO-4c for definition of terms). When the ratio H/H_c less than 0.3, a constant C_{SCW} of 3.33 is often used.

Equation SO-17 provides the discharge equation for sharp-crested weirs with end contractions (illustrated in Figure SO-4b). As stated above, the value of the coefficient C_{SCW} varies with the ratio H/H_c and becomes a constant 3.33 when H/H_c is less than 0.3.

$$Q = C_{scw}(L - 0.2H)H^{1.5} \qquad \text{(SO-17)}$$

Another form of sharp crested weir is the *Cipoletti* weir. It is a trapezoidal weir with sides slope at 1-horizontal to 4-vertical. The equation for this weir is:

$$Q = 3.367 \cdot L \cdot H^{1.5} \qquad \text{(SO-17a)}$$

Sharp-crested weirs will be affected by submergence when the tailwater rises above the weir crest elevation, as shown in Figure SO-4d. The result will be that the discharge over the weir will be reduced.

The discharge equation for a submerged sharp-crested weir is:

$$Q_s = Q_r \left[1 - \left(\frac{H_2}{H_1} \right)^{1.5} \right]^{0.385} \qquad \text{(SO-18)}$$

in which:

Q_s = submerged flow (cfs)

Q_r = un-submerged weir flow from Equation SO-15 or SO-16 (cfs)

H_1 = upstream head above crest (ft)

H_2 = downstream head above crest (ft)

Flow over the top edge of the riser pipe can initially be treated, until the throat of the pipe takes over the hydraulic control, as flow over a sharp-crested weir with no end constrictions. Equation SO-17 should be used for this case.

Broad-Crested Weir: The equation typically used for a broad-crested weir is:

$$Q = C_{BCW} L H^{1.5} \qquad \text{(SO-18)}$$

in which:

Q = discharge (cfs)

C_{BCW} = broad-crested weir coefficient (This ranges from 2.38 to 3.32 as per Brater and King (1976). A value of 3.0 is often used in practice.)

L = broad-crested weir length (ft)

H = head above weir crest (ft)

V-Notch Weir: The discharge through a V-notch or triangular weir is shown in Figure SO-5 and can be calculated from the following equation:

$$Q = C_t \tan\left(\frac{\theta}{2} \right) H^{2.5} \qquad \text{(SO-20)}$$

in which:

C_t = Coefficient for Triangular Weir taken from the table below

Q = discharge (cfs)

θ = angle of V-notch in degrees

H = head above the apex of V-notch (ft)

Depth H in feet	Coefficient C_t for V-Notch Angle θ			
	20^o	45^o	60^o	90^o
0.2	2.81	2.66	2.62	2.57
0.4	2.68	2.57	2.53	2.51
0.6	2.62	2.53	2.51	2.49
0.8	2.60	2.52	2.50	2.48

3.4.4 Preliminary Design

The preliminary design stage consists of refining the design of the basin (size, shape and elevation) and outlet structure (type, size, configuration). At this time, the basin's bottom may be sloped as needed to provide drainage to the outlet and/or trickle channel to prevent the bottom from becoming boggy and habitat for mosquito breeding. Preliminary design is an iterative process that determines the detention basin's outflow characteristics given the stage-storage-discharge parameters of the basin and the inflow hydrograph(s). The stage-storage-discharge characteristics are modified as needed after each model run until the outflow from the basin meets the specified flow limit. No description of the theory of reservoir routing is provided in this *Manual*. The subject is well described in many hydrology reference books (Viessman and Lewis 1996; Guo 1999b).

Reservoir modeling can be carried out in a number of different ways. The EPA SWMM model provides for reservoir routing. The modeler provides a stage-discharge relationship for a reservoir outlet junction and the stage-surface area relationship for the storage junction of the model or the detention facility. The stage-surface area relationship is determined by finding the water surface areas of the basin at different depths or elevations, which are then used by the model to calculate the incremental volumes used as the stage rises and falls. If the storage facility is modeled as part of a larger system being addressed through a master planning effort, the SWMM model must be used. For the design of individual detention sites that goes into greater detail than used in watershed master planning model, the District's UD POND software provides a reliable and relatively easy tool to facilitate detention basin design.

3.4.5 Final Design

The final design of the storage facility entails detailed hydraulic, structural, geotechnical, and civil design. This includes detailed grading of the site, embankment design, spillway design, outlet works hydraulic and structural design, trash rack design, consideration of sedimentation and erosion potential within and downstream of the facility, liner design (if needed), etc. Collaboration between geotechnical engineers, structural engineers, hydrologic and hydraulic engineers, land planners, landscape architects, biologists, and/or other disciplines is encouraged during the preliminary and final design phases.

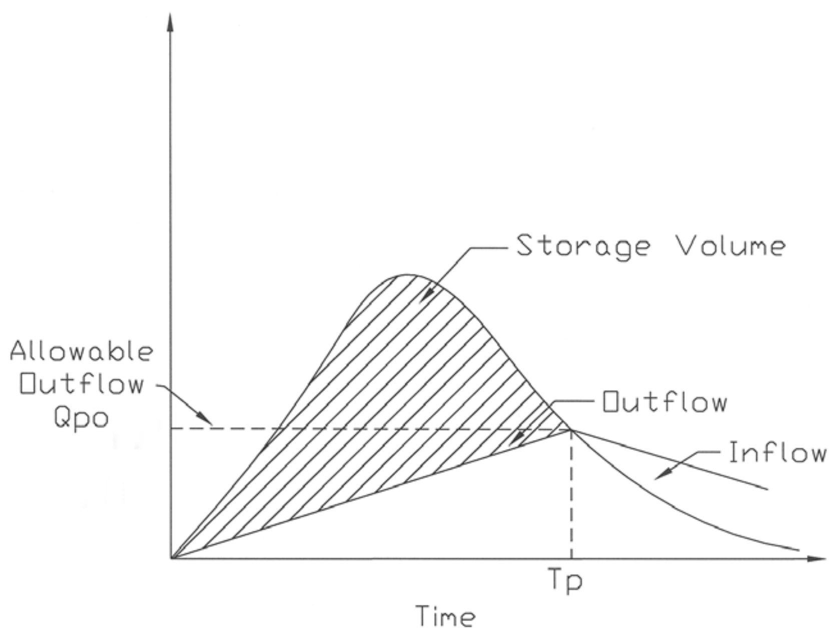
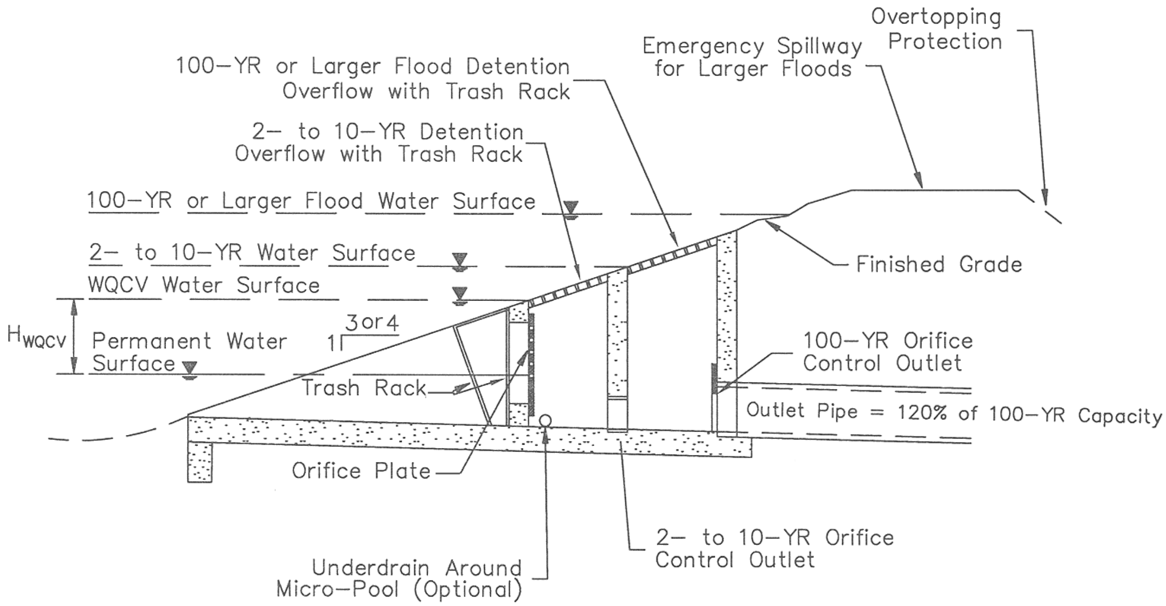
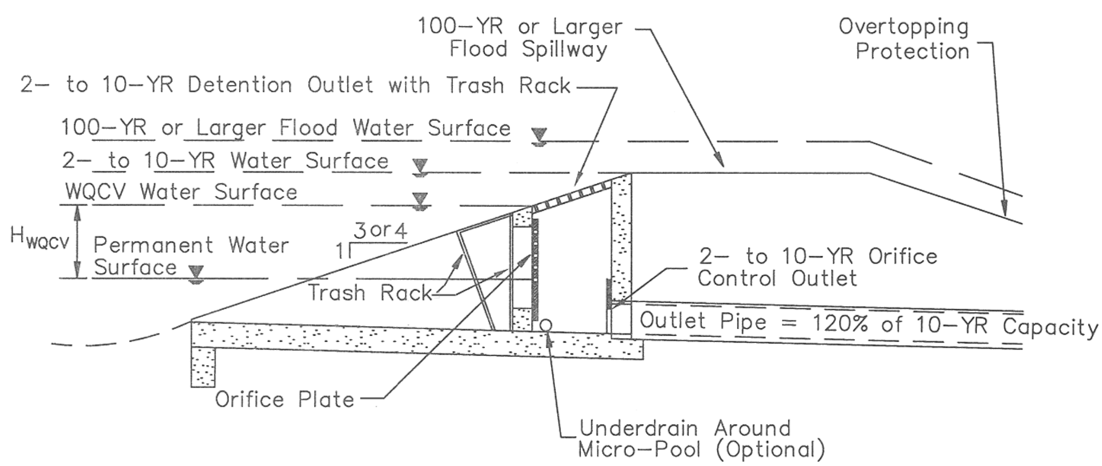


Figure SO-1—Hydrograph Volumetric Method for Initial Basin Pre-Sizing

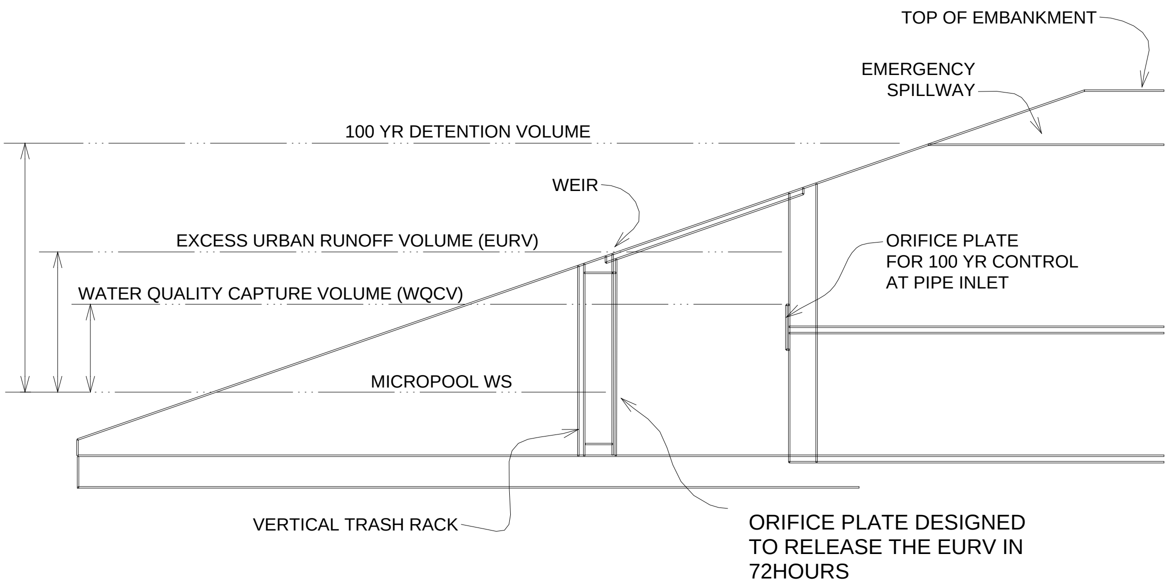


Drop Box Outlet Option



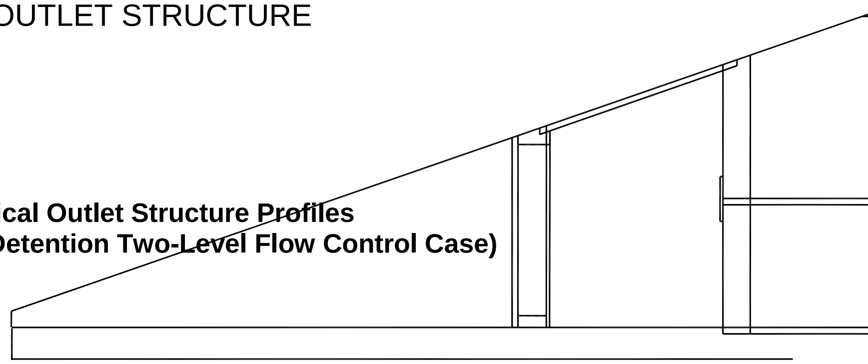
Overtopping Spillway Option

Figure SO-2—Typical Outlet Structure Profiles
(Sheet 1 of 2: Three-Level Peak Flow Control Case)



EXAMPLE OUTLET STRUCTURE

Figure SO-2—Typical Outlet Structure Profiles
(Sheet 2 of 2: Full Spectrum Detention Two-Level Flow Control Case)



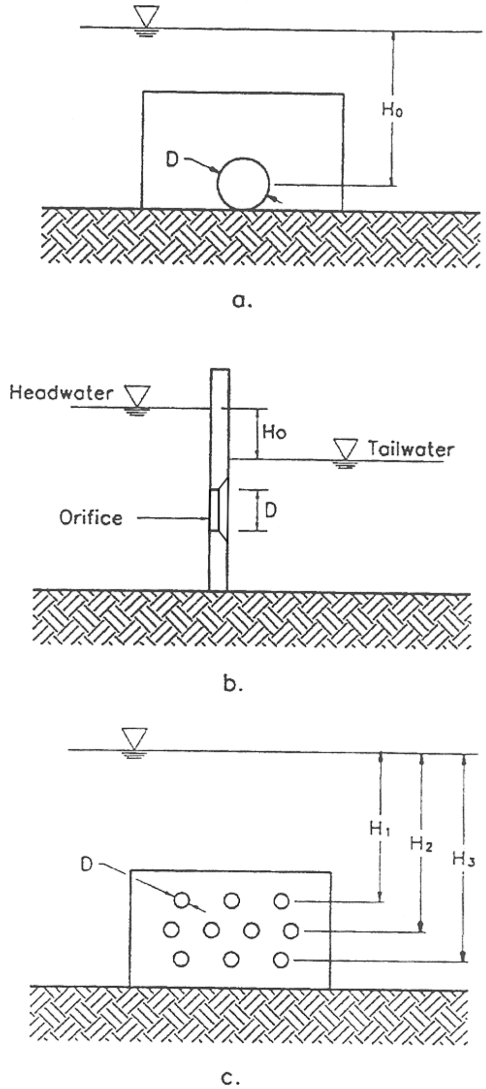


Figure SO-3—Illustration Defining Hydraulic Head for Flow through Orifice(s)

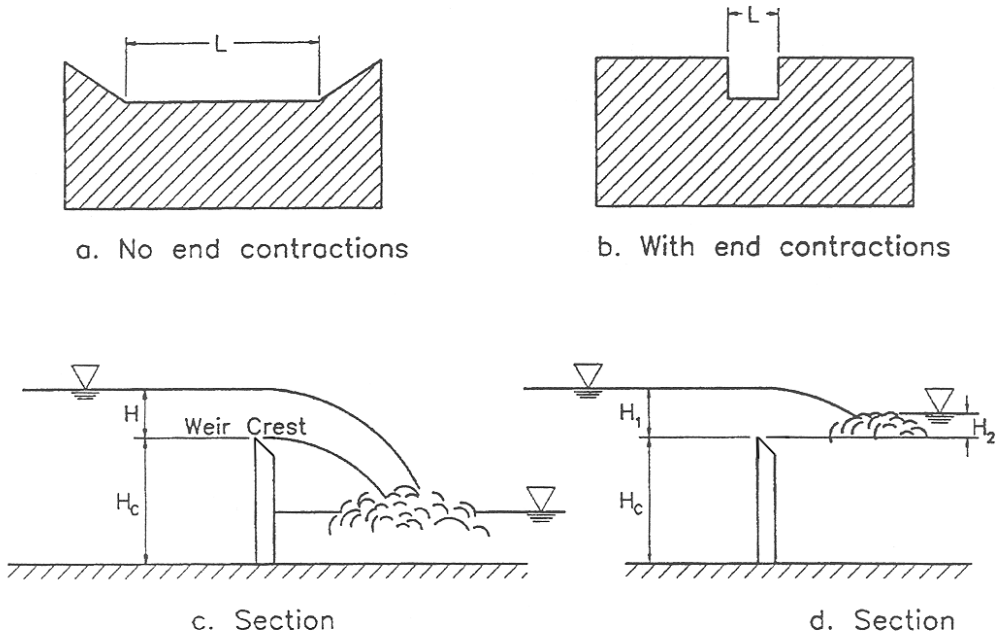


Figure SO-4—Sharp-Crested Weirs

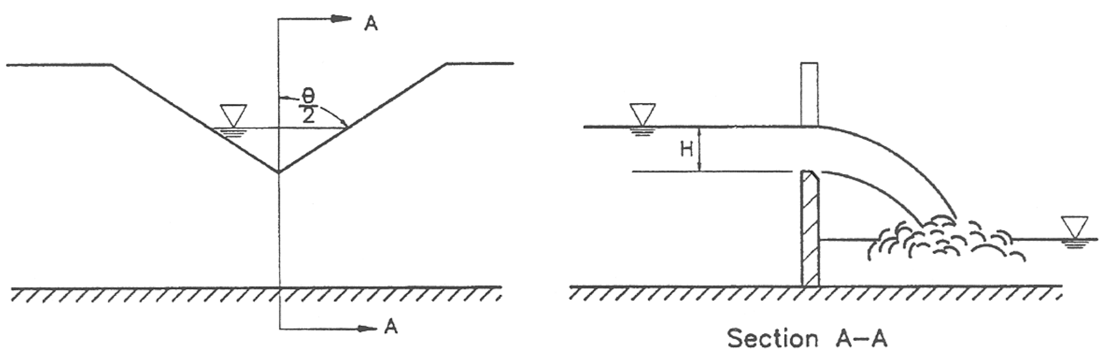


Figure SO-5—V-Notch Weir

4.0 FINAL DESIGN CONSIDERATIONS

Final design of a storage facility should recognize the kinds of considerations described in this section. It is beyond the scope of this *Manual* to provide detailed dam design guidance. There are many excellent references in this regard such as *Design of Small Dams* (U.S. Bureau of Reclamation 1987). The District urges all designers to review and adhere to the guidance in such references because the failure of even small embankments can have serious consequences for the public and the municipalities downstream of the embankment. General guidelines for the final design phase of detention or retention facilities follows.

4.1 Storage Volume

The determination of storage volume for quantity control is described earlier in this chapter. If the storage facility includes a WQCV, the appropriate flood storage volume should be provided, one that is in addition to the WQCV, as discussed under Sections 3.1.3 and 3.2.4. Determination of the WQCV is described in Volume 3 of the *Manual*. In the case of on-site detention, if the Excess Urban Runoff Volume is to be provided (i.e., Full Spectrum Detention) in conjunction with the 100-year volume obtained using empirical equations, no additional volume for WQCV needs to be provided within the 100-year basin. When using the Full Spectrum Detention concept with regional detention, the flood control volume has to be calculated using full hydrograph routing procedures.

4.2 Potential for Multiple Uses

Whenever desirable and feasible, incorporate water quality detention into a larger flood control facility. Also, when feasible, provide for other urban uses such as active or passive recreation and wildlife habitat. If multiple uses are being contemplated, use the multiple-stage detention basin to limit inundation of passive recreational areas to one or two occurrences a year and active recreation areas to once every two years. Generally, the area within the WQCV is not well suited for active or passive recreation facilities such as ballparks, playing fields, picnic areas, wildlife habitat, or hiking trails. These are best located above the water quality storage level.

4.3 Geometry of Storage Facilities

The geometry of a storage facility depends on specific site conditions such as adjoining land uses, topography, geology, preserving/creating wildlife habitat, volume requirements, etc. Several key features should be incorporated in all storage facilities located within the District (see Figure SO-6). These include (a) 4:1 or flatter side slopes of all banks, (b) low-flow or trickle-flow channel unless a permanent pool takes its place, (c) forebay, (d) pond bottom sloped at least 1.0 percent to drain toward the low-flow or trickle-flow channel or the outlet, (e) micro pool at the outlet for Extended Detention Basins in Volume 3 of this *Manual*, and (f) emergency spillway or fortification of the embankment to prevent catastrophic failure when overtopped.

It is desirable to shape the water quality portion of the facility with a gradual expansion from the inlet and a gradual contraction toward the outlet, thereby minimizing short-circuiting. Storage facility geometry and layout are best developed in concert with a land planner/landscape architect.

4.4 Embankments and Cut Slopes

If the storage facility is "jurisdictional," namely, subject to regulation by the Colorado State Engineer's Office (SEO), the embankment shall be designed, constructed and maintained to meet SEO most-current criteria for jurisdictional structures. The design for an embankment of a stormwater detention or retention storage facility should be based upon a site-specific engineering evaluation. In general, the embankment should be designed to not catastrophically fail during the 100-year and larger storms that the facility may encounter. The following criteria apply in many situations (ASCE and WEF 1992):

1. Side Slopes—For ease of maintenance, the side slopes of the embankment should not be steeper than 3H:1V, with 4H:1V preferred. The embankment's side slopes should be well vegetated, and soil-riprap protection (or the equivalent) may be necessary to protect it from wave action on the upstream face, especially in retention ponds.

2. Freeboard—The elevation of the top of the embankment shall be a minimum of 1 foot above the water surface elevation when the emergency spillway is conveying the maximum design or emergency flow. When the embankment is designed to survive its overtopping without failure, freeboard requirements may be waived. When relevant, all SEO dam safety criteria must be carefully considered when determining the freeboard capacity of an impoundment.

3. Settlement—The design height of the embankment should be increased by roughly 5 percent to account for settlement. All earth fill should be free from unsuitable materials and all organic materials such as grass, turf, brush, roots, and other organic material subject to decomposition. The fill material in all earth dams and embankments should be compacted to at least 95 percent of the maximum density based on the Modified Proctor method of ASTM D698 testing.

4. Emergency Spillway—An emergency spillway will often be needed to convey flows that exceed the primary outlet capacity, unless the embankment is designed to convey overtopped flows without failure (e.g., buried soil cement, grouted boulders, concrete walls with splash pads, etc.).

4.5 Linings

A storage facility may require an impermeable clay or synthetic liner for a number of reasons. Stormwater detention and retention facilities have the potential to raise the groundwater level in the vicinity of the basin. If the basin is close to structures or other facilities that could be damaged by raising the groundwater level, consider lining the basin with an impermeable liner. An impermeable liner may also be warranted in a retention pond where the designer seeks to limit seepage from a permanent pool.

Alternatively, there are situations where the designer may seek to encourage infiltration of stormwater into the ground. In this situation, a layer of permeable material may be warranted.

4.6 Inlets

Inlets to the detention facility should incorporate energy dissipation to limit erosion. They should be designed in accordance with drop structure or impact stilling basin criteria in the HYDRAULIC STRUCTURES chapter of this *Manual*, or using other approved energy dissipation structures. In addition, incorporate forebays or sediment traps at all inflow points to detention facilities to deposit coarse sediment being delivered by stormwater to the facility. These forebays will need regular maintenance to lessen the sediment being transported and deposited on the storage basin's bottom.

4.7 Outlet Works

Outlet works should be sized and structurally designed to release at the specified flow rates without structural or hydraulic failure. The design guidance for outlet works used for water quality purposes is included in Volume 3 of the *Manual* and for full-spectrum detention earlier in this chapter.

4.8 Trash Racks

Provide trash racks of sufficient size that do not interfere with the hydraulic capacity of the outlet. See Figure SO-7 for minimum trash rack sizes.

4.9 Vegetation

The type of grass used in vegetating a newly constructed storage facility is a function of the frequency and duration of inundation of the area, soil types, whether native or non-native grasses are desired, and the other potential uses (park, open space, etc.) of the area. A planting plan should be developed for new facilities to meet their intended use and setting in the urban landscape. Generally, trees and shrubs are not recommended on dams or fill embankments (see the REVEGETATION chapter). However, use of trees on the sides of detention basins will not interfere with their flood control operation of increase maintenance need significantly. Also, sparse planting of tree on bottoms of larger regional detention basins may also be acceptable as long as they are not located near inlets and outlet or on the emergency spillway(s) and will not interfere significantly with maintenance. At the same time use of shrubs on the banks and bottom, while not affecting the flood routing, can increase maintenance significantly by providing traps for debris that are difficult to clean and obstructions for the mowing of grasses.

4.10 Operation and Maintenance

Maintenance considerations during design include the following (ASCE and WEF 1992).

1. Use of flat side slopes along the banks and the installation of landscaping that will discourage

entry (thick, thorny shrubs) along the periphery near the outlets and steeper embankment sections are advisable. Also, use of safety railings at vertical or very steep structural faces is needed to public safety. If the impoundment is situated at a lower grade than and adjacent to a highway, installation of a guardrail is in order. Providing features to discourage public access to the inlet and outlet areas of the facility should be considered.

2. The facility should be accessible to maintenance equipment for removal of silt and debris and for repair of damages that may occur over time. Easements and/or rights-of-way are required to allow access to the impoundment by the owner or agency responsible for maintenance.
3. Bank slopes, bank protection needs, and vegetation types are important design considerations for site aesthetics and maintainability.
4. Permanent ponds should have provisions for complete drainage for sediment removal or other maintenance. The frequency of sediment removal will vary among facilities, depending on the original volume set aside for sediment, the rate of accumulation, rate of growth of vegetation, drainage area erosion control measures, and the desired aesthetic appearance of the pond.
5. For facilities designed for multipurpose use, especially those intended for active recreation, the play area might need special consideration during design to minimize the frequency and periods of inundation and wet conditions. It may be advisable to provide an underground tile drainage system if active recreation is contemplated.
6. Adequate dissolved oxygen supply in ponds (to minimize odors and other nuisances) can be maintained by artificial aeration. Use of fertilizer and EPA approved pesticides and herbicides adjacent to the permanent pool pond and within the detention basin should be controlled.
7. Secondary uses that would be incompatible with sediment deposits should not be planned unless a high level of maintenance will be provided.
8. French drains or the equivalent are almost impossible to maintain, and should be used with discretion where sediment loads are apt to be high.
9. Underground tanks or conduits designed for detention should be sized and designed to permit pumping or multiple entrance points to remove accumulated sediment and trash.
10. All detention facilities should be designed with sufficient depth to allow accumulation of sediment for several years prior to its removal.
11. Permanent pools should be of sufficient depth to discourage excessive aquatic vegetation on the bottom of the basin, unless specifically provided for water quality purposes.

12. Often designers use trash racks and/or fences to minimize hazards. These may become trap debris, impede flows, hinder maintenance, and, ironically, fail to prevent access to the outlet. On the other hand, desirable conditions can be achieved through careful design and positioning of the structure, as well as through landscaping that will discourage access (e.g., positioning the outlet away from the embankment when the permanent pool is present, etc.). Creative designs, integrated with innovative landscaping, can be safe and can also enhance the appearance of the outlet and pond. Such designs often are less expensive initially.

13. To reduce maintenance and avoid operational problems, outlet structures should be designed with no moving parts (i.e., use only pipes, orifices, and weirs). Manually and/or electrically operated gates should be avoided. To reduce maintenance, outlets should be designed with openings as large as possible, compatible with the depth-discharge relationships desired and with water quality, safety, and aesthetic objectives in mind. One way of doing this is to use a larger outlet pipe and to construct orifice(s) in the headwall to reduce outflow rates. Outlets should be robustly designed to lessen the chances of damage from debris or vandalism. The use of thin steel plates as sharp-crested weirs is best avoided because of potential accidents, especially with children. Trash/safety racks must protect all outlets.

14. Clean out all forebays and sediment traps on a regular basis or when routine inspection shows them to be ¼ to ½ full.

See Volume 3 of this *Manual* for additional recommendations regarding operation and maintenance of water quality related facilities, some of which also apply to detention facilities designed to meet other objectives.

Photograph SO-8—Maintenance considerations must be carefully accounted for during design, with sediment accumulation a particular concern.

4.11 Access

All weather stable access to the bottom, inflow, forebay, and outlet works areas shall be provided for maintenance equipment. Maximum grades should be no steeper than 10 percent, and a solid driving surface of gravel, rock, concrete, or gravel-stabilized turf should be provided.

4.12 Geotechnical Considerations

The designer must take into full account the geotechnical conditions of the site. These considerations may include issues related to embankment stability, geologic hazards, seepage, and other site-specific issues.

It may be necessary to confer with a qualified geotechnical engineer during both design and construction, especially for the larger detention and retention storage facilities.

4.13 Environmental Permitting and Other Considerations

The designer must take into account environmental considerations surrounding the facility and the site during its selection, design and construction. These can include regulatory issues such as (a) whether the facility will be located on a jurisdictional wetland, (b) whether the facility is to be located on a waterway that is regulated by the U.S. Army Corps of Engineers as a “Waters of the U.S.”, and (c) whether there are threatened and endangered species or habitat in the area.

There are also non-regulatory environmental issues that should be taken into account. Detention facilities can become breeding grounds for mosquitoes unless they are properly designed, constructed and maintained. Area residents may view riparian habitat destruction necessary for construction of the facility objectionably. Considerations of this kind must be carefully taken into account and early discussions with relevant federal, state and local regulators are recommended.

In addition, under Colorado Water Law, storage impoundments can be subject to regulation from a water rights perspective by the SEO. For larger facilities, particularly those with permanent pools, the designer is encouraged to check with the SEO or a qualified water rights attorney to determine which water rights regulations apply.

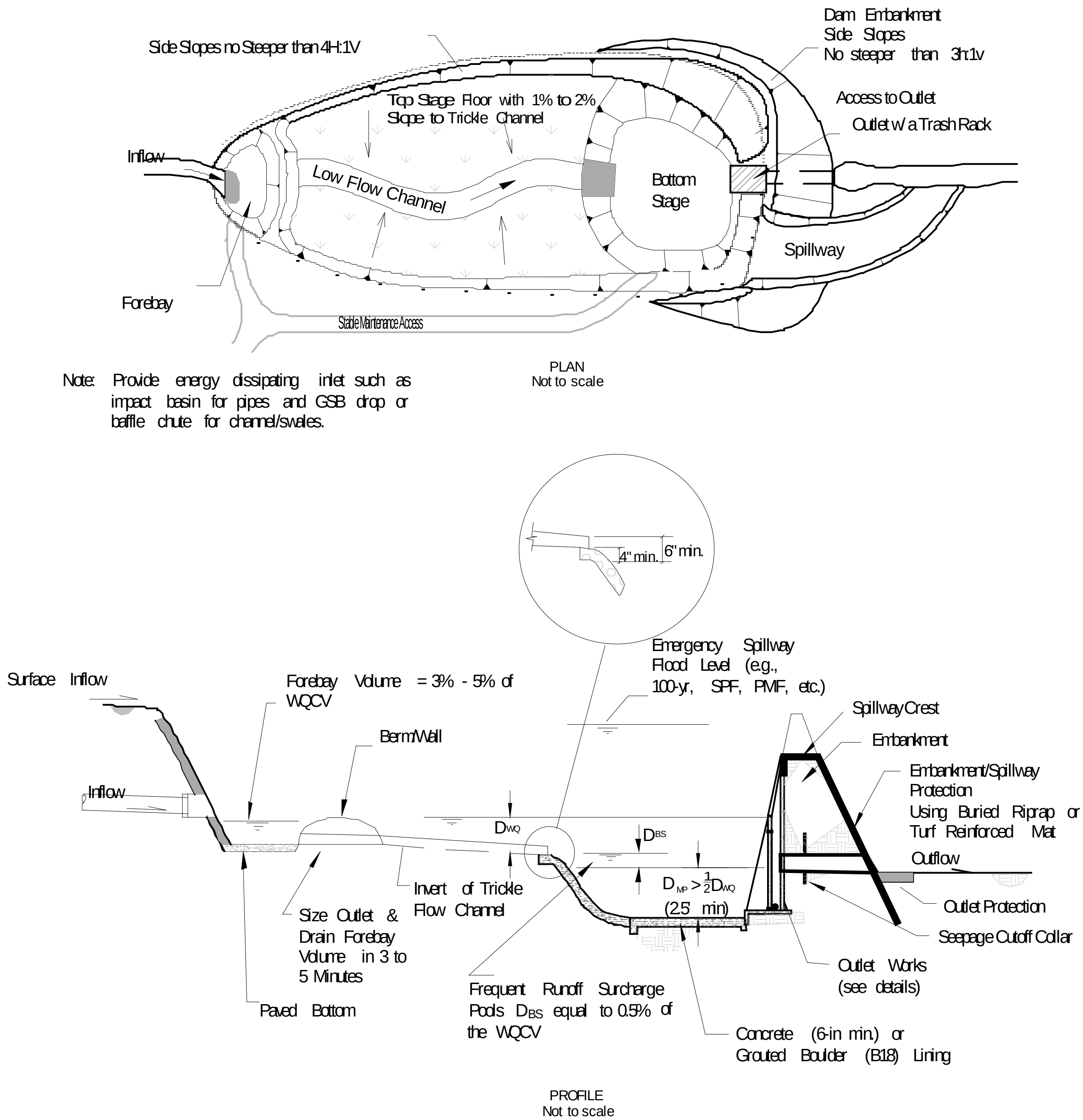


Figure SO-6—Plan and Profile of an Extended Detention Basin in a Flood Control Detention Basin

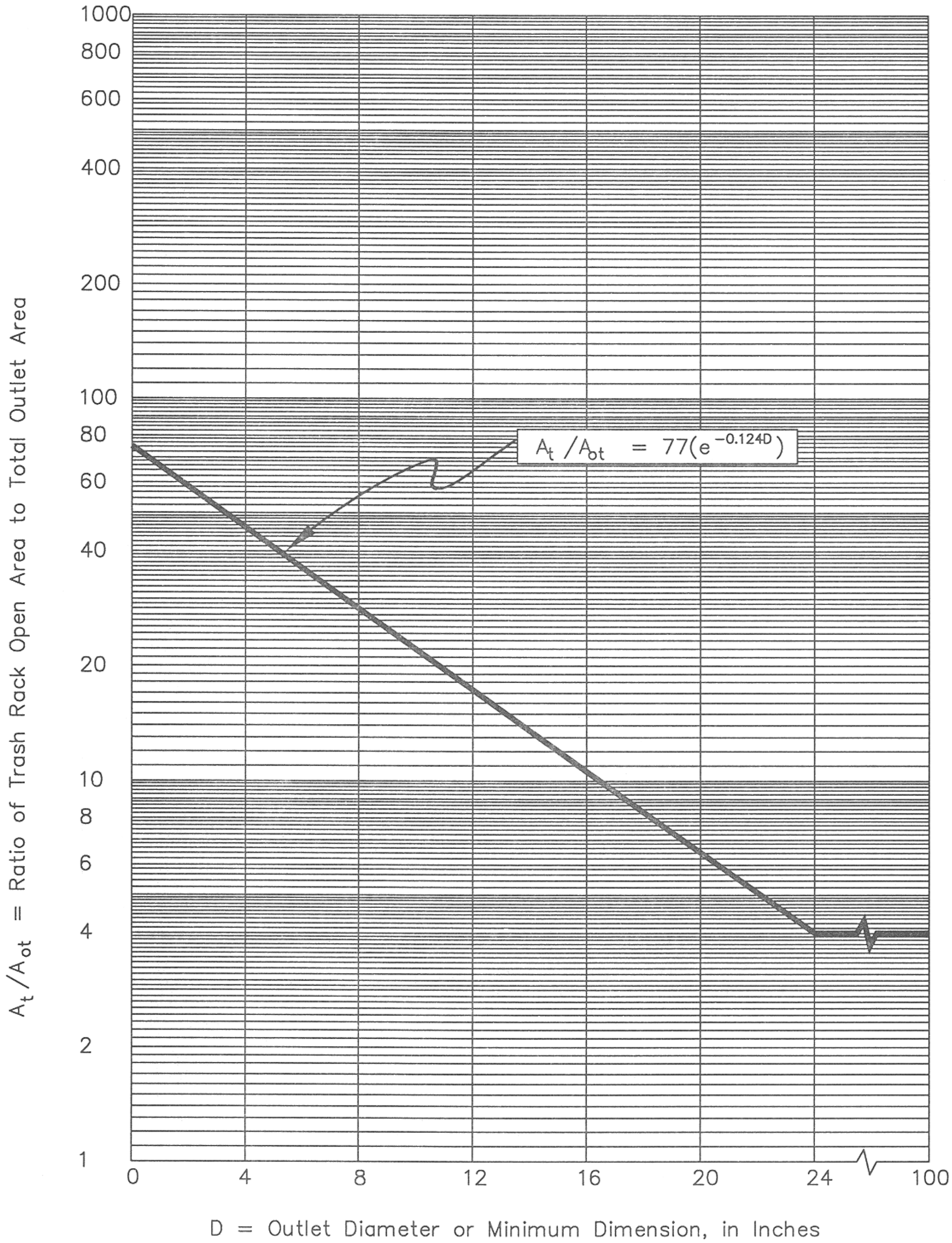


Figure SO-7—Minimum Trash Rack Open Area—Extended Range

Note: This figure was intentionally removed in December 2012 when equation SO-13 was improved and replaced. Use equation SO-13 for sizing EURV outlets.

Figure SO-8—Outlet Sizing for EURV Control with 72-hour Drain Time for On-Site Detention

5.0 DISTRICT MAINTENANCE ELIGIBILITY FOR DETENTION FACILITIES

The District has a program to assist local jurisdictions in the on-going maintenance of major drainage facilities including detention facilities. These guidelines change over time as budgets, priorities and needs of the community change. Check the District's Web site (www.udfcd.org) for the most-current maintenance eligibility requirements. Maintenance Eligibility Guidelines as of June 2001 are provided on the CD version of this *Manual*.

There are some common features for which the District's policy is unlikely to change over time. One is the requirement that the facility be owned by or be under control of a public body. "Public body" is defined as a local government (city or county), special district (such as a park district), or a metropolitan district that has a service plan that includes the maintenance and operation of drainage facilities. The public body has to have a reliable funding source to fund maintenance. Legal maintenance access to the detention facility must be made available to the District by the local jurisdiction in accordance with any of the following criteria:

1. The facility is owned by a public body that has accepted primary maintenance responsibility for it.
2. The ownership of the facility is by a private entity (such as a homeowners association owning common areas), but a body has accepted primary maintenance responsibility and has a maintenance access easement(s) that allows it to perform maintenance if the owner does not. Easements crossing individual lots are not acceptable.

6.0 DESIGN EXAMPLES

6.1 Example—Empirical Equations Sizing of a Detention Basin

Determine the required detention volume and allowable release rate for the 10-year and 100-year storm events for a 15-acre site that is in a jurisdiction that has adopted the empirical detention requirements and release rates shown in Section 3.2.1. The NRCS soil survey shows the site has hydrologic soil group B soils. The site will have a developed percentage imperviousness of 45%.

Examination of the District-approved master plan for the area indicates that the current empirical detention requirements for the area may be used. Examination of topographic mapping for the area indicates that no up-gradient off-site flows will traverse the site.

Determine the allowable release rates from Table SO-1:

10-year release rate = 0.23 · 15 acres = 3.45 cfs

100-year release rate = 0.85 · 15 acres = 12.75 cfs

Determine the 10-year required storage volume from Equations SO-1 and SO-3:

Using Equation SO-3, $K_{10} = \{(0.95 \cdot 45) - 1.9\}/1000 = 0.041$

Using Equation SO-1, $V_{10} = 0.041 \cdot 15$ acres = 0.61 acre-feet

The detention required for the 10-year storm is 0.61 acre-feet

Determine the 100-year required storage volume from Equations SO-1 and SO-2:

Using Equation SO-2, $K_{100} = \{(1.78 \cdot 45) - (0.002 \cdot 45^2) - 3.56\}/900 = 0.081$

Using Equation SO-1, $V_{100} = 0.081 \cdot 15$ acres = 1.21 acre-feet

The detention required for the 100-year storm is 1.21 acre-feet

6.2 Example—Rational Method Analysis

Use the FAA method to determine the required detention volume for the 100-year storm event for a 15-acre site that will have a developed percentage imperviousness of 45%. The NRCS soil survey shows the site has hydrologic soil group B soils. The allowable release rate from the basin has to be limited to the unit values in Table SO-1. The time of concentration has been calculated at 12 minutes. The 100-year, 1-hour point precipitation is 2.6 inches.

A runoff coefficient, C, of 0.51 is determined using Table RO-5 of the RUNOFF chapter (the 45% row and

100-year storm column of the type B soils table equals 0.51). The calculations are shown in spreadsheet form UD-Detention workbook in Table SO-2.

Table SO-2—FAA Method Calculations (From UD-Detention Workbook)

Determination of Detention Volume Using Modified FAA Method

Rainfall durations must be entered in an increasing order.

Rainfall Duration minutes (input)	Rainfall Intensity inch/hr (output)	Inflow Volume cubic feet (output)	Adjustment Factor (output)	Average Outflow cfs (output)	Outflow Volume cubic feet (output)	Storage Volume cubic feet (output)
0.00	12.02	0				
5.00	8.72	20,021	1.00	12.75	3,825	16,196
10.00	6.95	31,902	1.00	12.75	7,650	24,252
15.00	5.83	40,119	0.90	11.48	10,328	29,791
20.00	5.05	46,316	0.80	10.20	12,240	34,076
25.00	4.47	51,257	0.74	9.44	14,153	37,105
30.00	4.02	55,351	0.70	8.93	16,065	39,286
35.00	3.66	58,838	0.67	8.56	17,978	40,861
40.00	3.37	61,873	0.65	8.29	19,890	41,983
45.00	3.13	64,559	0.63	8.08	21,803	42,757
50.00	2.92	66,967	0.62	7.91	23,715	43,252
55.00	2.74	69,150	0.61	7.77	25,628	43,522
60.00	2.58	71,147	0.60	7.65	27,540	43,607
65.00	2.45	72,987	0.59	7.55	29,453	43,535
70.00	2.32	74,694	0.59	7.47	31,365	43,329
75.00	2.22	76,287	0.58	7.40	33,278	43,010
80.00	2.12	77,780	0.58	7.33	35,190	42,590
85.00	2.03	79,186	0.57	7.28	37,103	42,083
90.00	1.95	80,514	0.57	7.23	39,015	41,499
95.00	1.88	81,774	0.56	7.18	40,928	40,846
100.00	1.81	82,972	0.56	7.14	42,840	40,132
105.00	1.75	84,114	0.56	7.10	44,753	39,362
110.00	1.69	85,206	0.55	7.07	46,665	38,541
115.00	1.63	86,252	0.55	7.04	48,578	37,675
120.00	1.58	87,256	0.55	7.01	50,490	36,766
125.00	1.54	88,222	0.55	6.99	52,403	35,820
130.00	1.49	89,152	0.55	6.96	54,315	34,837
135.00	1.45	90,050	0.54	6.94	56,228	33,823
140.00	1.41	90,917	0.54	6.92	58,140	32,777
145.00	1.38	91,757	0.54	6.90	60,053	31,704
150.00	1.34	92,569	0.54	6.89	61,965	30,604
155.00	1.31	93,358	0.54	6.87	63,878	29,480
160.00	1.28	94,123	0.54	6.85	65,790	28,333
165.00	1.25	94,867	0.54	6.84	67,703	27,164
170.00	1.23	95,590	0.54	6.83	69,615	25,975
175.00	1.20	96,295	0.53	6.81	71,528	24,767
180.00	1.17	96,981	0.53	6.80	73,440	23,541

Stormwater Detention Volume (Cubic Feet) = 43,607

The required storage volume is 43,607 cubic feet (approx. 1.0 acre-foot). This compares to 1.21 acre-feet calculated for the same catchment in Design Example 6.1 using the empirical equations.

6.3 Example—Hydrograph Procedure Preliminary Sizing

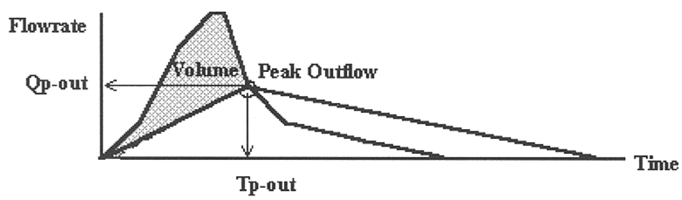
Use the hydrograph method to determine the preliminary size of a detention basin that will detain the 100-year peak flow to historic conditions for a catchment that has the following characteristics: area = 90 acres, length of catchment = 0.53 miles, length to centroid = 0.30 miles, impervious area = 67%, catchment slope = 0.0178 ft/ft, pervious retention = 0.35 inches, impervious retention = 0.05 inches, type B soils. 100-yr, 1-hour rainfall depth = 2.6 inches. The peak outflow is to be limited to the allowable unit release rates shown in Table SO-1.

The calculations are set up in tabular form similar to one illustrated in Table SO-3 that was taken from the UD-Detention workbook. The inflow hydrograph is calculated using the CUHP model and entered into the second column of the calculations. The preliminary sizing for the detention basin indicates a storage volume of 11.2 acre-feet.

Table SO-3—Detention Volume Estimate Using a Hydrograph
(From UD-Detention Workbook)

DETENTION VOLUME ESTIMATE USING A HYDROGRAPH

(For Catchments > 90 and < 160 acres)



a. Enter the inflow storm hydrograph with a time interval as:

time interval on hydrograph dt = 5.0 minutes

Time minutes (input)	Inflow hydrograph cfs (input)	**Outflow** Rising Hy cfs (output)	Incremental Volume acre-ft (output)	**Storage** Volume acre-ft (output)
0.0	0.00	0.00	0.00	0.00
5.0	0.00	4.78	0.00	0.00
10.0	3.00	9.56	0.00	0.00
15.0	16.00	14.34	0.01	0.01
20.0	41.00	19.13	0.15	0.16
25.0	84.00	23.91	0.41	0.58
30.0	179.00	28.69	1.04	1.61
35.0	312.00	33.47	1.92	3.53
40.0	333.00	38.25	2.03	5.56
45.0	289.00	43.03	1.69	7.25
50.0	240.00	47.81	1.32	8.58
55.0	196.00	52.59	0.99	9.56
60.0	160.00	57.38	0.71	10.27
65.0	135.00	62.16	0.50	10.77
70.0	115.00	66.94	0.33	11.10
75.0	88.00	71.72	0.11	11.22
80.0	64.00	76.50	0.00	11.22
85.0	47.00	----	----	----
90.0	36.00	----	----	----
95.0	29.00	----	----	----
100.0	25.00	----	----	----
105.0	23.00	----	----	----
110.0	21.00	----	----	----
115.0	20.00	----	----	----
120.0	20.00	----	----	----

b. You must provide the design information as:

Max. Allowable Peak Outflow Qp-out = 76.50 cfs
Time to Peak Outflow Tp-out= 80.00 minutes
(Qp-out,Tp-out) is a point on the recession of the inflow hydrograph.

C. Detention Storage Volume = 11.22 acre-ft

NOTE: THIS IS A FIRST APPROXIMATION ONLY

7.0 CHECKLIST

Criterion/Requirement	☐
If facility falls under State Engineer's jurisdiction, it must it meet all of State Engineer's requirements?	
Side slopes must be 4:1 or flatter.	
Embankment (dam fill) slopes must be 3:1 or flatter (4:1 or flatter preferred).	
Trickle channels are not required for retention ponds ("wet" ponds) and wetland basins, but the District will provide only limited maintenance assistance of these areas.	
The longitudinal slope for trickle channels shall be at least 0.4% for concrete bottoms and at least 1% for other bottoms.	
The pond bottom cross slope toward trickle channel or outlet shall be at least 1%.	
Maintenance access ramps to the pond bottom have at least 8 feet wide stabilized surface and have a 10%, or longitudinal flatter slope and turning radii that permit large maintenance equipment access.	
Provide an emergency spillway or embankment protection for flows that exceed primary outlet capacity.	
Provide a minimum 1-foot freeboard before embankment overtops.	
Outlet structures meter out the discharges as required by local municipality's criteria.	
Trash racks provided that do not interfere with the hydraulic capacity of the outlet.	
Tributary inflow points to the ponds have adequate energy dissipation and/or protection to prevent erosion.	
Designs consider the safety of the public.	
Pre-sedimentation forebay provided.	
WQCV is increased by 20% to account for sediment accumulation.	
Geotechnical considerations (embankment stability, geologic hazard, seepage) are taken into account and documented.	
Vegetation takes into account frequency and duration of inundation.	

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FLOOD PROOFING

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1.0 FLOOD PROOFING

1.1 Definition of Flood Proofing

Flood proofing is any combination of structural or nonstructural changes or adjustments incorporated in the design, construction, or alteration of individual buildings or properties that will reduce flood damages.

1.2 Overview of Flood-Proofing Methods

Some examples of flood proofing include the placement of walls or levees around individual buildings; elevation of buildings on fill, posts, piers, walls, or pilings; anchorage of buildings to resist floatation and lateral movement; watertight closures for doors and windows; reinforcement of walls to resist water pressure and floating debris; use of paints, membranes, and other sealants to reduce seepage of water; installation of check valves to prevent entrance of floodwaters at utility and sewer wall penetrations; and location of electrical equipment and circuits above expected flood levels.

1.2.1 Classification of Flood Proofing

Flood-proofing techniques can be classified on the basis of the type of protection that is provided as follows: (1) *permanent measures*—always in place, requiring no action if flooding occurs; (2) *contingent measures*—requiring installation prior to the occurrence of flood; and (3) *emergency measures*—improvised at the site when flooding occurs.

In the Denver metropolitan area, flood-proofing efforts should focus on permanent measures due to the rapid response of most of the Front Range stream systems. Contingent measures are more effective when combined with an early flood warning system or in areas not immediately adjacent to a stream channel.

1.2.2 FEMA Recommended Methods

The Federal Emergency Management Agency (FEMA) has published numerous references on the subject of flood proofing (FEMA 1984, 1986a, 1986b, 1991, 1993a, 1993b, 1993c, 1993d, 1993e, 1994, 1995, 1996). In several of these documents, FEMA outlines six methods of flood proofing as follows:

1. Elevation—Raising the structure so that the lowest floor is above the flood level.
2. Wet Flood Proofing—Making uninhabited portions of the structure resistant to flood damage and allowing water to enter during flooding.
3. Relocation—Moving the structure out of the floodplain to higher ground where it will not be exposed to flooding.
4. Dry Flood Proofing—Sealing the structure to prevent floodwaters from entering.
5. Levees and Floodwalls—Building a physical barrier around the structure to hold back floodwater.
6. Demolition—Tearing down the damaged structure and either rebuilding properly on the same

property or buying or building outside the floodplain.

1.3 Approach of *Manual* Relative to Flood-Proofing Guidance

Floodplain management includes all measurements for planning and actions that are needed to determine, implement, revise, and update comprehensive plans for the wise use of the floodplain and related water resources. This includes both corrective actions, as represented by most of the chapters of this *Manual*, and preventive actions as described in the POLICY and PLANNING chapters. Preventive measures cover a wide array of accepted and proven techniques ranging from floodplain regulation to flood forecasting to flood proofing. Due to the fact that flood proofing is often mentioned but little understood, this chapter is presented to assist drainage and flood control engineers in dealing effectively with existing development that is already flood prone.

1.4 Regulatory Considerations

Most regulations for flood proofing are based on the minimum standards of the National Flood Insurance Program (NFIP). The NFIP sets minimum regulatory standards for constructing, modifying, or repairing buildings located in the floodplain to keep flood losses to a minimum. The NFIP limits some flood proofing; for example, it prohibits obstructions, such as berms and floodwalls, in floodways.

The NFIP also requires flood proofing for a building that is substantially improved or substantially damaged. “Substantially damaged” is defined as “damage of any origin sustained by a structure whereby the cost of restoring the structure to its before damaged condition would equal or exceed 50 percent of the market value of the structure before the damage occurred.” Buildings that have been substantially damaged or are being substantially improved (renovated) must be elevated to or above the 100-year flood level. Nonresidential buildings must be elevated or dry flood proofed.

Other federal agencies, such as the U. S. Army Corps of Engineers (USACE), U. S. Geological Survey, and Natural Resources Conservation Service, also publish flood-proofing information, as do some state and local agencies. The USACE provides engineering and construction standards in the publication *Flood Proofing Regulations* (1995b). Additional USACE publications (1984, 1988, 1990, 1993, 1994, 1995a, 1996, 1998) provide information on case studies and detailed engineering applications of flood-proofing methods.

1.5 Flood Proofing In the Context of Overall Floodplain Management

Flood proofing is but one tool of an overall floodplain management strategy. With new development, the first option should always be to construct outside of the floodplain. If building outside of the floodplain is not practical for a site, then the structure should be constructed in compliance with local floodplain regulations. The remaining flood-proofing methods discussed in this chapter should be considered primarily for retrofitting existing structures.

2.0 WHEN TO FLOOD PROOF

2.1 How Flooding Can Damage Structures

To understand how flooding can damage a structure, there are six important flood characteristics: depth/elevation, flow velocity, frequency, rate of rise and rate of fall, duration, and debris load. The flood conditions at a particular site are determined largely by the combination of these characteristics.

2.1.1 Depth/Elevation of Flooding

The depth and elevation of flooding are so closely related that they can be viewed as a single characteristic for the purposes of this discussion. Flood depth is the height of the floodwater above the surface of the ground or other feature at a specific point. Flood elevation is the height of the floodwater above an established reference datum. The standard datums used by most federal agencies and many state and local agencies are the National Geodetic Vertical Datum (NGVD) and the North American Vertical Datum (NAVD); however, other datums are in use. The use of other datums is important because elevations of the ground, floodwaters, and other features cannot be meaningfully compared with one another unless they are based on the same datum.

When the elevation of the ground (or another surface such as the lowest floor of the building) and the elevation of the floodwater are both based on the same datum, the flood depth at any point is equal to the flood elevation at that point minus the elevation of the ground (or other surface) at that point. Figure FP-1 illustrates this relationship. An additional point to consider: ground elevations are established by surveys; flood elevations may be calculated, or they may be known from watermarks left by past floods.

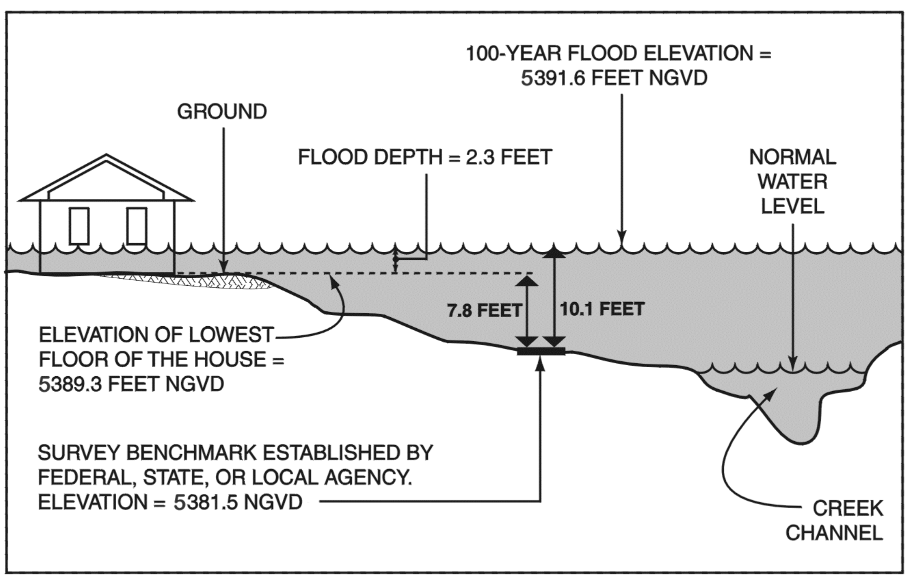

Figure FP-1—Schematic Representation of Flood Depth and Flood Elevation

The depth of flooding is important primarily because floodwaters, even when they are not moving, exert pressure on structural components such as walls and concrete floor slabs. The pressure exerted by still water is called hydrostatic pressure. It is caused by the weight of the water, so it increases as the depth of the water increases. As shown in Figure FP-2a, floodwater, including water that has saturated the soil under the building, pushes in on walls and up on floors. The upward force on floors is called buoyancy.

As shown in Figure FP-2b, water that has saturated the soil poses a special hazard for basement walls. Because hydrostatic pressure increases with the depth of the water, the pressure on basement walls is greater than the pressure on the walls of the upper floor, as indicated by the arrows in the figure. This pressure is made even greater by the weight of the saturated soil that surrounds the basement. The walls of buildings built according to standard construction practice are not designed to resist this pressure. Once the pressure exceeds the strength of the walls (including basement walls), it can push them in, cause extensive structural damage, and possibly cause the building to collapse.

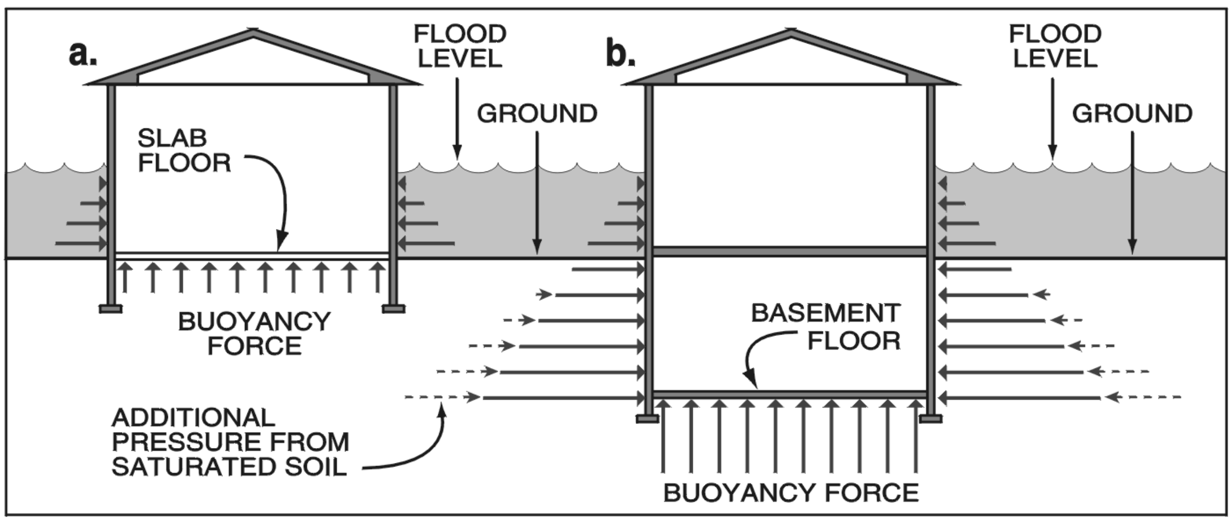

Figure FP-2 (a and b)—Hydrostatic Pressure Diagram With Dry Flood Proofing

Note that in the preceding illustration of hydrostatic pressure, no water is shown inside the building. If water is allowed to enter, the hydrostatic pressures on both sides of the walls and floor become the same, or equalized, and the walls are much less likely to fail (Figure FP-3).

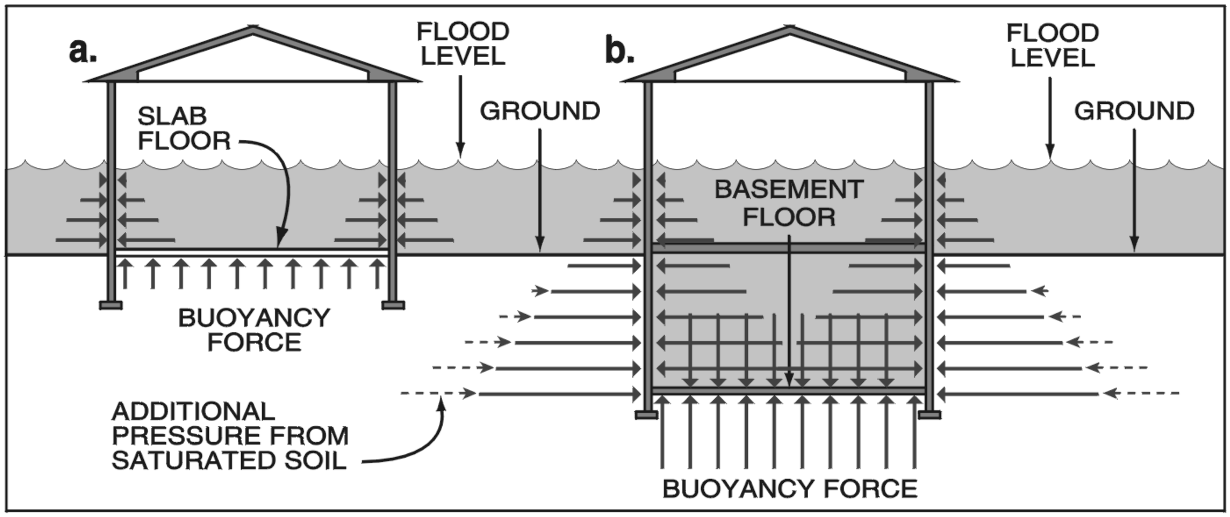

Figure FP-3—Hydrostatic Pressure Diagram With Wet Flood Proofing

2.1.2 Flow Velocity

Flow velocity is the speed at which floodwaters move. It is usually measured in feet per second (ft/sec). Flow velocities during riverine floods can easily reach 5 to 10 ft/sec, and in some situations may be even greater. Expressing velocities in ft/sec is common in floodplain studies and engineering analyses. Here, it may be helpful to relate ft/sec to a more familiar unit of measure. For example, 10 ft/sec is roughly equal to 7 miles per hour.

The velocity of riverine floodwaters depends on a number of factors; one of the most important is the slope of the stream channel and floodplain. As one might expect, floodwaters will generally move much faster along streams in steep mountainous areas than streams in flatter areas. Even within the same floodplain, however, flow velocity can still vary. As water flows over the ground, its velocity depends largely on the roughness of the ground surface. For example, water will flow more swiftly over parking lots, roads, and other paved surfaces and will flow more slowly over ground covered with large rocks, trees, dense vegetation, or other obstacles. Also, flow velocities in the floodplain will usually be higher nearer the stream channel than at the outermost fringes of the floodplain, where water may flow very slowly or not at all.

If a building is in an area where floodwaters are flowing, especially if they are moving more than about 5 ft/sec, the flow velocity is important for several reasons. Flowing water pushes harder on the walls of a building than still water. So instead of just the hydrostatic pressure caused by the weight of the floodwater resting against the walls, there is the additional pressure of moving water, referred to as hydrodynamic pressure. As water flows around the building, it pushes against the side that faces the flow (the upstream side). As it flows past the sides, it creates friction that can tear at wall coverings, such as siding. On the side of the building that faces away from the flow (the downstream side) the water may tend to create a suction that pulls on walls. In some situations, the combination of these forces can destroy one or more walls, cause the building to shift on its foundation, or even sweep the building away.

Flowing water can also cause erosion and scour. Erosion is the removal of soil that lowers the ground surface across an area. Scour is the removal of soil around objects that obstruct flow, such as foundation walls. Both erosion and scour can weaken the structure by removing supporting soil and undermining the foundation. In general, the greater the flow velocity and the larger the building, the greater the extent and depth of erosion and scour. Also, any objects being carried by floodwaters will be moving at roughly the same speed as the water. The dangers associated with these objects are discussed in Section 2.1.6.

2.1.3 Flood Frequency

Flood frequencies are usually determined through statistical and engineering analyses performed by floodplain management agencies and other organizations who need information on which to base engineering designs and flood insurance rates. The results of those analyses define the probability, expressed as a percentage, that a flood of a specific size on a specific stream will be equaled or exceeded in any year.

The 100-year flood is particularly important for homeowners because it is the basis of National Flood Insurance Program (NFIP) flood insurance rates and regulatory floodplain management requirements. In the NFIP, the 100-year flood is referred to as the base flood, the 100-year flood elevation as the base flood elevation (BFE), and the floodplain associated with the base flood as the special flood hazard area (SFHA). Other federal agencies, such as the USACE, use the 100-year flood for planning and engineering design, as do many state and local agencies.

2.1.4 Rate of Rise and Rate of Fall

Floodwaters with high flow velocities, such as those in areas of steep terrain, and water released by the failure of a dam or levee usually rise and fall more rapidly than slower-moving floodwaters, such as those in more gently sloping floodplains. In the floodplains of streams with high rates of rise, homeowners may have only a few hours' notice of a coming flood or perhaps none at all. If the flood protection method chosen depends partly on action the homeowner must take each time flooding threatens (i.e., contingent measures), warning time is especially important.

Rate of rise and rate of fall are important also because of their effect on hydrostatic pressure. As explained in the discussion of flood depth/elevation, hydrostatic pressure is most dangerous for a building when the internal and external pressures are not equalized. This situation occurs when the level of water inside is significantly higher or lower than the level outside. When floodwaters rise rapidly, water may not be able to flow into a building quickly enough for the level in the building to rise as rapidly as the level outside. Conversely, when floodwaters fall rapidly, water that has filled a building may not be able to flow out quickly enough, and the level inside will be higher than the level outside. In either situation, the unequalized hydrostatic pressures can cause serious structural damage, possibly to the extent that the building collapses.

2.1.5 Duration

Duration is related to rate of rise and rate of fall. Generally, water that rises and falls rapidly will recede more rapidly, and water that rises and falls slowly will recede more slowly.

Duration is important because it determines how long the structural members (such as the foundation, floor joists, and wall studs), interior finishes (such as drywall and paneling), service equipment (such as furnaces and hot water heaters), and building contents will be affected by floodwaters. Long periods of inundation are more likely to cause damage than short periods. In addition, long duration flooding can saturate soils (Figure FP-2), increasing the pressure on the foundation. Duration can also determine how long a building remains uninhabitable.

2.1.6 Debris Impact

Floodwaters can pick up and carry objects of all types (from small to large, from light to heavy) including trees, portions of flood-damaged buildings, automobiles, boats, storage tanks, mobile homes, and even entire buildings. Dirt and other substances such as oil, gasoline, sewage, and various chemicals can also be carried by floodwaters. All of these types of debris add to the dangers of flooding. Even when flow velocity is relatively low, large objects carried by floodwaters can easily damage windows, doors, walls, and more importantly critical structural components of a building. As velocity increases, so does the danger of greater damage from debris. If floodwaters carrying large amounts of dirt or hazardous substances enter the building, cleanup costs are likely to be higher and cleanup time greater.

2.2 When Flood Proofing is Not Appropriate

Many factors influence the decision-making process for determining the feasibility of flood-proofing options. However, there are certain situations in which flood proofing should not be considered, with the exception of relocation and/or demolition. For example, structures located within a regulatory floodway cannot be retrofitted with substantial improvements that would result in any increase in flood levels during the base flood discharge. Under these conditions, the structure should be relocated out of the floodway and, preferably, out of the floodplain.

2.3 Typical Causes of Flooding Problems

Flooding in the Denver metropolitan area typically results from heavy rains during the spring and summer months. Intense rainfall can lead to flooding in several ways and is exacerbated by the increasing percentage of impervious cover associated with urban development. The time of concentration is reduced as water is conveyed via a network of gutters and storm sewers yielding increased peak flows in the drainageways. Flooding can occur at any point in the drainage system and is aggravated if debris inhibits the flow.

2.3.1 Inadequate Street Conveyance

As discussed in the STREETS/INLETS/STORM SEWERS chapter, the minor drainage system should be designed to convey between the 2- and 10-year design storms. Over time, the street conveyance capacity can diminish due to pavement overlays reducing the gutter depth and altering the design slopes. As a result, even during minor storms, flows can pond or exceed the gutter capacity resulting in localized flooding.

2.3.2 Inadequate Storm Sewer Conveyance

Older sections of the metropolitan area predate drainage criteria. In many cases, the storm sewer capacity is limited to the 2-year or less frequency design.

2.3.3 Inadequate Drainage Channel Conveyance

Prior to current floodplain and drainage criteria, development often encroached on natural drainageways resulting in the reduced capacity of open channel conveyance. Over-bank flooding is the most dangerous type due to the combination of velocity and depth of the floodwaters.

2.3.4 Sewage Backup

Flooding can often inundate and overload sanitary sewer systems and combined sanitary/storm sewer systems. As a result, water can flow backward through sewer lines and out through toilets or floor drains. The best solution to this problem is usually to install a backflow valve.

3.0 FLOOD PROOFING METHODS

3.1 Overview of Six Methods Identified by FEMA

The following sections describe the retrofitting methods, explain how they work and where they are appropriate, and list their advantages and disadvantages.

3.1.1 Elevation

Elevating a building to prevent floodwaters from reaching living areas is an effective retrofitting method. The goal of the elevation process is to raise the lowest floor to or above the flood protection elevation (FPE) as shown in Figure FP-4. This can be done by elevating the entire building, including the floor, or by leaving the building in its existing position and constructing a new, elevated floor within the building. The method used depends largely on construction type, foundation type, and flooding conditions.

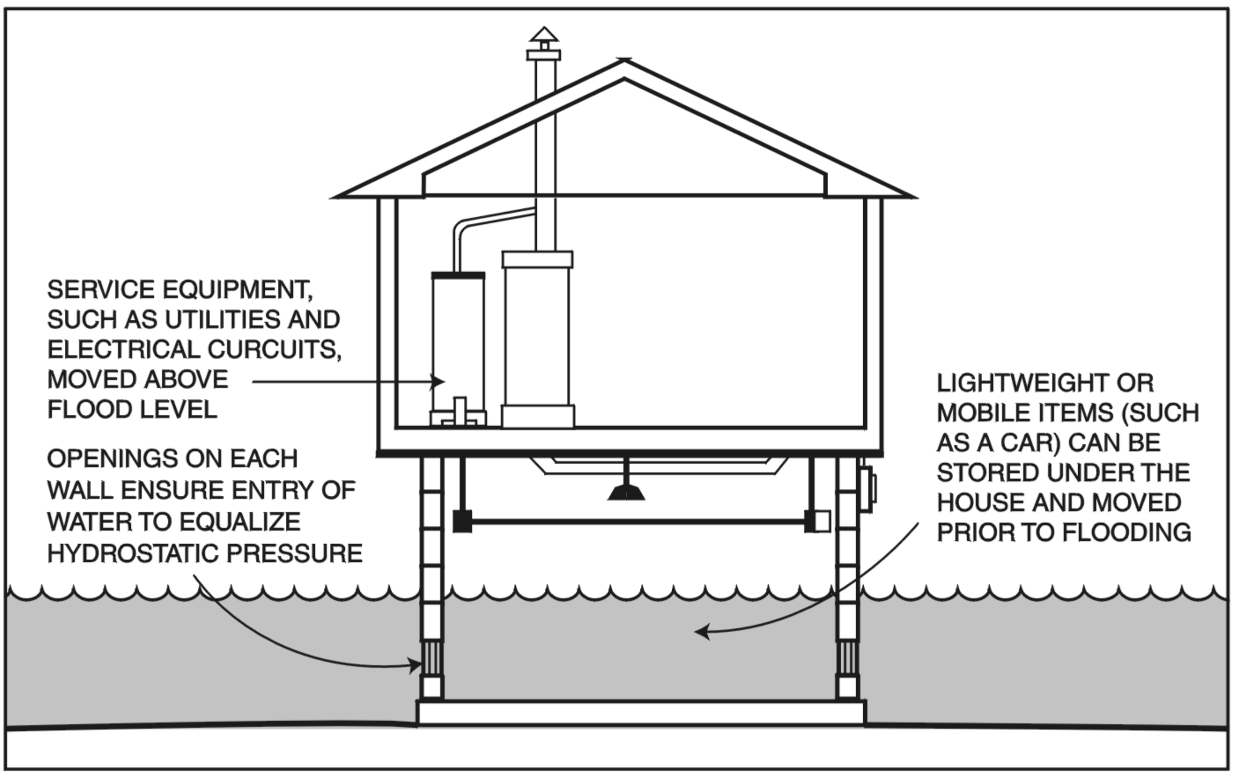

Figure FP-4—Example of a Structure Elevated on Continuous Foundation Walls

During the elevation process, most buildings are separated from their foundations, raised on hydraulic jacks, and held by temporary supports while a new or extended foundation is constructed below. This method works well for buildings originally built on basement, crawl space, and open foundations. As explained later in this section, the new or extended foundation can consist of continuous walls or separate piers, posts, columns, or pilings.

A variation of this method is used for buildings on slab-on-grade foundations. In these buildings, the slab forms both the foundation and the floor of the building. Elevating these buildings is easier if the building is left attached to the slab foundation and both are lifted together. After the building and slab are lifted, a new foundation is constructed below the slab.

Alternative techniques are available for masonry buildings on slab-on-grade foundations. These techniques do not require the lifting of the building. Instead, they involve raising the floor within the building or moving the living space to an upper story.

Although elevating a building can help protect it from floodwaters, other hazards need to be considered before choosing this method (Table FP-1). The walls and roof of an elevated building may be more susceptible to wind forces because they are higher and more exposed. In addition, both continuous wall foundations and open foundations can fail as a result of damage caused by erosion and the impact of debris carried by floodwaters. If portions of the original foundation, such as the footings, are used to support new walls or other foundation members or a new second story, they must be capable of safely carrying the additional loads imposed by the new construction and the expected flood and wind forces.

Table FP-1—Advantages and Disadvantages of Elevation

Advantages	Disadvantages
• Elevation to or above the FPE allows a substantially damaged or substantially improved building to be brought into compliance with the community's floodplain management ordinance or law. • Elevation reduces the flood risk to the building and its contents. • Except where a lower floor is used for storage, elevation eliminates the need to move vulnerable contents to areas above the water level during flooding. • Elevation often reduces flood insurance premiums. • Elevation techniques are well known, and qualified contractors are often readily available. • Elevation does not require the additional land that may be needed for construction of floodwalls or levees. • Elevation reduces the physical, financial, and emotional strain that accompanies floods.	• Cost may be prohibitive. • The appearance of the building may be adversely affected. • Access to the building may be adversely affected. • The building must not be occupied during a flood. • Unless special measures are taken, elevation is not appropriate in areas with high-velocity flows, waves, fast-moving ice or debris flow, or erosion. • Additional costs are likely if the building must be brought into compliance with current code requirements for plumbing, electrical, and energy systems. • Potential wind and earthquake loads must be considered.

3.1.2 Wet Flood Proofing

Wet flood proofing a building is done by modifying the uninhabited portions (such as a crawl space or an unfinished basement) so that floodwaters will enter but not cause significant damage to either the building or its contents. The purpose of allowing water into portions of the building is to ensure that the interior and exterior hydrostatic pressures will be equal (Figure FP-5). Allowing these pressures to equalize greatly reduces the likelihood of wall failures and structural damage. Wet flood proofing is often used when all other retrofitting methods are either too costly or are not feasible, but it is practical in only a limited number of situations. The advantages and disadvantages of wet flood proofing are summarized in Table FP-2.

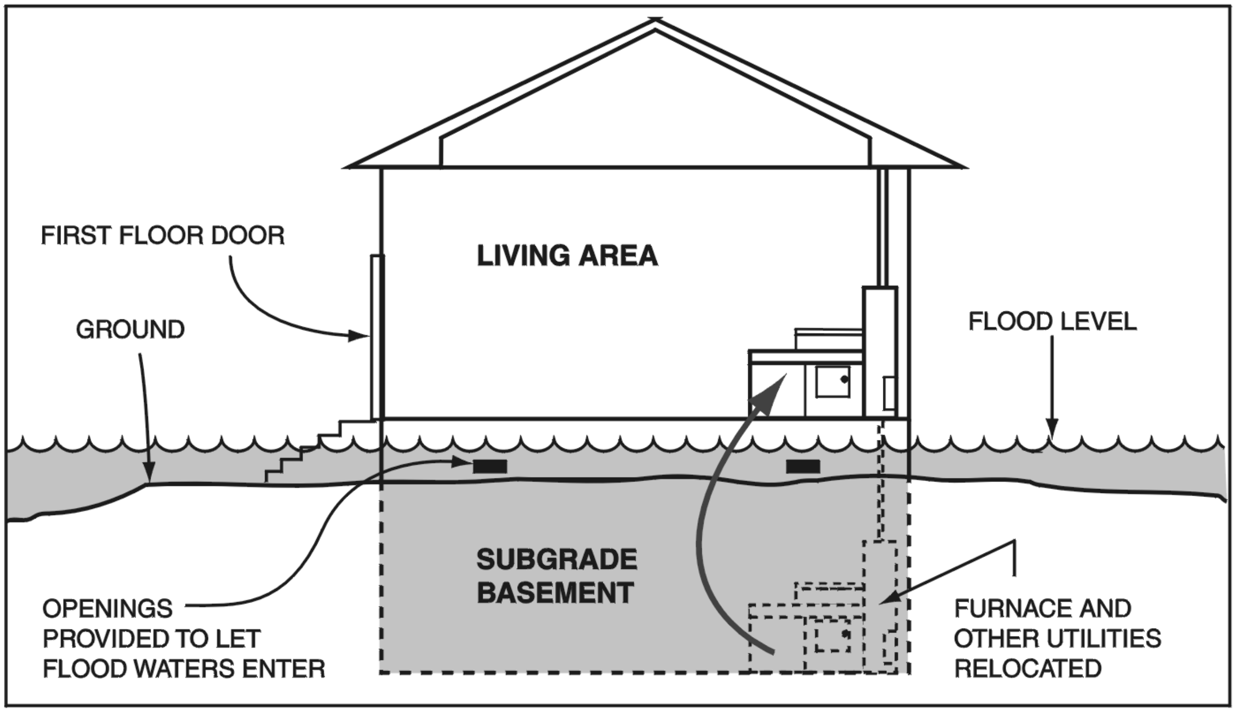

Figure FP-5—Example of a Building With a Wet Flood-Proofed Subgrade Basement

Because wet flood proofing allows floodwaters to enter the building, all construction and finishing materials below the FPE must be resistant to flood damage. For this reason, wet flood proofing is practical only for portions of a building that are not used for living space, such as a basement as defined by the NFIP regulations, a walkout-on-grade basement, crawl space, or attached garage. It would not be practical for most slab-on-grade buildings, in which the living space is at or very near the ground level. Whether or not wet flood proofing is appropriate for a building will depend on the flood conditions, the FPE selected, the design and construction of a building, and whether the building has been substantially damaged or is being substantially improved.

Table FP-2—Advantages and Disadvantages of Wet Flood Proofing

Advantages	Disadvantages
• No matter how small the effort, wet flood proofing can, in many instances, reduce flood damage to a building and its contents. • Because wet flood proofing allows internal and external hydrostatic pressures to equalize, the loads on walls and floors will be less than in a dry flood-proofed building (discussed later in this section). • Costs for moving or storing contents (except basement contents) after a flood warning is issued are covered by flood insurance in some circumstances. • Wet flood-proofing measures are often less costly than other types of retrofitting. • Wet flood proofing does not require the additional land that may be needed for floodwalls and levees (discussed later in this section). • The appearance of the building is usually not adversely affected. • Wet flood proofing reduces the physical, financial, and emotional strains that accompany floods.	• Wet flood proofing may be used to bring a substantially damaged or substantially improved building into compliance with a community's floodplain management ordinance or law <u>only</u> if the areas of the building below the FPE are used solely for parking, storage, or building access. • Preparing the building and its contents for an impending flood requires human intervention and adequate warning time. • The building will get wet inside and possibly be contaminated by sewage, chemicals, and other materials borne by floodwaters. Extensive cleanup may be necessary. • The building must not be occupied during a flood, and it may be uninhabitable for some time afterward. • It will be necessary to limit the uses of the floodable area of the building. • Periodic maintenance may be required. • Pumping floodwaters out of a wet flood-proofed basement too soon after a flood may lead to structural damage.* • Wet flood proofing does nothing to minimize the potential damage from high-velocity flood flow and wave action.

* **WARNING.** After floodwaters recede from the area around a building with a wet flood-proofed basement, the owner will usually want to pump out the water that filled the basement during the flood. If the soil surrounding the basement walls and below the basement floor is still saturated with water, however, removing the water in the basement too quickly can be dangerous. As the water level in the basement drops, the outside pressure on the basement walls and flood becomes greater than the inside pressure. As a result, the walls can collapse and the floor can be pushed up or cracked.

3.1.3 Dry Flood Proofing

In some situations, a building can be made watertight below the FPE, so that floodwaters cannot enter. This method is called dry flood proofing. Making the building watertight requires sealing the walls with waterproof coatings, impermeable membranes, or supplemental layers of masonry or concrete. Also, doors, windows, and other openings below the FPE must be equipped with permanent or removable shields, and backflow valves must be installed in sewer lines and drains (Figure FP-6). The flood characteristics that affect the success of dry flood proofing are flood depth, flood duration, flow velocity, and the potential for wave action and flood-borne debris.

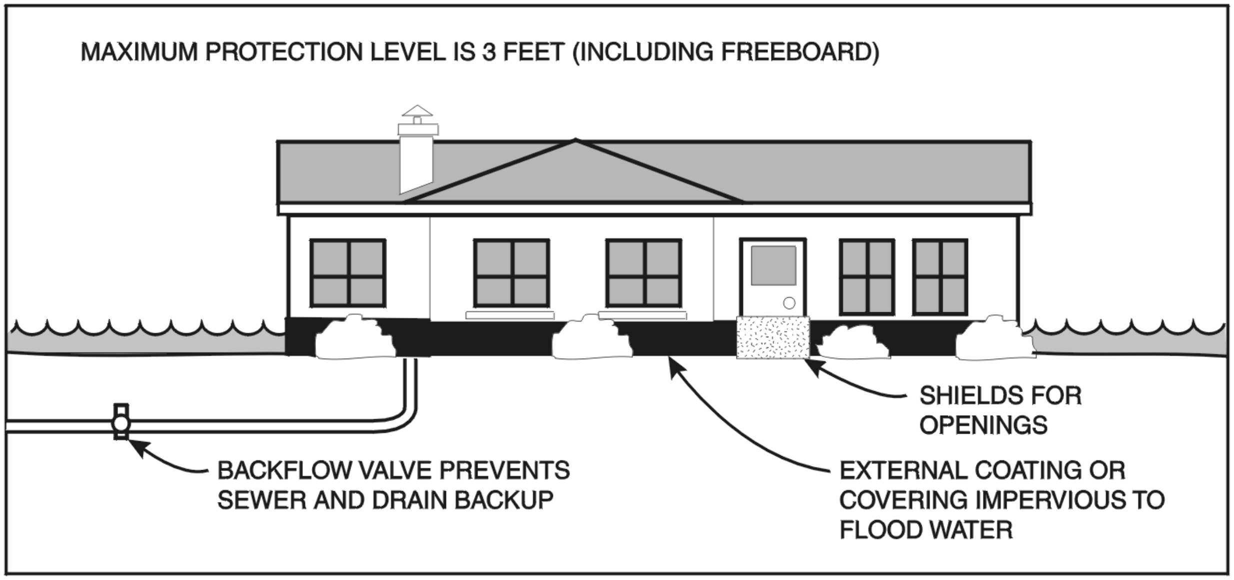

Figure FP-6—Example of a Dry Flood-Proofed House

Flood depth is important because of the hydrostatic pressure that floodwaters exert on walls and floors. Because water is prevented from entering a dry flood-proofed building, the exterior pressure on walls and floors is not counteracted as it is in a wet flood-proofed building. The ability of building walls to withstand the pressure exerted by floodwaters depends partly on how the walls are constructed. Typical masonry and masonry veneer walls, without reinforcement, can usually withstand the pressure exerted by water up to about 3 feet deep. When flood depths exceed 3 feet, unreinforced masonry and masonry veneer walls are much more likely to crack or collapse. An advantage of masonry and masonry veneer walls is that their exterior surfaces are resistant to damage by moisture and can be made watertight relatively easily with sealants. In contrast, typical frame walls are likely to fail at lower flood depths, are more difficult to make watertight, and are more vulnerable to damage from moisture. As a result, wet flood proofing is not recommended for buildings with frame walls that will be damaged by moisture.

Dry flood proofing may not be used to bring a substantially damaged or substantially improved building into compliance with a community’s floodplain management ordinance or law. The advantages and disadvantages of dry flood proofing are summarized in Table FP-3.

Table FP-3—Advantages and Disadvantages of Dry Flood Proofing

Advantages	Disadvantages
• Dry flood proofing reduces the flood risk to the building and its contents. • Dry flood proofing may be less costly than other retrofitting methods. • Dry flood proofing does not require the additional land that may be needed for levees and floodwalls (discussed later in this chapter). • Dry flood proofing reduces the physical, financial, and emotional strains that accompany floods.	• Dry flood proofing may not be used to bring a substantially damaged or substantially improved building into compliance with a community's floodplain management ordinance or law. • Ongoing maintenance is required. • Flood insurance premiums are not reduced for residential structures. • Installing temporary protective measures, such as flood shields, requires human intervention and adequate warning time.* • If the protective measures fail or the FPE is exceeded, the effect on the building will be the same as if there were no protection at all. • If design loads are exceeded, walls may collapse, floors may buckle, and the building may even float, potentially resulting in more damage than if the building was allowed to flood. • The building must not be occupied during a flood. • Flood shields may not be aesthetically pleasing. • Damage to the exterior of the building and other property may not be reduced. • Shields and sealants may leak, which could result in damage to the building and its contents. • Dry flood proofing does nothing to minimize the potential damage from high-velocity flood flow and wave action.

* **WARNING.** Because dry flood proofing requires human intervention, one must be willing and able to install all flood shields and carry out all other activities required for the successful operation of the dry flood-proofing system. As a result, not only must one be physically capable of carrying out these activities, one must be in the building or able to go there in time to do so before floodwaters arrive.

3.1.4 Relocation

Moving a building to high ground, outside the flood hazard area, is the most effective of the retrofitting methods described in this *Manual*. Retrofitting literature commonly refers to this method as relocation. When space permits, it may even be possible to move a building to another location on the same piece of property.

Relocating a building usually involves jacking it up and placing it on a wheeled vehicle, which delivers it to the new site. The original foundation cannot be moved, so it is demolished and a new foundation is built at the new site. The building is installed on the new foundation and all utility lines are connected.

Relocation is particularly appropriate in areas where the flood hazard is severe. Relocation is also appropriate for those who want to be free of worries about damage from future floods that may exceed a selected FPE.

Although similar to elevation, relocation requires additional steps that usually make it more expensive. These include moving the building, buying and preparing a new site (including building the new foundation and providing the necessary utilities), and restoring the old site (including demolishing the old foundation and properly capping and abandoning old utility lines). The advantages and disadvantages of relocation are summarized in Table FP-4.

Table FP-4—Advantages and Disadvantages of Relocation

Advantages	Disadvantages
• Relocation allows a substantially damaged or substantially improved building to be brought into compliance with a community's floodplain management ordinance or law. • Relocation significantly reduces flood risk to the building and its contents. • Relocation can either eliminate the need to purchase flood insurance or reduce the amount of the premium. • Relocation techniques are well known, and qualified contractors are often readily available. • Relocation reduces the physical, financial, and emotional strains that accompany flood events.	• Cost may be prohibitive. • A new site (preferably outside the flood hazard area) must be located and purchased. • The flood-prone lot on which the building was located must be sold or otherwise disposed of. • Additional costs are likely if the building must be brought into compliance with current code requirements for plumbing, electrical, and energy systems.

3.1.5 Levees and Floodwalls

Levees and floodwalls are types of flood protection barriers. A levee is typically a compacted earthen structure; a floodwall is an engineered structure usually built of concrete, masonry, or a combination of both (Figure FP-7). When these barriers are built to protect a building, they are usually referred to as residential, individual, or on-site levees and floodwalls. The practical heights of these levees and floodwalls are usually limited to 6 feet and 4 feet, respectively. These limits are the result of the following considerations:

- As the height of a levee or floodwall increases, so does the depth of water that can build up behind it. Greater depths result in greater water pressures, so taller levees and floodwalls must be designed and constructed to withstand the increased pressures. Meeting this need for additional strength greatly increases the cost of the levee or floodwall, usually beyond what an individual homeowner can afford.

- Because taller levees and floodwalls must be stronger, they must also be more massive, so they usually require more space than is likely to be available on an individual lot. This is especially true of levees.

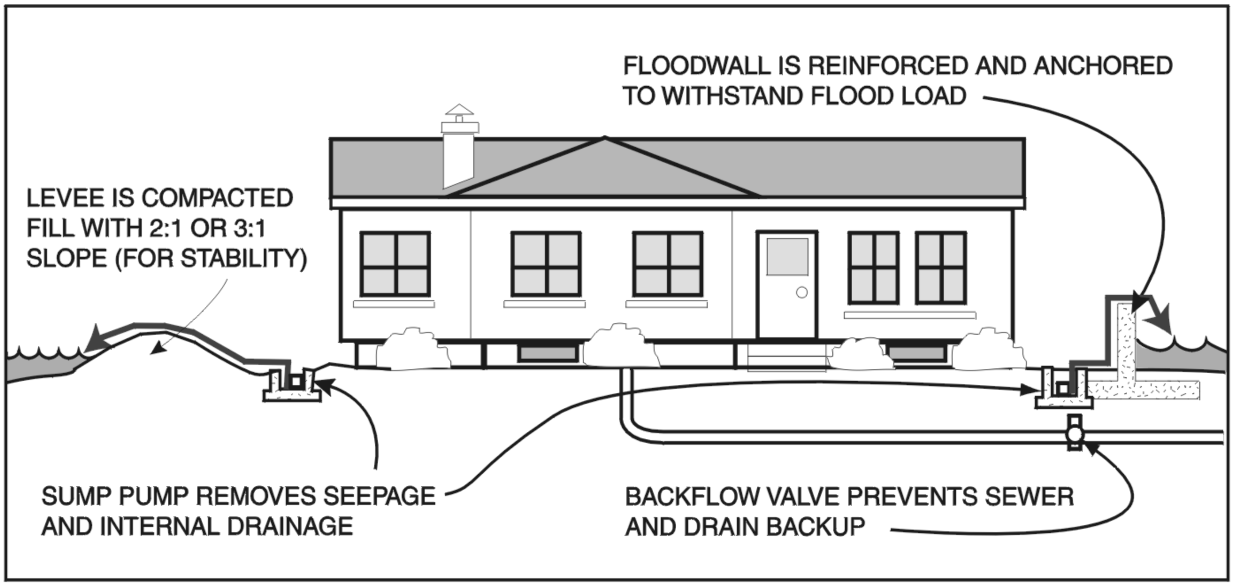

Figure FP-7—Example of Levee and Floodwall Protection

Both levees and floodwalls should provide at least 3 feet of freeboard. For example, if building a levee to protect a building from the base flood, the top of the levee should be 1 foot above the FPE.

For a levee to be effective over time, it must be constructed of soils that cannot be easily penetrated by floodwaters; it must have proper side slopes for stability, and it must be periodically inspected and maintained. In areas where high flow velocities could erode the surface of a levee, the side of the levee exposed to floodwater is usually protected with riprap or with other erosion-resistant material. Levees can surround a building, or they may be built only across low areas and tied into existing high ground.

A floodwall can surround a building, or, depending on flood depths, site topography, and design preferences, it can protect isolated openings such as doors, windows, and basement entrances, including entry doors and garage doors in walkout-on-grade basements. When built with decorative bricks or blocks or as part of garden areas, floodwalls can become attractive architectural or landscaping features. But they can also be built solely for utility, usually at a much lower cost.

Because a floodwall is made of concrete or masonry rather than compacted earth, it is more resistant to erosion than a levee and generally requires less space than a levee that provides the same level of protection; however, floodwalls are usually more expensive. As a result, floodwalls are normally considered only for sites where there is not enough room for a levee or where high flow velocities may erode a levee. Also, some homeowners prefer floodwalls because they can be more aesthetically pleasing and allow for the preservation of existing site features, such as trees.

An interior drainage system, including a sump pump, must be installed in the area protected by a levee or floodwall. The purpose of the system is to remove rainwater trapped inside the protected area and, during flooding, to remove water that enters through seepage or infiltration.

Special design considerations are necessary when levees or floodwalls are built to protect a building with a basement. Even though the surface water is kept from coming into contact with the building, the soil below the levee or floodwall and around the building can become saturated, especially during floods of long duration. The resulting pressure on basement walls and floors can cause them to crack, buckle, or even collapse.

3.1.6 Demolition

Demolition, as a retrofitting method, is tearing down a damaged building and either rebuilding properly somewhere on the same property or moving to a building on other property outside the regulatory floodplain. This retrofitting method may be the most practical of all those described in this *Manual* when a building has sustained extensive damage, especially severe structural damage.

Whether rebuilding or moving, the damaged building must be torn down and the site restored. Site restoration usually involves filling in a basement, grading, and landscaping. As a result, the services of a demolition contractor will probably be needed.

All demolition, construction, and site restoration work must be done according to the regulatory requirements of the community. Permits may be required for all or part of this work. If the new structure is built on the site of the old building, it must be rebuilt properly, which means ensuring that the lowest floor of the new building is at or above the FPE and that the new building is located outside the floodway. This can be accomplished by elevating the new building on an extended foundation as described in Section 3.1.1 or on compacted fill dirt. If the property includes an alternative building site outside the regulatory floodplain, a better approach is to build on that site, where standard construction practices, including the construction of a basement, can be used. If the building is reconstructed on the existing site within the regulatory floodplain, the community's floodplain management ordinance or law will not allow the new building to have a basement (as defined by the NFIP regulations).

The advantages and disadvantages of demolition depend on the decision of where to rebuild the structure (Table FP-5). If one of the flood-proofing methods is used, such as relocation or elevation, then the advantages and disadvantages of those methods will apply.

Table FP-5—Advantages and Disadvantages of Levees and Floodwalls

Advantages	Disadvantages
• The building and the area around it will be protected from inundation, and no significant changes to the building will be required. • Floodwaters cannot reach the building or other structures in the protected area and, therefore, will not cause damage through inundation, hydrodynamic pressure, erosion, scour, or debris impact. • The building can be occupied during construction of levees and floodwalls. • Levees and floodwalls reduce the flood risk to the building and its contents. • Levees and floodwalls reduce the physical, financial, and emotional strains that accompany flood events.	• Levees and floodwalls may not be used to bring a substantially damaged or substantially improved building into compliance with a community's floodplain management ordinance or law. • Cost may be prohibitive. • Periodic maintenance is required. • Human intervention and adequate warning time are required to close any openings in a levee or floodwall. • If a levee or floodwall fails or is overtopped by floodwaters, the effect on the building will be the same as if there were no protection at all. • An interior drainage system must be provided. • Local drainage can be affected, possibly creating or worsening flood problems for others. • The building must not be occupied during a flood. • Access to the building may be restricted. • Levees and floodwalls do not reduce flood insurance rates. • Floodplain management requirements may make levees and floodwalls violations of codes and/or regulations. • A large area may be required for construction, especially for levees. • Hydrostatic pressure on below-ground portions of a building may still be a problem, so levees and floodwalls are not good retrofitting methods for buildings with basements.

3.2 Engineering Aspects

Engineering aspects of flood proofing include evaluating the site and building characteristics, determining the flooding characteristics, and analyzing the potential loads on the structure during a flood event.

3.2.1 Analysis of Flood Hazards

Determining the potential depth of flooding is the first and most logical step in assessing flood hazards, since it is often the primary factor in evaluating the potential for flood damage. The depth of flooding is also critical in determining the extent of retrofitting that will be needed, and which method(s) will be the most appropriate for a given site. Detailed flood information is given in Flood Insurance Studies (FISs) and Flood Insurance Rate Maps (FIRMs) where such studies are available, and can be obtained from the District or local community in the form of Flood Hazard Area Delineations (FHADs).

The next step is to calculate the forces acting upon a structure during a flood. These forces include hydrostatic, hydrodynamic, and impact loads. Hydrostatic forces include lateral water pressure, saturated soil pressures, combined water and soil pressures, equivalent hydrostatic pressures due to low velocity flows (< 10 ft/sec), and buoyancy pressures. Hydrodynamic forces consist of frontal impact by the mass of moving water against the projected width and height of the obstruction represented by the structure, drag effect along the sides of the structure, and eddies or negative pressures on the downstream side of the structure. Impact loads are imposed on the structure by objects carried by moving water.

3.2.2 Site Characteristics

Important site characteristics to evaluate include the location of the structure relative to sources of potential flooding and geotechnical considerations. The site location should be evaluated with respect to mapped floodplains and floodways and the potential for local flooding from stormwater conveyance elements.

Soil properties during conditions of flooding are important factors in the design of any surface intended to resist flood loads. These properties include saturated soil pressures, allowable bearing capacity, potential for scour, frost zone location, permeability, and shrink-swell potential.

3.2.3 Building Characteristics

The building should be evaluated with respect to the type of construction and the condition of the structure. The type of foundation, foundation materials, wall materials, and the method of connection all play a role in deciding which retrofitting method will be most applicable. Operations involving a building in poor condition may easily wind up further damaging the building and costing more than its original value.

3.3 Selection of Flood-Proofing Techniques

In addition to the engineering aspects, the selection of the flood-proofing technique is a function of several factors that are dependent on the owner of the structure.

3.3.1 Regulatory Considerations

Federal, state, and local regulations may restrict the choice of retrofitting measures. Such regulations may include state and local building codes, floodplain management ordinances or laws, zoning ordinances, federal regulations concerning the alteration of buildings classified as historic structures, deed restrictions, and the covenants of homeowners associations.

State and local regulations may require that a retrofitted building be upgraded to meet current code requirements that were not in effect when the building was built. Portions of the electrical, plumbing, and heating/ventilation/air conditioning systems could be affected. For example, the electrical panel might have to be upgraded from fuses to circuit breakers. These changes are required for the safety of the homeowner. Other code-required upgrades include those necessary for increased energy efficiency. Any required upgrade can add to the scope and cost of the retrofitting project.

3.3.2 Appearance

The final appearance of a building and property after retrofitting will depend largely on the retrofitting method used and the FPE. For example, elevating a building several feet will change its appearance much more than elevating it only 1 or 2 feet, and a building elevated on an open foundation will not look the same as one elevated on extended foundation walls. However, a change in appearance will not necessarily be a change for the worse.

3.3.3 Accessibility

Accessibility refers to how easy or difficult it is to routinely reach and enter the building after the retrofitting project is completed. The retrofitting methods described in this *Manual* affect accessibility in different ways. For example, elevating a building will usually require the addition of stairs, which may be unacceptable to some. Wet flood proofing will have little, if any, effect on accessibility. The effect of relocation on accessibility will depend on the location and configuration of the new site.

3.3.4 Human Intervention Required

For retrofitting methods that require human intervention, owners must be willing, able, and prepared to take the necessary action, such as operating a closure mechanism in a floodwall or placing flood barriers across the doors of a dry flood-proofed building. Also, the owner must always have adequate warning of a coming flood and must be present or near enough to reach the building and take the necessary action before floodwaters arrive. If these conditions cannot be met, retrofitting methods that require human intervention should be eliminated from consideration.

3.3.5 Benefit/Cost Analysis

The cost of retrofitting will depend largely on the retrofitting method used and the FPE. For some methods, the construction type (frame, masonry, etc.) and foundation type (crawl space, slab, etc.) will also affect the cost. In general, costs will increase as the FPE increases, but there may be tradeoffs between alternative methods. For example, elevating may be less expensive than relocating when a building is raised only 1 or 2 feet but may become more expensive at greater heights. The benefits considered in a flood-proofing measure are the future damages and losses that are expected to be avoided as a result of the measure.

3.3.6 Other

Building owners may need to consider other factors, such as the availability of federal, state, and local financial assistance; the current value of the building versus the inconvenience and cost of retrofitting; the amount of time required to complete the retrofitting project; and the need to move out of the building during construction (including the availability and cost of alternative housing).

4.0 PROVIDING ASSISTANCE TO PROPERTY OWNERS

4.1 Decision Making Process for Property Owners

The decision of which flood-proofing method to use will be based primarily on legal requirements, the technical limitations of the methods, and cost. Other considerations might include such things as the appearance of the building after retrofitting and any inconvenience resulting from retrofitting.

4.1.1 Determine Flood Hazards

Information about flooding in the area is available from the District and local officials. Local officials, design professionals, and contractors can use this information, along with the flood hazard information developed by FEMA and other agencies and organizations, to provide advice about retrofitting options.

4.1.2 Inspect Structure

The structure should be inspected to determine the construction method and the type of foundation. Four characteristics of a building that are particularly important in retrofitting are construction type, foundation type, lowest floor elevation, and condition. Key to the inspection is performing a "Low Point of Entry" determination as illustrated in Figure FP-8.

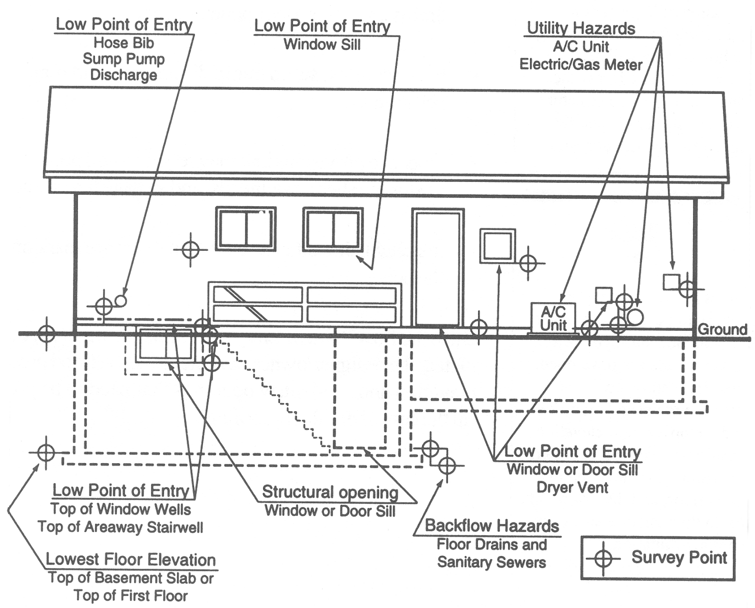

Figure FP-8—Example of a Low Point of Entry Survey

4.1.3 Contact Local Officials

The District and local officials have copies of the FIS and FIRM published for the community by FEMA. District or community officials can determine whether a building is in the regulatory floodplain and, if so, the FPE at the location of the building.

Local officials will provide federal, state, and local regulations, codes, and other requirements that can determine what retrofitting methods will be allowed. They can also provide information about federal, state, and local programs that provide financial assistance for homeowner retrofitting projects. If the property is 50 or more years old and receiving federal financial assistance for a retrofitting project, then the State Historic Preservation Office should also be contacted.

4.1.4 Consult With Professionals

The owner of a structure that needs flood proofing will need to consult with a design professional and a contractor in order to choose the appropriate flood-proofing method and ensure that the method is properly constructed. Table FP-6 shows the types of contractors and design professionals that may be required for each of the retrofitting methods.

4.2 Potential Sources of Financial Assistance at Federal, State, and Local Levels

FEMA and other federal agencies have a wide array of financial assistance programs that help states, communities, and individual property owners mitigate the negative effects of flood hazards. Property owners may be eligible to receive financial assistance through one or more of these programs that will help pay for the retrofitting project. If a presidential declaration of a major disaster has been issued for the area, property owners should seek information from FEMA and the state and local government representatives supporting the post-disaster recovery of the community.

The community's floodplain management ordinance or law includes requirements concerning construction in the community's regulatory floodplain. These requirements apply not only to new buildings but also to existing buildings that have been substantially damaged or that are being substantially improved. If the structure falls into one of the latter two categories, one of the following will be required:

- Elevate the building so that its lowest floor is at or above the FPE (Elevation).
- Move the building out of the regulatory floodplain (Relocation).
- Wet flood proof the part of the building that is below the FPE (Wet Flood Proofing). (This alternative is allowed only if the part of the building that is below the FPE is used solely for parking, storage, and building access and is not a basement as defined by the NFIP.)

Table FP-6—Requirements for Contractor and Design Professional Services

Method	Need for Contractor and/or Design Professional	Primary Services
Elevation	Design Professional	Evaluating the condition, stability, and strength of the existing foundation to determine whether it can support the increased load of the elevated building, including any wind and seismic loads
	Contractor: Building Elevation Contractor	Disconnecting utilities, jacking up the building, increasing the height of the foundation, and connecting utilities
Wet Flood Proofing	Design Professional	Designing any necessary replacements of vulnerable structural materials and relocated utility systems
	Contractor: General Construction Contractor	Replacing vulnerable structural and finishing materials below the FPE with flood-resistant materials, raising utilities and appliances to a location above the FPE, and installing openings required to allow the entry of floodwaters
Relocation	Design Professional	Designing any new building, foundation, and site improvements that may be required, such as new utility systems
	Contractor: Building Moving Contractor	Jacking up the building, moving it to the new site, and installing it on the new foundation
	Contractor: General Construction Contractor	Preparing the new site (including grading, foundation construction, and utilities) and cleaning up the old site (including demolition)
Dry Flood Proofing	Design Professional	For masonry walls to be dry flood proofed higher than 3 feet and for masonry veneer or frame walls to be dry flood proofed higher than 2 feet, evaluating the condition, stability, and strength of the existing walls to determine whether they can withstand the pressure from floodwaters at the FPE; designing or selecting flood shields for openings
	Contractor: General Construction Contractor	Applying waterproof sealants and membranes, installing flood shields over openings below the FPE, installing backflow valves in sewer and water lines, and, if necessary, bracing or modifying walls so that they can withstand the pressure from floodwaters at the FPE
Levees and Floodwalls	Design Professional	Assessing the adequacy of soils at the site and preparing the engineering design to ensure that the levee or floodwall, including any closures required, will be structurally stable under the expected flood loads and will be able to resist erosion, scour, and seepage
	Contractor: General Construction Contractor	Constructing the levee or floodwall
Demolition	Design Professional	Designing any new building, foundation, and site improvements that may be required, such as new utility systems
	Contractor: Demolition Contractor	Disconnecting and capping utility lines, tearing down the damaged building, hauling away debris, and cleaning up the old site
	Contractor: General Construction Contractor	Building the new building on the new site (May also be able to do all demolition work)

Communities with more restrictive floodplain management ordinances or laws may require a greater level of protection.

Although the substantial damage/substantial improvement requirement helps protect lives and property, it has at times placed an additional burden on property owners who were trying to repair their damaged buildings. Under the original terms and conditions of the NFIP Standard Flood Insurance Policy (SFIP), the owner of a substantially damaged building was reimbursed for the costs of repairing the damage but not for the costs of complying with state and local requirements concerning substantially damaged structures. For example, the homeowner would not have been reimbursed for the cost of elevating the building, even though state or local ordinances or laws required elevating.

In 1997, to provide relief for the owners of houses substantially damaged by flooding, Congress authorized the inclusion of Increased Cost of Compliance (ICC) coverage in the SFIP. With this change in effect, the SFIP reimburses homeowners not only for the cost of repairing flood damage but also for the additional cost, up to a maximum amount stated in the SFIP, of meeting certain state and local floodplain management requirements concerning substantial damage and repetitive losses. Other sources of assistance include:

- Small Business Administration (SBA)—In areas declared a major disaster area by the President, the SBA provides low-interest disaster assistance loans to individuals for both businesses and private residences. These loans cover the cost of rebuilding a damaged building, including the cost of bringing the building into compliance with applicable ordinances and laws. The loans can pay for retrofitting of substantially damaged buildings required by ordinances or laws (including elevating flood-prone buildings and rebuilding badly damaged flood-prone buildings at an alternative location), as well as some mitigation projects that are not required by ordinances or laws. At the applicant’s request, the amount of the loan may be increased by up to 20 percent for hazard mitigation measures not required by the community’s ordinances or laws.

- Department of Housing and Urban Development (HUD)—In an area declared a major disaster area by the President, HUD may provide additional, or allow for the reprogramming of existing, community development block grants. If a community wishes, these grants may be used for retrofitting substantially damaged or substandard buildings (including elevating flood-prone buildings and acquiring badly damaged flood-prone buildings).

- U.S. Army Corps of Engineers (USACE)—The USACE has the statutory authority to participate in flood protection projects that may include residential retrofitting (including elevating flood-prone buildings and acquiring badly damaged flood-prone buildings).

- Natural Resources Conservation Service (NRCS)—The NRCS has the statutory authority to participate in small watershed flood protection projects that may include residential retrofitting.

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REVEGETATION

CONTENTS

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1.0 INTRODUCTION

This chapter provides information on methods and plant materials needed for revegetation of drainage facilities within the Urban Drainage and Flood Control District (District). Establishment of a robust cover of vegetation is critical to the proper functioning of drainage structures such as grass-lined channels, detention basins, retention ponds, and wetlands. Vegetation serves multiple purposes, including stabilization of structures to prevent excessive erosion and removal of pollutants in stormwater. The semi-arid nature of the climate, prevalence of introduced weeds, and variety of soil types encountered in the District virtually mandate prompt implementation of a revegetation plan to achieve revegetation success.

2.0 SCOPE OF THIS CHAPTER AND RELATION TO OTHER RELEVANT DOCUMENTS

This chapter provides guidelines and recommendations for plant materials and methods for revegetation of components of the drainage system that are to be vegetated. Such components include:

- Natural channels
- Grass-lined channels
- Detention ponds
- Retention ponds
- Constructed wetlands/wetland channels
- Streambank stabilization and grade control structures

This chapter addresses the different revegetation requirements of the various parts of these facilities. For example, the bottom, side slopes and areas immediately adjacent to a facility have different moisture regimes and, therefore, should be planted with different plant species. Different plant forms (e.g., grasses, shrubs, trees) may also be limited to specific areas to enable proper functioning of the facility. For example, planting trees and shrubs along the bottom of a channel can reduce the hydraulic capacity of the channel, increase maintenance requirements, and cause the plugging of downstream bridges and culverts when uprooted by higher flows.

Additional information on revegetation methods in the District can be found in *Guidelines for Development and Maintenance of Natural Vegetation* (Don Godi and Associates 1984) and in *Design Workbook for Establishment of Natural Vegetation* (Don Godi and Associates 1993). Establishment of temporary and permanent vegetation for construction BMPs is addressed in the CONSTRUCTION BMPs chapter in Volume 3 of this *Manual*.

Although the information in this chapter is generally consistent with the information in these other documents, certain areas and topics have been updated (e.g., recommended seed mixes). Refer to the other documents listed for additional information, especially on factors to consider in preparing a revegetation plan.

3.0 GENERAL GUIDELINES FOR REVEGETATION

The guidelines below should be followed when developing a revegetation plan to the extent feasible.

3.1 Plant Materials

- The form(s) of vegetation and species used should be adapted to the soil and moisture conditions and use (e.g., conveyance of flow, side slopes, etc.) of the area.
- Native, perennial species should be used to the extent possible.
- Use of bluegrass and other species requiring irrigation and high maintenance should be avoided except along formal park settings.
- Sod-forming grasses are preferred over bunch grasses.
- Containerized nursery stock should be used for wetlands, trees, and shrubs to the extent feasible.
- Wetland plantings should not include cattails.
- Maintenance requirements should be considered in plant selection (e.g., tall grasses should not be used in urban areas unless regular mowing will occur).
- Live stakes, willow bundles, and cottonwood poles should be obtained from local, on-site sources, whenever possible (see Section 4.7.1).

3.2 Site Preparation

- All areas to be planted should have at least 6 inches of “topsoil” suitable to support plant growth (Don Godi and Associates 1984). Native topsoil should be stripped and saved for this purpose whenever a site is graded.
- The upper 3 inches of the soils in areas to be seeded should not be heavily compacted and should be in a friable condition. An 85% standard proctor density is acceptable.
- When necessary, soil amendments should be added to correct topsoil deficiencies (e.g., soil texture, pH or percent organic matter). (If topsoil and native seed mixes are used, fertilizer is often not needed.)
- Fertilizer should be used if specified by a soil analysis. Slow-release type fertilizers should be used to reduce weed growth and protect water quality. Fertilizer should be worked into soil during seedbed preparation.

3.3 Seeding and Planting

- Seed mixtures should be sown at the proper time of year specified for the mixture.
- Recommended seeding rates specified as “pounds pure live seed per acre” (lbs PLS/acre) should be used.
- Seed should be drill seeded, whenever possible.
- Broadcast seeding or hydro-seeding may be substituted on slopes steeper than 3(H):1(V) or on other areas not practical to drill seed.
- Seeding rates should be doubled for broadcast seeding or increased by 50% if using a Brillion drill or hydro-seeding.
- Broadcast seed should be lightly hand raked into the soil.
- Seed depth should be ⅓ to ½ inch for most mixtures.
- All seeded areas should be mulched, and the mulch should be adequately secured.
- If hydro-seeding is conducted, mulching should be conducted as a separate, second operation.
- All containerized nursery stock should be kept in a live and healthy condition prior to installation.
- Containerized trees and shrubs should be installed according to the planting details provided in Section 4.4.
- Live stakes, poles and willow bundles should be installed when dormant (late winter and early spring) according to the planting details in Section 4.7.
- Beaver protection should be provided for trees and shrubs for species known to be attractive to beavers if beavers are known to be in the area (see Figure RV-6).

3.4 Maintenance

- Sites should be routinely inspected following planting to implement follow-up measures to increase success. Immediate attention to a problem (e.g., weed infestation, failure of seed to germinate) can prevent total failure later.
- Access to and grazing on recently revegetated areas should be limited with temporary fencing and signage while plants are becoming established (normally the first year).
- Weed infestations should be managed using appropriate physical, chemical, or biological methods as soon as possible. (See the other documents referenced for details on weed

management options.)

- Stakes and guy wires for trees should be maintained, and dead or damaged growth should be pruned.
- Beaver protection cages should be used around tree plantings.
- Mulch should be maintained by adding additional mulch and redistributing mulch, as necessary.
- Areas of excessive erosion should be repaired and stabilized.
- Planted trees and shrubs should be watered monthly or as needed from April through September until established.

4.0 PREPARATION OF A PLANTING PLAN

4.1 General

A plan (drawings and specifications) needs to be prepared for revegetation work. The plan should address the following:

- Soil bed preparation
- Species, types, and sizes of materials to be planted
- Planting methods
- Mulching/fertilization
- Planting schedule

Figure RV-1 is a matrix that shows the steps involved in the revegetation process. Additional information on planning and design of a revegetation plan is included in *Design Workbook for Establishment of Natural Vegetation* (Don Godi and Associates 1993). This includes a “design analysis revegetation matrix” and several “checklists.” This and other relevant documents should be consulted for details on preparation of a planting plan. In addition, refer to the DESIGN EXAMPLES chapter of this *Manual* for more information on planting plans.

4.2 Soil Amendments

Native topsoil should be stripped and saved for revegetation. If this is not appropriate due to poor soil quality or for some other reason, then subsoil can be made conducive for plant growth through the use of amendments. Since soil pH is typically suitable within the District, amendments are usually needed for increasing organic matter content or providing nutrients in the form of fertilizers. Consideration should be given to importing topsoil, instead of amending poor quality subsoil, as this may be less expensive.

Peat moss, composted manure, composted organic materials, grass clippings, and plowed-in green crops can be used to increase the organic matter content of a soil. Several of these also provide a source of nutrients. Inorganic and organic fertilizers are commonly used to increase the nutrient content of soils. Deficiencies with trace elements also occur on occasion. Soil samples should be sent to a laboratory for testing (e.g., Colorado State University Soils Test Laboratory), and fertilizer recommendations followed.

Detailed information on the types and amounts of soil amendments and fertilizers needed is beyond the scope of this document and can be found in the documents previously referenced. However, information is provided on the use of humate soil conditioner and biosol fertilizer. Both of these materials are relatively new and show promise as soil conditioners and sources of slow-release fertilizers for revegetation work in the District.

4.2.1 Humate Conditioner

1. Utilize natural humic acid-based concentrated solution or granular material with the following characteristics:

   - Maximum of 10% retained on a #50 mesh screen
   - 4% N, 20% P as P_2O_5, 20% K as K_20
   - 1% Ca, 0.4% Fe, 0.4% S, humic acid 45%

2. Apply granular humate at a rate of 750 pounds/acre in a uniform manner prior to tilling soils for seeding.

3. Apply soluble concentrate at 1.0 pound/acre.

4. Thoroughly mix into soil to increase organic matter and nutrient content.

4.2.2 Biosol

1. Utilize organic fertilizer with the following characteristics:

   - 6% N, 1% P as P_2O_5, 3% K as K_2O
   - 90% fungal biomass

2. Apply at a rate of 1,200 pounds/acre in a uniform manner prior to tilling soils for seeding.

3. Thoroughly mix into soil to increase nutrients.

4.3 Recommended Seed Mixes

Unlined drainage facilities and all areas disturbed during construction should be actively revegetated. Seed mixes should be selected to match the conditions where they will be used. Seed mixes can be developed for the revegetation plan consistent with the guidelines in Section 3.0, or the mixes presented in this section can be used.

Recommended seed mixes for the bottom (wet soils) and side slopes of drainage facilities within the District are included in Tables RV-1 and RV-2. Mixes for different soil conditions in upland areas are provided in Tables RV-3 to RV-6. The seeding rates in these mixes are recommended minimum rates that should be used for drill seeding. These rates should be doubled for broadcast seeding and increased by 50% if a Brillion drill or hydro-seeding is used.

The recommended seed mixes are suitable for the Colorado Front Range for sites from 4,500 to 7,000 feet in elevation and latitude 38° to 42° North. Applications outside these ranges should be made after consultation with a qualified revegetation specialist.

Table RV-1—Recommended Seed Mix for High Water Table Conditions[1]

Common Name (Variety)	Scientific Name	Growth Season	Growth Form	Seeds/Lb	Lbs PLS/Acre
Redtop*	*Agrostis alba*	Warm	Sod	5,000,000	0.1
Switchgrass (Pathfinder)	*Panicum virgatum*	Warm	Sod/bunch	389,000	2.2
Western wheatgrass (Arriba)	*Pascopyrum smithii*	Cool	Sod	110,000	7.9
Inland saltgrass	*Distichlis spicata*	Warm	Sod	520,000	1.0
Wooly sedge	*Carex lanuginose*	Cool	Sod	400,000	0.1
Baltic rush	*Juncus balticus*	Cool	Sod	109,300,000	0.1
Prairie cordgrass	*Spartina pectinata*	Coll	Sod	110,000	1.0
					12.4
Wildflowers					
Nuttall's sunflower	*Helianthus nuttallii*	---	---	250,000	0.10
Wild bergamot	*Monarda fistulosa*	---	---	1,450,000	0.12
Yarrow	*Achillea millefolium*	---	---	2,770,000	0.06
Blue vervain	*Verbena hastata*	---	---		0.12
					0.40

[1] For areas of facilities located near or on the bottom or where wet soil conditions occur. Planting of potted nursery stock wetland plants 2-foot on-center is recommended for sites with wetland hydrology.

* Nonnative.

Table RV-2—Recommended Seed Mix for Transition Areas[1]

Common Name (Variety)	Scientific Name	Growth Season	Growth Form	Seeds/Lb	Lbs PLS/Acre
Sheep fescue (Durar)	*Festuca ovina*	Cool	Bunch	680,000	1.3
Western wheatgrass (Arriba)	*Pascopyrum smithii*	Cool	Sod	110,000	7.9
Alkali sacaton	*Spolobolus airoides*	Warm	Bunch	1,758,000	0.5
Slender wheatgrass	*Elymus trachycaulus*	Cool	Bunch	159,000	5.5
Canadian bluegrass (Ruebens)*[2]	*Poa compressa*	Cool	Sod	2,500,000	0.3
Switch grass (Pathfinder)	*Panicum virgatum*	Warm	Sod/bunch	389,000	1.3
					16.8
Wildflowers					
Blanket flower	*Gaillardia aristata*	---	---	132,000	0.25
Prairie coneflower	*Ratibida columnaris*	---	---	1,230,000	0.20
Purple prairie clover	*Petalostemum purpurea*	---	---	210,000	0.20
Gayfeather	*Liatris punctata*	---	---	138,000	0.06
Flax	*Linum lewisii*	---	---	293,000	0.20
Penstemon	*Penstemon strictus*	---	---	592,000	0.20
Yarrow	*Achillea millefolium*	---	---	2,770,000	0.03
					1.14

[1] For side slopes or between wet and dry areas.

[2] Substitute 1.7 lbs PLS/acre of inland salt grass (*Distichlis spicata*) in salty soils.

* Nonnative.

Table RV-3—Recommended Seed Mix for Alkali Soils

Common Name (Variety)	Scientific Name	Growth Season	Growth Form	Seeds/Lb	Lbs PLS/Acre
Alkali sacaton	*Sporobolus airoides*	Cool	Bunch	1,750,000	0.5
Streambank wheatgrass (Sodar)	*Agropyron riparium*	Cool	Sod	156,000	5.6
Inland salt grass	*Distichlis stricta*	Warm	Sod	520,000	1.7
Western wheatgrass (Arriba)	*Pascopyrum smithii*	Cool	Sod	110,000	7.9
Blue grama (Hachita)	*Chondrosum gracile*	Warm	Sod	825,000	4.0
Buffalograss	*Buchloe dactyloides*	Warm	Sod	56,000	2.0
					21.7
Wildflowers					
Blanket flower	*Gaillardia aristata*	---	---	132,000	0.25
Prairie coneflower	*Ratibida columnaris*	---	---	1,230,000	0.20
Purple prairie clover	*Petalostemum purpurea*	---	---	210,000	0.20
Gayfeather	*Liatris punctata*	---	---	138,000	0.06
Blue Flax	*Linum lewisii*	---	---	293,000	0.20
Rocky Mountain penstemon	*Penstemon strictus*	---	---	592,000	0.20
Yarrow	*Achillea millefolium*	---	---	2,770,000	0.03
					1.14

Table RV-4—Recommended Seed Mix for Loamy Soils

Common Name (Variety)	Scientific Name	Growth Season	Growth Form	Seeds/Lb	Lbs PLS/Acre
Sheep fescue (Durar)	*Festuca ovina*	Cool	Bunch	680,000	0.6
Canby bluegrass	*Poa canbyi*	Cool	Bunch	926,000	0.5
Thickspike wheatgrass (Critana)	*Elymus lanceolatus*	Cool	Sod	154,000	5.7
Western wheatgrass (Arriba)	*Pascopyrum smithii*	Cool	Sod	110,000	7.9
Blue grama (Hahita)	*Chondrosum gracile*	Warm	Sod/bunch	825,000	1.1
Switchgrass (Pathfinder)	*Panicum virgatum*	Warm	Sod/bunch	389,000	1.0
Sideoats grama (Butte)	*Boutelou curtipendula*	Warm	Sod	191,000	2.0
					18.8
Wildflowers					
Blanket flower	*Gaillardia aristata*	---	---	132,000	0.25
Prairie coneflower	*Ratibida columnaris*	---	---	1,230,000	0.20
Purple prairie clover	*Petalostemum purpurea*	---	---	210,000	0.20
Gayfeather	*Liatris punctata*	---	---	138,000	0.06
Flax	*Linum lewisii*	---	---	293,000	0.20
Penstemon	*Penstemon strictus*	---	---	592,000	0.20
Yarrow	*Achillea millefolium*	---	---	2,770,000	0.03
					1.14

Table RV-5—Recommended Seed Mix for Sandy Soils

Common Name (Variety)	Scientific Name	Growth Season	Growth Form	Seeds/Lb	Lbs PLS/Acre
Blue grama (Hachita)	*Chondrosum gracile*	Warm	Sod/bunch	825,000	2.1
Little bluestem (Camper)	*Schizachyrium scoparium*	Warm	Bunch	260,000	3.0
Prairie sandreed	*Calamovilfa longifolia*	Warm	Sod	274,000	3.0
Sand dropseed	*Sporobolus cryptandrus*	Warm	Bunch	5,298,000	0.3
Sideoats grama (Vaughn)	*Bouteloua curtipendula*	Warm	Sod/bunch	191,000	5.6
Western wheatgrass (Arriba)	*Pascopyrum smithii*	Cool	Sod	110,000	8.0
					22.0
Wildflowers					
Blanket flower	*Gaillardia aristata*	---	---	132,000	0.25
Prairie coneflower	*Ratibida columnifera*	---	---	1,230,000	0.20
Purple prairie clover	*Petalostemum purpurea*	---	---	210,000	0.20
Gayfeather	*Liatris punctata*	---	---	138,000	0.06
Flax	*Linum lewisii*	---	---	293,000	0.20
Penstemon	*Penstemon strictus*	---	---	592,000	0.20
Yarrow	*Achillea millefolium*	---	---	2,770,000	0.03
					1.14

Table RV-6—Recommended Seed Mix for Clay Soils

Common Name (Variety)	Scientific Name	Growth Season	Growth Form	Seeds/Lb	Lbs PLS/Acre
Buffalograss	*Buchloe dactyloides*	Warm	Sod	56,000	14.0
Sideoats grama (Vaughn)	*Bouteloua curtipendula*	Warm	Sod	191,000	3.0
Blue grama (Hachita)	*Chondrosum gracile*	Warm	Sod/bunch	825,000	1.1
Western wheatgrass (Arriba)	*Pascopyrum smithii*	Cool	Sod	110,000	7.9
Alkali sacaton	*Sporobolus airoides*	Warm	Bunch	1,758,000	1.0
Inland saltgrass	*Distichlis stricta*	Warm	Sod	520,000	2.0
					29.0
Wildflowers					
Gayfeather	*Liatris punctata*	---	---	138,000	0.1
Purple prairie clover	*Petalostemum purpureum*	---	---	210,000	0.1
Scarlet globemallow	*Sphaeralcea coccinea*	---	---	500,000	0.1
Rocky Mountain beeplant	*Cleome serrulata*	---	---	65,900	0.1
					0.4

The seed mixes in Tables RV-1 through RV-6t06 also include recommended wildflowers that can be included in a mix when wildflowers are desired. The wildflower seeds can be sown at the same time or after the grass seed mix. Table RV-7 includes a general wildflower seed mix that can be used in sunny locations. This mix stresses more drought tolerant, native perennials, and can be sown at the same time as a grass seed mix, or after. The mix in Table RV-7 is recommended instead of the species shown in Tables RV-1 through RV-6, when more wildflowers are desired.

Table RV-7—Wildflower Mix (to be seeded with grass seed mix)[1]

Common Name (Variety)	Scientific Name	Flower Color	Seeds/Lb	Lbs PLS/Acre
Scarlet globemallow	*Sphaeralcea coccinea*	Red/orange	500,000	0.6
Blue flax	*Linum lewisii*	Blue	293,000	0.6
Purple prairie clover	*Petalostemum purpureum*	Red-purple	210,000	0.7
White prairie clover	*Petalostemum candidum*	White	354,000	0.6
California poppy	*Eschscholtzia californica*	Orange	293,000	0.3
Blanket flower	*Gaillardia aristata*	Yellow/red	132,000	1.0
Prairie aster	*Aster tanacetifolius*	Violet	496,000	0.3
Blackeyed Susan	*Rudbeckia hirta*	Yellow	1,710,000	0.3
Purple coneflower	*Echinacea purpurea*	Purple	117,000	0.9
Yarrow	*Achillea millefolium*	White	2,770,000	0.1
Gayfeather	*Liatris punctata*	Rose/purple	138,000	0.6
Total				6.0

[1] This is a general mix for the District that stresses native perennials that do well in a range of soil types in sunny locations.

4.4 Trees, Shrubs and Wetland Plantings

Trees and shrubs add diversity to a planting plan and value for wildlife and birds. Trees and shrubs that impede flow and reduce the capacity of the structure should not be planted in the bottom of a drainage channel. It is recommended that containerized stock of the species listed in Table RV-8 be planted, as shown on Figures RV-2 and RV-3. Alternatively, cottonwood pole plantings and coyote (or sandbar) willow cuttings may be used to establish cottonwood trees and willows especially in soils with a shallow groundwater table.

The species of trees and shrubs to be planted should be chosen carefully to meet specific site conditions. For example, a shrub species that requires moderate to high soil moisture (e.g., sandbar willow) should not be planted on a dry hillside or upper streambank unless there is evidence of a high groundwater table or another continuous water source.

Table RV-8—Recommended Shrubs and Trees[1]

Common Name	Scientific Name	Height (ft)	Sun/Shade	Planting Zone	Notes
Shrubs					
Saskatoon serviceberry	*Amelanchier alnifolia*	3 – 15	Sun	Upland	Good for wildlife
Lead plant	*Amorpha fruticosa*	3 – 8	Sun	Upland	Drought tolerant
Rubber rabbitbrush	*Chrysothamnus nauseosus*	2 – 3	Sun	Upland	Drought tolerant
Wild plum	*Prunus Americana*	5 – 20	Sun/shade	Transition	Forms thickets
Chokecherrry	*Prunus virginiana*	5 – 20	Sun/shade	Transition	Forms thickets
Smooth sumac	*Rhus glabra*	4 – 7	Sun/shade	Upland	Good for wildlife
Oakbrush sumac	*Rhus trilobata*	2 – 6	Sun/shade	Upland	Drought tolerant
Wax currant	*Ribes cereum*	3 – 5	Sun/shade	Transition	Good for wildlife
Redosier dogwood	*Cornus stolonifera*	3 – 9	Shade	Wetland	Drought tolerant
Sandbar willow	*Salix exigua*	6 – 10	Sun	Transition or wetland	Requires more water
Snowberry	*Symphoricarpos oreophilus*	2 – 5	Sun/shade	Transition	Prefers moist area
Spanish bayonet	*Yucca glauca*	1 – 2	Sun	Upland	Drought tolerant
Woods rose	*Rosa woodsii*	2 – 3	Sun	Upland	Establishes quickly
Silver buffaloberry	*Shepherdia argentea*	6 – 13	Sun	Upland	Drought tolerant
Trees					
Narrow leaf cottonwood	*Populus angustifolia*	10 – 30	Sun	Transition or wetland	Requires more water
Plains cottonwood	*Populus deltoides*	50	Sun	Transition	Requires more water
Rocky Mountain juniper	*Juniperus scopulorum*	5 – 15	Sun	Upland	Drought tolerant
Colorado blue spruce	*Picea pungens*	60 – 100	Sun	Transition	Requires more water
Ponderosa pine	*Pinus ponderosa*	75 – 100	Sun	Upland	Drought tolerant
Peach leaf willow	*Salix amygdaloides*	15 – 30	Sun	Wetland	Requires more water

[1] Trees and shrubs should not be planted in the bottoms of drainage channels or where they could impede flow and decrease channel capacity. It is recommended that containerized stock (e.g., 2-gallon, 5-gallon) be used for trees and shrubs.

Wetland vegetation should be established in constructed wetlands, wetland bottom channels and, at times, along the shoreline of retention ponds. Such vegetation serves multiple functions, including assistance with pollutant removal, shoreline stabilization, aesthetics, and wildlife and bird habitat. Wetland plants should be planted in “zones” based on water depth. A common problem with establishing wetlands within the District is invasion by cattails. Actively planting a constructed wetland and keeping open areas with a water depth greater than 2 feet will discourage cattail invasion. Recommended plants for wetlands are shown in Table RV-9 by water depth. It is recommended that containerized stock be used for wetland plantings. Additional information on design of constructed wetlands and retention ponds can be found in Volume 3 of this *Manual*.

Table RV-9—Recommended Plants for Constructed Wetlands and Retention Pond Shelf[1]

Depth of Water (ft)	Common Name	Scientific Name	Notes
0 - 1.5	Soft stem bulrush Hard stem bulrush Arrowhead Alkali bulrush Smart weed	*Scirpus validus* *Scirpus acutus* *Sagittaria latifolia* *Scirpus maritimus* *Polygonum persicaria*	• Planted plants should extend above water • Plants will invade deeper water with time
0.25 - 0.5	Three-square Spike rush	*Scirpus americanus* *Eleocharis palustris*	• Planted plants should extend above water
0 - 0.25	Rice cut grass Nebraska sedge Soft rush Baltic rush Torrey's rush Foxtail barley	*Leersia oryzoides* *Carex nebrascensis* *Juncus effuses* *Juncus balticus* *Juncus torreyi* *Hordeum jubatum*	• Species will adjust to moisture conditions with time
Height above water 0 – 1 0 – 3	 Milkweed Switchgrass Prairie cordgrass Beebalm	 *Asclepias incarnata* *Panicum virgatum* *Spartina pectinata* *Monarda fistulosa*	 • Best to plant near water where soil is wet • Colorful wildflower

[1] It is recommended that containerized stock be used for wetland plantings. It is not recommended that cattails be planted since they will invade naturally.

4.5 Mulching

All planted areas should be mulched preferably immediately following planting, but in no case later than 14 days from planting. Mulch conserves water and reduces erosion. The most common type of mulch used is hay or grass that is crimped into the soil to hold it. However, crimping may not be practical on slopes steeper than 3:1.

The following guidelines should be followed with mulching:

- Only weed-free and seed-free straw mulch should be used (grass hay often contains weedy exotic species). Mulch should be applied at 2 tons/acre and adequately secured by crimping, tackifier, netting, or blankets.

- Crimping is appropriate on slopes of 3:1 or flatter and must be done so as to tuck mulch fibers into the soil 3 to 4 inches deep.
- Tackifier or netting and blankets anchored with staples should be used on slopes steeper than 3:1.
- Hydraulic mulching may also be used on steep slopes or where access is limited. Wood cellulose fibers mixed with water at 2,000 to 2,500 pounds/acre and organic tackifier at 100 pounds per acre should be applied with a hydraulic mulcher.
- Wood chip mulch should be applied to planted trees and shrubs, as shown in Figures RV-2 and RV-3.

Additional details on mulching can be found in Volume 3 of this *Manual*.

4.6 Bioengineering

Willow bundles, live stakes, and cottonwood poles are plant materials that can be used to revegetate drainage facilities. Willow bundles can be placed to provide bank protection along lower slopes of channels. Live stakes and poles can be planted near the toe of a slope where there is a source of high groundwater. They are especially applicable for vegetating large riprap and boulders filled with soil. Information is provided below on methods for collecting and planting willow bundles, live stakes and cottonwood poles. In addition, see Section 4.5, Bioengineered Channels, in the MAJOR DRAINAGE chapter of this *Manual* for additional information and figures.

4.7 Collection of Live Stakes, Willow Cuttings, and Poles

Live stakes, willow cuttings, and poles are straight branches or saplings that have been cut and pruned from dormant living plant material (plants that have lost their leaves).

Single live stakes: The live branches which shall be trimmed and cut to length for this installation shall be a minimum of 2½ feet long and a minimum of ½ inch in diameter for bare ground installation, and a minimum of 3½ feet long for riprap joint planting. These units shall be free from all side branches. The terminal bud must remain undamaged. The "root" end of each cutting shall be cut at a 45-degree angle. This serves as an indicator of which end of the stake to tamp into the ground or riprap and also facilitates the tamping process.

Willow bundling: The live branches, which shall be trimmed and cut to length for this installation, shall be a minimum of 4 feet long and a minimum of ⅜ inch in diameter. These units shall be free from all side branches. The "root" end of each cutting shall be cut at a 45-degree angle. This serves as an indicator of which end of the stake to insert into the ground or riprap.

Cottonwood poling: The live saplings or straight branches, which shall be trimmed and cut to length for this installation, shall be a minimum of 10 feet long and a minimum of 1 inch in diameter. These units shall be free from all side branches. The "root" end of each pole shall be cut at a 45-degree angle. This serves as an indicator of which end of the pole to insert into the ground or riprap.

4.7.1 Harvest Procedure

1. Timing of harvest and Installation: All live willow staking, bundling, and poling shall be performed between February 1 and April 1, prior to leafing out.

2. Source and species of live cut materials: Live cuttings shall be taken from approved, existing, natural, native-growing sites. All cuttings shall be taken from a dormant plant. Willow species shall be *Salix exigua* (Sandbar willow) or approved equivalent. Cottonwood species shall be *Populus deltoides* (Plains cottonwood) or equivalent. Willow cuttings shall be at least ¼ inch in diameter, and cottonwood poles no less than ¾ inch in diameter.

3. Cutting: The use of weed whips with metal blades, loppers, brush cutters, and pruners is recommended, provided that they are used in such a manner that they leave clean cuts. The use of chain saws is not recommended. Live plant materials shall be cut and handled with care to avoid bark stripping and trunk wood splitting. Cuts shall be made 8 to 10 inches from the ground when cutting from the approved sites. Cuts shall be made flat or at a blunt angle.

   All cuttings should be placed in water deep enough to cover at least the lower 6 inches of the cutting immediately after harvest.

4. Harvesting site: No more than 30% of available branches should be harvested at a site. The harvesting site must be left clean and tidy. Excess woody debris should be removed from the site and disposed of properly, or could be cut up into 16-inch lengths and evenly distributed around the site.

5. Binding and storage: Live branch cuttings shall be bound together securely with twine at the collection site, in groups, for easy handling and for protection during transport. Live branch cuttings shall be grouped in such a manner that they stay together when handled. Outside storage locations shall be continually shaded and protected from the wind. Cuttings shall be held in moist soils or kept in water until ready for planting. Cuttings shall be protected from freezing and drying at all times.

6. Transportation: During transportation, the live cuttings shall be placed on the transport vehicles in an orderly fashion to prevent damage and to facilitate handling. The live cuttings shall be kept wet and covered with a tarp or burlap material during transportation.

7. Arrival time: All cuttings shall arrive on the job site within 8 hours of cutting. Upon arrival at the installation site, cuttings shall be inspected for acceptability. Cuttings not installed on the day of arrival at the job site shall be stored and protected (kept in water and in cold storage) until installation. All cuttings shall be installed within 24 hours of harvesting.

8. Inspection and approval: Upon arrival at the construction site, live branch cuttings shall be

inspected for acceptability. Live cuttings shall be collected from sources that shall be approved prior to the commencement of cutting operations.

4.7.2 Installation

Single live stakes: Live stakes shall be planted in three rows starting 0.5 feet above the ordinary high water line, at 1 foot spacing. Stakes shall be installed in a 2-feet by 2-feet grid pattern. Live stakes shall be tamped directly into the soil or between rock riprap and shall protrude 4 to 8 inches from the soil surface. Live stakes shall be installed at least 12 inches into the soil and at least 6 inches into saturated soil. In no case will the live stakes protrude more than 8 inches above the soil surface. In the case of joint planting in riprap, the protruding measurement shall be taken from the soil level between the rocks and not from the top of rock. Only dead blow hammers or rubber mallets shall be used to tamp the live stakes into the soil. Care shall be taken to prevent splitting the stakes due to impact from the hammers. Sledgehammers shall not be used to tamp the live stakes into the soil. In cases where the soil is too hard to tamp the live stake in directly, a metal rod of ½- to ¾-inch-diameter may be driven in first to prepare a pilot hole. Backfill around the installed live stake with the original soil to eliminate air voids, then tamp the ground lightly around the stake with a hammer to hold it securely in place. A slight “saucer” shall be formed around each cutting to capture and hold precipitation. This saucer should be filled with water after planting. After the stakes are fully tamped into the soil, the top 1 to 2 inches of each live stake shall be pruned to a clean, non-damaged cut. Figure RV-4 shows a typical installation of live willow staking.

Willow bundling: Bundles shall consist of five to seven cuttings bound together into a 2- to 3-inch-diameter. Bundles shall be planted in rows starting 0.5 feet above the ordinary high water line at 4-foot spacing. Bundles shall be inserted directly into the soil or between rock riprap and shall protrude 4 to 8 inches from the soil surface. Bundles shall be installed at least 12 inches into the soil and reach at least 6 inches into saturated soil. In no case should the cuttings protrude more than 8 inches above the soil surface. In the case of joint planting in riprap, the protruding measurement shall be taken from the soil level between the rocks and not from the top of rock. If tamping is necessary, care shall be taken to prevent splitting the cuttings. Backfill around the installed bundle with the original soil to eliminate air voids, then tamp the ground lightly around the bundle with a hammer to hold it securely in place. A slight saucer shall be formed around each bundle to capture and hold precipitation. This saucer should be filled with water after planting. After the bundles are fully inserted into the soil, the top 1 to 2 inches of each cutting shall be pruned if necessary to a clean, non-damaged cut. Figure RV-5 shows a typical installation of willow bundling.

Cottonwood poling: All branches must be trimmed from the pole except those at the tip. Prepare the pilot hole by using an auger, stinger, or probe to bore to a minimum depth of 5 feet or as needed to penetrate groundwater. Poles should pass through 18 inches of aerated soil before penetrating the water table.

The pilot hole shall be of sufficient diameter to facilitate easy insertion of a cottonwood pole. Backfill around the installed pole with loose sand to eliminate air voids, then tamp the ground lightly around the pole with a hammer to hold it securely in place. A slight saucer shall be formed around each pole to capture and hold precipitation. This saucer should be filled with water after planting. Cottonwood poles should be protected against beaver damage by the installation of a 30-inch-diameter beaver protection sleeve made from an 8½-foot length of 48-inch-wide 2-inch by 2-inch welded wire fabric fastened with wire or hogring fasteners. Figure RV-6 shows a typical installation of cottonwood poles.

1. Site Analysis

- Existing soil conditions
- Purpose(s) and type(s) of facilities
- Identification of planting zones

2. Design—Planting Plan

- Soil preparation—Section 4.2
- Plant/seed lists—Sections 4.3, 4.4 and 4.6
- Planting methods—Sections 3.0, 4.4

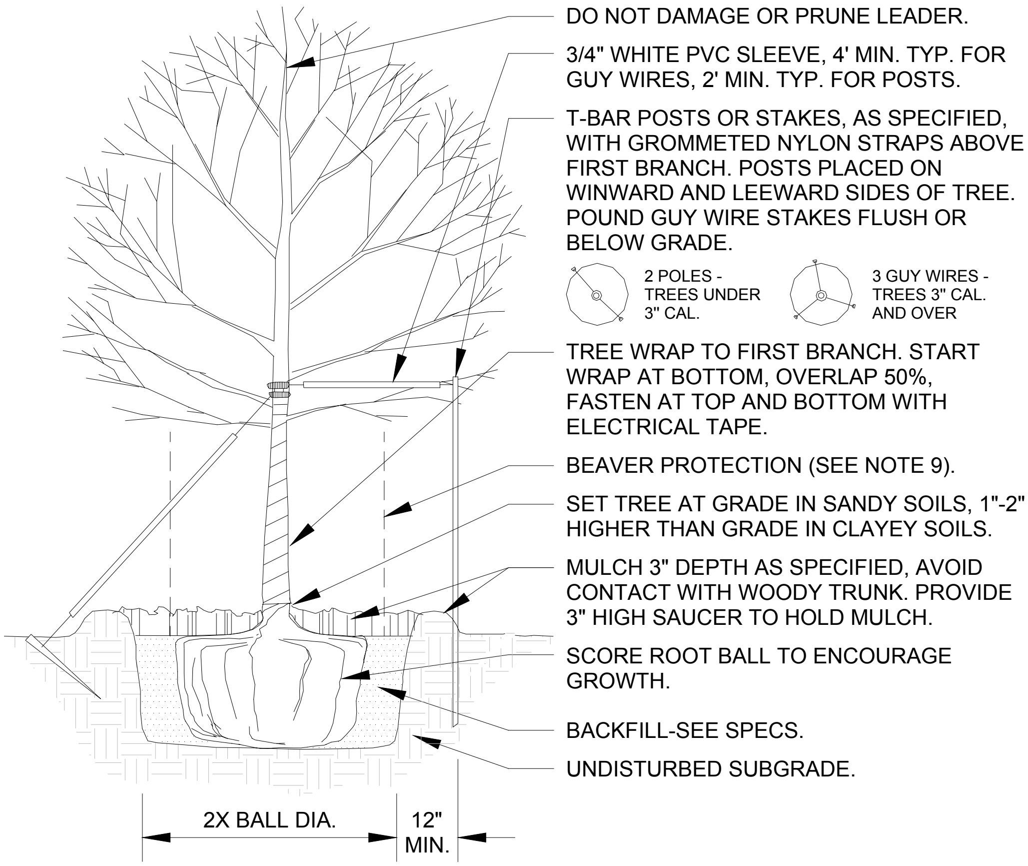
3. Construction

- Verify final grades

4. Maintenance—Section 5.0

- Inspect routinely
- Implement any corrective actions
- Conduct routine mowing and other required maintenance, including watering trees and shrubs until established

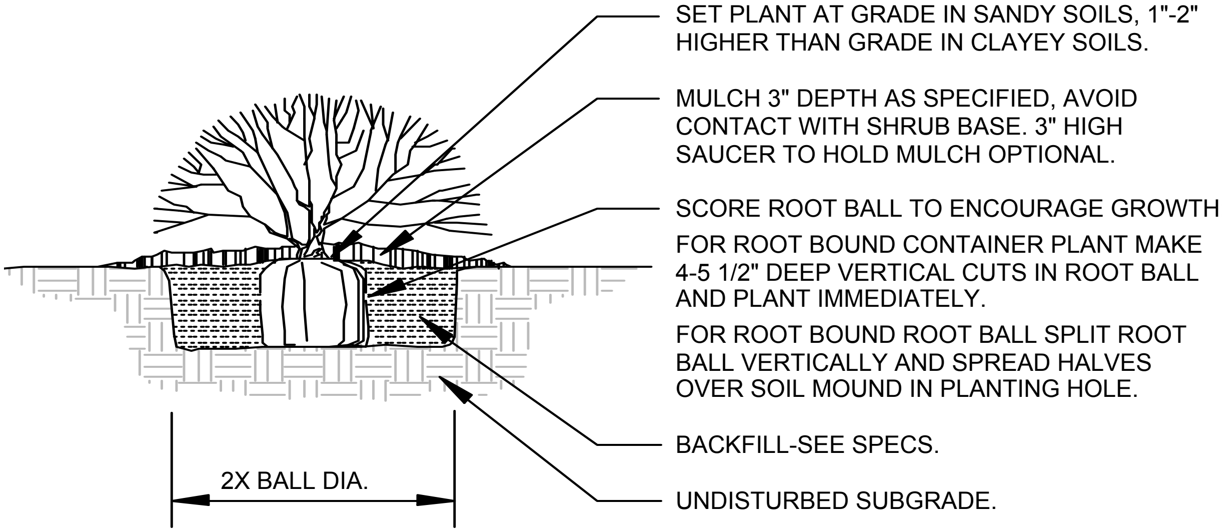
Figure RV-1—Revegetation Process Chart



NOTES:

1. SEE SPECS FOR ADDITIONAL PLANTING REQUIREMENTS.
2. KEEP PLANTS MOIST AND SHADED UNTIL PLANTED. PRUNE ALL DAMAGED AND DEAD BRANCHES AND WEAK OR NARROW CROTCHES. DO NOT REMOVE LOWER LIMBS AND SPROUTS FOR AT LEAST TWO GROWING SEASONS.
3. PLUMB AND ORIENT PLANTS FOR BEST APPEARANCE.
4. REMOVE ALL TWINE AND WIRE FROM ROOT BALL AND FOLD BURLAP BACK 2/3. REMOVE ALL RESTRAINING MATERIAL AFTER TREE IS SET IN PLANTING HOLE.
5. ROOT BALL SHALL REST ON FIRM, UNDISTURBED SOIL. IN SANDY SOIL PLANTING HOLE SHALL BE NO DEEPER THAN ROOT BALL. IN CLAYEY SOIL PLANTING HOLE SHALL BE 1"-2" SHALLOWER THAN ROOTBALL.
6. SCARIFY VERTICAL SLOPES INSIDE HOLE WITH SPADE.
7. ON SLOPES GREATER THAN 5:1 SET ROOT BALL 2" ABOVE LINE OF SLOPE AT GRADE. PROVIDE SAUCER RIM ON DOWNHILL SIDE OF ROOT BALL, 2:1 MAX. SLOPE, COVER EXPOSED ROOT BALL MIN. 6".
8. WATER ALL PLANTS WELL AT PLANTING.
9. PROVIDE A BARRIER PROTECTION SLEEVE (SEE FIGURE RV-6) WHENEVER BEAVER ARE SUSPECTED TO LIVE OR ARE EXPECTED TO MOVE INTO THE PROJECT AREA LATER.

Figure RV-2—Tree Planting Details



NOTES:

1. SEE SPECS FOR ADDITIONAL PLANTING REQUIREMENTS.
2. KEEP PLANTS MOIST AND SHADED UNTIL PLANTED. PRUNE ALL DAMAGED AND DEAD WOOD.
3. PLUMB AND ORIENT PLANTS FOR BEST APPEARANCE.
4. REMOVE ALL TWINE FROM ROOT BALL AND FOLD BURLAP BACK 2/3. REMOVE PLASTIC BURLAP, CONTAINERS AND WIRE BASKETS ENTIRELY.
5. ROOT BALL SHALL REST ON FIRM, UNDISTURBED SOIL. IN SANDY SOIL PLANTING HOLE SHALL BE NO DEEPER THAN ROOT BALL. IN CLAYEY SOIL PLANTING HOLE SHALL BE 1"-2" SHALLOWER THAN ROOTBALL.
6. SCARIFY VERTICAL SLOPES INSIDE HOLE WITH SPADE.
7. ON SLOPES GREATER THAN 5:1 SET ROOT BALL EVEN WITH LINE OF SLOPE AT GRADE. PROVIDE SAUCER RIM ON DOWNHILL SIDE OF ROOT BALL, 2:1 MAX. SLOPE, COVER EXPOSED ROOT BALL MIN. 6".
8. WATER ALL PLANTS WELL AT PLANTING.

Figure RV-3—Shrub Planting Details

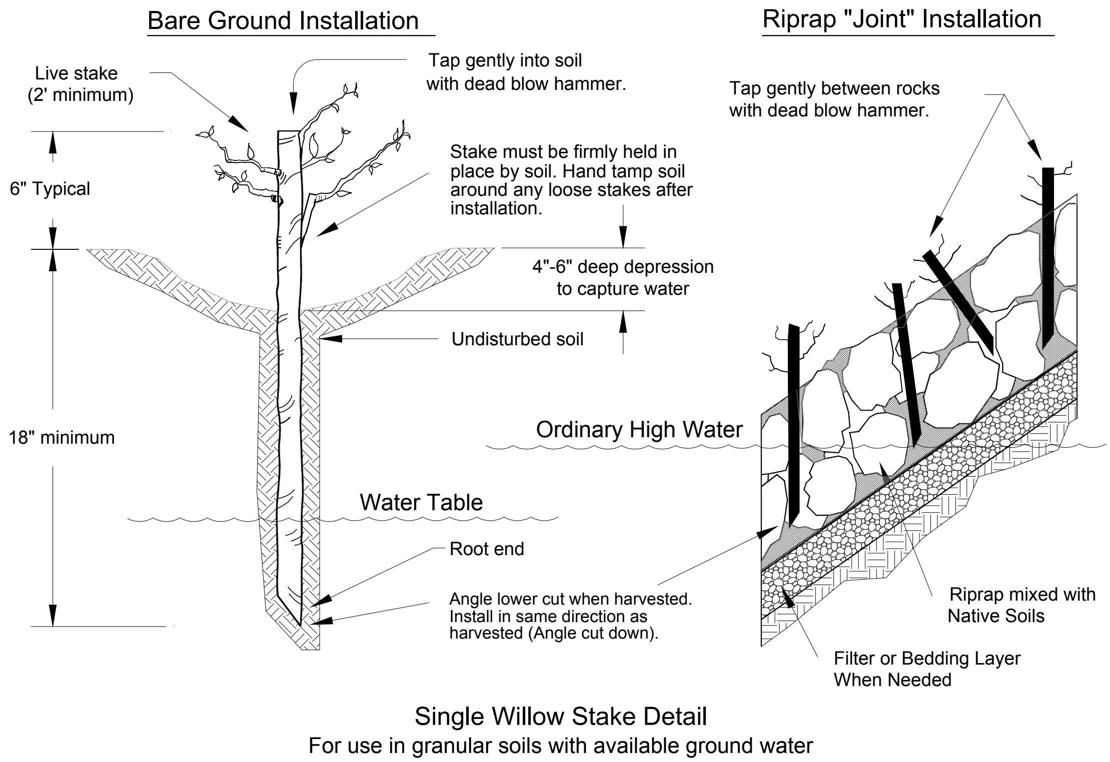


Figure RV-4—Single Willow Stake Detail for Use in Granular Soils With Available Groundwater

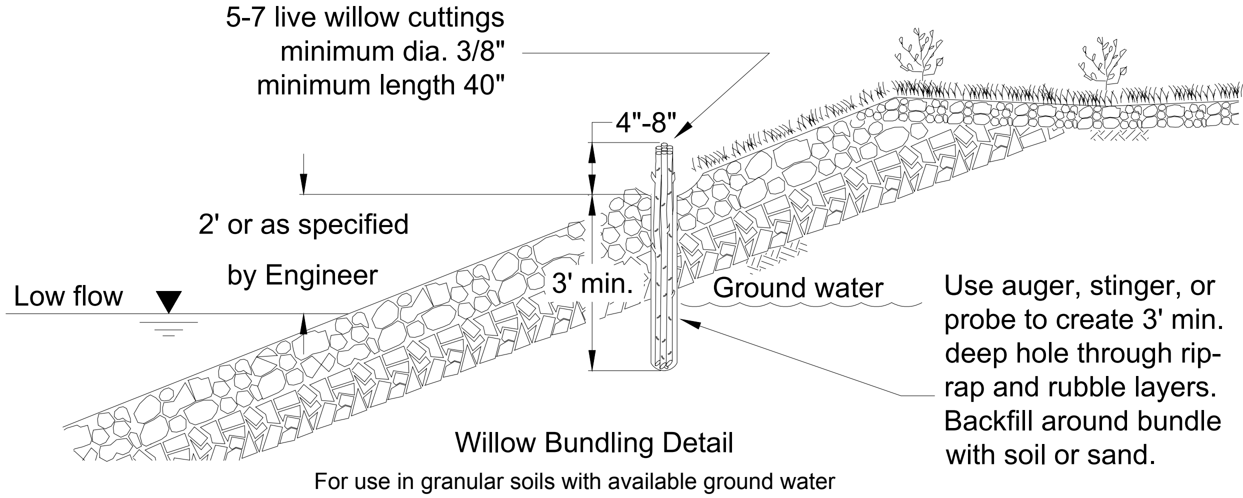
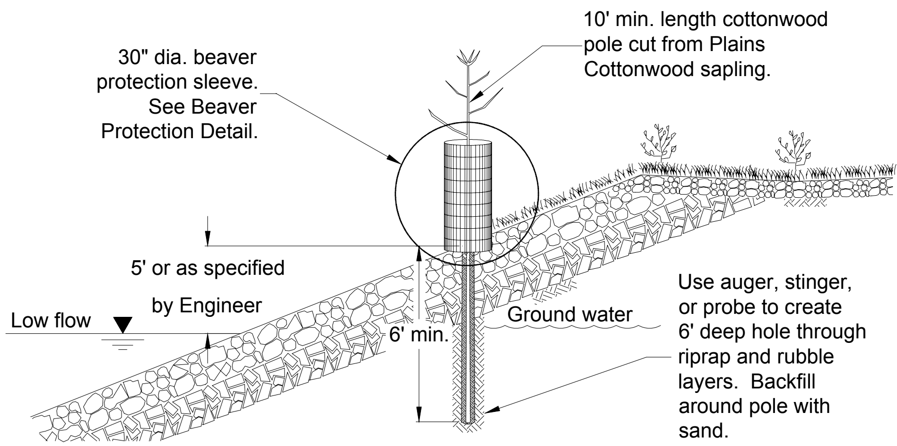
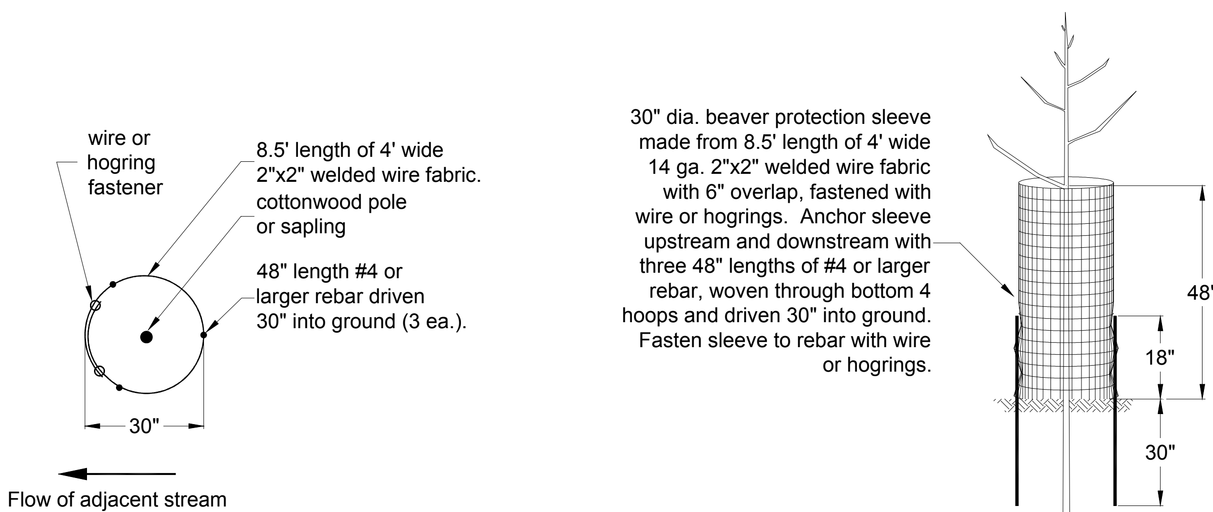


Figure RV-5—Willow Bundling Detail



Cottonwood Poling Detail

For use in granular soils with available ground water



Plan View

Elevation View

Beaver Protection Detail

Figure RV-6—Cottonwood Poling Details

5.0 POST-CONSTRUCTION MONITORING

Monitoring is necessary to check the status of revegetation work and to implement any follow-up measures needed, such as mowing, weed control, watering, overseeding, etc. This is especially important for establishing native species since it may take several years for vegetation to become adequately established. Sites should be observed several times during their first two growing seasons and at least once a year thereafter. The guidelines in Section 3.4 should be followed.

6.0 REFERENCES

Don Godi and Associates. 1984. *Guidelines for Development and Maintenance of Natural Vegetation*. Denver, CO: Urban Drainage and Flood Control District.

———. 1993. *Design Workbook for Establishment of Natural Vegetation*. Denver, CO: Urban Drainage and Flood Control District.

DESIGN EXAMPLES

CONTENTS

Tables

1.0 INTRODUCTION

The purpose of this chapter is to provide design examples, featuring actual projects in the Denver metropolitan area. These examples were prepared by the consulting engineers and landscape architects/planners listed in Table DE-1.

Table DE-1—List of Design Examples

Section	Case Study Name and Location	Prepared By
2.0	**Stapleton Redevelopment**[1] in the City and County of Denver	Matrix Design Group BRW
3.0	**Willow Creek** in Arapahoe County	Muller Consulting Engineers Wenk Associates
4.0	**Rock Creek** in Superior	McLaughlin Water Engineers, Ltd. The Norris/Dullea Company
5.0	**Sand Creek** in Adams County	Aquatic and Wetland Consultants Camp Dresser and McKee (CDM)
6.0	**Goldsmith Gulch** at Bible Park in the City and County of Denver	Sellards & Grigg Consulting Engineers Wenk Associates
7.0	**Greenwood Gulch** in the City of Greenwood Village	Water & Waste Consulting Engineers Sellards & Grigg Consulting Engineers Design Concepts, Inc.
8.0	**Lena Gulch** in Wheatridge	Taggart Engineering Associates EDAW

[1] Comprehensive design example with calculations.

DISCLAIMER

Several design examples are presented in this chapter to illustrate specific problem-solving approaches for projects having particular circumstances and drainage characteristics. The design examples have been selected to represent typical District situations and to show application of drainage principles and design criteria as described in Chapters 1 through 12. The design examples represent standard District technology and application and, for the most part, have been approved by the District and responsible governmental agencies leading to construction. Nonetheless, the designs shown shall be used at the sole risk of the user, and the District and the contributing consultants do not warrant these designs for any particular application. None of the examples represent proprietary design criteria or information and may be freely used as guidelines and examples, as with an engineering textbook. The designs and/or calculations shown represent methods and techniques recommended by the District and are in the public domain.

DESIGN EXAMPLES—SECTION 2

CONTENTS

Tables for Section 2

Figures for Section 2

2.0 CASE STUDY—STAPLETON REDEVELOPMENT

2.1 Project Setting

The following example illustrates application of this *Manual* for the design of conveyance and detention facilities, including use of computational spreadsheets described in pertinent sections of the *Manual*. Redevelopment of the former Stapleton International Airport in Denver poses significant opportunities and challenges for stormwater management. Like many airports, the site was graded to create gentle grades for runway operations. A formal storm sewer system was installed to control minor storm events, while major 100-year storms were conveyed via sheet flow or by overflow open channels. Consequently, significant drainage infrastructure improvements were needed. The challenge was to strike a balance between conveyance and detention to optimize the reuse of the existing system and minimize grading and demolition.

Figure 1 shows the project location and hydrologic setting for the *Stapleton East-West Linear Park Flood Control Project*. As indicated on Figure 2, the project incorporates a watershed of 104.0 acres that has been delineated into Sub-Basins "031" and "032". The mixture of residential, park, and school uses represents an average surface imperviousness of 44%. This assignment involved providing preliminary-level engineering for a sub-regional detention pond and associated outfall sewer and overflow channel. It is expected to be constructed by 2002 to support redevelopment of the Stapleton site near Yosemite Boulevard and 26th Avenue. The pond had to be designed to meet both detention volume requirements and enable reuse of an existing 54-inch storm sewer that outfalls to Westerly Creek. As a result, the detention volume had to be computed by V=KA, the modified Federal Aviation Administration (FAA) Method and a synthetic unit hydrograph to determine the controlling criteria.

2.2 Project Objectives

A multi-disciplinary team of engineers, landscape architects, planners, and scientists was formed to plan and design facilities to achieve the following objectives:

Provide a detention facility that offers multiple benefits, including park and recreation uses, flood control, water quality enhancement, and educational benefits.

Minimize demolition in and grading of the sub-basin by designing detention facilities to enable a retrofit and reuse of an existing 54-inch storm sewer.

Perform hydraulic engineering to determine the capacity of the existing outfall system and preliminarily size new collection and conveyance systems required to support land development at Stapleton.

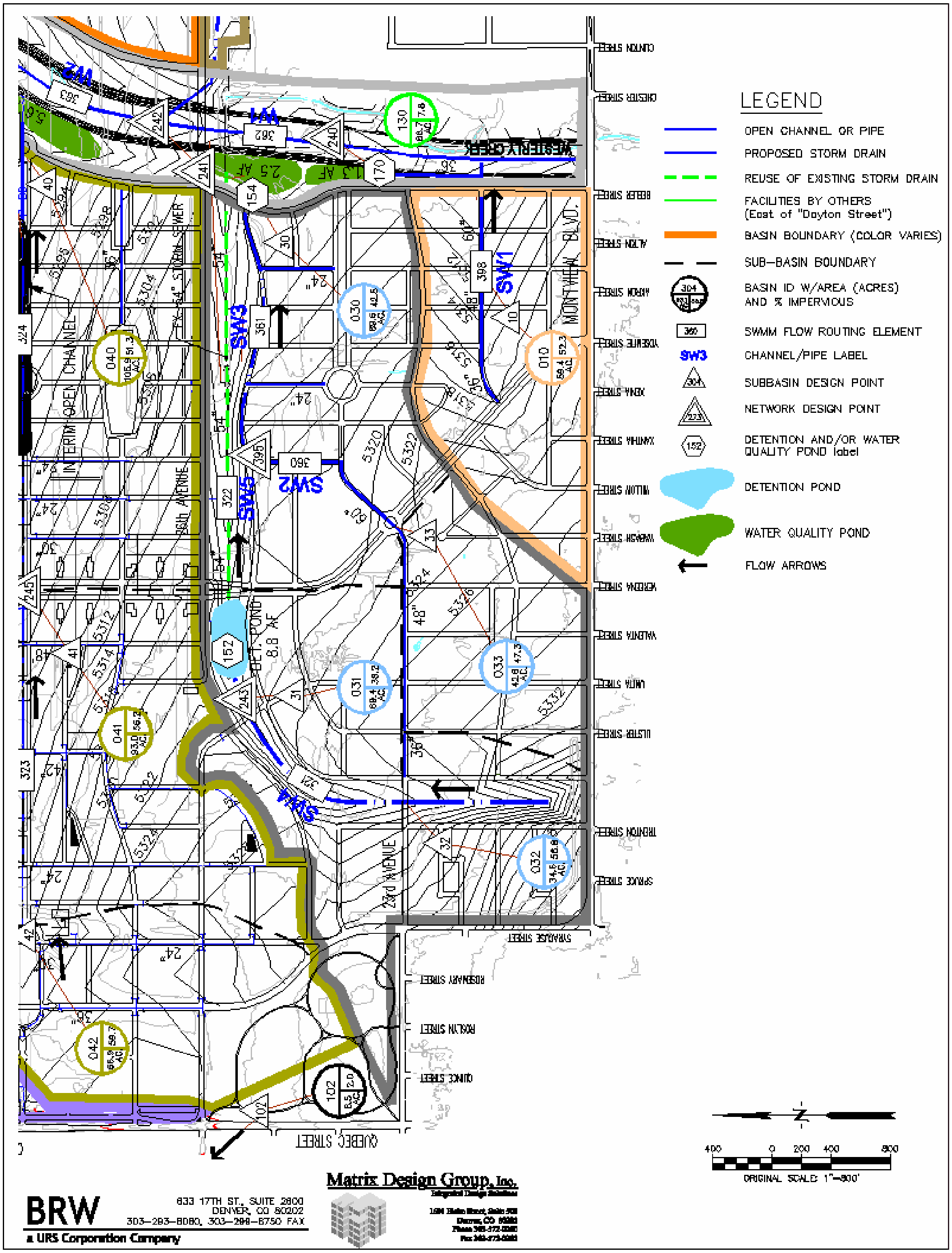


Figure 1—Stapleton Redevelopment Drainage Map

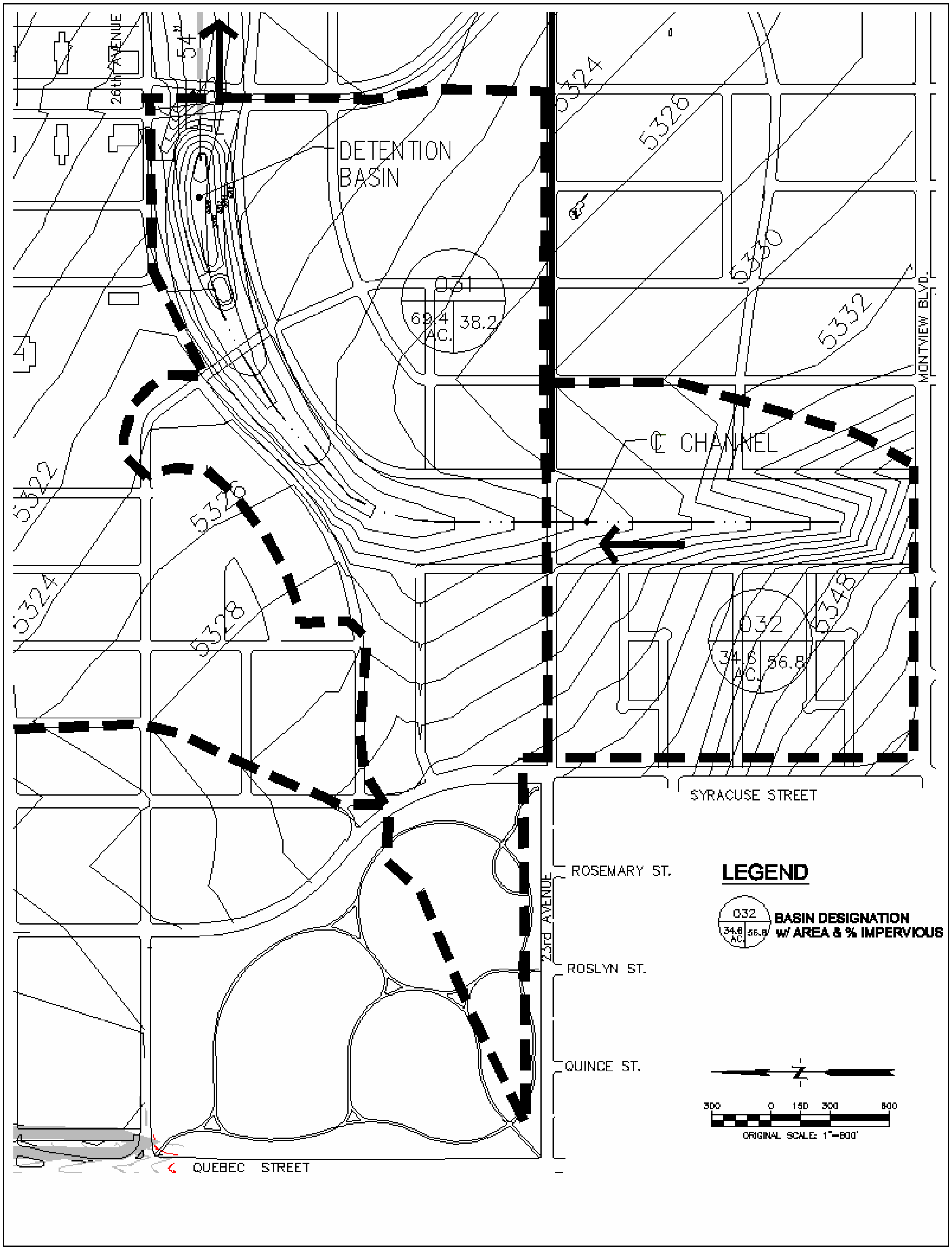


Figure 2—Stapleton Redevelopment Drainage Catchment Map

2.3 Hydrologic Evaluation For Detention Pond Sizing

Three hydrologic methods were used to establish the required detention pond size:

1. The Colorado Urban Hydrograph Procedure (CUHP) and UDSWM
2. The modified FAA Method
3. The V=KA approach

Because of the basin area (greater than 90 acres) and the need to match discharges with the established capacity of an outfall system, the utilization of a more detailed assessment with a synthetic hydrograph generated by CUHP and UDSWM was required. All three methods were used to verify reasonableness of the results and to ensure that appropriate local detention sizing criteria were satisfied.

2.3.1 CUHP and UDSWM

Input data for CUHP and UDSWM are shown in Table 1. Two discharge rates were considered for the pond routing: the allowable release rate and the flow capacity of the 54-inch storm sewer. The allowable release for the 104-acre basin was 88.4 cfs, relating to 0.85 cfs per acre for Type B Soils. The capacity of the 54-inch RCP (n=0.013, slope=0.38%) was 121 cfs and, consequently, the allowable release rate governed the design of the detention volume. Storage characteristics were developed with a preliminary grading plan to enable stage-storage-discharge data to be used in UDSWM routing.

Table 2 presents the modeling results with the required storage volumes for attenuation of flows to the allowable release rate. Figure 3 graphs the inflow and pond discharge hydrographs for the 100-year storm and shows the required minimum detention volume of 8.8 acre-feet.

Table 1—CUHP and UDSWM Input

CUHP Basin Data

Basin	Area (acres)	Imperviousness	Slope	Length (ft)	Time of Concentration (min)	Centroid Length (ft)
031	69.4	38.2%	0.8%	3820	31.2	1600
032	34.6	56.8%	2.0%	1240	16.9	590

Note: Hydrologic Soil Group B Soils are used in this example.

UDSWM Pond Routing Data

Elevation (Feet)	Depth (Feet)	Storage (Acre-feet)	Discharge (cfs)
5308.7	0.0	0.00	0.0
5310.0	1.3	1.99	0.1
5310.0	1.3	2.00	20.0
5312.2	3.5	4.50	23.9
5312.3	3.6	4.60	88.4
5314.0	5.3	8.78	88.4
5314.1	5.4	8.80	90.0
5316.0	7.3	20.00	5000.0

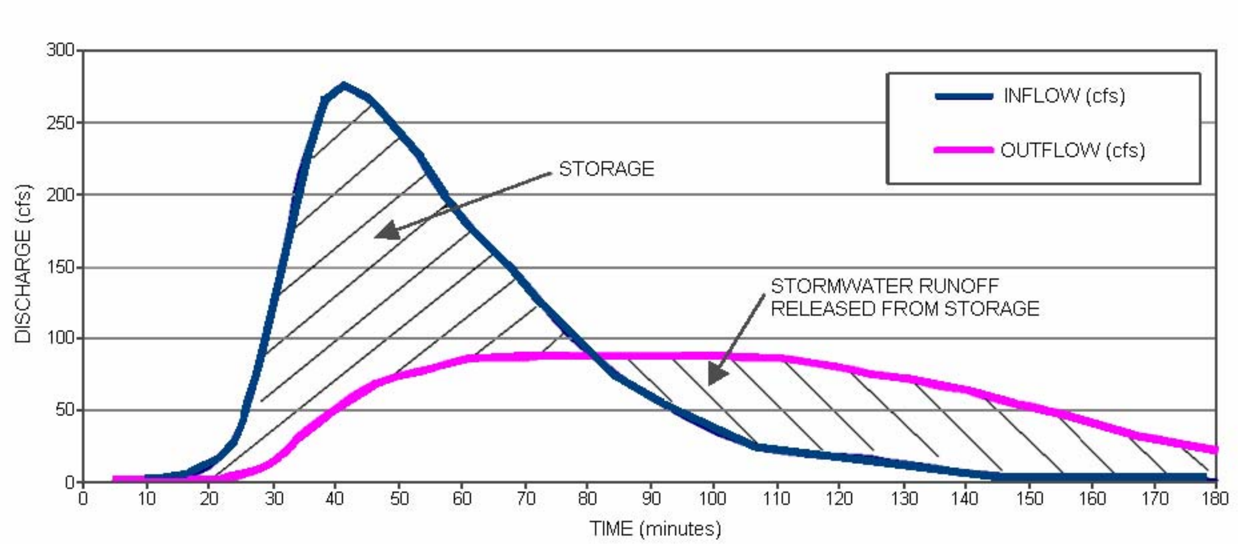


Figure 3—Detention Pond Inflow/Outflow Hydrographs

Table 2—CUHP and UDSWM Modeling Results

Return Period	**Q_{in} (cfs)**	**Q_{out} (cfs)**	**Detention Storage Volume (acre-feet)**
2	44	20	2.1
5	83	22	3.3
10	106	24	4.3
50	222	88	7.0
100	273	88	8.8

2.3.2 Rational Method Hydrology

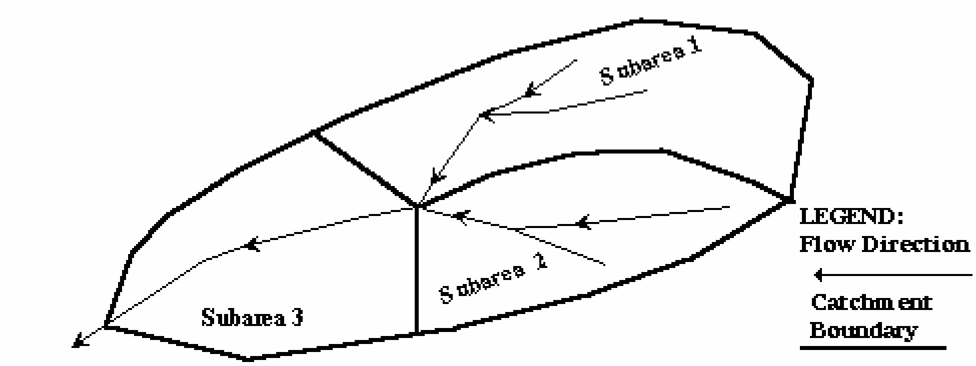
For purposes of this design example, the basin was also analyzed using the Rational Method. Figures 4 and 5 are spreadsheets used to determine the composite runoff coefficients for the basin; they show the 10-year composite runoff coefficient to be 0.55 and the 100-year composite runoff coefficient to be 0.65. By evaluating the basin runoff coefficients, overland flow path, and concentrated flow path, the resulting time of concentration is 35 minutes.

The time of concentration is related to rainfall intensity for use in the Rational Method. By inputting the basin area, runoff coefficients, and rainfall intensity into the Rational Method equation, Q=CIA. Figures 6 and 7 show the 10-year and 100-year peak discharges into the detention pond from the 104-acre drainage basin to be 131 cfs and 250 cfs, respectively.

Area-Weighting for Runoff Coefficient Calculation

Project Title =	Stapleton Redevelopment Area
Catchment ID =	31.1, 31 and 32
Return Period =	10yr (initial event), 100yr (major event)

Illustration



Instructions: For each catchment Sub area, enter values for A and C.

(10-yr Event)

Subarea ID input	Area acres A input	Runoff Coeff C input	Product CA output
31.1A	5.23	0.50	2.62
31.1B	1.10	0.60	0.66
31.1C	1.19	0.50	0.60
31.1D	0.26	0.50	0.13
31.1E	0.42	0.50	0.21
31	61.20	0.50	30.60
32	34.60	0.65	22.49
Sum:	**104.00**	**Sum:**	**57.30**

(100-yr Event)

Subarea ID input	Area acres A input	Runoff Coeff C input	Product CA output
31.1A	5.23	0.60	3.14
31.1B	1.10	0.70	0.77
31.1C	1.19	0.60	0.71
31.1D	0.26	0.60	0.16
31.1E	0.42	0.60	0.25
31	61.20	0.60	36.72
32	34.60	0.75	25.95
Sum:	**104.00**	**Sum:**	**67.70**

Weighted Runoff Coeffecient
(sum CA / sum A) = **0.55** (10-yr) **0.65** (100-yr)

Figures 4 & 5—Area-Weighting for Runoff Coefficient Calculation

CALCULATION OF A PEAK RUNOFF USING RATIONAL METHOD

I. Catchment Hydrologic Data

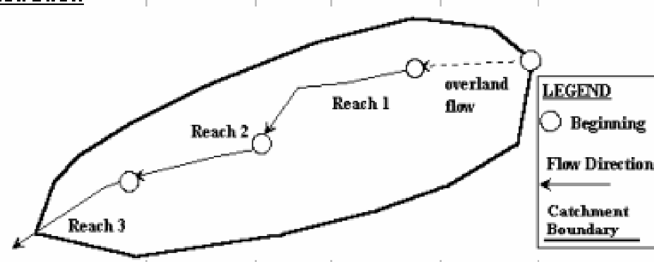
(10-yr Event)			(100-yr Event)	
Catchment ID =	31, 32	(input)	Catchment ID	31, 32
Area (A) =	104.00	(input)	Area (A) =	104.00
Runoff Coeff (C)=	0.55	(input)	Runoff Coeff (C	0.65

II. Rainfall Information I (inch/hr) = C1 * P1 /(C2 + Td)^C3

Tr =	10	years	(input)	Tr =	100	years
C1 =	28.50		(input)	C1 =	28.50	
C2=	10.00		(input)	C2=	10.00	
C3=	0.786		(input)	C3=	0.786	
P1=	1.61	inches	(input)	P1=	2.60	inches

III. Analysis of Flow Time (Time of Concentration) for a Catchment

Illustration



SCS Type	Heavy Meadow	Tillage Field	Forest Woods	Short Pasture	Bare Soil	Grass Swale	Paved Flow
Conveyance	2.50	5.00	5.00	7.00	10.00	15.00	20.00

Calculations:

Reach ID	Slope S ft/ft input	Length L ft input	5-yr Runoff Coeff C-5 input	SCS Convey-ance input	Flow Velocity V fps output	Flow Time Tf minutes output
Overland	0.0050	60.00	0.45		0.09	11.43
1	0.0060	550.00		20.00	1.55	5.92
2	0.0200	1000.00		20.00	2.83	5.89
3	0.0084	3000.00		20.00	1.83	27.28
4						
5						
Sum		4610.00				50.51
					Regional Tc =	35.61
					Min value of Regional Tc and Flow Time is Recommended Tc =	35.61

IV. Peak Runoff Prediction

(10-yr Event)			(100-yr Event)		
I =	2.28	inch/hr	I =	3.68	inch/hr
Qp =	130.56	cfs	Qp =	249.10	cfs

Figures 6 and 7—Calculation of a Peak Runoff Using Rational Method

2.3.3 FAA Method

The modified FAA Method utilizes the Rational Method to estimate detention volumes using a mass diagram. It is appropriate for basins smaller than 160 acres without multiple detention ponds or unusual watershed storage characteristics. Table 3 highlights key input data for use of the FAA Method.

Table 3—FAA Method Input Data

	Area (acres)	Runoff Coefficient C	SCS Soil Type	T_c (min)	Release Rate (cfs/acre)	1-Hour Precip. (in)
10-Year	104	0.55	B	35	0.23	1.60
100-Year	104	0.65	B	35	0.85	2.60

Figure 8 shows the computation of the 10-year storage volume using the FAA method. The plot of mass inflow versus mass outflow is depicted on Figure 9. Figures 10 and 11 show the corresponding information for the 100-year storage volume. The vertical difference between the plots of the 100-year inflow and modified outflow relates to a minimum detention volume of 382,399 cubic feet (8.8 acre-feet).

2.3.4 Denver Regression Equation

For checking purposes, the use of the formula V=KA is required in the Denver Metropolitan area. The formulae for the coefficient, K, for initial and major storm events are stated below.

$K_{10} = (0.95I – 1.90)/1000$

$K_{100} = (1.78I –0.002[I]^2 – 3.56)/1000$

where I = Basin Imperviousness (%)

For a 104-acre basin with an imperviousness of 44%, the corresponding detention volumes are as shown below in Table 4.

Table 4—Detention Volume

	BASIN 031		BASIN 032		TOTAL	
Area =	69.40	acres	34.60	acres	**104.00**	**acres**
Imp. =	38%		57%		**44.4%**	
K_{10} =	0.034		0.052		**0.040**	
K_{100} =	0.062		0.091		**0.072**	
VOL_{10} =	2.387	acre-feet	1.801	acre-feet	**4.188**	**acre-feet**
VOL_{100} =	4.269	acre-feet	3.152	acre-feet	**7.421**	**acre-feet**

For catchments less than 160 acres only. For larger catchments, use hydrograph routing methods.
(Note: for catchments larger than 90 acres, CUHP hydrograph and routing are recommended).

10-YEAR

Design Information (Input)

Catchment Drainage Area	A =	104.00	acres
Runoff Coefficient	C =	0.55	
Predevelopment NRCS Soil Group	Type =	B	B
Return Period for Detention Control	T =	10	10
Time of concentration of Watershed	Tc =	35	minutes
Allowable Unit Release Rate (See Table A)	q =	0.23	Default
One-hour Precipitation	P1 =	1.60	inches

Design Rainfall IDF Formula I = C1* P1/(C2+Td)^C3

Coefficient one	C1 =	28.50	Click Here to Accept Denver Area Default Values
Coefficient two	C2 =	10.00	
Coefficient three	C3 =	0.79	

Determination of Average Outflow from the Basin (Calculated)

Inflow Peak Runoff	Qp-in =	128.92	cfs
Allowable Peak Outflow Rate	Qp-out =	23.92	cfs
Ratio of Qp-out/Qp-in	Ratio =	0.19	

Recommended Unit Flow Release Rate in cfs/acre of tributary catchment within UDFCD boundaries.

Design Frequency	NRCS (SCS) Hydrologic Soil Group		
	A	B	C & D
2-year	0.02	0.03	0.04
5-year	0.07	0.13	0.17
10-year	0.13	0.23	0.30
25-year	0.24	0.41	0.52
50-year	0.33	0.56	0.68
100-year	0.50	0.85	1.00

100-YEAR

Design Information (Input)

Catchment Drainage Area	A =	104.00	acres
Runoff Coefficient	C =	0.65	
Predevelopment NRCS Soil Group	Type =	B	B
Return Period for Detention Control	T =	100	10
Time of concentration of Watershed	Tc =	35	minutes
Allowable Unit Release Rate (See T	q =	0.85	Default
One-hour Precipitation	P1 =	2.60	inches

Design Rainfall IDF Formula I = C1* P1/(C2+Td)^C3

Coefficient one	C1 =	28.50	Click Here to Accept Denver Area Default Values
Coefficient two	C2 =	10.00	
Coefficient three	C3 =	0.79	

Determination of Average Outflow from the Basin (Calculated)

Inflow Peak Runoff	Qp-in =	247.59	cfs
Allowable Peak Outflow Rate	Qp-out =	88.40	cfs
Ratio of Qp-out/Qp-in	Ratio =	0.36	

Recommended Unit Flow Release Rate in cfs/acre of tributary catchmer within UDFCD boundaries.

Design Frequency	NRCS (SCS) Hydrologic Soil Group		
	A	B	C & D
2-year	0.02	0.03	0.04
5-year	0.07	0.13	0.17
10-year	0.13	0.23	0.30
25-year	0.24	0.41	0.52
50-year	0.33	0.56	0.68
100-year	0.50	0.85	1.00

Determination of Detention Volume Using Modified FAA Method

Rainfall duration must be entered in an increasing order.

10-YEAR

Rainfall Duration minutes (input)	Rainfall Intensity inch/hr (output)	Inflow Volume cubic feet (output)	Adjustment Factor (output)	Average Outflow cfs (output)	Outflow Volume cubic feet (output)	Storage Volume cubic feet (output)
5.00	5.37	92,123	1.00	23.92	7,176	84,947
10.00	4.28	146,791	1.00	23.92	14,352	132,439
15.00	3.59	184,600	1.00	23.92	21,528	163,072
20.00	3.10	213,116	1.00	23.92	28,704	184,412
25.00	2.75	235,851	1.00	23.92	35,880	199,971
30.00	2.47	254,687	1.00	23.92	43,056	211,631
35.00	2.25	270,734	1.00	23.92	50,232	220,502
40.00	2.07	284,699	0.94	22.43	53,820	230,879
45.00	1.92	297,056	0.89	21.26	57,408	239,648
50.00	1.80	308,136	0.85	20.33	60,996	247,140
55.00	1.69	318,180	0.82	19.57	64,584	253,596
60.00	1.59	327,368	0.79	18.94	68,172	259,196
65.00	1.51	335,836	0.77	18.40	71,760	264,076
70.00	1.43	343,692	0.75	17.94	75,348	268,344
75.00	1.36	351,020	0.73	17.54	78,936	272,084
80.00	1.30	357,891	0.72	17.19	82,524	275,367
85.00	1.25	364,359	0.71	16.88	86,112	278,247
90.00	1.20	370,471	0.69	16.61	89,700	280,771
95.00	1.15	376,267	0.68	16.37	93,288	282,979
100.00	1.11	381,779	0.68	16.15	96,876	284,903
105.00	1.07	387,035	0.67	15.95	100,464	286,571
110.00	1.04	392,059	0.66	15.77	104,052	288,007
115.00	1.01	396,873	0.65	15.60	107,640	289,233
120.00	0.97	401,493	0.65	15.45	111,228	290,265
125.00	0.95	405,937	0.64	15.31	114,816	291,121
130.00	0.92	410,218	0.63	15.18	118,404	291,814
135.00	0.89	414,348	0.63	15.06	121,992	292,356
140.00	0.87	418,339	0.63	14.95	125,580	292,759
145.00	0.85	422,200	0.62	14.85	129,168	293,032
150.00	0.83	425,940	0.62	14.75	132,756	293,184
155.00	0.81	429,568	0.61	14.66	136,344	293,224
160.00	0.79	433,089	0.61	14.58	139,932	293,157
165.00	0.77	436,512	0.61	14.50	143,520	292,992
170.00	0.75	439,841	0.60	14.42	147,108	292,733
175.00	0.74	443,082	0.60	14.35	150,696	292,386
180.00	0.72	446,241	0.60	14.29	154,284	291,957
185.00	0.71	449,321	0.59	14.22	157,872	291,449

Stormwater Detention Volume (Cubic Feet) = 293,224
Stormwater Detention Volume (Acre Feet) = 6.731

100-YEAR

Rainfall Duration minutes (input)	Rainfall Intensity inch/hr (output)	Inflow Volume cubic feet (output)	Adjustment Factor (output)	Average Outflow cfs (output)	Outflow Volume cubic feet (output)	Storage Volume cubic feet (output)
5.00	8.72	176,919	1.00	88.40	26,520	150,399
10.00	6.95	281,905	1.00	88.40	53,040	228,865
15.00	5.83	354,515	1.00	88.40	79,560	274,955
20.00	5.05	409,280	1.00	88.40	106,080	303,200
25.00	4.47	452,942	1.00	88.40	132,600	320,342
30.00	4.02	489,114	1.00	88.40	159,120	329,994
35.00	3.66	519,932	1.00	88.40	185,640	334,292
40.00	3.37	546,751	0.94	82.88	198,900	347,851
45.00	3.13	570,482	0.89	78.58	212,160	358,322
50.00	2.92	591,761	0.85	75.14	225,420	366,341
55.00	2.74	611,051	0.82	72.33	238,680	372,371
60.00	2.58	628,695	0.79	69.98	251,940	376,755
65.00	2.45	644,958	0.77	68.00	265,200	379,758
70.00	2.32	660,044	0.75	66.30	278,460	381,584
75.00	2.22	674,119	0.73	64.83	291,720	382,399
80.00	2.12	687,313	0.72	63.54	304,980	382,333
85.00	2.03	699,735	0.71	62.40	318,240	381,495
90.00	1.95	711,473	0.69	61.39	331,500	379,973
95.00	1.88	722,604	0.68	60.48	344,760	377,844
100.00	1.81	733,189	0.68	59.67	358,020	375,169
105.00	1.75	743,283	0.67	58.93	371,280	372,003
110.00	1.69	752,932	0.66	58.26	384,540	368,392
115.00	1.63	762,176	0.65	57.65	397,800	364,376
120.00	1.58	771,049	0.65	57.09	411,060	359,989
125.00	1.54	779,583	0.64	56.58	424,320	355,263
130.00	1.49	787,804	0.63	56.10	437,580	350,224
135.00	1.45	795,736	0.63	55.66	450,840	344,896
140.00	1.41	803,400	0.63	55.25	464,100	339,300
145.00	1.38	810,816	0.62	54.87	477,360	333,456
150.00	1.34	817,999	0.62	54.51	490,620	327,379
155.00	1.31	824,965	0.61	54.18	503,880	321,085
160.00	1.28	831,728	0.61	53.87	517,140	314,588
165.00	1.25	838,301	0.61	53.58	530,400	307,901
170.00	1.23	844,695	0.60	53.30	543,660	301,035
175.00	1.20	850,920	0.60	53.04	556,920	294,000
180.00	1.17	856,985	0.60	52.79	570,180	286,805
185.00	1.15	862,900	0.59	52.56	583,440	279,460

Stormwater Detention Volume (Cubic Feet) = 382,399
Stormwater Detention Volume (Acre Feet) = 8.779

Figures 8 and 9—Detention Volume by Modified FAA Method

(See Chapter 5-Runoff of this *Manual* for description of method)

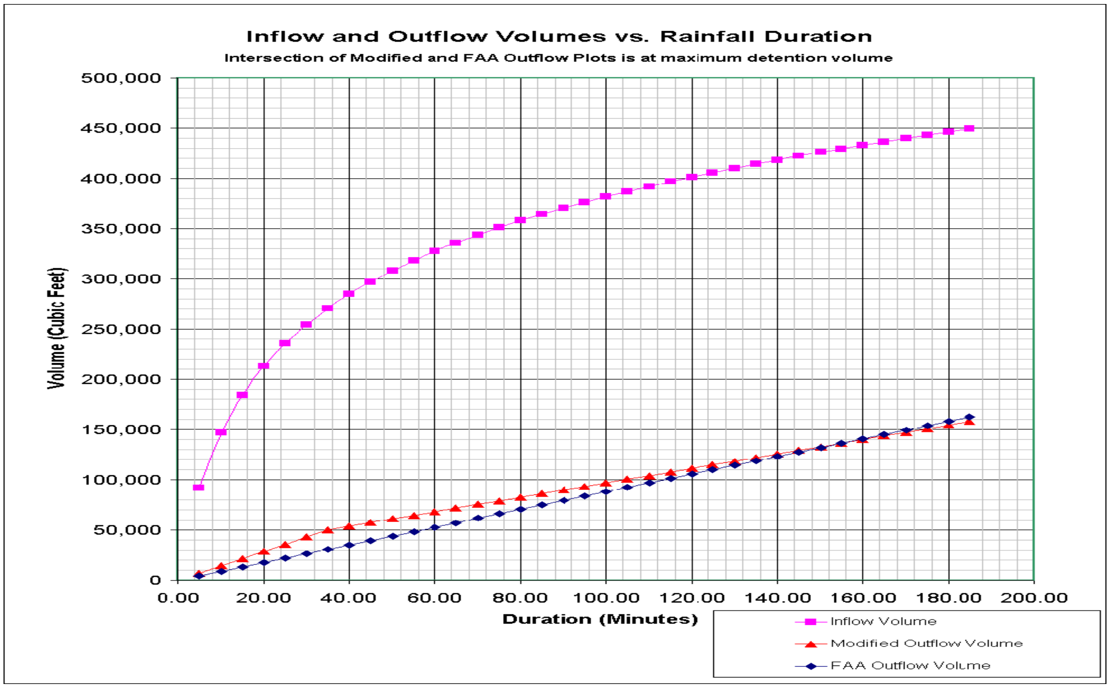


Figure 10—10-Year Modified FAA Method

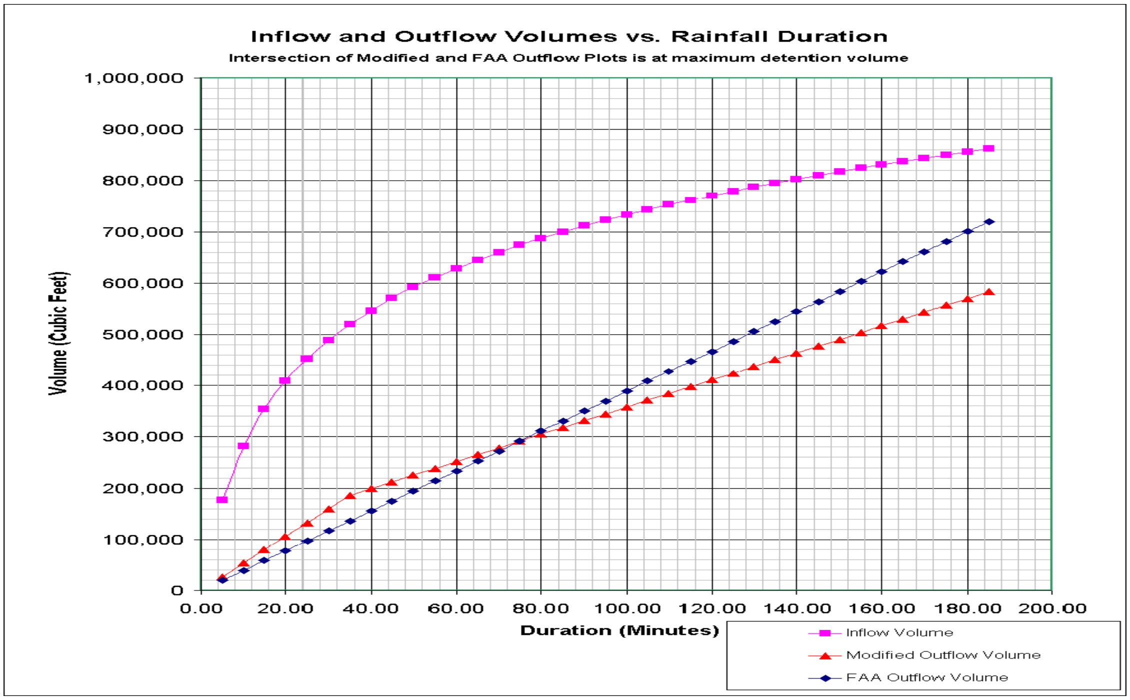


Figure 11—100-Year Modified FAA

2.3.5 Comparison of the Sizing Methodologies

Table 5 offers a comparison of the modeling results for detention sizing.

Table 5—Summary Comparison of Sizing Methodologies

	V=KA (Acre-Feet)	FAA Method (Acre-Feet)	CUHP/SWM (Acre-Feet)
10-Year	4.2	6.7	4.3
100-Year	7.4	8.8	8.8

For the purposes of this design, the results of the CUHP/UDSWM analysis were used with a required storage volume of 8.8 acre-feet.

2.4 Detention Pond Outlet Configuration

A more detailed grading plan and storm sewer layouts for the detention pond area and adjacent roadways are illustrated on Figure 12. In order to prepare a design for the detention pond, it was necessary to confirm the adequacy of pond volume and establish related water surface depths. The outlet had to be designed to restrict discharges to the design criteria for each storm event and corresponding depth (and hydraulic head) condition. Additionally, the water quality capture volume (WQCV) had to be computed and included in the design volume.

Other objectives of the pond design included:

- For aesthetic purposes, the landscape architect determined that a more elongated and contoured shape was desirable.
- In order to provide for safety and to address the potential risk associated with the adjacent elementary school site, a dry detention pond scheme was selected. A maximum depth of 6 ft was provided and a more flatly graded perimeter area was chosen as a safety shelf.
- A multi-stage outlet was designed to control discharges of the WQCV, 10-year, and 100-year events.
- An overflow spillway and overland channel to Westerly Creek had to be provided for events greater than the 100-year storm and emergency operations.
- Due to the embankment height of less than 10 feet, the Colorado State Engineer did not regulate the pond and a Probable Maximum Flood (PMF) analysis was not required. However, in final design the emergency spillway must be designed for the un-attenuated inflow peak 100-year flow rate of 273 cfs or more and the embankment stability checked for a total flow of 273 cfs.

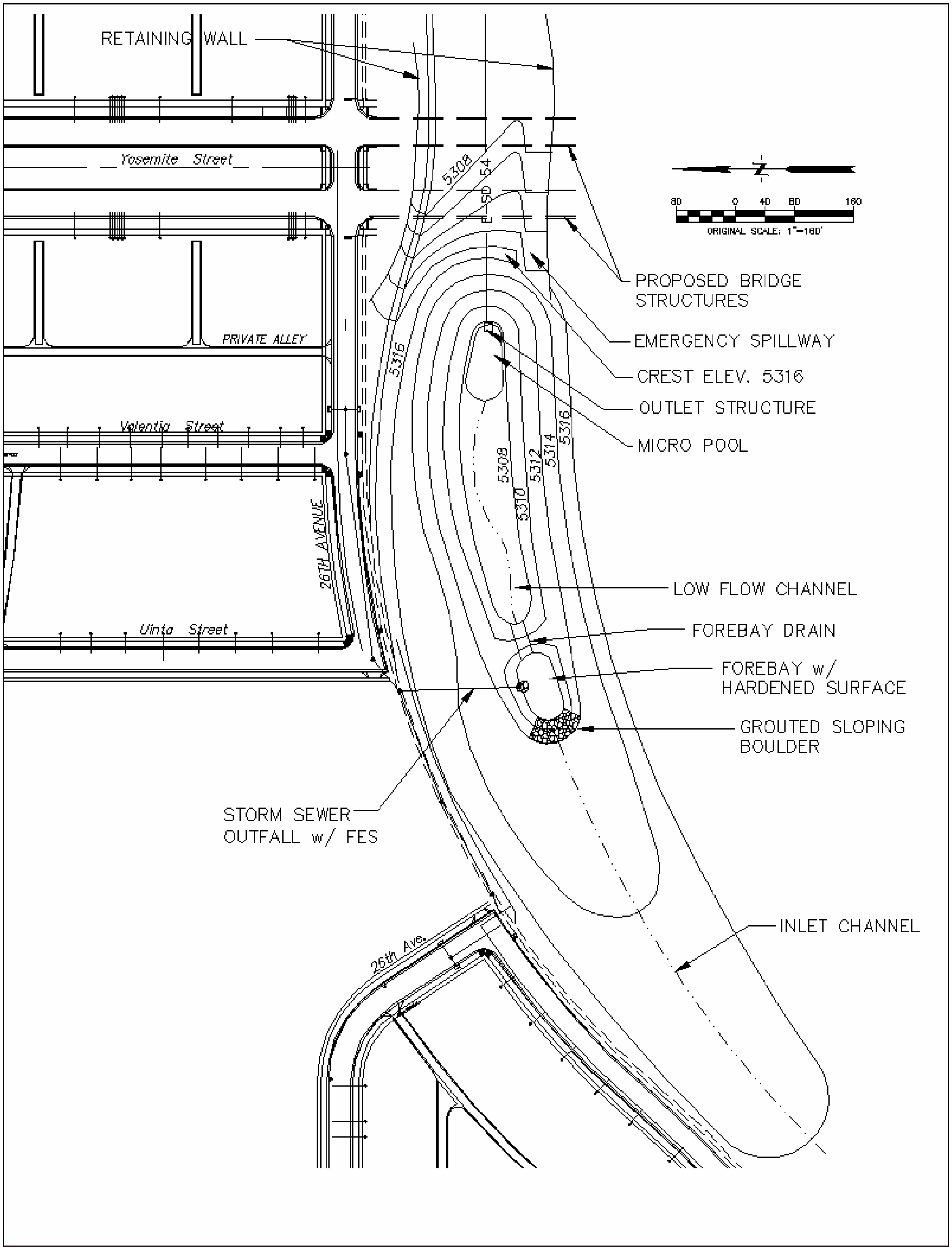


Figure 12—Stapleton Redevelopment Detention Pond Detail

2.4.1 Stage-Storage Relationships

To properly size the outlet works, it is important to develop depth versus cumulative storage volume relationships for the final detention pond configuration, as shown on Table 6. Figure 13 graphically shows the rating curve for the pond.

Table 6—Stapleton East-West Detention Pond Cumulative Volume Analysis

Contour (feet)	Area (sq. ft.)	Avg Area (sq. ft.)	Volume (cu. ft.)	Cum. Vol. (cu. ft.)	Cum. Vol. (ac-ft)
5306	2,788				
		10,992	21,984	21,984	0.50
5308	22,303				
		28,992	57,983	79,967	1.84
5310	36,242				
		52,065	104,131	184,098	4.23
5312	69,696				
		102,551	205,102	389,200	8.93
5314	139,392				
		188,602	377,203	766,403	17.59
5316	242,542				

2.4.2 Water Quality Volume Requirements

The WQCV must also be determined and incorporated into the pond design. Figure 14 (3 pages) shows the computation of the WQCV from the **Extended Dry Detention Spreadsheet** of Volume 3 of this *Manual.* This computation includes the analysis of the perforated plate, trash rack, forebay, micro-pool and outlet structure components for proper operation. As indicated on line 1(D), a volume of 1.99 acre-feet will be required. Figure 15 is the same analysis of the perforated plate for WQCV using the newly developed spreadsheet from Volumes 1 and 2 of this *Manual*. This computation shows a total of 20 holes (1.50-inch diameter with 5 columns and 4 rows) that will release runoff at the appropriate rate for water quality treatment. Figure 16 is the analysis of the 10-year pond outlet orifice to accomplish the desired release rate of 0.23 cfs/acre (Type B soils), or 24 cfs for a 104-acre drainage basin. Figure 17 is the computation form for the 100-year release rate of 0.80 cfs/acre (Type B soils), or 88 cfs for the drainage catchment area.

2.4.3 Final Pond Outlet Configuration

The final recommended outlet configuration is shown in plan and section view in Figure 18. As shown the WQCV of 2.0 acre-feet will require a ponded depth of 1.3 feet. The 100-year detention volume of 8.8 acre-feet will pond to a depth of 5.3 feet (excluding the micro-pond). These include the WQCV released over a 40-hour period. A horizontal grate at elevation 5313 controls the 100-year event.

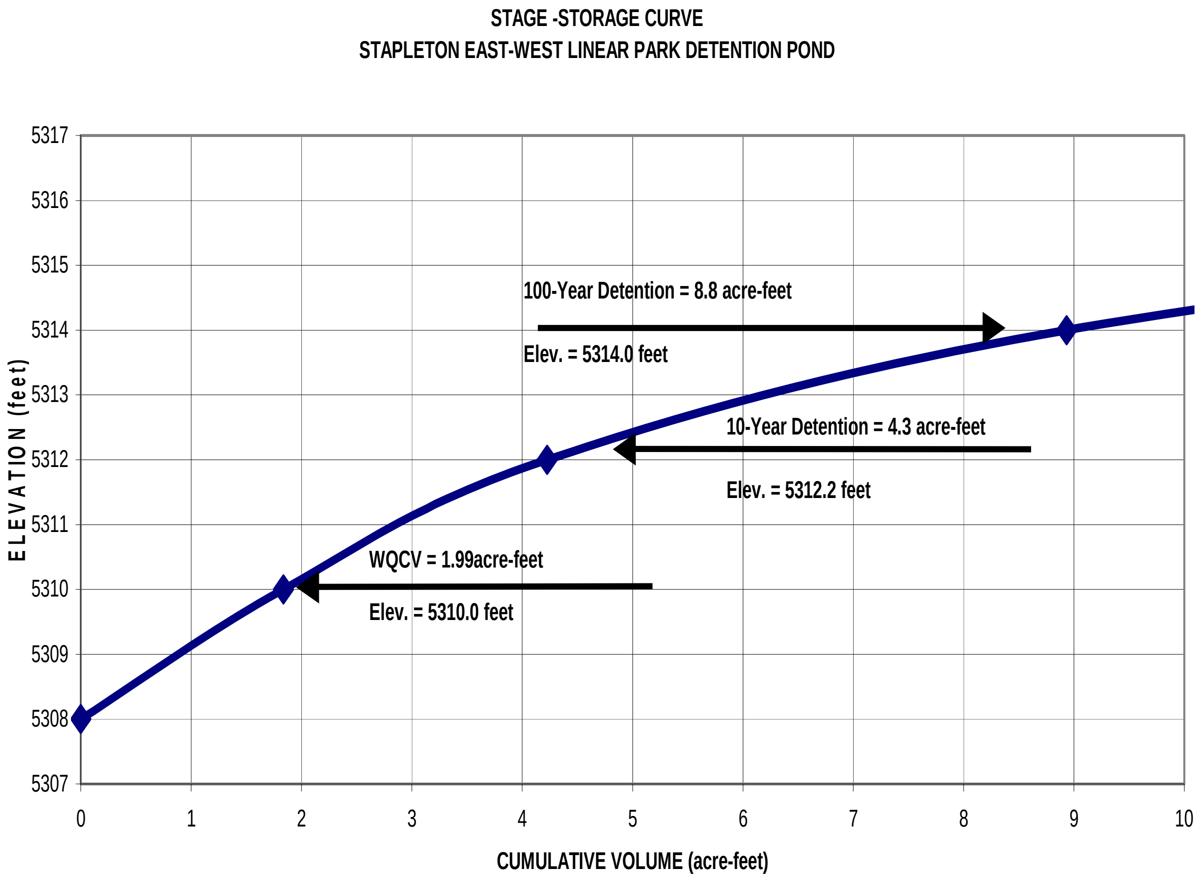


Figure 13—Stage-Storage Curve Stapleton East-West Linear Park Detention Pond

Sheet 1 of 3

Figure 14

Designer:	
Company:	
Date:	**February 9, 2001**
Project:	**UDFCD Example**
Location:	**Stapleton Redevelopment**

1. Basin Storage Volume			
	I_a =	44.40	%
A) Tributary Area's Imperviousness Ratio ($i = I_a / 100$)	i =	0.44	
B) Contributing Watershed Area (Area)	Area =	104.00	acres
C) Water Quality Capture Volume (WQCV) (WQCV =1.0 * (0.91 * I^3 - 1.19 * I^2 + 0.78 * I))	WQCV =	0.19	watershed inches
D) Design Volume: Vol = (WQCV / 12) * Area * 1.2	Vol =	1.990	acre-feet
2. Outlet Works			
A) Outlet Type (Check One)	X	Orifice Plate	
		Perforated Riser Pipe	
		Other:	
B) Depth at Outlet Above Lowest Perforation (H)	H =	1.30	feet
C) Required Maximum Outlet Area per Row, (A_o)	A_o =	9.57	square inches
D) Perforation Dimensions **(enter one only)**:			
i) Circular Perforation Diameter **OR**	D =	1.5000	inches, **OR**
ii) 2" Height Rectangular Perforation Width	W =		inches
E) Number of Columns (nc, See Table 6a-1 For Maximum)	nc =	5	number
F) Actual Design Outlet Area per Row (A_o)	A_o =	8.84	square inches
G) Number of Rows (nr)	nr =	4	number
H) Total Outlet Area (A_{ot})	A_{ot} =	34.46	square inches
3. Trash Rack			
A) Needed Open Area: A_t = 0.5 * (Figure 7 Value) * A_{ot}	A_t =	1,102	square inches
B) Type of Outlet Opening (Check One)	X	≤ 2" Diameter **Round**	
		2" High **Rectangular**	
		Other:	
C) For 2", or Smaller, **Round Opening** (Ref.: Figure 6a):			
i) Width of Trash Rack and Concrete Opening (W_{conc}) from Table 6a-1	W_{conc} =	60	inches
ii) Height of Trash Rack Screen (H_{TR})	H_{TR} =	40	inches

Figure 14—Design Procedure For Extended Detention Basin Sedimentation Facility

Design Procedure Form: Extended Detention Basin (EDB) - Sedimentation Facility

Sheet 2 of 3

Designer:
Company:
Date: **February 9, 2001**
Project: **UDFCD Example**
Location: **Stapleton Redevelopment**

Figure 14

iii) Type of Screen (Based on Depth H), Describe if "Other"	**X**	S.S. #93 VEE Wire (US Filter)	
		Other:	
iv) Screen Opening Slot Dimension, Describe if "Other"	**X**	0.139" (US Filter)	
		Other:	
v) Spacing of Support Rod (O.C.)	1.00	inches	
Type and Size of Support Rod (Ref.: Table 6a-2)	**TE 0.074 in. x 1.00 in.**		
vi) Type and Size of Holding Frame (Ref.: Table 6a-2)	**1.25 in. x 1.50 in. angle**		
D) For 2" High **Rectangular Opening** (Refer to Figure 6b):			
I) Width of Rectangular Opening (W)	W =		inches
ii) Width of Perforated Plate Opening (W_{conc} = W + 12")	W_{conc} =		inches
iii) Width of Trashrack Opening ($W_{opening}$) from Table 6b-1	$W_{opening}$ =		inches
iv) Height of Trash Rack Screen (H_{TR})	H_{TR} =		inches
v) Type of Screen (based on depth H) (Describe if "Other")		Klemp™ KPP Series Aluminum	
		Other:	
vi) Cross-bar Spacing (Based on Table 6b-1, Klemp™ KPP Grating). Describe if "Other"		inches	
		Other:	
vii) Minimum Bearing Bar Size (Klemp™ Series, Table 6b-2) (Based on depth of WQCV surcharge)			
4. Detention Basin length to width ratio		4.00	(L/W)
5 Pre-sedimentation Forebay Basin - Enter design values			
A) Volume (5 to 10% of the Design Volume in 1D)		0.199	acre-feet
B) Surface Area		0.199	acres
C) Connector Pipe Diameter (Size to drain this volume in 5-minutes under inlet control)		24	inches
D) Paved/Hard Bottom and Sides		**Y**	yes/no

Design Procedure Form: Extended Detention Basin (EDB) - Sedimentation Facility

Sheet 3 of 3

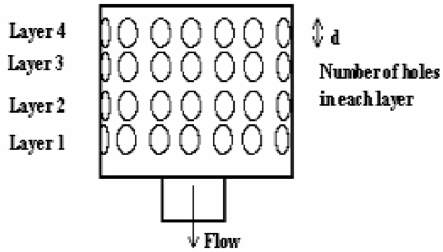
Figure 14

Designer:	
Company:	
Date:	**February 9, 2001**
Project:	**UDFCD Example**
Location:	**Stapleton Redevelopment**

6. Two-Stage Design			
A) Top Stage (D_{WQ} = 2' Minimum)	D_{WQ} =	2.00	feet
	Storage=	1.493	acre-feet
B) Bottom Stage ($D_{BS} = D_{WQ}$ + 1.5' Minimum, D_{WQ} + 3.0' Maximum,	D_{BS} =	3.50	feet
Storage = 5% to 15% of Total WQCV)	Storage=	0.299	acre-feet
	Surf. Area=	0.085	acres
C) Micro Pool (Minimum Depth = the Larger of	Depth=	2.50	feet
0.5 * Top Stage Depth or 2.5 Feet)	Storage=	0.214	acre-feet
	Surf. Area=	0.086	acres
D) Total Volume: Vol_{tot} = Storage from 5A + 6A + 6B	Vol_{tot} =	1.990	acre-feet
Must be $\geq$ Design Volume in 1D			
7. Basin Side Slopes (Z, horizontal distance per unit vertical)	Z =	4.00	(horizontal/vertical)
Minimum Z = 4, Flatter Preferred			
8. Dam Embankment Side Slopes (Z, horizontal distance	Z =	4.00	(horizontal/vertical)
per unit vertical) Minimum Z = 4, Flatter Preferred			
9. Vegetation (Check the method or describe "Other")	X	Native Grass	
		Irrigated Turf Grass	
		Other:	

Notes:

Flow Capacity of a Riser (Inlet Control)



1 Description of Riser

Diameter of holes	d =	1.500	in
Number of holes per layer	n =	5	
Vertical Distance Between Holes	h =	4	in
Orifice Discharge Coefficient	C_o =	0.60	
Total opening area at each layer	A_o =	8.8358	sq in
		0.0614	sq ft

2 Calculation of Collection Capacity

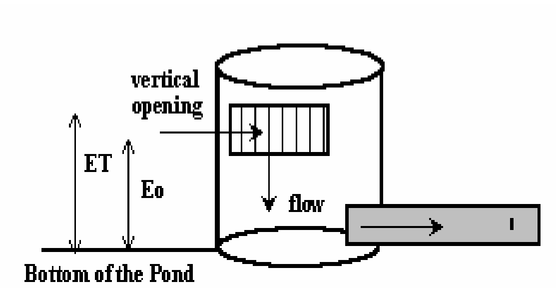
The starting elevation of water surface must be >= central elevation of the first layer.

Starting with Layer 1, enter the central elevation of each layer of holes (up to 10 layers) to calculate flow rates.

Elevations of water surface must be entered in an increasing order.

	Water Surface Elevation ft (input)	Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6	Layer 7	Layer 8	Layer 9	Layer 10	Flow Rate cfs
	Central Elevations of Layers of Holes **in feet**	5308.76	5309.09	5309.43	5309.76							
		Collection Capacity for Each Layer of Holes in cfs										
start	5308.80	0.06	0.00	0.00	0.00							**0.06**
	5308.90	0.11	0.00	0.00	0.00							**0.11**
	5309.00	0.14	0.00	0.00	0.00							**0.14**
	5309.10	0.17	0.02	0.00	0.00							**0.20**
	5309.20	0.20	0.10	0.00	0.00							**0.29**
	5309.30	0.22	0.13	0.00	0.00							**0.35**
	5309.40	0.24	0.16	0.00	0.00							**0.40**
	5309.50	0.25	0.19	0.08	0.00							**0.52**
	5309.60	0.27	0.21	0.12	0.00							**0.60**
	5309.70	0.29	0.23	0.15	0.00							**0.67**
	5309.80	0.30	0.25	0.18	0.06							**0.79**
	5309.90	0.32	0.27	0.20	0.11							**0.89**
	5310.00	0.33	0.28	0.22	0.14							**0.98**
	5311.00	0.44	0.41	0.37	0.33							**1.55**
	5312.00	0.53	0.50	0.47	0.44							**1.95**
	5313.00	0.61	0.58	0.56	0.53							**2.28**
	5314.00	0.68	0.65	0.63	0.61							**2.57**
	5315.00	0.74	0.72	0.70	0.68							**2.83**
	5316.00	0.79	0.78	0.76	0.74							**3.07**

Figure 15—Flow Capacity of a Riser (Inlet Control)



1 Description of Vertical Orifice

Net Opening Area	A_o =	4.2	sq ft
Orifice Coefficient	C_o =	0.65	
Top Elevation of Orifice Opening Area	E_t =	5312.00	ft
Center Elevation of Orifice Opening	E_o =	5311.00	ft

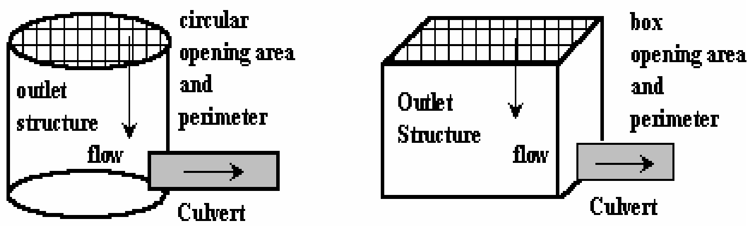
2 Calculation of Collection Capacity

The starting elevation of water surface >= top of the orifice opening.

Elevations of water surface must be entered in an increasing order.

	Water Surface Elevation ft (input)	Collection Capacity cfs (output)
start	5312.00	21.91
	5312.10	22.98
	5312.20	24.00
	5312.30	24.98
	5312.40	25.92
	5312.50	26.83
	5312.60	27.71
	5312.70	28.56
	5312.80	29.39
	5312.90	30.20
	5313.00	30.98

Figure 16—Collection Capacity of Vertical Orifice (Inlet Control)



1 Description of Horizontal Orifice

Net Opening Area (after Trash Rack Reduction)	A_o =	50.0	sq ft
Net Perimeter as Weir Length	L_w =	30.0	ft
Orifice Coefficient	C_o =	0.560	
Weir Coefficient	C_w =	3.000	
Center Elevation of Orifice Opening	E_o =	5313.00	ft

2 Calculation of Collection Capacity

The starting elevation of water surface must be >= Eo
Elevations of water surface must be entered in an increasing order.

	Water Surface Elevation ft (input)	Weir Flow cfs (output)	Orifice Flow cfs (output)	Collection Capacity cfs (output)
start	5313.00	0.00	0.00	0.00
	5313.10	2.85	71.06	2.85
	5313.20	8.05	100.49	8.05
	5313.30	14.79	123.07	14.79
	5313.40	22.77	142.11	22.77
	5313.50	31.82	158.89	31.82
	5313.60	41.83	174.05	41.83
	5313.70	52.71	188.00	52.71
	5313.80	64.40	200.98	64.40
	5313.90	76.84	213.17	76.84
	5314.00	90.00	224.70	90.00

Figure 17—Collection Capacity of Horizontal Orifice (Inlet Control)

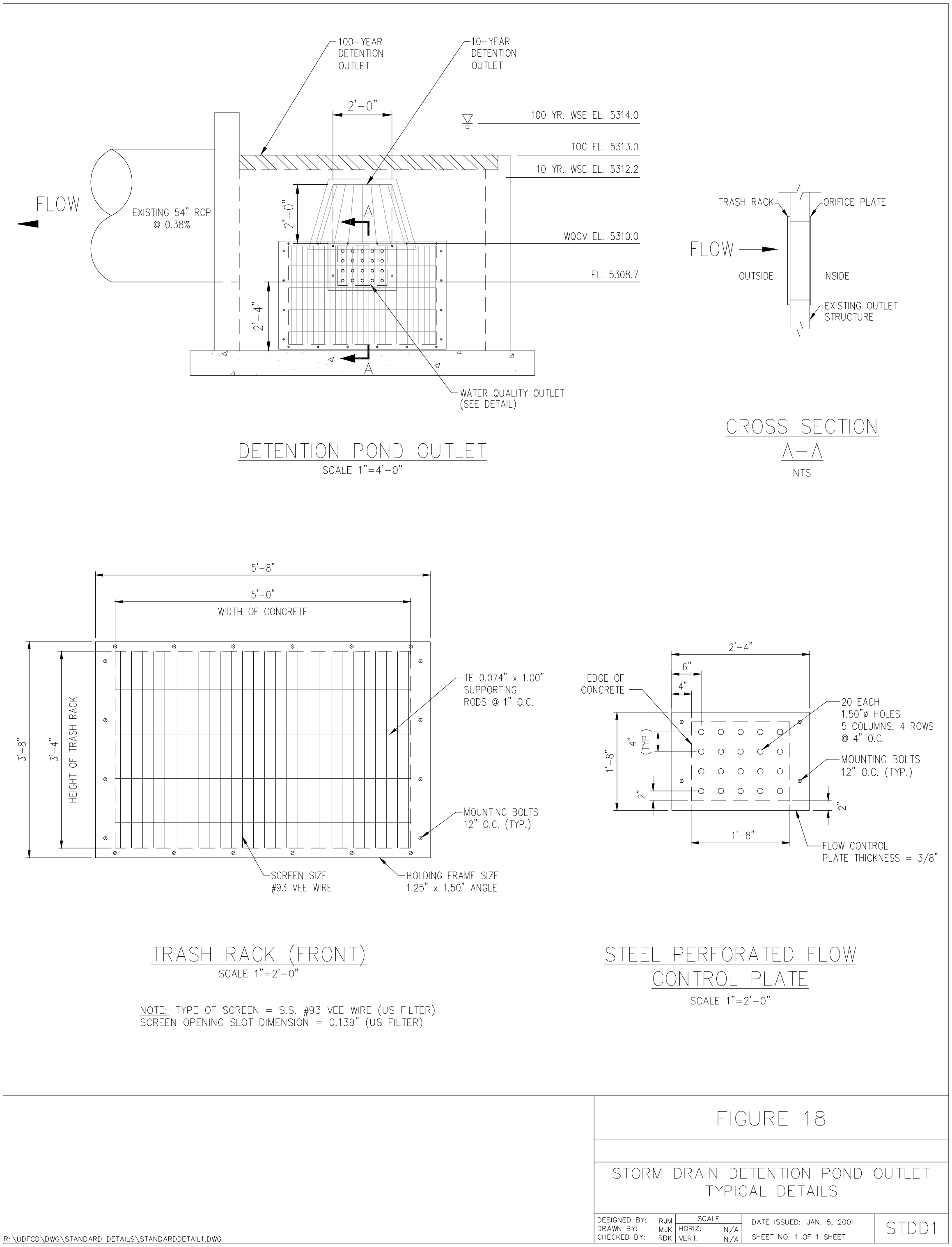


Figure 18—Detention Pond Outlet

2.5 Hydraulic Analysis And Capacity Verification Of The Existing Outfall

The capacity of the existing 54-inch storm sewer is a critical consideration in the design of the East-West Linear Park drainage system. Because the system outfalls to a major drainageway (Westerly Creek) that may create a tailwater control during peak flood flow conditions, a more detailed standard-step backwater analysis was performed. Figure 19 presents the profile of the existing pipeline.

The standard-step backwater is based on Manning's Equation to compute friction losses. Minor (form) losses should also be accounted for using the equations and factors described in the STREETS/INLETS/ STORM SEWERS chapter of this *Manual*. Figure 20 tabulates the computational process for the 100-year storm and a discharge rate of 88.4 cfs. The 100-year Westerly Creek floodplain elevation at the outfall of 5,304 ft is used as the beginning water surface elevation. Figure 21 provides a plot of the computed hydraulic grade line (HGL) and energy grade line (EGL) for the system. As indicated by an HGL above the crown of the pipe, a pressure flow condition exists for the 100-year storm. Because the 100-year HGL at the inlet is below the crown of pipe (outlet controlled), the allowable release rate of 88.4 cfs was used in the design of a multi-stage outlet (versus a restricting pipe capacity).

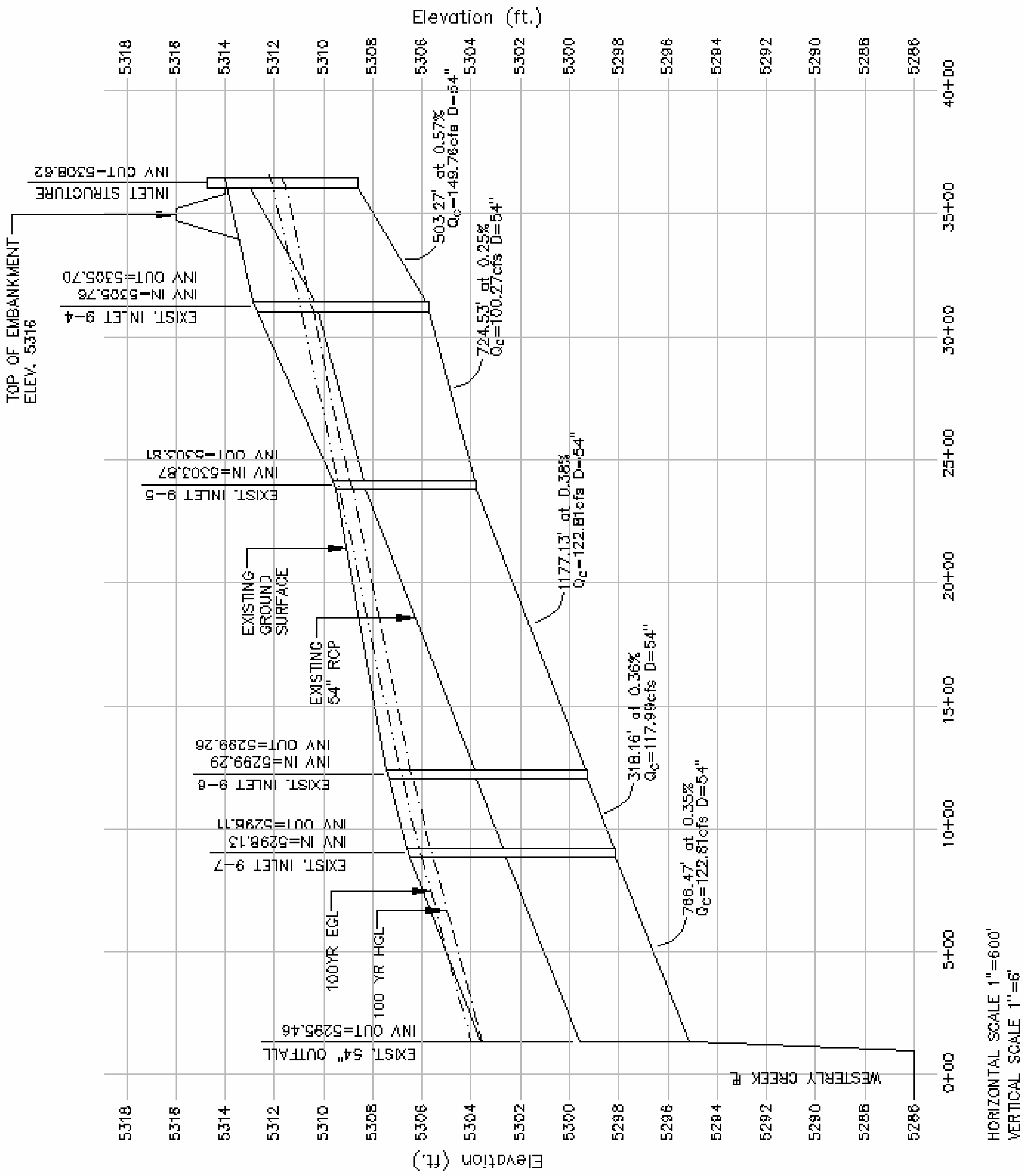


Figure 19—54” Pipe Outfall Profile

HYDRAULIC DESIGN OF STORM SEWER SYSTEMS

STANDARD STEP BACKWATER ANALYSIS FOR FULL PIPE GEOMETRY

PROJECT: Stapleton East-West Linear Park Outfall

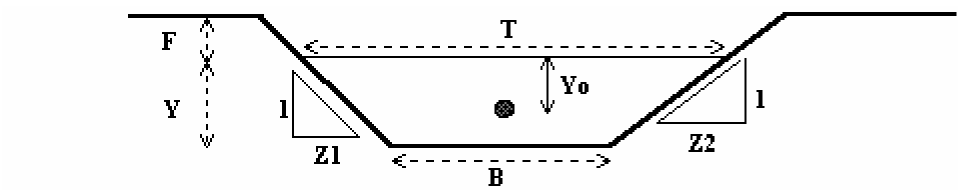
Manning's N-Value = 0.013 Full Flow Factor = 0.9

NOTES:
1 Computed values shown in Italics. All other values are required input
2 Freeboard criteria: HGL at or below rim or grnd.
3 Starting EGL set at Westerly Creek 100-Year floodplain elevation, assuming velocity head in Westerly Creek is negligible at culvert entrance

Design Point	Rim or Grnd. Elev.	Inv.	Sewer Grade	E.G.L.	U/S pipe dia.	Area	Q	Vel.	Vel. Hd.	H.G.L	Friction Slope	Pipe Length	Frict. Loss	Junction Loss		Exit/Form Loss		Total Losses		Freel
									Hv		Sf	L	Hf	Km	Hm	Ke	He	frict.	other	HGL
	(ft)	(ft)	%	(ft)	(in)	(sq.ft)	(cfs)	(fps)	(ft)	(ft)	(ft/ft)	(ft)	(ft)		(ft)		(ft)	(ft)	(ft)	(ft)
Westerly Creek	5305.0	5295.45		5304.00	54	*15.90*	*88.4*	*5.6*	*0.48*	5303.52										1.5
			0.35								0.00201	766.5	*1.54*	0	*0.00*	1	*0.48*	*1.54*	*0.48*	
Inlet #9-7, d/s	5306.6	5298.11		5306.02	54	*15.90*	*88.4*	*5.6*	*0.48*	5305.54										1.0
			n/a								0.00201	0.1	*0.00*	0.75	*0.12*	0.05	*0.02*	*0.00*	*0.14*	
Inlet #9-7, u/s	5306.6	5298.13		5306.17	54	*15.90*	*88.4*	*5.6*	*0.48*	5305.69										0.9
			0.36								0.00201	318.2	*0.64*	1	*0.00*		*0.00*	*0.64*	*0.00*	
Inlet #9-6, d/s	5307.4	5299.26		5306.80	54	*15.90*	*88.4*	*5.6*	*0.48*	5306.33										1.1
			n/a								0.00201	0.1	*0.00*	0.75	*0.12*	0.05	*0.02*	*0.00*	*0.14*	
Inlet #9-6, u/s	5307.4	5299.29		5306.95	54	*15.90*	*88.4*	*5.6*	*0.48*	5306.47										0.9
			0.38								0.00201	1177.1	*2.37*	1	*0.00*		*0.00*	*2.37*	*0.00*	
Inlet #9-5, d/s	5309.5	5303.81		5309.32	54	*15.90*	*88.4*	*5.6*	*0.48*	5308.84										0.7
			n/a								0.00201	0.1	*0.00*	0.75	*0.12*	0.05	*0.02*	*0.00*	*0.14*	
Inlet #9-5, u/s	5309.5	5303.87		5309.46	54	*15.90*	*88.4*	*5.6*	*0.48*	5308.98										0.5
			0.25								0.00201	724.5	*1.46*	1	*0.00*		*0.00*	*1.46*	*0.00*	
Inlet #9-4, d/s	5312.8	5305.70		5310.92	54	*15.90*	*88.4*	*5.6*	*0.48*	5310.44										2.4
			n/a								0.00201	0.1	*0.00*	0.75	*0.12*	0.05	*0.02*	*0.00*	*0.14*	
Inlet #9-4, u/s	5312.8	5305.76		5311.06	54	*15.90*	*88.4*	*5.6*	*0.48*	5310.58										2.2
			0.57								0.00201	503.3	*1.01*	1	*0.00*		*0.00*	*1.01*	*0.00*	
Inlet #9-3, d/s	5314.0	5308.62		5312.07	54	*15.90*	*88.4*	*5.6*	*0.48*	5311.59										2.4
			n/a								0.00201	0.1	*0.00*	0.75	*0.12*	0.05	*0.02*	*0.00*	*0.14*	
Inlet #9-3, u/s	5314.0	5308.77		5312.22	54	*15.90*	*88.4*	*5.6*	*0.48*	5311.74										2.2

Figure 20—Hydraulic Design of Storm Sewer Systems

Project = STAPLETON REDEVELOPMENT
Channel ID = DETENTION POND EMERGENCY OVERFLOW CHANNEL



Design overflow channel for 100-year peak inflow without attenuation (273 cfs).

Design Information (Input)			
Channel Invert Slope	So =	0.0030	ft/ft
Channel Manning's N	N =	0.038	
Bottom Width	B =	30.0	ft
Left Side Slope	Z1 =	4.0	ft/ft
Right Side Slope	Z2 =	4.0	ft/ft
Freeboard Height	F =	1.0	ft
Design Water Depth	Y =	**2.25**	ft
Normal Flow Condition (Calculated)			
Discharge	**Q =**	**279.6**	cfs
Froude Number	**Fr =**	**0.42**	
Flow Velocity	**V =**	**3.2**	ft
Flow Area	A =	87.8	ft
Top Width	T =	48.0	sq ft
Wetted Perimeter	P =	48.6	ft
Hydraulic Radius	R =	1.8	fps
Hydraulic Depth	D =	1.8	ft
Specific Energy	Es =	2.4	ft
Centroid of Flow Area	Yo =	1.0	ft
Specific Force	Fs =	7.4	klb's

Figure 21—Normal Flow Analysis - Trapezoidal Channel

2.6 Local Storm Sewer Design

The detention facility will adequately provide subregional storage for sub-basins 031 and 032 to protect downstream structures and control discharges to Westerly Creek. It will be essential to provide a conveyance system within the local sub-basins to collect and safely transport stormwater to the detention pond. Similar to most drainage systems, the Stapleton East-West Linear Park Flood Control Project utilizes a combination of roadway, open channel, and formal storm sewers for these purposes.

Figure 22 illustrates local basin 031 with further delineation of tributary areas (031.1A through 031.1E) to allow computation of hydrologic and hydraulic conditions at major intersections and inlet locations. An enlarged view of the storm sewer layout is shown on Figure 23, including an initial set of inlets at the intersection of 24^{th} and 26^{th} Avenues and installation of 24-inch RCP for conveyance to the detention pond.

2.6.1 Determination of Allowable Street Capacity

Inlets are provided to drain intersections without excessive encroachment and at street locations where needed to maintain allowable inundation depths for the initial and major storm events. Figure 24 shows computation of street capacity for the initial storm (2-year) with a normal depth, Y, to the top of curb. The corresponding capacity, Q_{max}, is 7.06 cfs. A similar calculation is performed in Figure 25 for the major storm for the specific roadway cross-section being constructed using Manning's Equation and the allowable depths indicated in this *Manual*. The corresponding capacity, Q_{max}, is 87.5 cfs.

2.6.2 Determination of Inlet Hydrology

The Rational Method is used to determine peak discharges for the local tributary area to each inlet. Figure 26 shows computation of the 2-year discharge for sub-basin 0.31.1B and the corresponding flow rate of 1.06 cfs. A check of the flow conditions in the street is provided on Figure 27 for 1.1 cfs and computation of the V_sD (velocity times depth product) to be 0.61 ft^2/sec.

2.6.3 Inlet Capacity Calculations

Figure 28 demonstrates use of the **UDINLET** spreadsheet for a **Curb Opening Inlet in a Sump** for inlet 26-5A. For the 2-year discharge of 1.1 cfs, a 6-foot curb opening in a sump condition will provide full capture (with a maximum capacity of 6.8 cfs).

2.6.4 Street and Storm Sewer Conveyance Computations

To determine the appropriate combination of inlet, storm sewer, and street conveyance capacity, a detailed hydrologic and hydraulic analysis must be performed for each tributary area under initial (2-year) and major (100-year) conditions. The computational spreadsheets shown on Figures 29 and 30 present these analyses for the local street and storm sewer system.

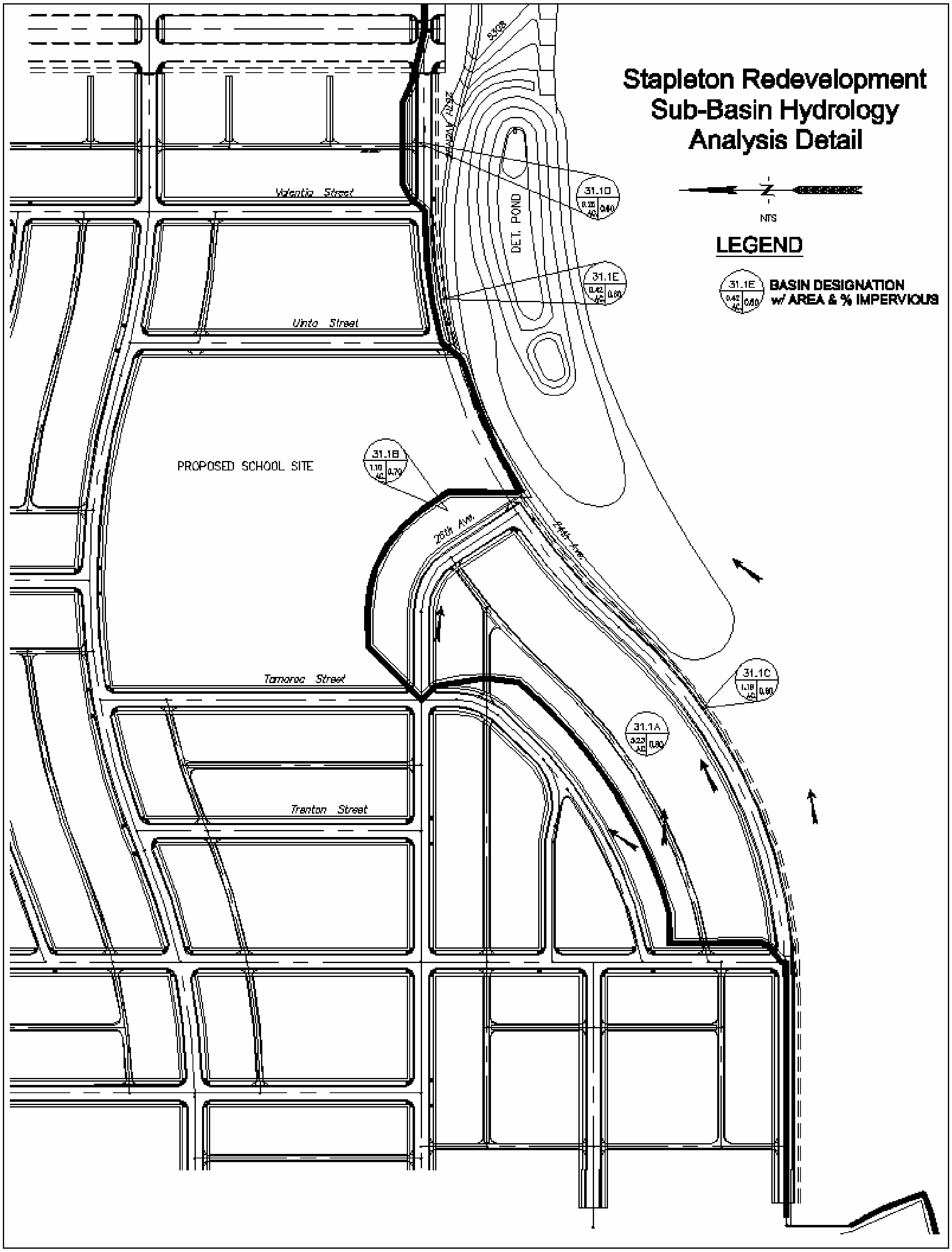


Figure 22—Sub-Basin Hydrology Analysis Detail

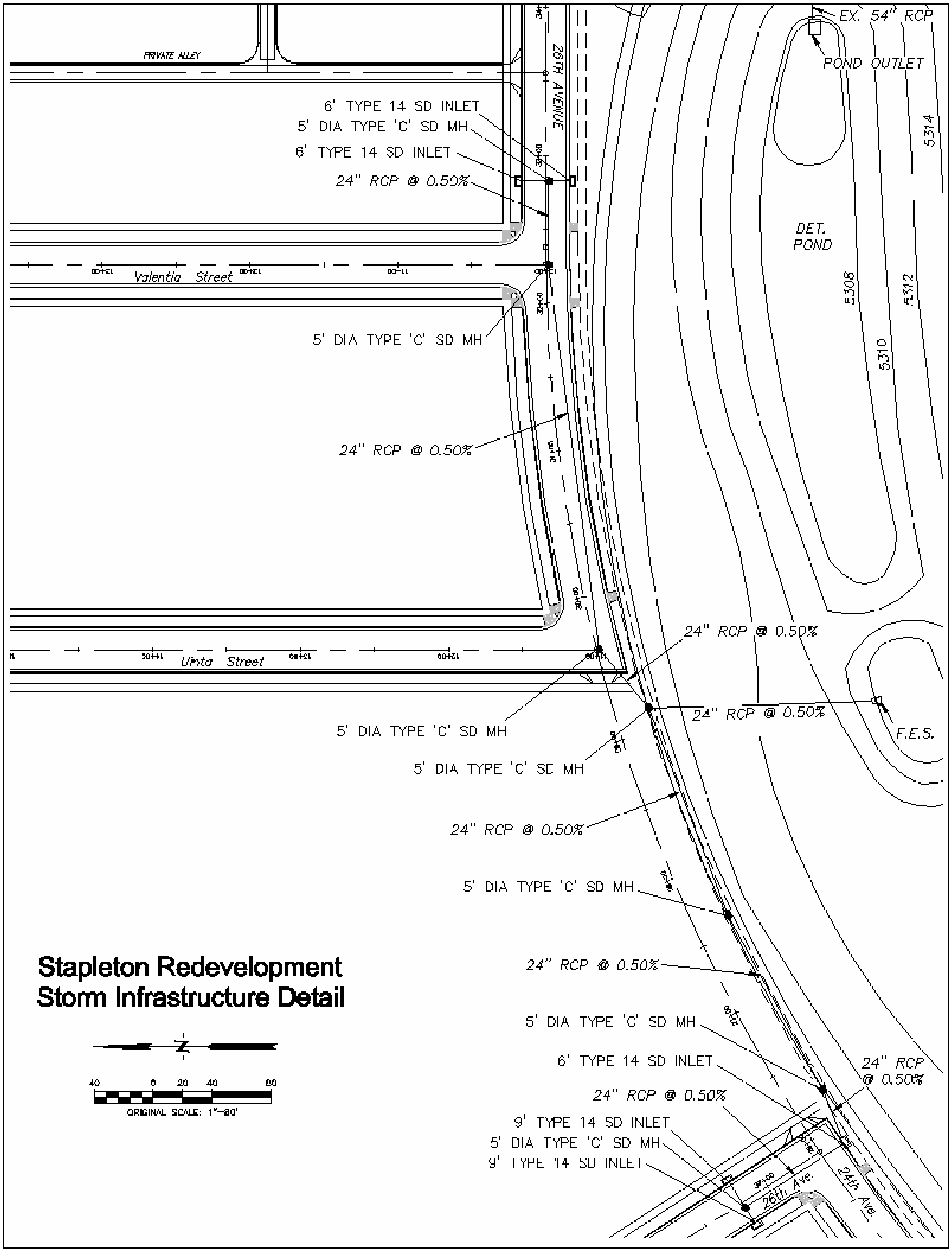
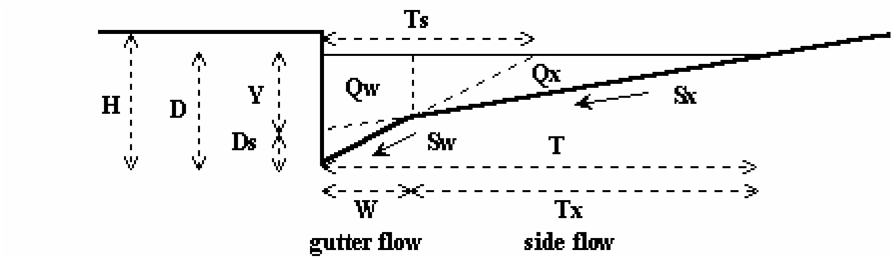


Figure 23—Storm Infrastructure Detail

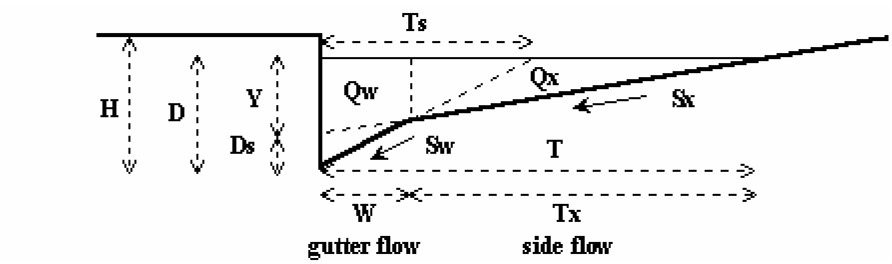
Project =	Stapleton Redevelopment
Street ID =	26th Avenue (32' Fl - Fl Local Street)



Gutter Geometry			
Curb Height	H =	6.00	inches
Gutter Width	W =	2.00	ft
Gutter Depression	Ds =	2.00	inches
Street Transverse Slope	Sx =	0.0200	ft/ft
Street Longitudinal Slope	So =	0.0050	ft/ft
Gutter Cross Slope:	Sw =	0.0833	ft/ft
Manning's Roughness	N =	0.016	
Maximum Allowable Water Spread for Major Event	T =	16.00	ft
Gutter Conveyance Capacity Based On Maximum Water Spread			
Water Depth without Gutter Depression	Y =	0.32	ft
Water Depth with a Gutter Depression	D =	0.49	ft
Spread for Side Flow on the Street	Tx =	14.00	ft
Spread for Gutter Flow along Gutter Slope	Ts =	5.84	ft
Flowrate Carried by Width Ts	Qws =	4.3	cfs
Flowrate Carried by Width (Ts - W)	Qww =	1.4	cfs
Gutter Flow	Qw =	2.9	cfs
Side Flow	Qx =	4.1	cfs
Maximum Spread Capacity	**Q-Tm =**	**7.1**	**cfs**
Gutter Full Conveyance Capacity Based on Curb Height			
Spread for Side Flow on the Street	Tx =	16.67	ft
Spread for Gutter Flow along Gutter Slope	Ts =	6.00	ft
Flowrate Carried by Width Ts	Qws =	4.7	cfs
Flowrate Carried by Width (Ts - W)	Qww =	1.6	cfs
Gutter Flow	Qw =	3.1	cfs
Side Flow	Qx =	6.6	cfs
Gutter Full Capacity	**Q-full =**	**9.7**	**cfs**
Gutter Design Conveyance Capacity Based on Min(Q-Tm, R*Q-full)			
Reduction Factor for Minor Event	R-min =	1.00	
Gutter Design Conveyance Capacity for Minor Event	**Q-min =**	**7.1**	**cfs**

Figure 24—Gutter Stormwater Conveyance Capacity for Initial Event

Project =	Stapleton Redevelopment
Street ID =	26th Avenue (32' Fl - Fl Local Street)

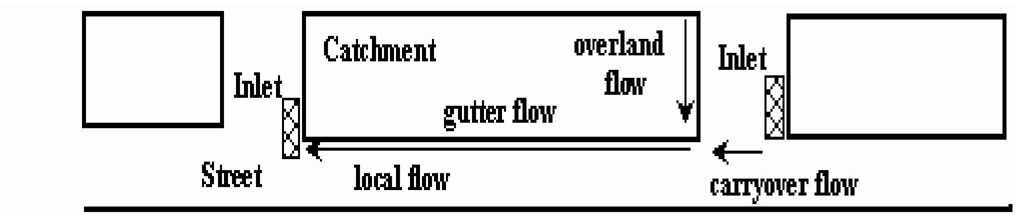


Gutter Geometry			
Curb Height	H =	12.00	inches
Gutter Width	W =	2.00	ft
Gutter Depression	Ds =	2.00	inches
Street Transverse Slope	Sx =	0.0200	ft/ft
Street Longitudinal Slope	So =	0.0050	ft/ft
Gutter Cross Slope	Sw =	0.0833	ft/ft
Manning's Roughness	N =	0.016	
Maximum Water Spread for Major Event	T =	16.00	ft
Gutter Conveyance Capacity Based On Maximum Water Spread			
Water Depth without Gutter Depression	Y =	0.32	ft
Water Depth with a Gutter Depression	D =	0.49	ft
Spread for Side Flow on the Street	Tx =	14.00	ft
Spread for Gutter Flow along Gutter Slope	Ts =	5.84	ft
Flowrate Carried by Width Ts	Qws =	4.3	cfs
Flowrate Carried by Width (Ts - W)	Qww =	1.4	cfs
Gutter Flow	Qw =	2.9	cfs
Side Flow	Qx =	4.1	cfs
Maximum Spread Capacity	**Q-Tm =**	**7.1**	**cfs**
Gutter Full Conveyance Capacity Based on Curb Height			
Spread for Side Flow on the Street	Tx =	41.67	ft
Spread for Gutter Flow along Gutter Slope	Ts =	12.00	ft
Flowrate Carried by Width Ts	Qws =	29.7	cfs
Flowrate Carried by Width (Ts - W)	Qww =	18.3	cfs
Gutter Flow	Qw =	11.4	cfs
Side Flow	Qx =	76.1	cfs
Gutter Full Capacity	**Q-full =**	**87.5**	**cfs**
Gutter Design Conveyance Capacity Based on Min(Q-Tm, R*Q-full)			
Reduction Factor for Major Event	R-maj =	1.00	
Gutter Design Conveyance Capacity for Major Event	**Q-maj =**	**7.1**	**cfs**

Figure 25—Gutter Stormwater Conveyance Capacity for Major Event

Design Flow = Local Flow + Carryover Flow

Project =	Stapleton Redevelopment
Street ID =	26th Avenue (32' Fl-Fl Local Street)
Return Period	2 year (Basin 31.1B)



A.LOCAL FLOW ANALYSIS

Area (A) =	1.10	acres	(input)
Runoff Coeff (C) =	0.45		(input)

Rainfall Information I (inch/hr) = 28.5 * P1 /(10 + Td)^0.786

P1 =	0.95	inches	(input one-hr precipitation)

Calculations of Time of Concentration

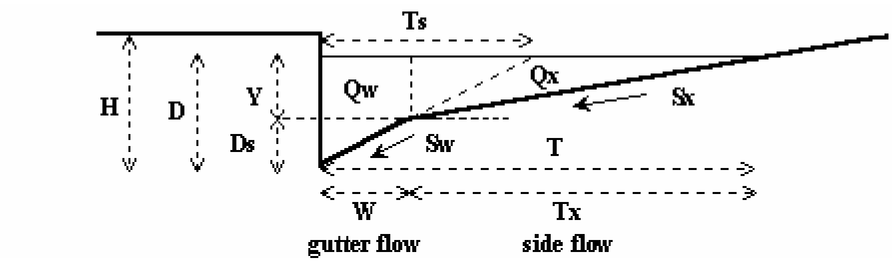
Reach **ID**	**Slope** **ft/ft** input	**Length** **ft** input	**5-yr Runoff Coeff** input	**Flow Velocity fps** output	**Flow Time minutes** output
Overland Flow	0.0150	50.00	0.50	0.12	6.70
Gutter Flow	0.0050	900.00		1.41	10.61
Sum		950.00			17.31
	Regional Tc =	15.28	minutes		
	Recommended Tc =	15.28	minutes		
	Enter Design Tc =	15.28	minutes		

B.LOCAL PEAK FLOW

Design Rainfall	I =	2.14	inch/hr	(output)
Local Peak Flow	Qp =	1.06	cfs	(output)
C.CARRYOVER FLOW	Qco =	0.00	cfs	(input)
D.DESIGN PEAK FLOW	Qs =	**1.06**	cfs	(output)

Figure 26—Determination Of Design Peak Flow On The Street

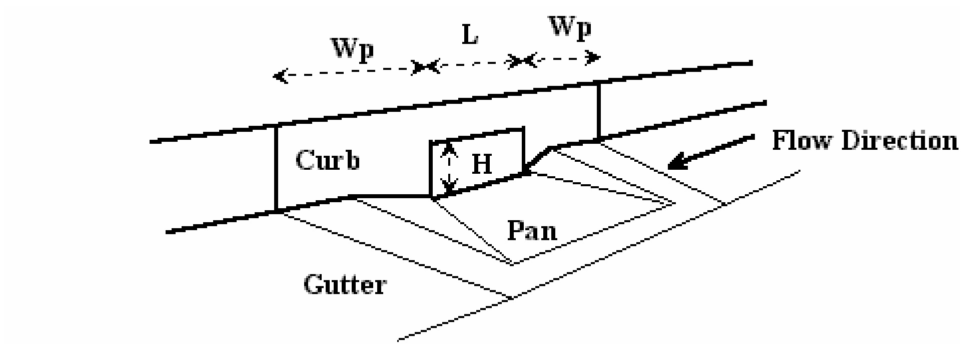
Project =	Stapleton Redevelopment
Street ID =	26th Avenue (32' Fl-Fl Local Street)



Street Geometry (Input)			
Design Discharge in the Gutter	**Qo =**	**1.1**	**cfs**
Curb Height	H =	6.00	inches
Gutter Width	W =	2.00	ft
Gutter Depression	Ds =	2.00	inches
Street Transverse Slope	Sx =	0.0200	ft/ft
Street Longitudinal Slope	So =	0.0100	ft/ft
Gutter Cross Slope	Sw =	0.0833	ft/ft
Manning's Roughness	N =	0.016	
Gutter Conveyance Capacity			
Water Spread Width	T =	4.32	ft
Water Depth without Gutter Depression	Y =	0.09	ft
Water Depth with a Gutter Depression	D =	0.25	ft
Spread for Side Flow on the Street	Tx =	2.32	ft
Spread for Gutter Flow along Gutter Slope	Ts =	3.04	ft
Flowrate Carried by Width Ts	Qws =	1.07	cfs
Flowrate Carried by Width (Ts - W)	Qww =	0.06	cfs
Gutter Flow	Qw =	1.01	cfs
Side Flow	Qx =	0.05	cfs
Total Flow (Check against Qo)	**Qs =**	**1.1**	**cfs**
Gutter Flow to Design Flow Ratio	Eo =	0.95	
Equivalent Slope for the Street	Se =	0.10	
Flow Area	As =	0.35	sq ft
Flow Velocity	Vs =	3.00	fps
VsD product	**VsD =**	**0.76**	**ft^2/s**

Figure 27—Gutter Conveyance Capacity

Project =	Stapleton Redevelopment
Inlet ID =	6' Type 14 (Basin 31.1B)



Design Information (Input)			
Design discharge on the street (from Street Hy)	**Qo =**	**1.1**	**cfs**
Length of a unit inlet	Lu =	6.00	ft
Side Width for Depression Pan	Wp =	2.00	ft
Clogging Factor for a Single Unit	Co =	0.20	
Height of Curb Opening	H =	0.50	ft
Orifice Coefficient	Cd =	0.65	
Weir Coefficient	Cw =	2.30	
Water Depth for the Design Condition	Yd =	0.55	ft
Angle of Throat	Theta =	1.05	rad
Number of Curb Opening Inlets	N =	1	
Curb Opening Inlet Capacity in a Sump			
As a Weir			
Total Length of Curb Opening Inlet	L =	6.00	ft
Capacity as a Weir without Clogging	Qwi =	9.0	cfs
Clogging Coefficient for Multiple Units	Clog-Coeff =	1.00	
Clogging Factor for Multiple Units	Clog =	0.20	
Capacity as a Weir with Clogging	Qwa =	7.9	cfs
As an Orifice			
Capacity as an Orifice without Clogging	Qoi =	9.0	cfs
Capacity as an Orifice with Clogging	Qoa =	7.2	cfs
Capacity for Design with Clogging	**Qa =**	**7.2**	**cfs**
Capture %age for this inlet = Qa/Qs =	**C% =**	**682.80**	**%**

Figure 28—Curb Opening Inlet In A Sump

STORM DRAINAGE SYSTEM COMPUTATION FORM **Design Storm: 2 yr**

Location: Stapleton Filing No 2

Computed by:

Checked by:

									Overland Time (ti)			Travel Time (tt)					tc Check		Runoff		
Basin ID	Inlet No.	Area	Cumulative Area	**Coefficient "C2"**	Coefficient "C5"	Coefficient "C100"	CA	Cumulative CA	Length (300' max)	Slope	ti	Length	Slope	Velocity (Fig. 3-2)	tt	tc=ti+tt	Total length	tc = (l/180+10)	Final tc	Intensity "I"	Total Peak Discharge "Q"
		acres	acres						ft	%	min.	ft	%	fps	min	min	ft	min	min	in/hr	cfs
31.1A	26-5A	5.23		0.40	0.45	0.60	2.09		60	0.5%	11.4	325	0.5%	1.5	3.6	15.0	385	12.1	12.1	2.37	5.0
31.1B	26-5B	1.10		0.45	0.50	0.70	0.50		70	0.5%	11.4	1017	0.5%	1.5	11.3	22.7	1087	16.0	16.0	2.09	1.0
PIPE			6.33	0.41				2.59											16.0	2.09	5.4
31.1C	26-5C	1.19		0.87	0.89	0.91	1.04		50	1.5%	2.3	1017	0.5%	1.5	11.3	13.6	1067	15.9	13.6	2.25	2.3
PIPE			7.52	0.48				3.62											16.0	2.09	7.6
PIPE			7.52	0.48				3.62											16.0	2.09	7.6
PIPE			7.52	0.48				3.62											16.0	2.09	7.6
31.1D	26-10B	0.26		0.40	0.45	0.60	0.10		50	0.5%	10.4	220	0.5%	1.5	2.4	12.9	270	11.5	11.5	2.43	0.3
31.1E	26-10A	0.42		0.87	0.89	0.91	0.37		50	0.5%	3.3	810	0.5%	1.5	9.0	12.3	860	14.8	12.3	2.36	0.9
PIPE			0.68	0.69				0.47											12.3	2.36	1.1
PIPE			0.68	0.69				0.47											12.3	2.36	1.1
PIPE			8.20	0.50				4.09											16.0	2.09	8.5
			8.20	**0.50**				**4.09**											**16.0**	**2.09**	**8.5**

Sub-Basin Data		Street				Inlet		System		Pipe											
Basin ID	Slope	Allowable Capacity (half of street)	Bypassed Flow	New Flow	Total Street Flow	Length	Allowable Capacity	Intercepted Flow	Street Flow	Pipe Identification (Upstream - Downstream)	Length	Slope	Size	Allowable Capacity (0.80 Capacity)	Pipe Flow	q/Q(0.8 Full)	v/V(Full) (from Fig 8-1)	Velocity	Pipe Flow Time	Enough Capacity?	Remarks
	%	cfs	cfs	cfs	cfs	ft	cfs	cfs	cfs		(ft)	%	in	cfs	cfs			fps	min		
31.1A	0.50%	5.8	0.0	5.0	5.0	9	8.6	5.0	0.0	26-5A 26-5	20	0.50%	18	5.9	5.0	0.84	1.00	3.4	0.1	no	
31.1B	0.63%	8.1	0.0	1.0	1.0	9	8.6	1.0	0.0	26-5B + 26-5	12	0.83%	18	7.7	1.0	0.14	0.58	2.5	0.1	no	
PIPE								5.4	0.0	26-5 + 26-5C	79	0.50%	24	12.8	5.4	0.42	0.80	3.3	0.4	no	
31.1C	0.50%	5.8	0.0	2.3	2.3	6	2.9	2.3	0.0	26-5C + 26-6	35	0.50%	24	12.8	2.3	0.18	0.63	2.6	0.2	no	
PIPE								7.6	0.0	26-6 + 26-7	117	0.50%	24	12.8	7.6	0.59	0.89	3.6	0.5	no	
PIPE								7.6	0.0	26-7 + 26-8	150	0.50%	24	12.8	7.6	0.59	0.89	3.6	0.7	no	
PIPE								7.6	0.0	26-8 + 26-9	54	0.50%	24	12.8	7.6	0.59	0.89	3.6	0.2	no	
31.1D	0.53%	6.5	0.0	0.3	0.3	6	5.8	0.3	0.0	26-11B + 26-11	18	0.60%	18	6.5	0.3	0.04	0.45	1.7	0.2	no	
31.1E	0.53%	6.5	0.0	0.9	0.9	6	5.8	0.9	0.0	26-11A + 26-11	14	0.79%	18	7.5	0.9	0.12	0.56	2.4	0.1	no	
PIPE								1.1	0.0	26-11 + 26-10	262	0.50%	24	12.8	1.1	0.09	0.52	2.1	2.1	no	
PIPE								1.1	0.0	26-10 + 26-9	57	0.50%	24	12.8	1.1	0.09	0.52	2.1	0.4	no	
PIPE								8.5	0.0	26-9 + OUTLET	180	0.61%	30	25.6	8.5	0.33	0.75	3.9	0.8	no	
	-	-						**8.5**	**0.0**			**0.61%**	**30**	**25.6**	**8.5**					yes	OUTLET TO DETENTION BASIN

Figure 29—Storm Drainage System Computation Form—2 Year

STORM DRAINAGE SYSTEM COMPUTATION FORM **Design Storm: 100 yr**

Location: Stapleton Filing No 2

Computed by:

Checked by:

Sub-Basin Data									Overland Time (ti)			Travel Time (tt)					tc Check		Runoff		
Basin ID	Inlet No.	Area	Cumulative Area	Coefficient "C2"	Coefficient "C5"	**Coefficient "C100"**	CA	Cumulative CA	Length (300' max)	Slope	ti	Length	Slope	Velocity (Fig. 3-2)	tt	tc=ti+tt	Total length	tc = (l/180+10)	Final tc	Intensity "I"	Total Peak Discharge "Q"
		acres	acres						ft	%	min.	ft	%	fps	min	min	ft	min	min	in/hr	cfs
31.1A	26-5A	5.23		0.40	0.45	0.60	3.14		60	0.5%	11.4	325	0.5%	1.5	3.6	15.0	385	12.1	12.1	6.49	20.4
31.1B	26-5B	1.10		0.45	0.50	0.70	0.77		70	0.5%	11.4	1017	0.5%	1.5	11.3	22.7	1087	16.0	16.0	5.72	4.4
PIPE			6.33			0.62		3.91											16.0	5.72	22.3
31.1C	26-5C	1.19		0.87	0.89	0.91	1.08		50	1.5%	2.3	1017	0.5%	1.5	11.3	13.6	1067	15.9	13.6	6.17	6.7
PIPE			7.52			0.66		4.99											16.0	5.72	28.5
PIPE			7.52			0.66		4.99											16.0	5.72	28.5
PIPE			7.52			0.66		4.99											16.0	5.72	28.5
31.1D	26-10B	0.26		0.40	0.45	0.60	0.16		50	0.5%	10.4	220	0.5%	1.5	2.4	12.9	270	11.5	11.5	6.65	1.0
31.1E	26-10A	0.42		0.87	0.89	0.91	0.38		50	0.5%	3.3	810	0.5%	1.5	9.0	12.3	860	14.8	12.3	6.45	2.5
PIPE			0.68			0.79		0.54											12.3	6.45	3.5
PIPE			0.68			0.79		0.54											12.3	6.45	3.5
PIPE			8.2			0.67		5.53											16.0	5.72	31.6
			8.20			**0.67**		**5.53**											**16.04**	**5.72**	**31.6**

Sub-Basin Data		Street				Inlet					Pipe										
Basin ID	Slope	Allowable Capacity*	Bypassed Flow (Negative flows indicates bypass flow to another DP system (See Remarks))	New Flow	Total Street Flow	Length	Allowable Capacity	Intercepted Flow (if inlet is in series, less intercepted flow is possible -- see remarks)	Bypassed Street Flow	Pipe Identification (Upstream-Downstream)	Length	Slope	Size	Allowable Capacity	Pipe Flow	q/Q(Full)	v/V(Full)	Velocity	Pipe Flow Time	Enough Capacity (Street + Storm Sewer)?	Remarks
	%	cfs	cfs	cfs	cfs	ft	cfs	cfs	cfs		(ft)	%	in	cfs	cfs			fps	(min)		
31.1A	0.50%	23.3	0.0	20.4	20.4	9	12.2	7.4	13.0	26-5A 26-5	20	0.5%	18	7.4	7.4	1.0	1.01	4.3	0.1	yes	
31.1B	0.63%	32.1	13.0	4.4	17.4	9	12.2	9.6	7.8	26-5B + 26-5	12	0.8%	18	9.6	9.6	1.0	1.01	5.5	0.0	yes	
PIPE				22.3				16.0	6.3	26-5 + 26-5C	79	0.5%	24	16.0	16.0	1.0	1.01	5.2	0.3	yes	
31.1C	0.50%	23.3	6.3	6.7	13.0	6	12.2	12.2	0.8	26-5C + 26-6	35	0.5%	24	16.0	12.2	0.8	0.98	3.8	0.2	yes	
PIPE				28.5				16.0	12.5	26-6 + 26-7	117	0.5%	24	16.0	16.0	1.0	1.01	5.2	0.4	yes	
PIPE				28.5				16.0	12.5	26-7 + 26-8	150	0.5%	24	16.0	16.0	1.0	1.01	5.2	0.5	yes	
PIPE				28.5				16.0	12.5	26-8 + 26-9	54	0.5%	24	16.0	16.0	1.0	1.01	5.2	0.2	yes	
31.1D	0.53%	25.8	0.0	1.0	1.0	6	8.2	1.0	0.0	26-11B + 26-1	18	0.6%	18	8.1	1.0	0.1	0.57	0.3	0.9	yes	
31.1E	0.53%	25.8	0.0	2.5	2.5	6	8.2	2.5	0.0	26-11A + 26-1	14	0.8%	18	9.3	2.5	0.3	0.70	1.0	0.2	yes	
PIPE				3.5				3.5	0.0	26-11 + 26-10	262	0.5%	24	16.0	3.5	0.2	0.66	0.7	5.9	yes	
PIPE				3.5				3.5	0.0	26-10 + 26-9	57	0.5%	24	16.0	3.5	0.2	0.66	0.7	1.3	yes	BYPASS FLOW TO LOW POINT @ INLET
PIPE				31.6				19.5	0.0	26-9 + OUTLET	180	0.6%	30	32.0	19.5	0.6	0.90	3.6	0.8	yes	
				31.6				**19.5**	**0.0**			**0.6%**	**30**	**32.0**	**19.5**						OUTLET TO DETENTION BASIN

Figure 30—Storm Drainage System Computation Form—100 Year

DESIGN EXAMPLES—SECTION 3

CONTENTS

Figures for Section 3

3.0 CASE STUDY—WILLOW CREEK

Willow Creek meanders through a natural open space park in Southern Arapahoe County (Figure 1). The low-flow channel carries about 200 cfs, and almost the entire open space is within the 100-year floodplain. The basin tributary to Willow Creek is 8.10 square miles; the lower portion is fully developed and the upper portion is actively being developed. Because of the changes in the basin runoff characteristics, Willow Creek is experiencing higher low-flow volumes. Frequent storms and increased base flows have created a 30-foot-high vertical cliff where the open space borders a residential development. If nothing was done, the house at the top of the cliff was in imminent danger (Photo 1).

Photo 1. Cliff Created by Erosion from Creek

Summary of Flows

Base Flow	> 5 cfs
2-year Storm	1,650 cfs
5-year Storm	3,000 cfs
10-year Storm	4,100 cfs
50-year Storm	5,500 cfs
100-year Storm	6,100 cfs

Because of these safety issues and potential loss of private property, Arapahoe County and South Suburban Parks and Recreational District requested assistance from the Urban Drainage and Flood Control District (District). The sponsors selected Muller Engineering Company, who teamed with Wenk Associates, to design the Willow Creek Channel Improvements. It was agreed at the outset that bioengineering techniques should be explored for this channel improvement project. The client team and the design team both saw this as a great opportunity to try new approaches to channel and bank stabilization.

3.1 Design

Designing a retaining wall to stabilize the cliff was one alternative considered by the client team, but it was rejected because of the cost, safety issues, and "hard" unnatural characteristics. The final design was chosen because it best satisfied the project goals for safety, aesthetics, habitat improvement, and affordability. The design included moving the creek from the south side to the north side of the existing stand of cottonwood trees. The trees' root systems would provide some stabilization for what would then be the outer bend of the meander. The trees would still receive sufficient water from the relocated stream. With the creek now 60 feet from the toe of the cliff, a safer 2:1 slope could be built to replace it (Figure 2).

Photo 2. Existing "Texas" Low-flow Crossing

Photo 3. New Grouted Boulder Structure & Pedestrian Bridge

Although moving the creek made it feasible to fill in the vertical cliff, it also reduced the amount of area to mitigate to about 0.5 acres of

Photo 4. Biolog & Erosion Mat Installation

wetlands. Wenk designed a wetland backwater area inside the meander to accommodate the additional area needed. The water pools up during a storm event and then slowly drains, creating a good wetland water regime. A temporary wetland drain pipe from the creek was installed to feed the area until the plants were established (Figure 2).

Photo 5. Reconstructed Slope with Wrapped Soil Lifts at Toe

The realignment of the creek shortened the total length of channel and increased its slope. Two grouted boulder grade control structures, with 1-foot drops, were incorporated as permanent "hard" improvements to establish a stable channel slope of 0.5% (for bioengineered channels a milder slope of 0.3 to 0.4% is normally recommended by the District). Adjacent to the grade control structures, box culvert/pedestrian bridges were built to replace the existing slippery "Texas" low-flow crossings, which had been high maintenance for South Suburban as well as being a safety hazard (Photos 2 & 3).

Photo 6. Construction of Brush Layering

Incorporating "hard" grade control structures with the new bridges allowed the rest of the project area to have improvements with a "soft" appearance (Figure 3). Wenk designed a "biolog" or coir-roll stream edge for the outer bank of the low-flow channel. Two biologs, stacked almost on top of each other, laid next to and above a buried rock blanket, line the edge of the new low-flow channel between the bridges. The biologs were partially buried, staked, tied, and overlapped so that they could not be dislodged during a storm event. Willow stakes were also planted through them. Permanent erosion control mat was placed on the bank above the biologs (Photo 4). The inner bank of the meander was covered with a plastic permanent "enkamat" geotextile, designed to trap sediment that is washed around the bend and encourage wetland and riparian plant growth (Figures 4, 6, & 7).

Photo 7. Completed Slope with Brush Layering, Erosion Mat, and Wrapped Soil Lifts

Bioengineering techniques were also used to stabilize and help establish vegetation on the 2:1 fill slope of the 30-foot vertical cliff. Extra stabilization was needed at the toe of the new slope to protect up to the 100-year water surface elevation. Six layers of wrapped soil lifts made of a double layer of coir fabric encasing a 6-inch lift of soil protects the soil from erosion at the toe of the slope while still allowing vegetation to grow (Photo 5 & 9). The upper portion of the

Photo 8. Complete Channel with Plantings

slope is a test area for both brush layering and traditional erosion control matting. For the brush layering, Wenk specified that willow and cottonwood branches be placed horizontally in the slope with about 3 inches of the tips sticking out. These little "fingers" of the mostly dead branches collect leaves and natural debris while breaking up the water that trickles down the slope, preventing rill erosion (Photo 6). The brush layering was used on half of the new fill slope, and the other half received a temporary erosion control blanket. These two methods will be compared over the years to see if one is more successful than the other (Photo 7 and Figure 5).

The channel edges and the wrapped soil lifts were then planted with willow stakes. Cottonwood whips were also planted within the meander and around the check structures (Photo 8). All the willow stakes, the cottonwood whips, and even the brush for the brush layering were harvested from the immediate area.

As an added precaution, the District asked Muller to design modified riprap bank protection, which was buried behind the biologs as a secondary line of defense. Also, to save several existing cottonwood trees, huge boulders were placed as retaining walls to hold back the fill slope from the bases of these trees.

3.2 Criteria

District criteria were followed for the design of this project to the maximum extent possible. As within many District projects that address existing problems, right-of-way limitations often dictate a need to deviate from some of the criteria, knowing full well that had the criteria been followed, the problems that had to be addressed would not have materialized. The new channel slope is 0.5%, and the radius of the new curve is 150 feet. Buried riprap was placed on the downstream side of the box culvert/pedestrian bridge in accordance with the District. The riprap bank protection behind the biologs was slimmed down from the District criteria since it was installed as a precautionary measure. Reference materials obtained from an International Erosion Control Association seminar and from King County, Washington entitled "Guidelines for Bank Stabilization Projects" were used to assist in the design of the bioengineering. However, at the time of the design, there were no established design criteria available for the bioengineering aspects of the project.

Photo 9. Construction of Wrapped Soil Lifts

3.3 Construction

L&M Enterprises was awarded the contract for the construction of this channel project. It was necessary to use small equipment to build the wrapped soil lifts and the brush layering, which made the job go slower than expected. It was also difficult to compact the slope with the brush layering inside of it. The biggest challenge during construction was dealing with higher than anticipated creek flows due to a wet winter and spring. Construction began in October 1998, and in early 1999 there were spring storms that tested the channel before the vegetation took root. Overall, the channel held up well.

In retrospect, it was determined that wider rolls of geotextiles would function better and would be easier to install. The permanent "enkamat" geotextile came in 3-foot-wide rolls, and after the pieces were overlapped, there was little left to cover the ground. Also, there would have been fewer areas of failure if the trees were planted prior to installing the geotextile.

3.4 Success

The Willow Creek Channel Improvement Project continues to be a success story. The new channel has seen numerous storm events, and sediment has deposited on the inside of the bend without eroding the outside. Almost every willow stake has sprouted. Many of the cottonwood whips are growing. The biologs are secure with their double-tied stakes and will soon be permanently anchored by the willows and grasses growing in them. The secondary riprap protection acts as a backup measure for protection during very large flood events. The most surprising success was the cottonwood branches that were placed in the brush layering even without irrigation. The very next season, sprouts were already 3 feet tall. Also, the wetland backwater idea has been incorporated into other projects because of its success.

Willow Creek is once again a meandering creek in this reach with two check structures that mimic splashing waterfalls which are enjoyed by the trail users and the residential neighbors. The looming 30-foot cliff and the slippery channel crossings are gone, and a safe and beautiful Colorado open space was created.

Photo 10. Relocated Channel & New Pedestrian Bridge

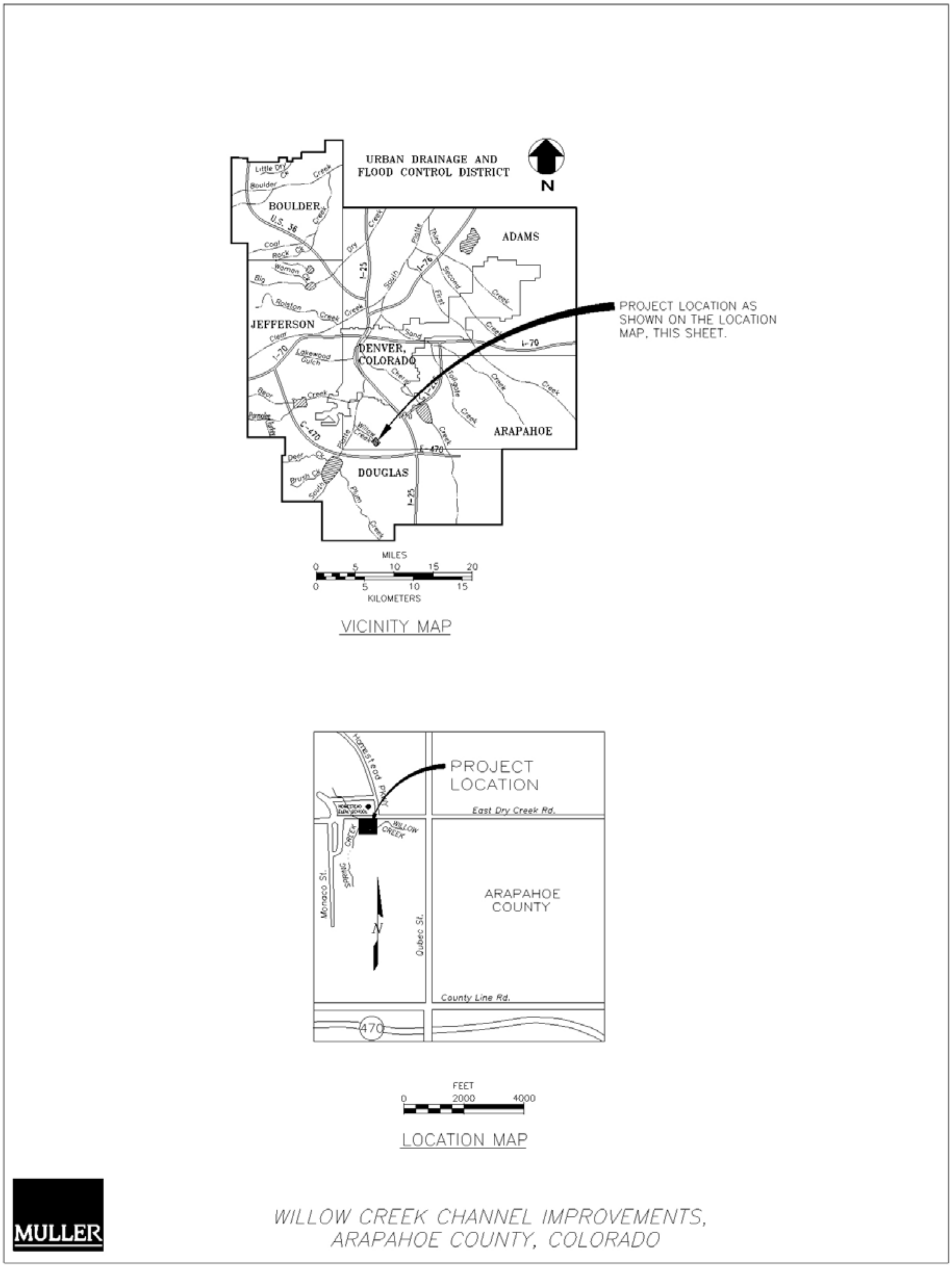


Figure 1—Location Map

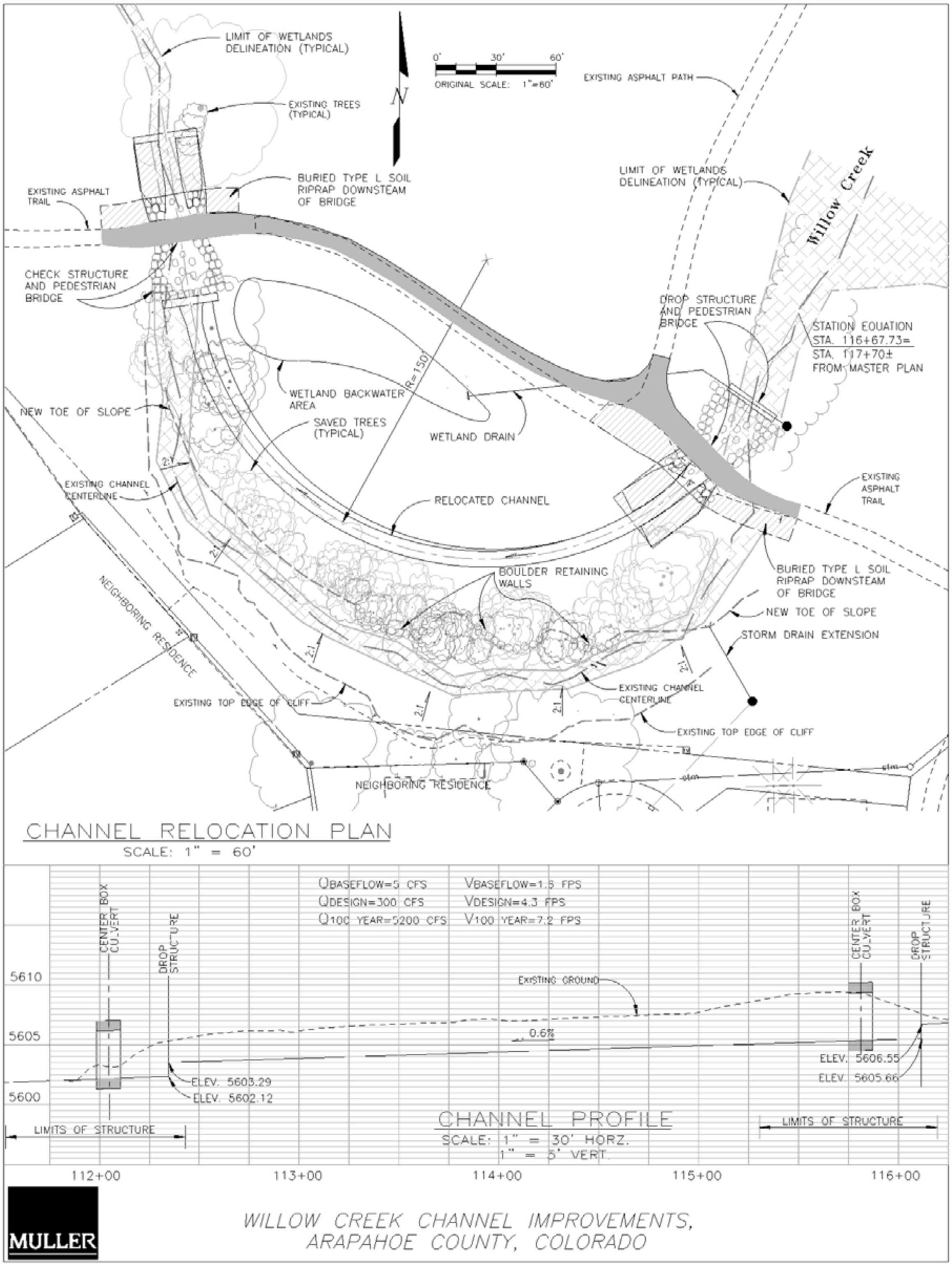


Figure 2—Channel Relocation Plan

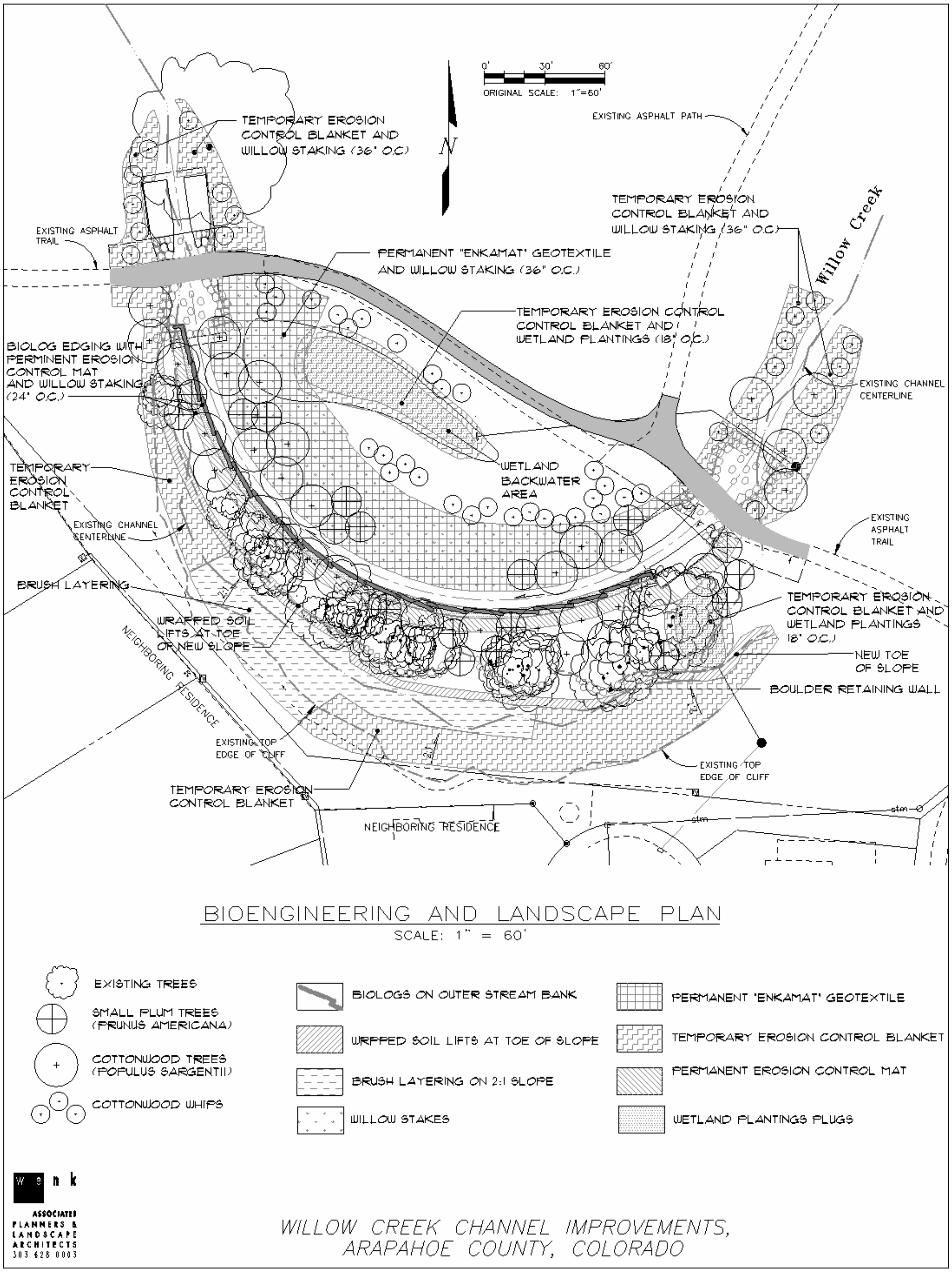


Figure 3—Bioengineering and Landscape Plan

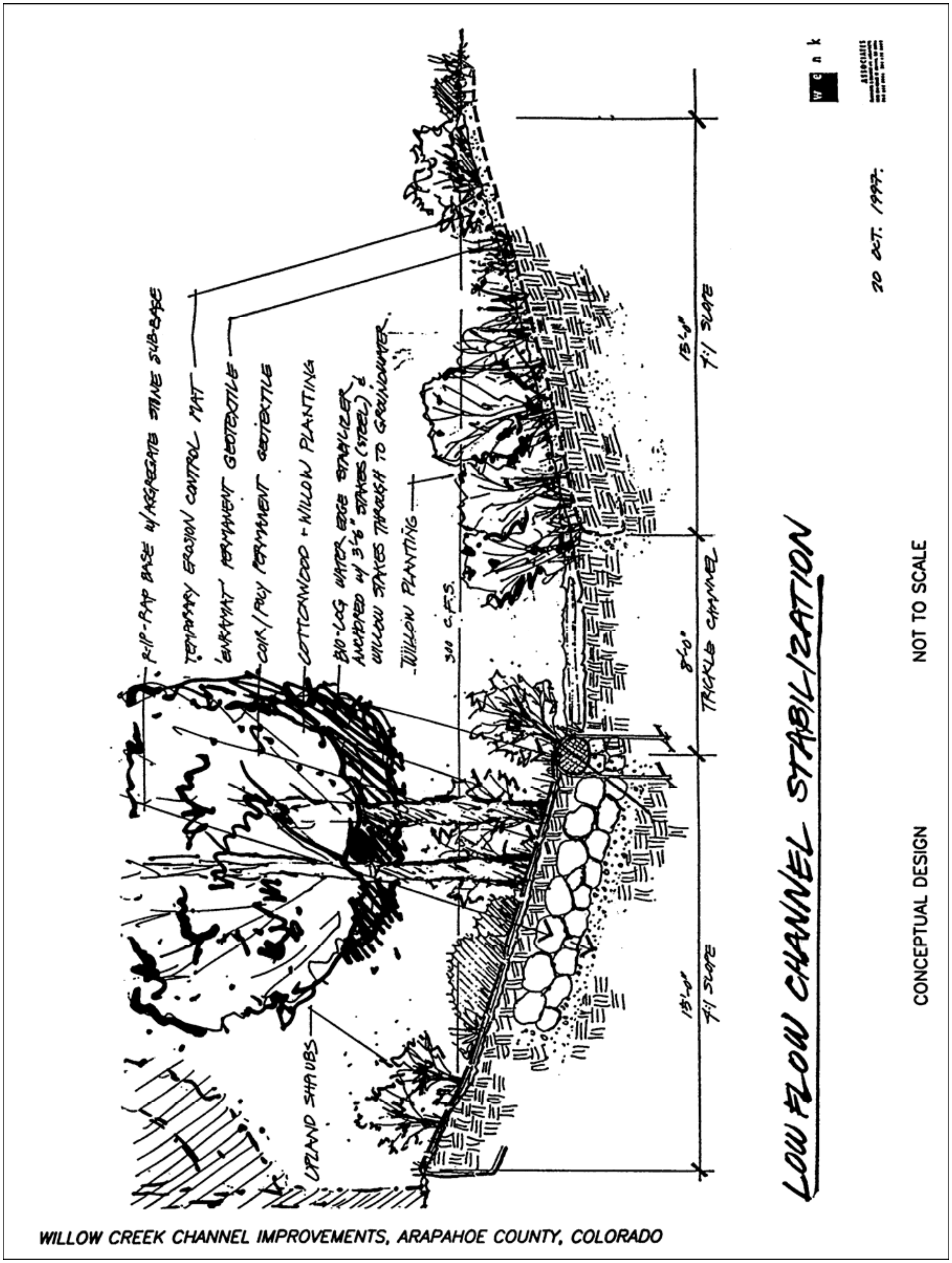


Figure 4—Low-Flow Channel Stabilization

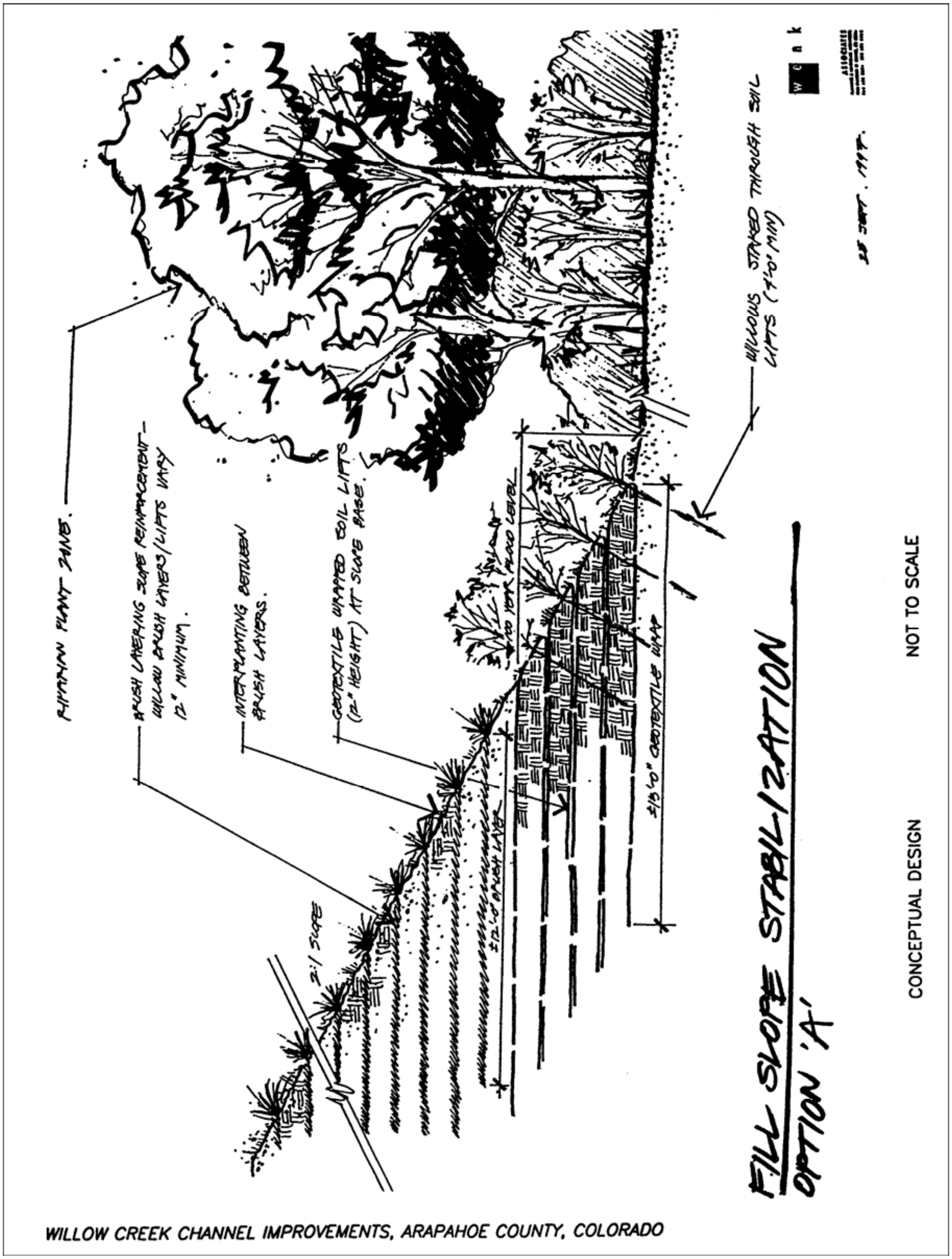


Figure 5—Fill Slope Stabilization Option A

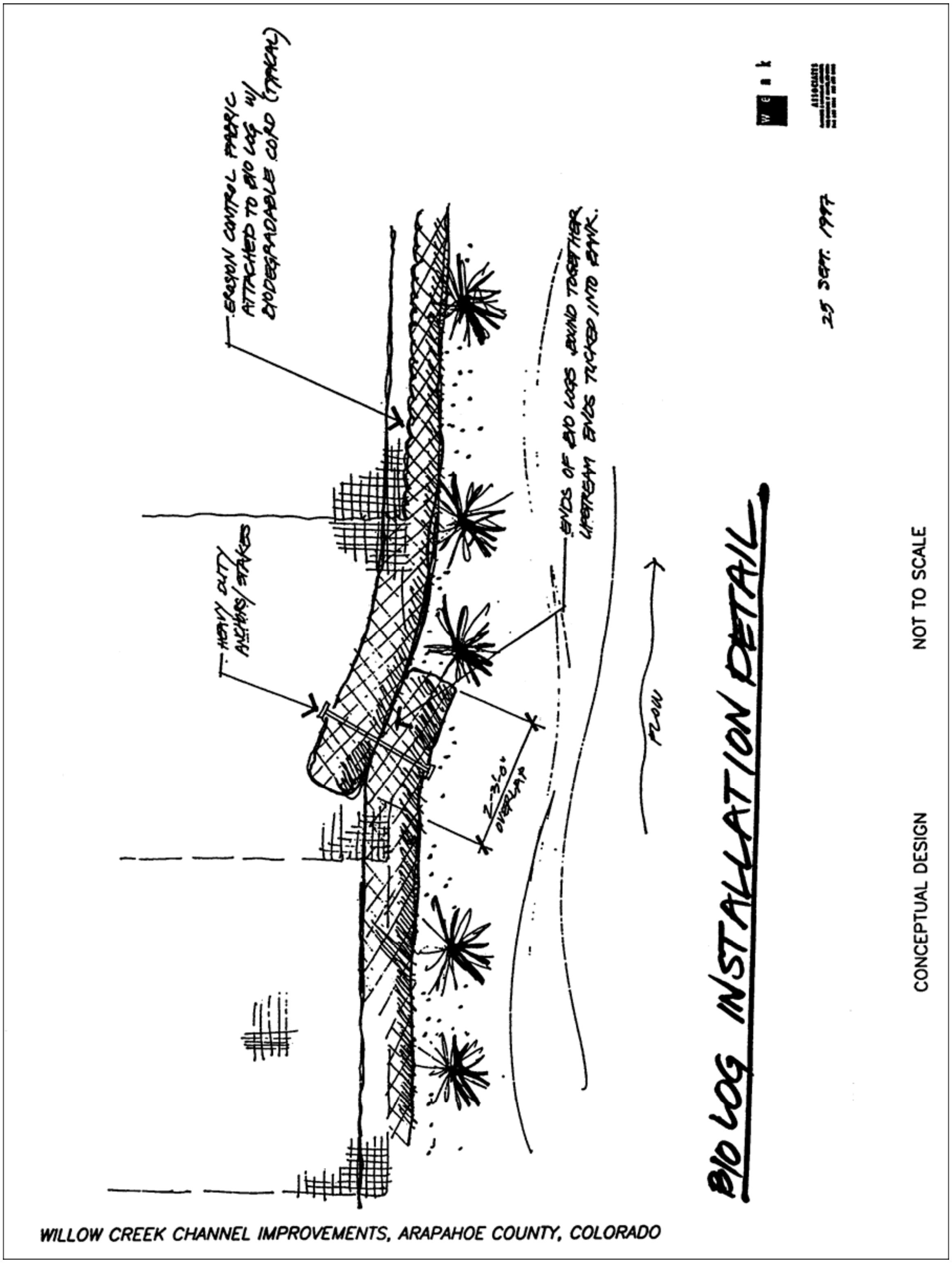


Figure 6—Biolog Installation Detail

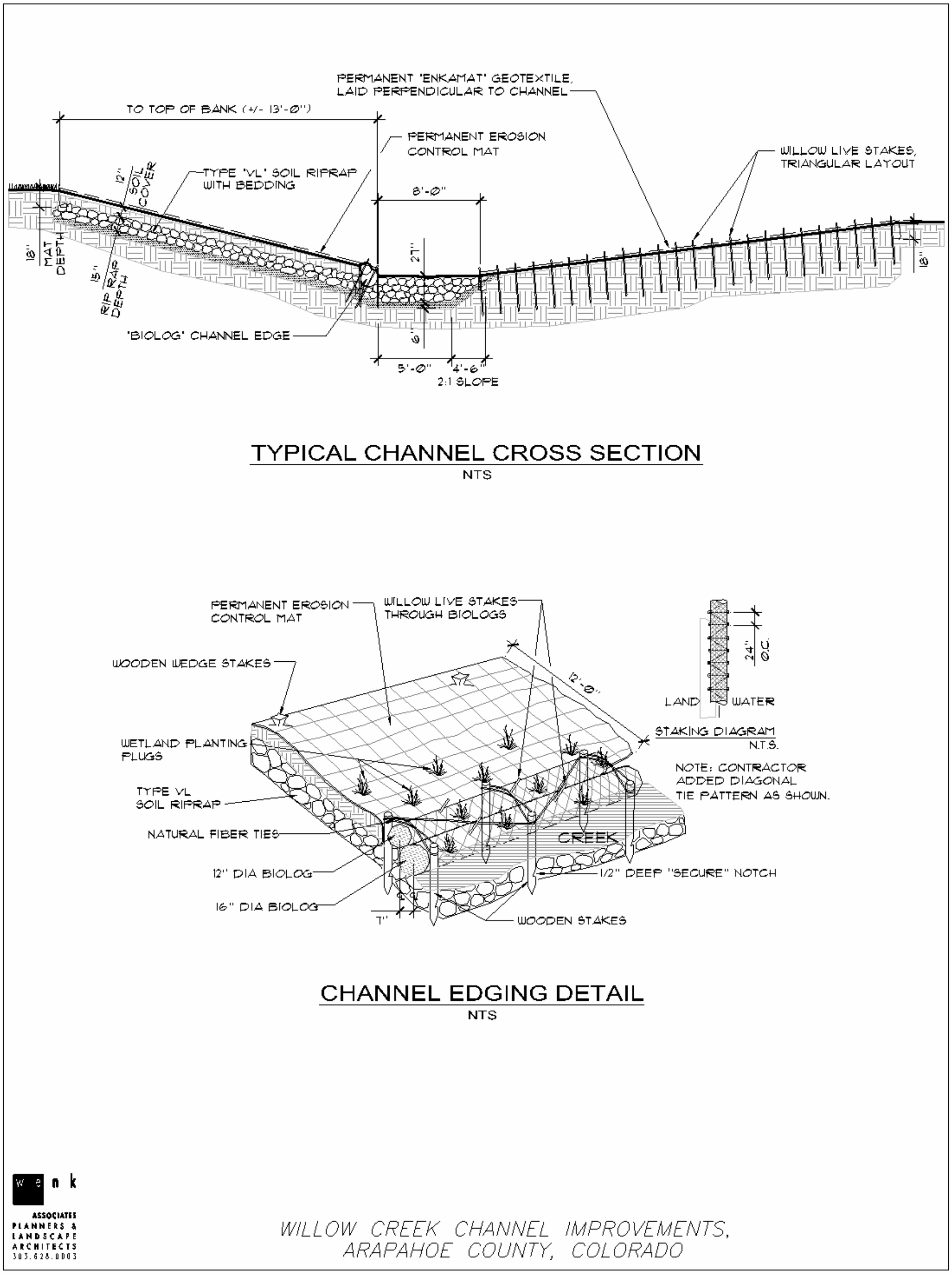


Figure 7—Typical Channel Cross Section and Channel Edging Detail

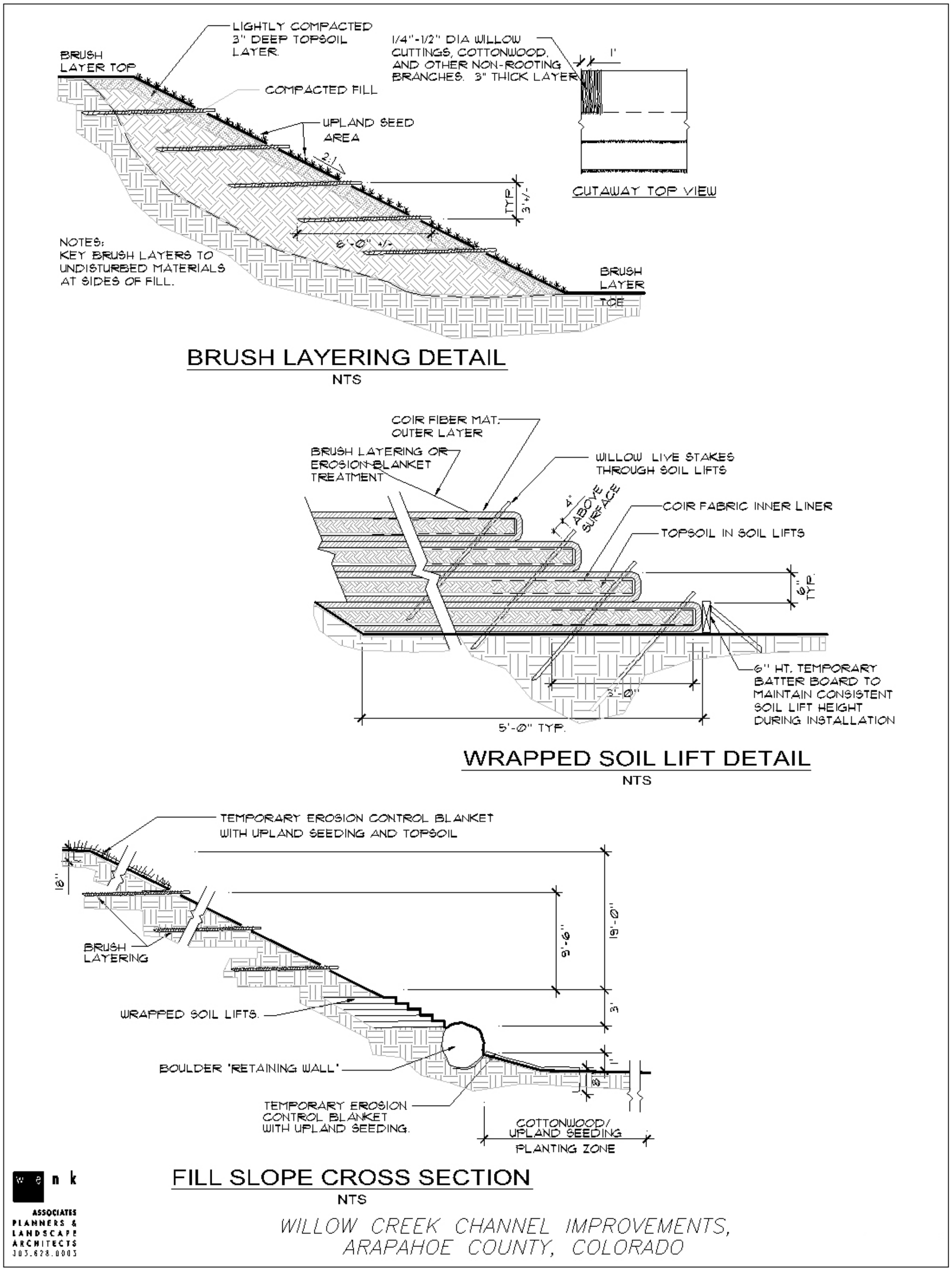


Figure 8—Brush Layering Detail, Wrapped Soil Lift Detail, and Fill Slope Cross Section

DESIGN EXAMPLES—SECTION 4

CONTENTS

4.0 CASE STUDY—ROCK CREEK

The purpose of this case study is to demonstrate the following features:

- Detention facility
- Grouted boulder drop structure
- Grouted boulder check structures and wetland bottom channels
- Stream bank stabilization including grouted boulder check structure with low-water crossing, slope flattening and revegetation

McLaughlin Water Engineers, Ltd. (MWE) and the Norris/Dullea Company (both based in Denver) prepared the attached drawings. Photographs are provided for each of the facilities featured in this case study.

The formal names of these projects are:

1. Tributary LB-3 Channel and Flor Storage Facility
2. Rock Creek Stabilization, Tributary RB-3 Outfall Pipe, and Community Pond East

The client for MWE and Norris/Dullea Company was Superior Metropolitan District No. 1 and the relevant drawings were prepared in 1994 and 1997.

Public reaction to the facilities shown on the attached pages and to the overall drainage plan has been extremely positive due to the aesthetic nature of the facilities, the fact that they nicely integrate into the community, their environmentally-sensitive nature and multi-purpose benefits. There is no question that the drainage system in Rock Creek substantially enhances community character and the value of residential properties.

Detention Facility—Flor Storage Facility

Detention Facility—Flor Storage Facility

and Interpretive Sign

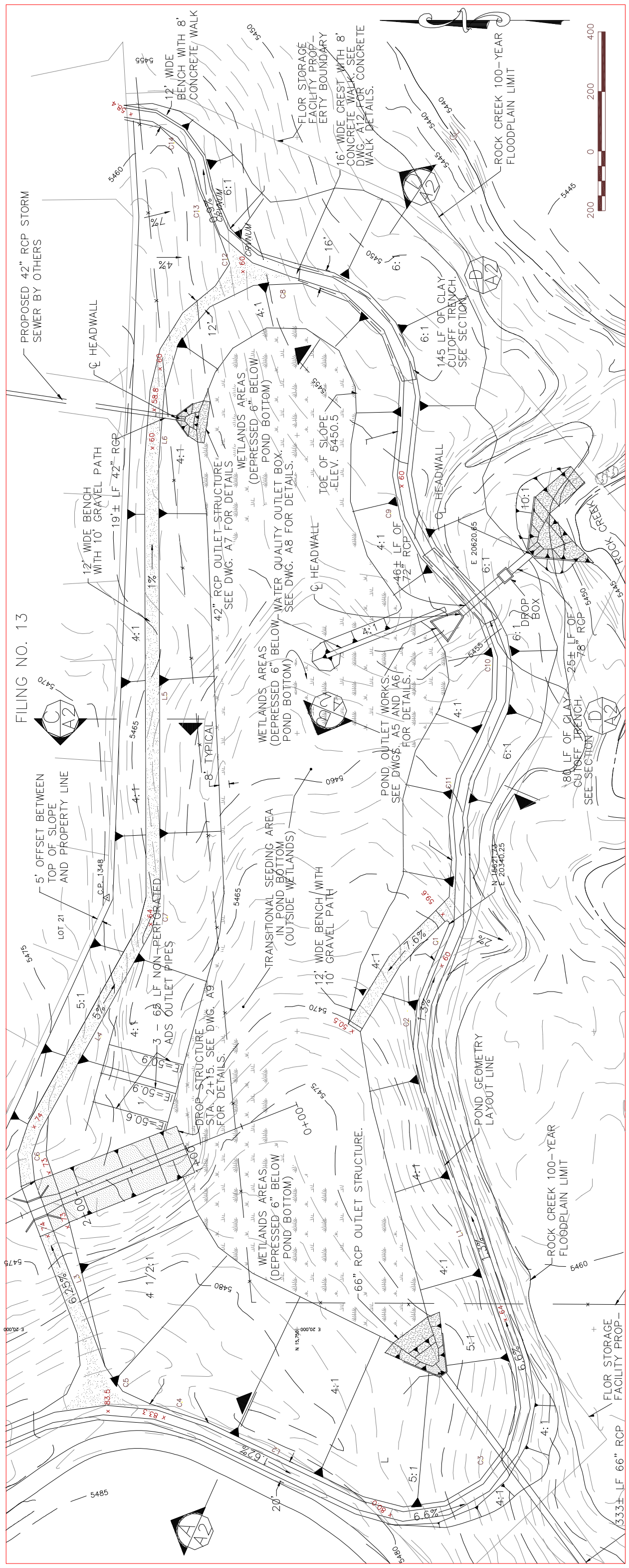


Figure 1—Flor Storage Facility Plan View

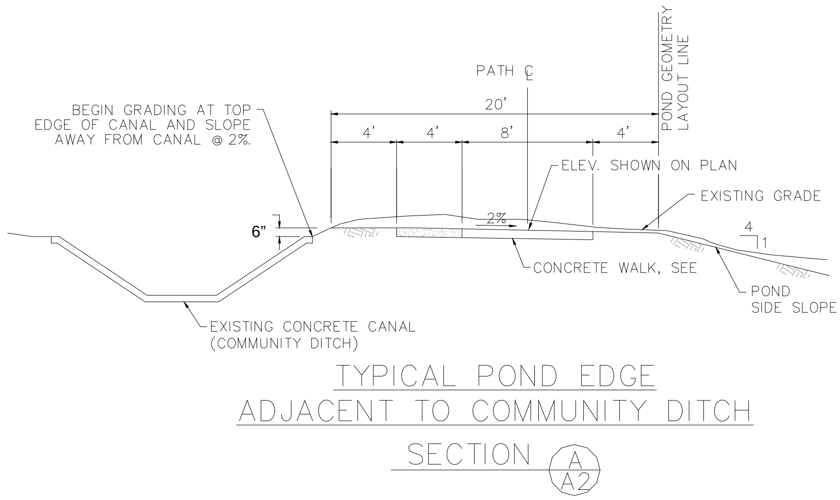


Figure 2—Typical Pond Edge Adjacent to Community Ditch

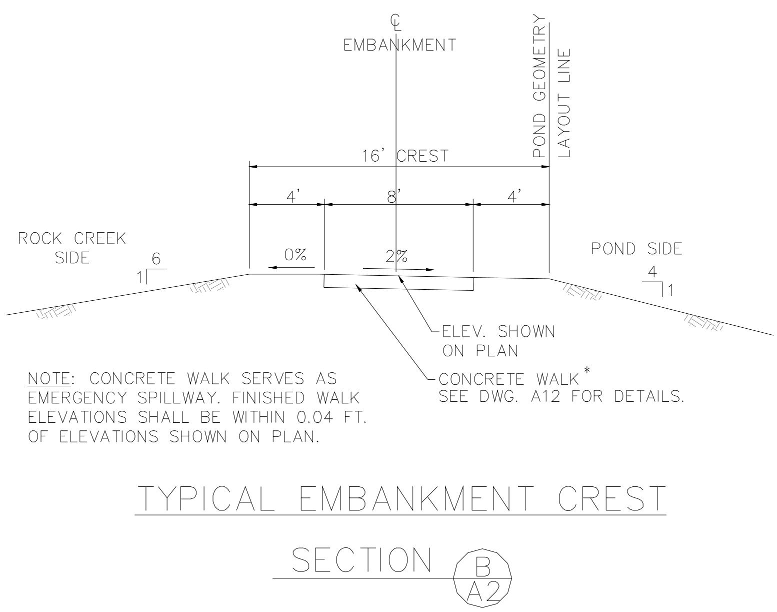


Figure 3—Typical Embankment Crest

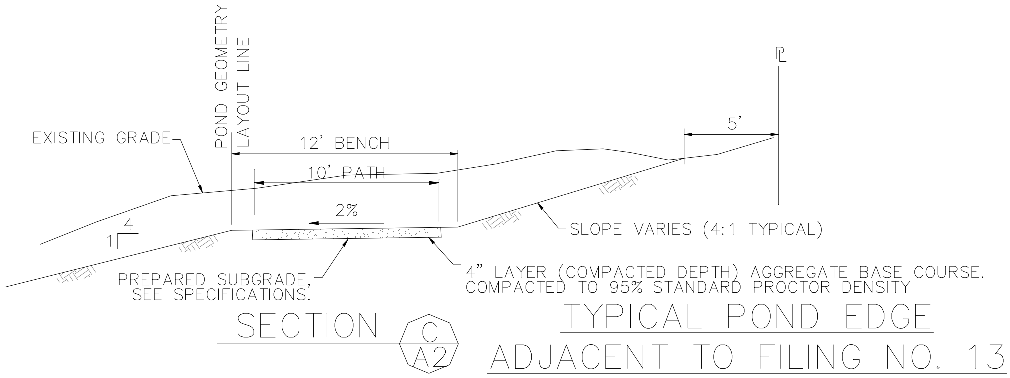


Figure 4—Typical Pond Edge Adjacent to Filing No. 13

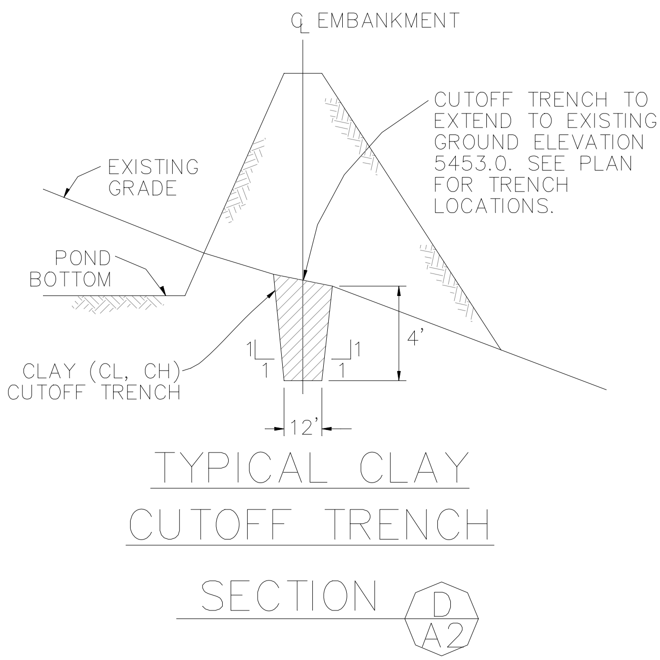


Figure 5—Typical Clay Cutoff Trench

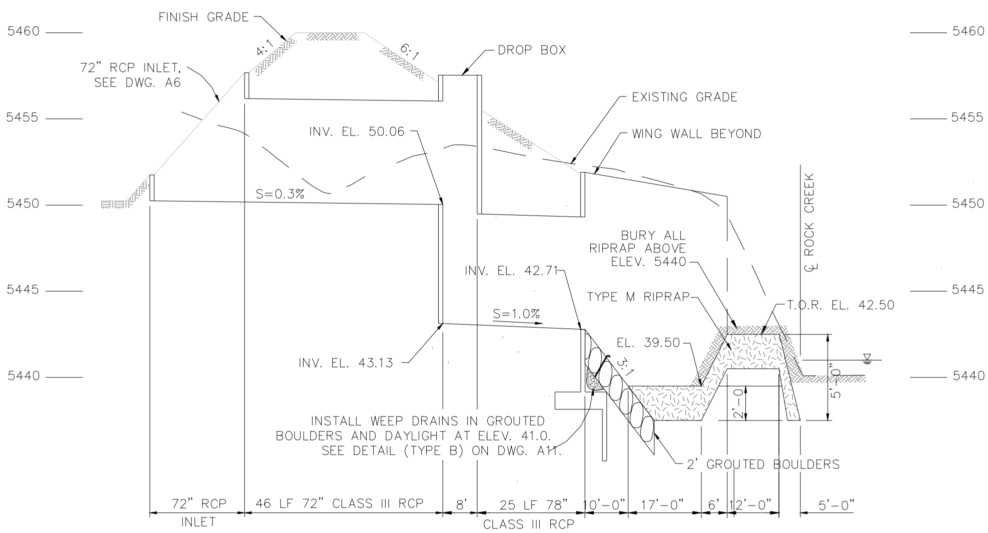


Figure 6—Profile Pond Outlet Works

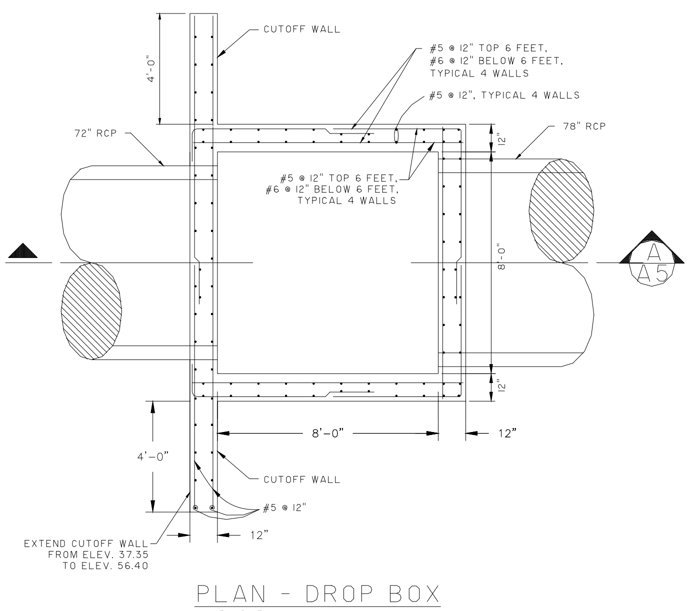


Figure 7—Plan Drop Box

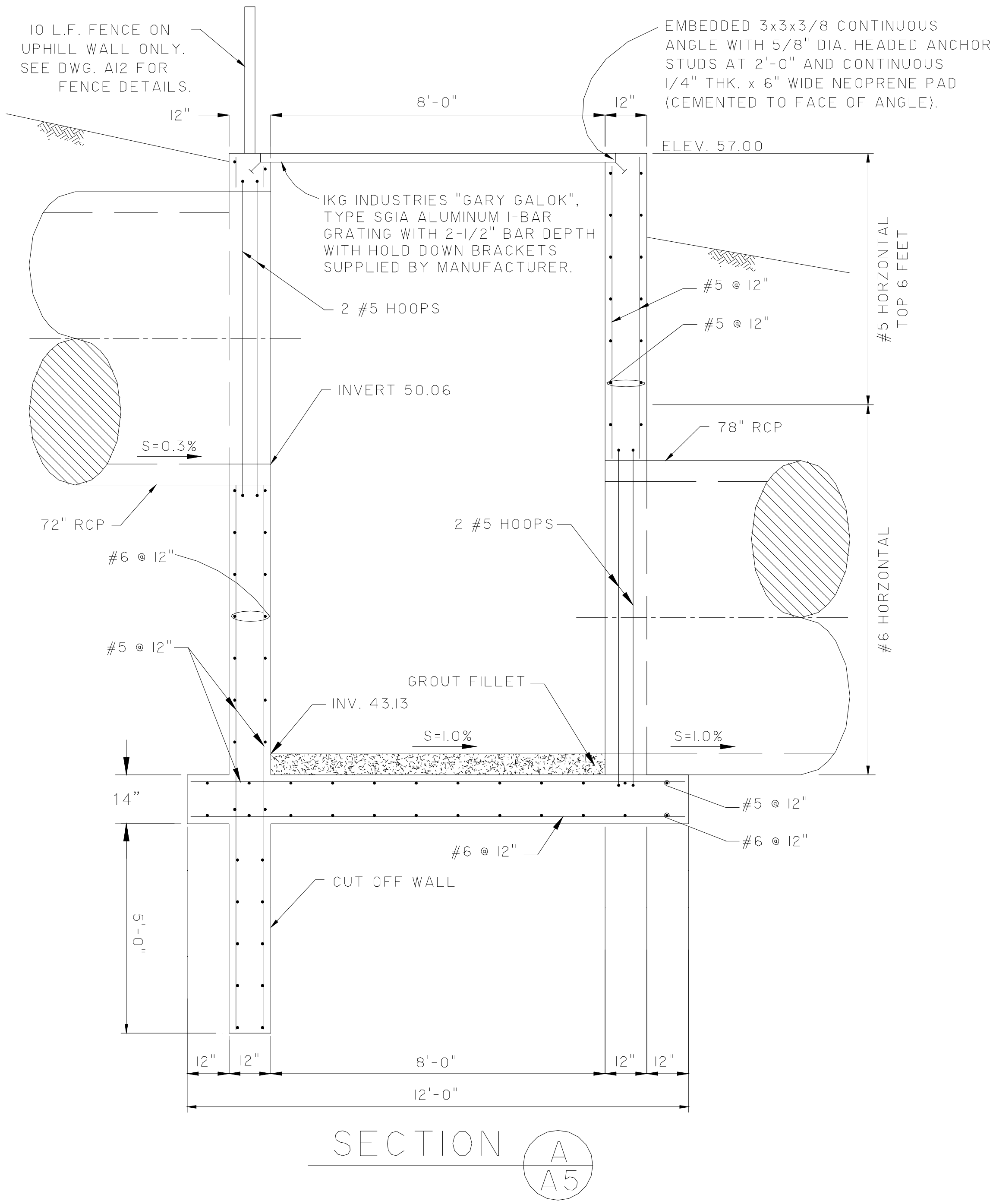


Figure 8—Section Drop Box

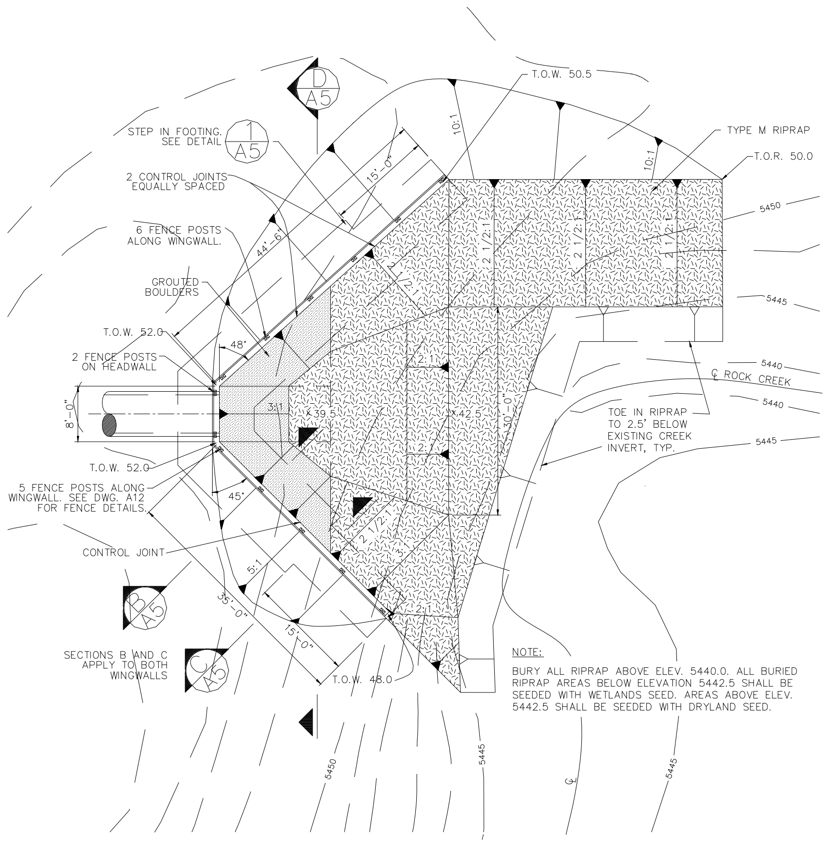


Figure 9—Plan 78" RCP Outlet

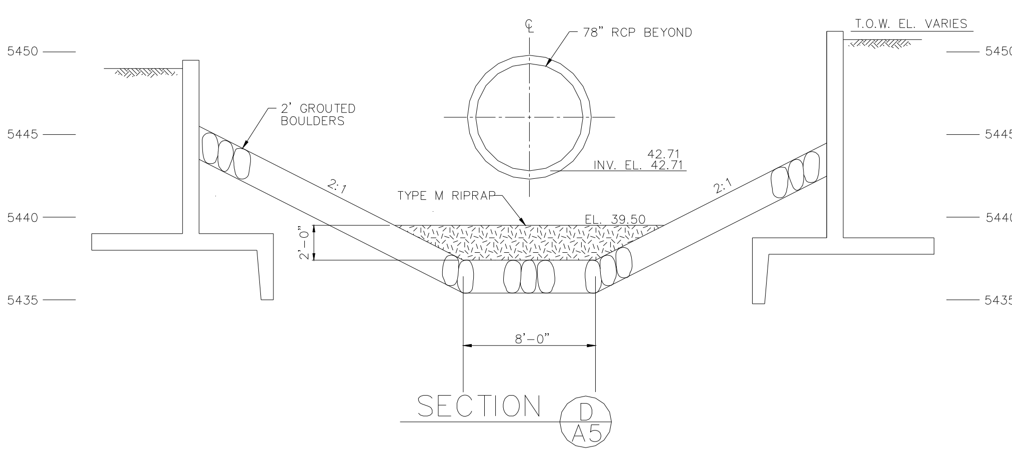


Figure 10—Section 78" RCP Outlet

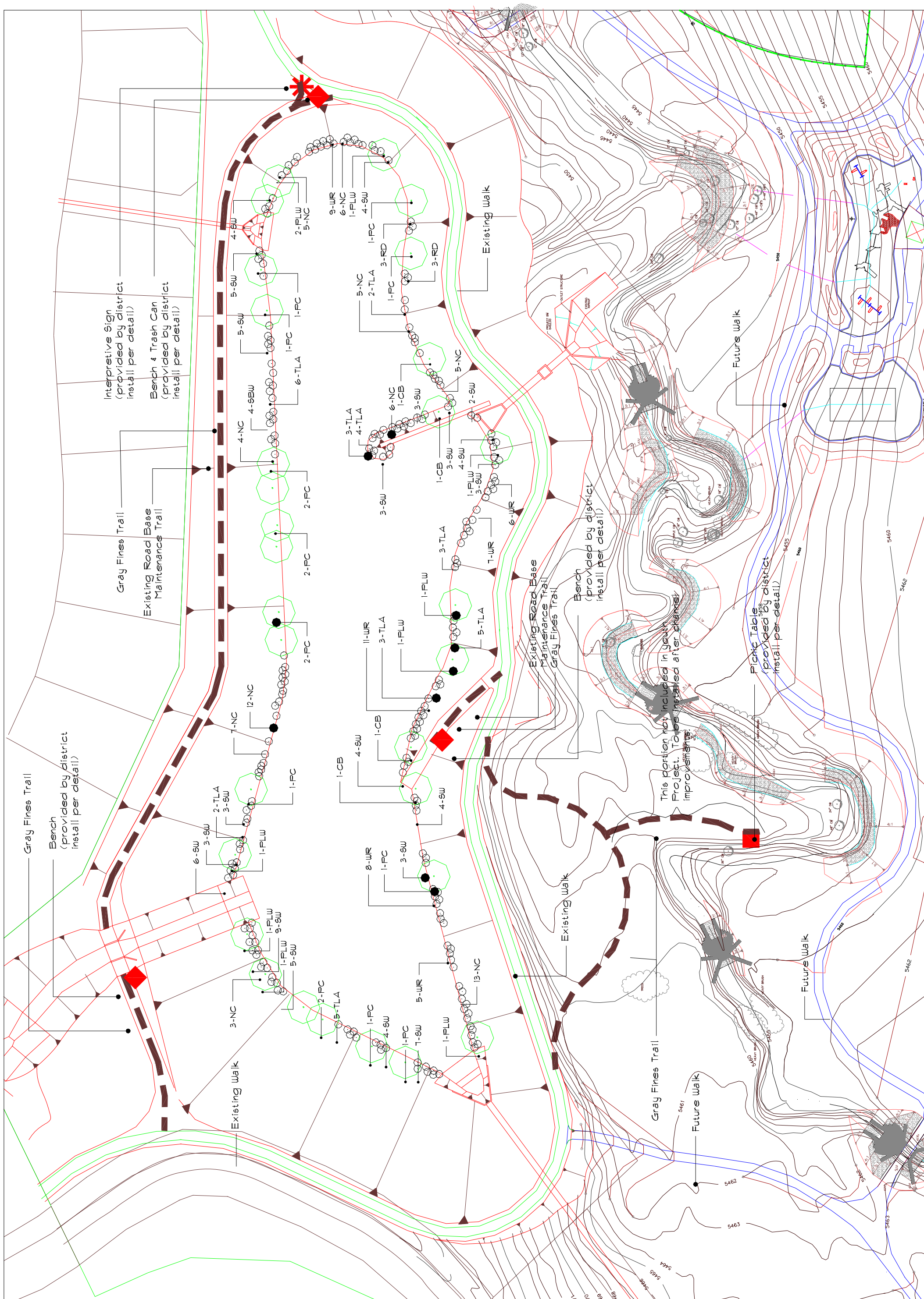


Figure 11—Rock Creek Flor Storage and Landscape Plan

Construction Notes

1. All plant material, trial surface material, and site furniture will be provided and delivered to project area by the district. Installation of these items shall be by the Boulder County Youth group.
2. Trail cutting by district contractor. Trail surface installation by Boulder Co. Youth. Minimize disturbance to grades and vegetation by removing cut material from the site. Do not pile soil on existing vegetation adjacent to the trial. Seed disturbed areas with native seed.
3. Notify The Norris Dullea Company for Trial, Plant, and Site Furniture locations prior to installation.
4. Trees and shrubs to be planted per detail. Install plant material in general locations shown on plan. Adjust location relative to moisture conditions for each species. Refer to Plant Legend for optimal moisture conditions for each species.
5. Water all planted material every 2 days for the first 30 days after planting. Reduce watering to every 4 days from 31 days to 90 days after planting. fully saturate soil within planting saucer with pond water. Minimize disturbance to the pond bank while obtaining water. Suspend watering schedule if pond floods. Resume watering after water levels reaches normal pool level.
6. Compact crusher fines in 2" lifts with sod roller. Maintain crusher fine material within the 6' cut area. Mound crusher fines 1" above edges in center of trail.

Plant Legend

Sym.	Plant Name	Size/Condition	Planting Instructions
PC	Plains Cottonwood	1 1/2" Cal. B&B	1'-2' above water table no water in planting pit not in water quality zone
PLW	Peach Leaf Willow	1 1/2" Cal. B&B	1'-2' above water table no water in planting pit not in water quality zone
CB	Common Boxelder	1 1/2" Cal. B&B	.5'-1.5' above water table no water in planting pit
TLA	Thin-Leaf Alder	5 Gal.	.5'-1.5' above water table no water in planting pit
NC	Native Chokecherry	5 Gal.	2'-3' above water table no water in planting pit not in water quality zone
SW	Sandbar Willow	5 Gal.	.5'-1' above water table no water in planting pit
RD	Redosier Dogwood	5 Gal.	.5'-1' above water table no water in planting pit
WR	Woods Rose	5 Gal.	2'-4' above water table no water in planting pit not in water quality zone

Figure 12—Landscape Plan Construction Notes and Plant Legend

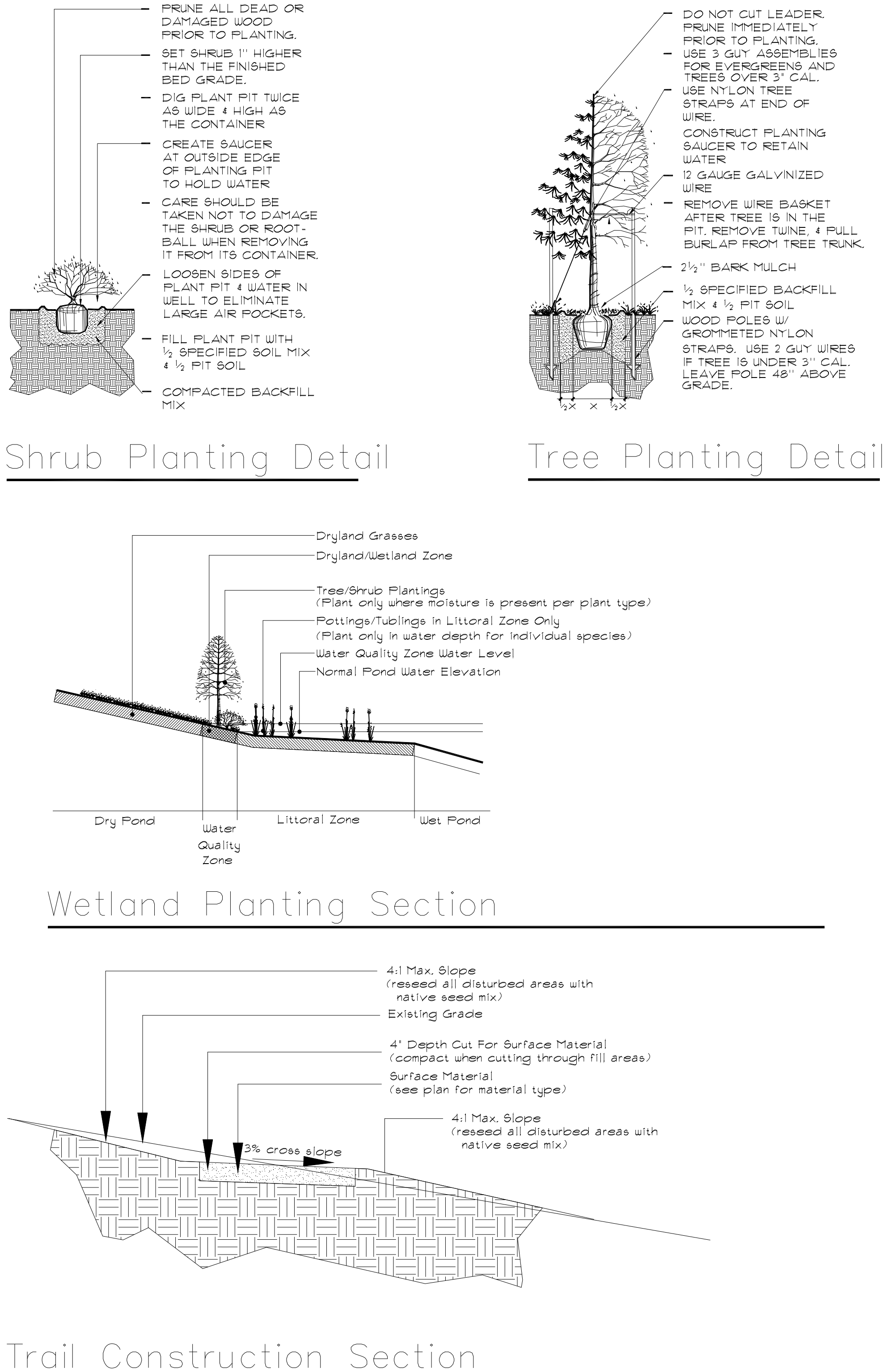


Figure 13—Planting and Trail Details

Grouted Boulder Drop Structures on Tributary LB-3 Channel

Grouted Boulder Drop Structures on Tributary LB-3 Channel

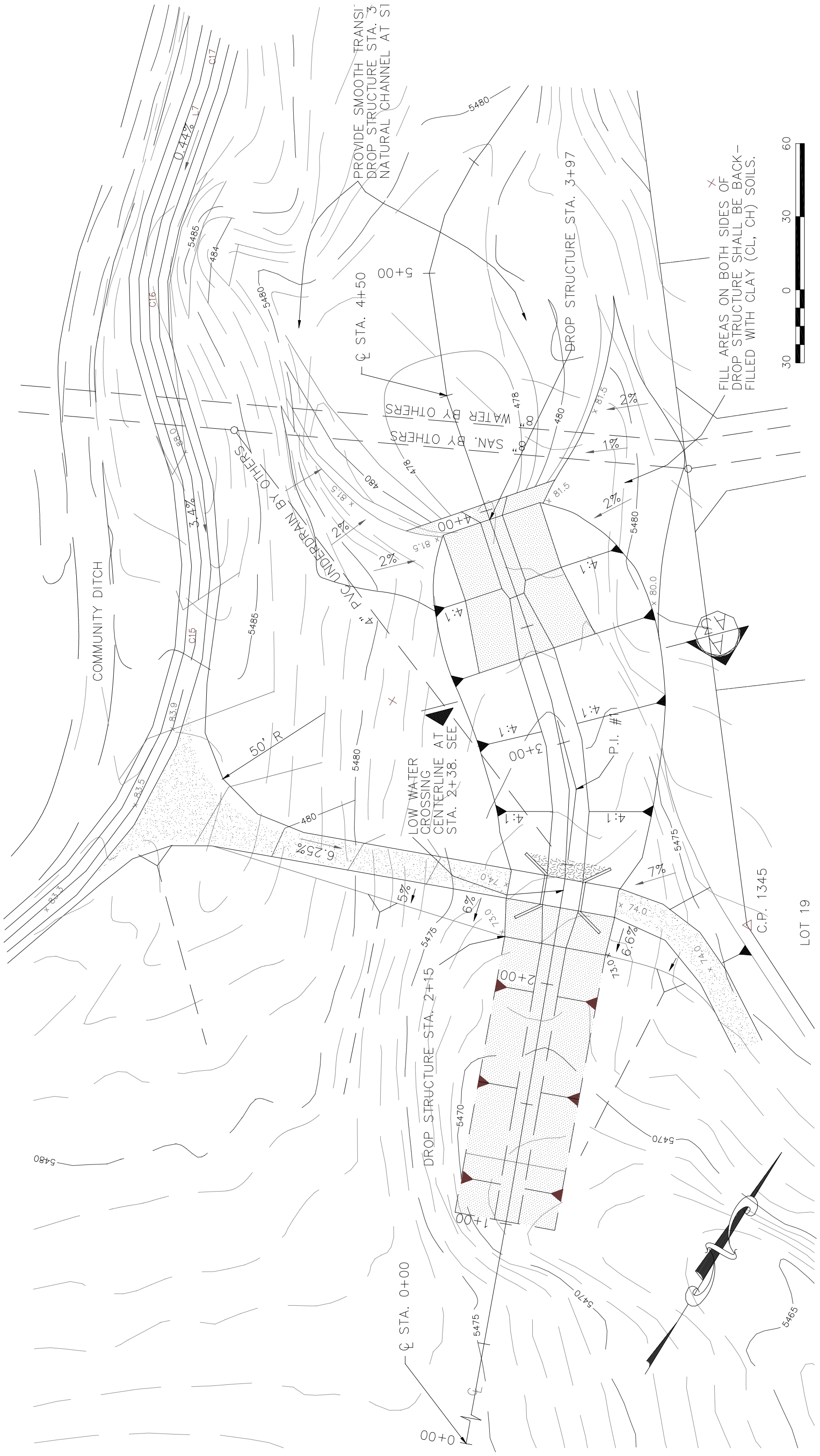


Figure 14—Grouted Boulder Drop Structures

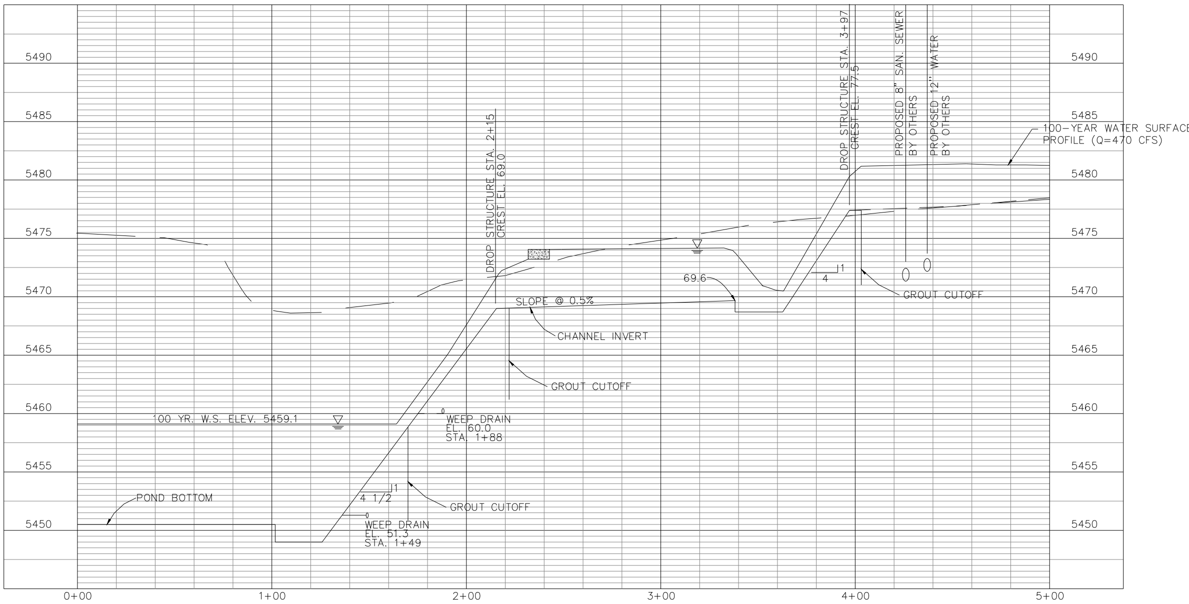


Figure 15— LB3 Channel Profile

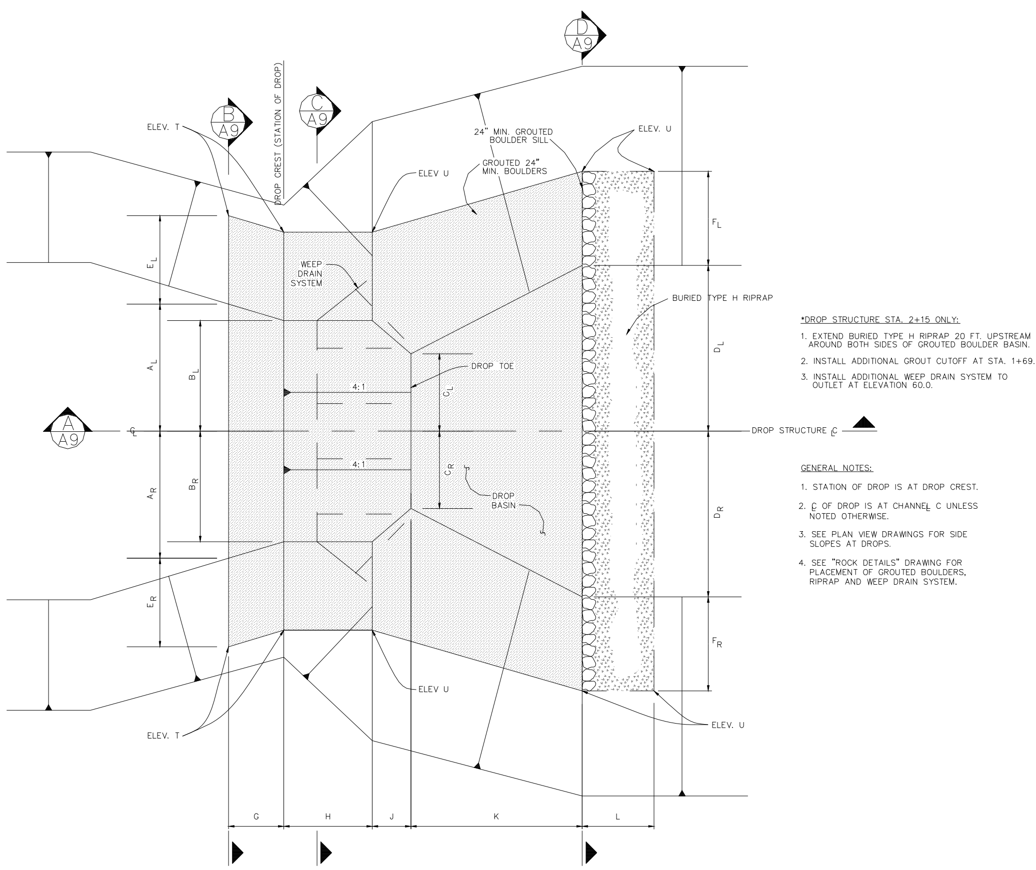


Figure 16—Typical Drop Structure

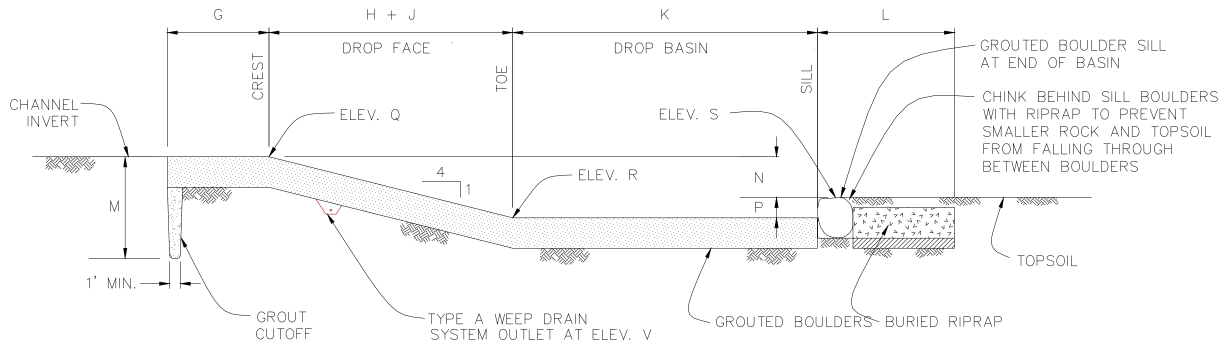


Figure 17—Grout Cutoff Section

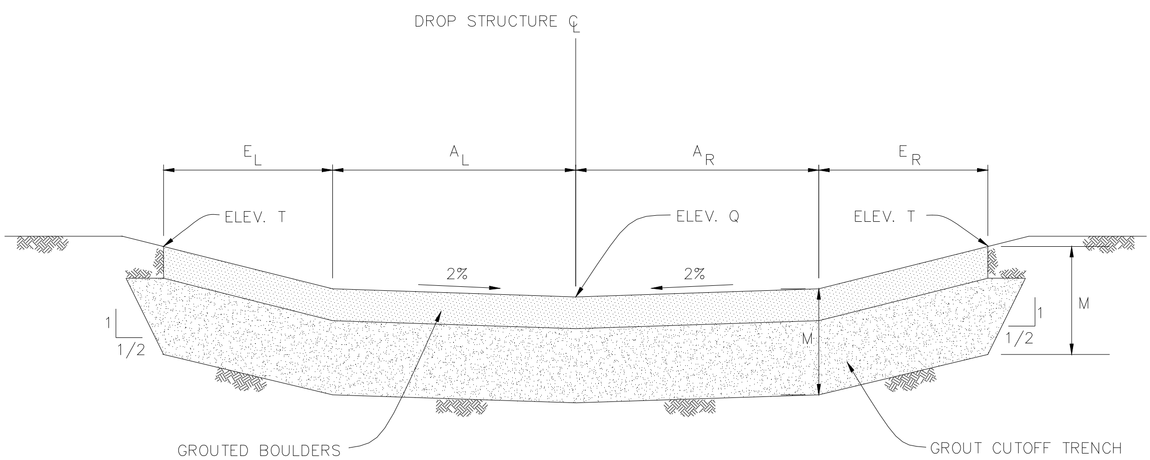


Figure 18—Drop Structure Profile

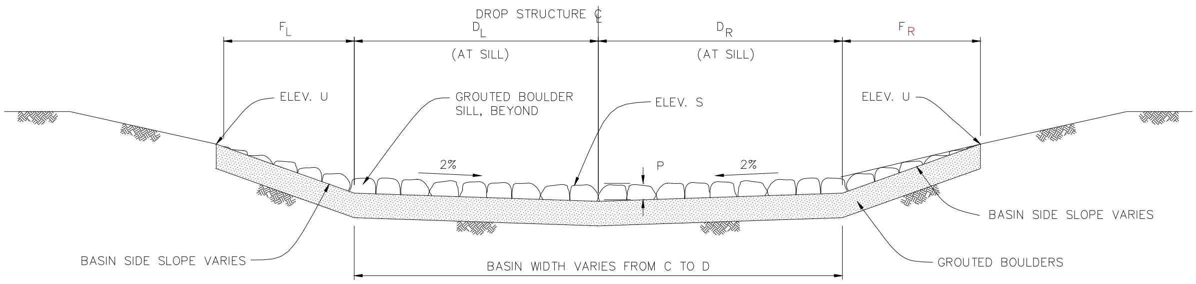


Figure 19—Typical Drop Basin Section and Sill

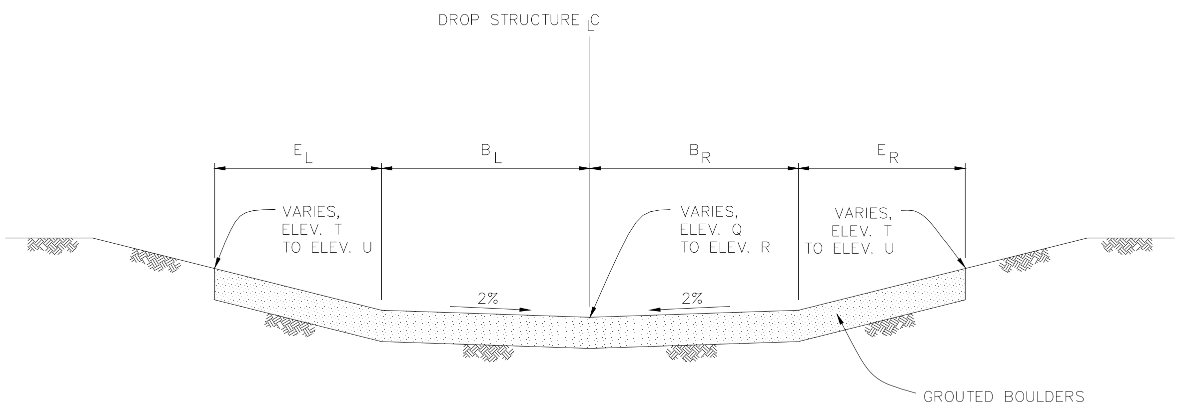


Figure 20—Typical Drop Face Section

DROP STRUCTURE MEASUREMENT TABLE

(ALL DIMENSIONS ARE IN FEET)

STATION OF DROP	A_L	A_R	B_L	B_R	C_L	C_R	D_L	D_R	E_L	E_R	F_L	F_R	G	H	J	K	L	M	N	P	ELEV. Q	ELEV. R	ELEV. S	ELEV. T	ELEV. U	ELEV. V	REMARKS
2+15	6.7	6.7	5	5	5	5	5	5	16	16	16	16	7	83.25	6.75	25	10	8	18.5	1.5	69.0	49.0	50.5	73.0	54.5	51.3 & 60.0	FACE SLOPE IS 4-1/2:1 ELEV. T = 73.7 AT UPSTREAM END OF DROP; ELEV. U = 50.5 AT SILL AND DOWNSTREAM
3+97	11	11	5	5	3	3	7.5	7.5	16	16	16	16	6	31.2	4	25	10	6.5	7.9	1.0	77.5	68.7	69.6	81.5	73.7	70.4	

Figure 21—Drop Structure Measurement Table

Grouted Boulder Check Structures And Wetland Bottom Channels

Grouted Boulder Check Structures And Wetland Bottom Channels

Wetland Bottom Channel

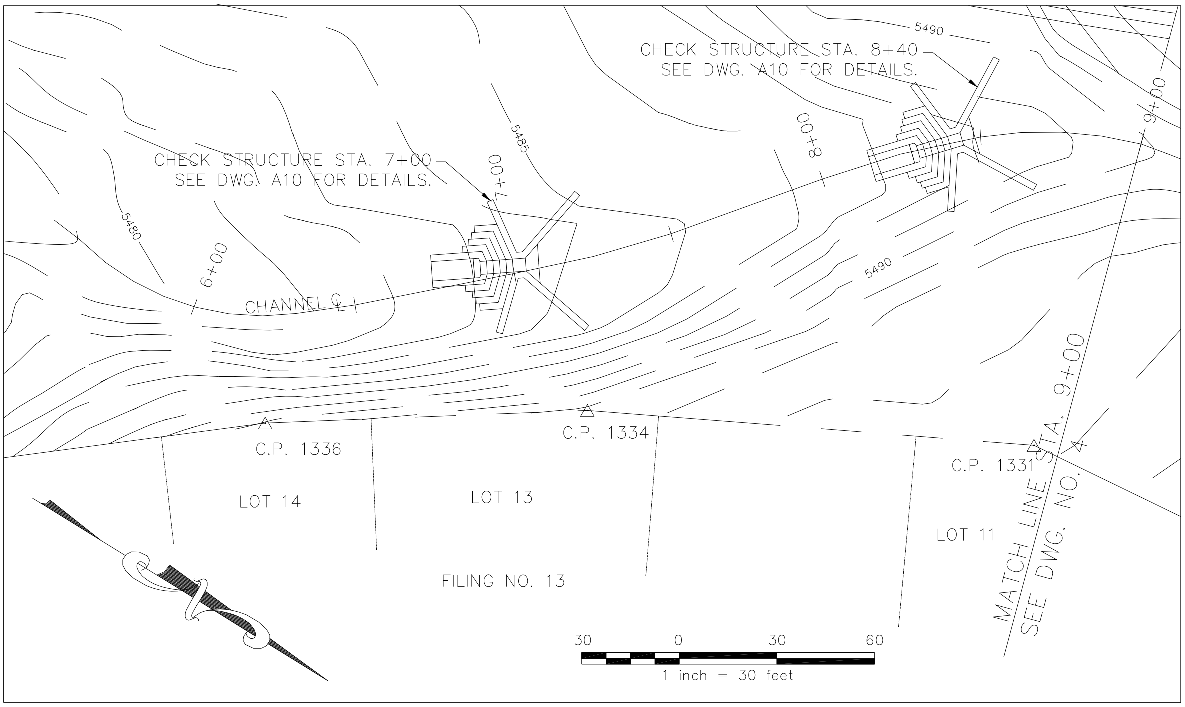


Figure 22—LB3 Channel Plan

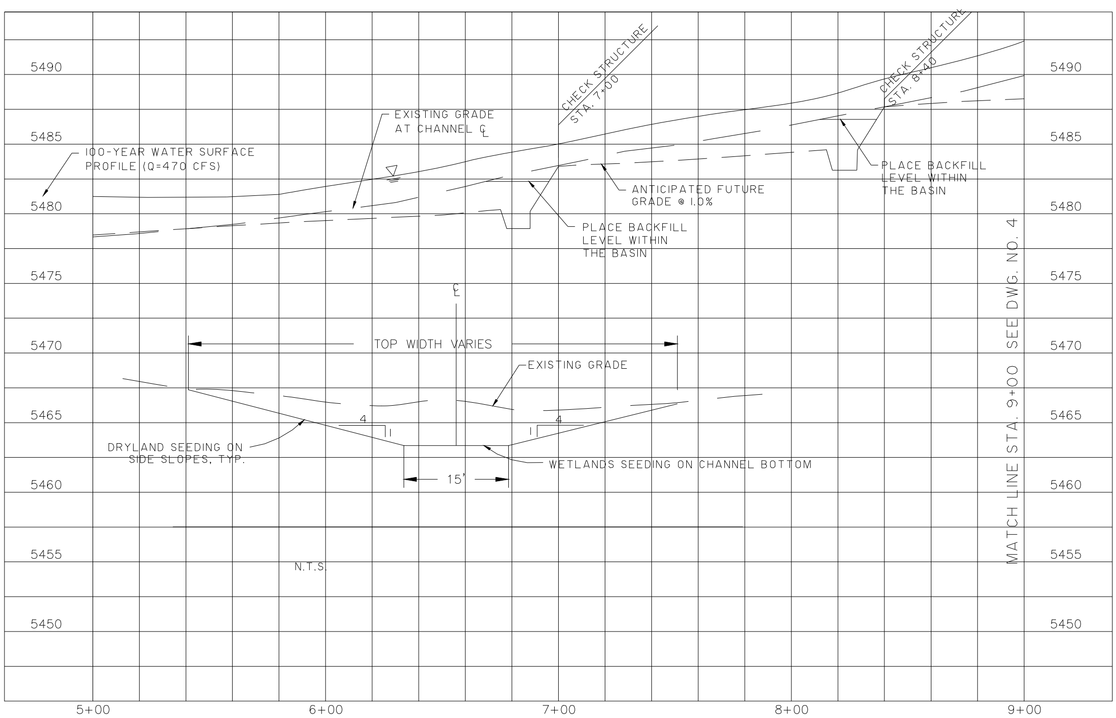


Figure 23—Typical Wetland Channel Section and LB3 Channel Profile

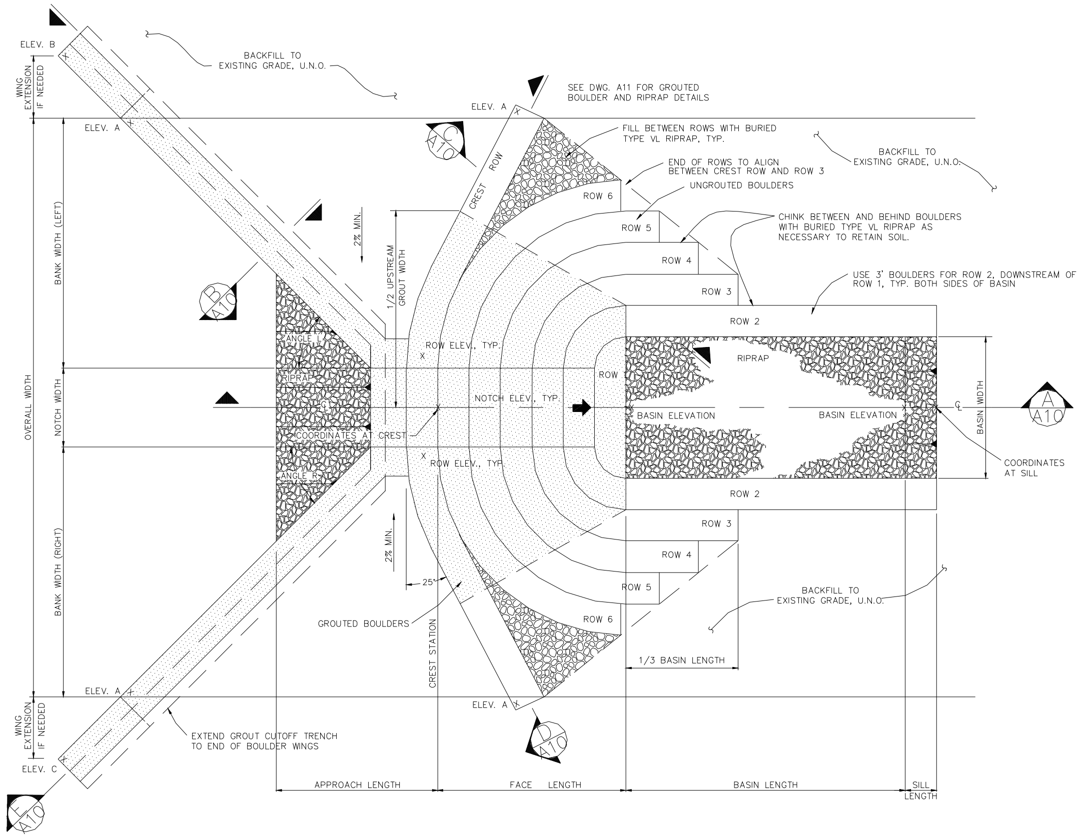


Figure 24—Check Structure Plan

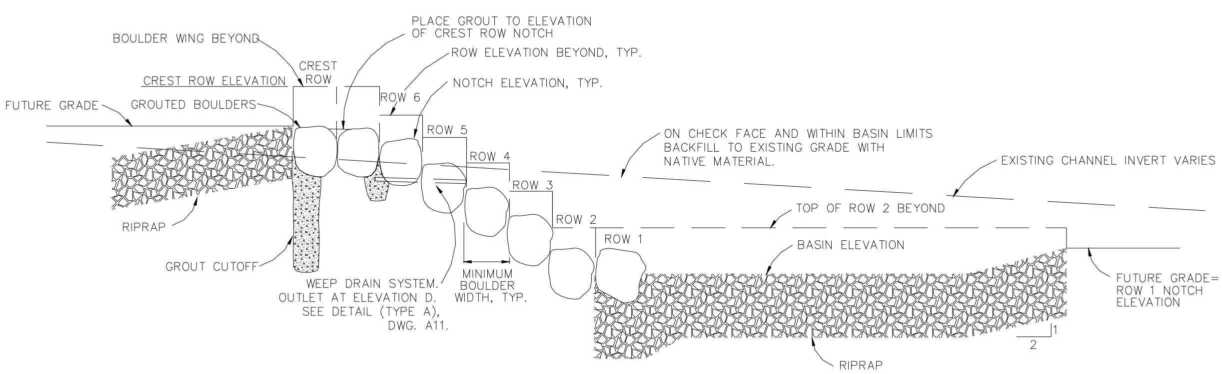


Figure 25—Check Structure Profile

CHECK STRUCTURE LAYOUT TABLE (all dimensions in feet, U.N.O.)

CHECK STRUCTURE DESIGNATION	STA. 7+00	STA. 8+40	STA. 10+50
SILL LENGTH	3'	3'	3'
BASIN LENGTH	10'	10'	10'
FACE LENGTH	12'	12'	12'
APPROACH LENGTH	8'	8'	8'
NOTCH WIDTH	4'	4'	4'
BANK WIDTH	18' L/18' R	18' L/18' R	18' L/18' R
WING EXTENSION	0' L/0' R	0' L/0' R	0' L/0' R
OVERALL WIDTH	40'	40'	40'
BASIN WIDTH	6'	6'	6'
1/2 UPSTREAM GROUT WIDTH	13' L/13' R	13' L/13' R	13' L/13' R
MINIMUM BOULDER SIZE	2'	2'	2'
RIPRAP TYPE FOR BASIN AND APPROACH	H	H	H
COORDINATES AT CREST			
NORTHING	16,333	16,427.5	16,628
EASTING	19,849	19,747	19,729
COORDINATES AT SILL			
NORTHING	16,313	16,411	16,603.5
EASTING	19,864	19,766	19,723
BASIN ELEVATION	79.2	83.5	88.5
NOTCH ELEVATIONS/ROW ELEVATIONS			
ROW 1	80.4/80.4	84.7/84.7	89.7/89.7
ROW 2	80.4/80.9	84.7/85.2	89.7/90.2
ROW 3	81.0/81.6	85.3/85.9	90.3/90.9
ROW 4	81.7/82.3	86.0/86.6	91.0/91.6
ROW 5	82.4/83.0	86.7/87.3	91.7/92.3
ROW 6	83.1/83.7	87.4/88.0	92.4/93.0
CREST ROW	83.4/84.4	87.7/88.7	92.7/93.7
BOULDER WING ELEVATIONS			
ELEVATION A	84.7	89.0	94.0
ELEVATION B	NA	NA	NA
ELEVATION C	NA	NA	NA
CUTOFF DEPTH	5'	5'	5'
ANGLE L (in degrees)	45°	45°	45°
ANGLE R (in degrees)	45°	45°	45°
WEEP DRAIN OUTLET-ELEVATION D	80.9	85.2	90.2

* IF NA APPEARS IN TABLE, THEN THAT PARTICULAR ROW OR ELEVATION IS NOT REQUIRED IN THE CHECK STRUCTURE.

Figure 26—Check Structure Layout Table

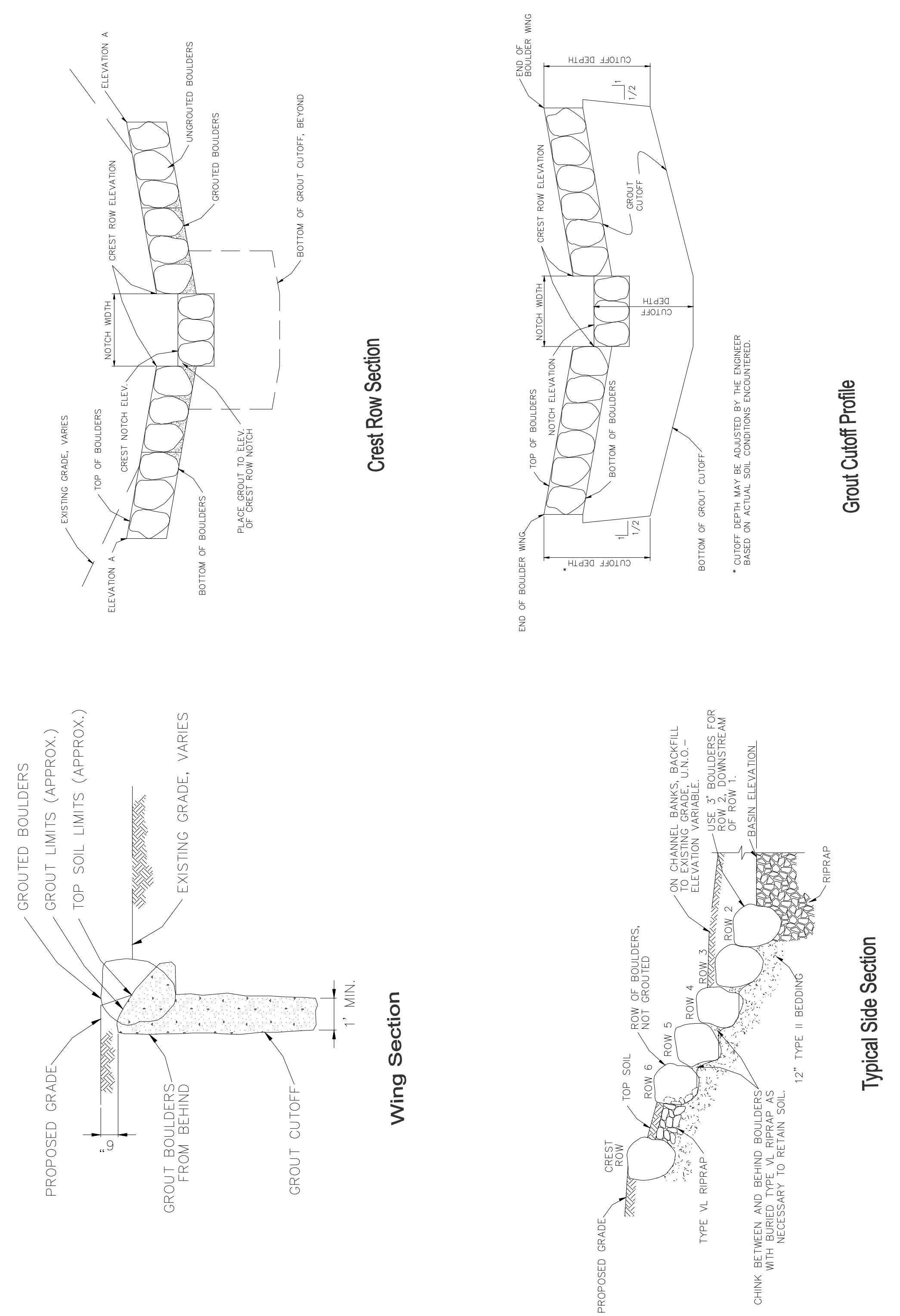
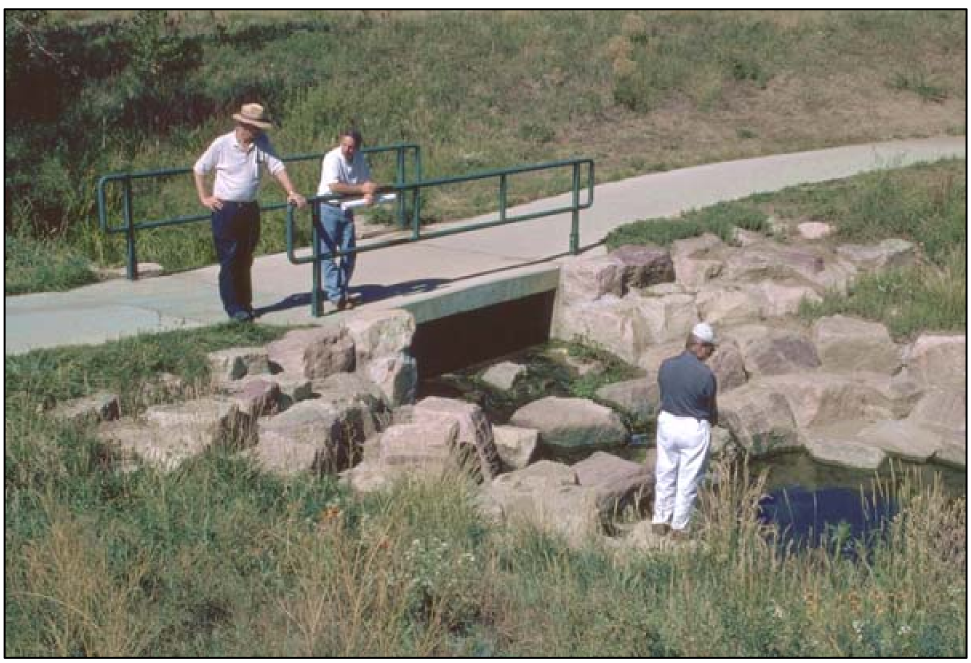


Figure 27—Check Structure Details

Stream Bank Stabilization Including Grouted Boulder Check Structure With Low-Water Crossing, Slope Flattening, And Revegetation



Stream Bank Stabilization Including Grouted Boulder Check Structure With Low-Water Crossing, Slope Flattening, And Revegetation

Stream Bank Stabilization Including Grouted Boulder Check Structure With Low-Water Crossing, Slope Flattening, And Revegetation

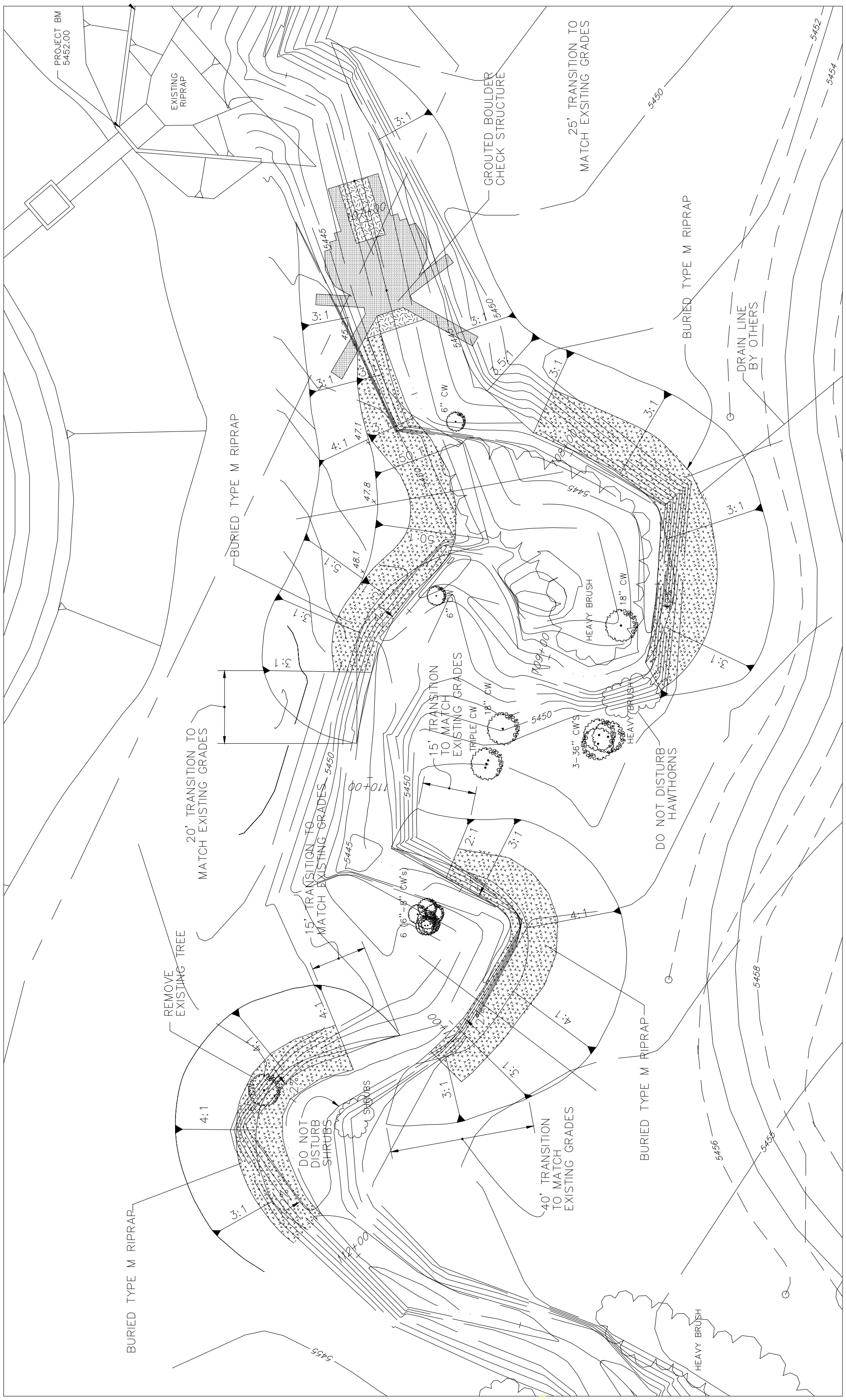


Figure 28—Stream Stabilization Plan

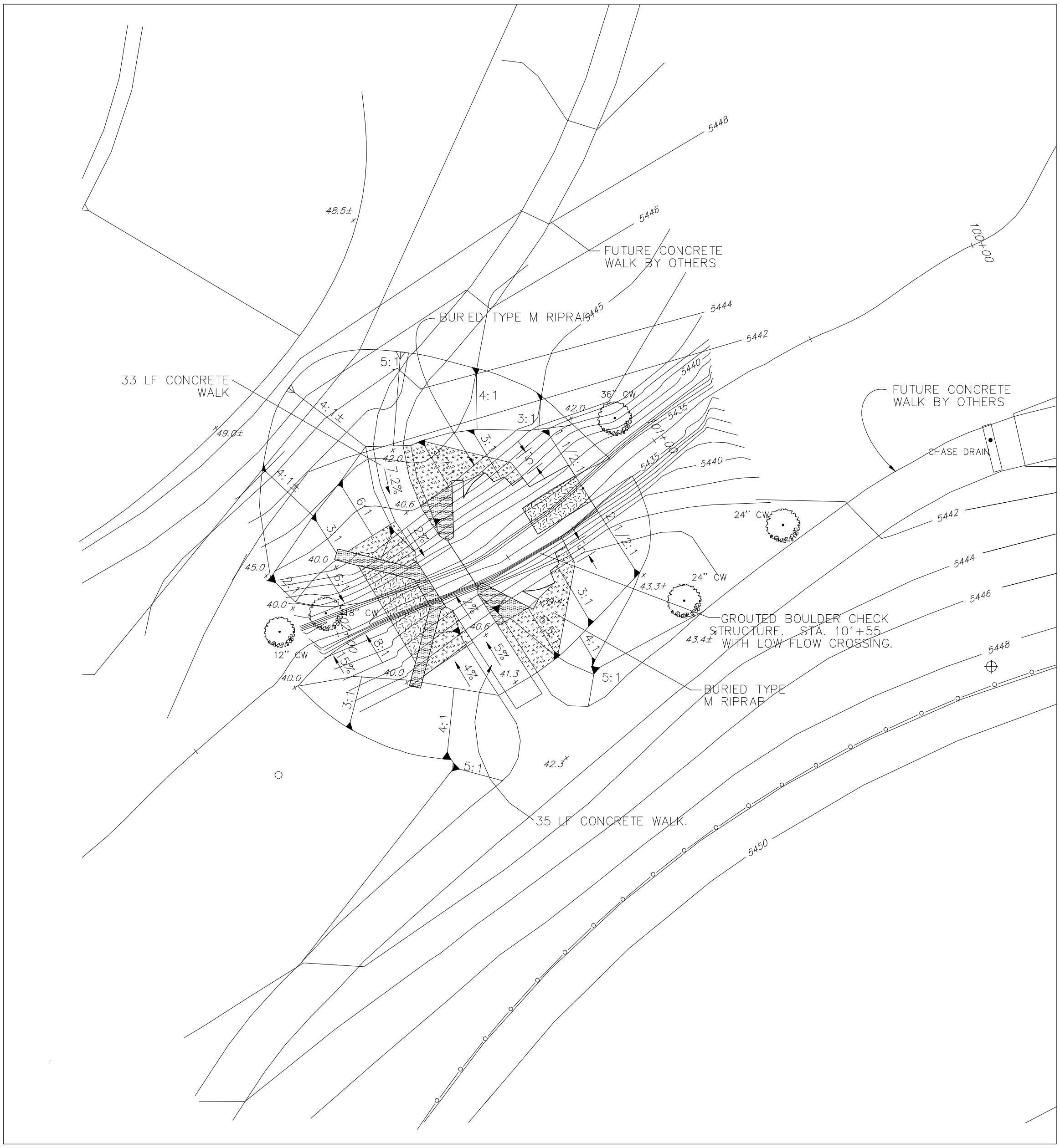


Figure 29—Grouted Boulder Check Structure with Low-Water Crossing Site Plan

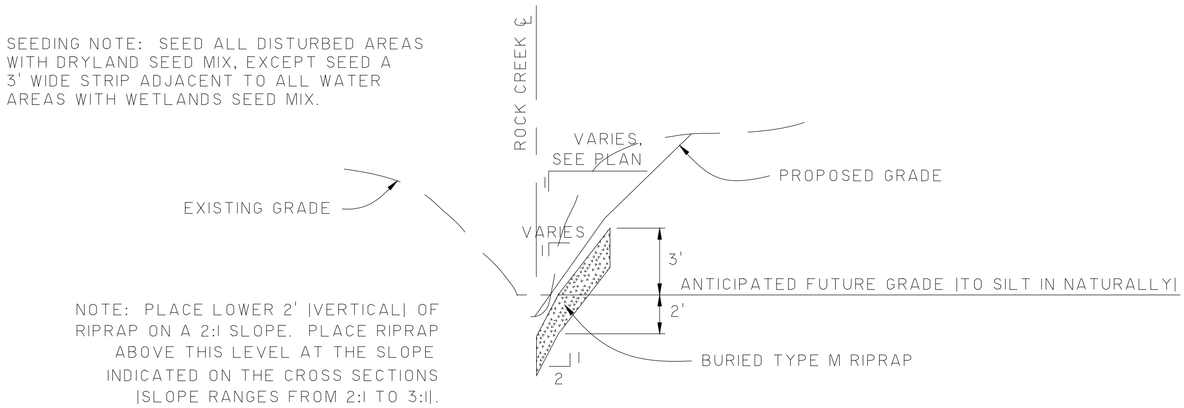


Figure 30—Typical Stream Stabilization Detail

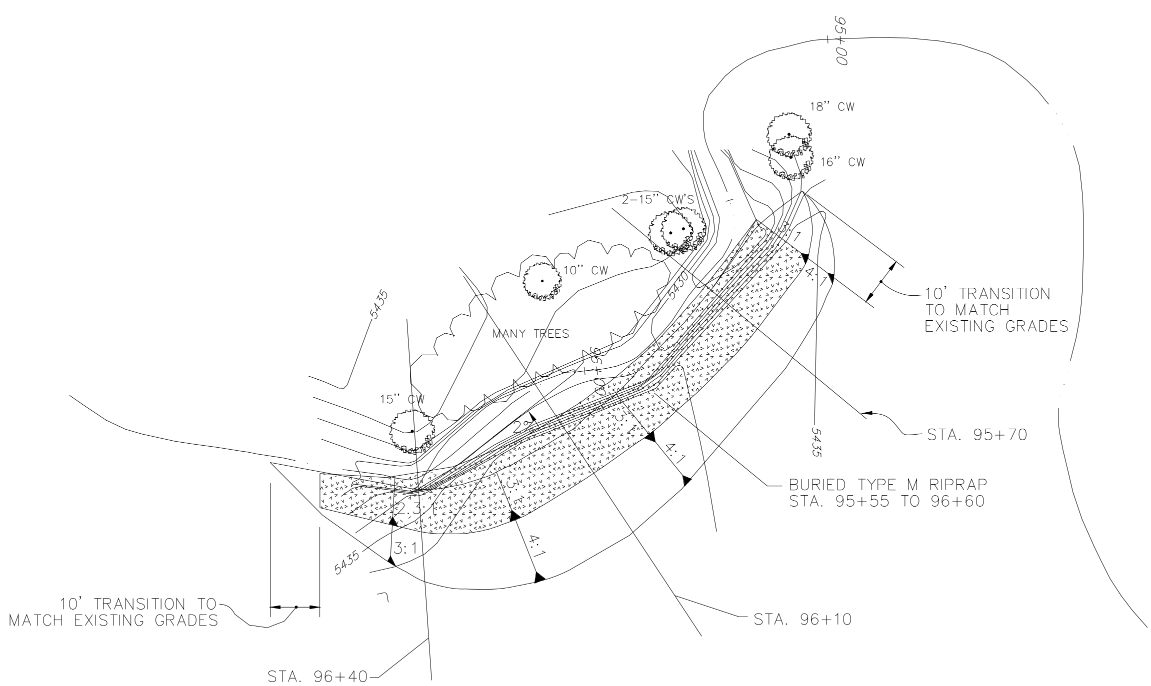


Figure 31—Stream Stabilization Site Plan

DESIGN EXAMPLES—SECTION 5

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Figures for Section 5

5.0 CASE STUDY—SAND CREEK

Bluff Lake Park is a 123-acre park that was created by the City and County of Denver to serve as an educational facility for Denver public school children and also as a gateway to the Sand Creek corridor. The site is located within the former Stapleton Airport operations area and includes the 9-acre Bluff Lake impoundment, 30 acres of wetland areas, abundant shortgrass prairie habitat, a remnant cottonwood gallery, and 3600 feet of Sand Creek, a major South Platte River tributary. The park is both a recreational area (pedestrians, joggers, etc.) and an educational facility where students learn about water quality, wetland, and riverine habitats. Improvements to this reach of Sand Creek include channel stabilization and enhancement of biological function along the corridor.

Historically Sand Creek contained very little or no base flow, with intermittent surface flows occurring in response to precipitation events. Most of the Sand Creek drainage flowed underground as part of the alluvial aquifer. Urbanization of the upstream watershed had impacted the creek, resulting in a flashy hydrograph, increased recurrence of flood flows, and development of a base flow. Erosion and deposition was occurring along the channel bed and banks as the channel tried to conform to the altered hydrology. The bank erosion threatened several structures adjacent to the creek, so bank-hardening treatments had been installed in localized areas as protection. The hardened banks were impeding the channel's natural tendency to meander and exacerbating the erosion problems. The altered flow conditions, the constrictions, and the channel's inclination to restore its wide, shallow, meandering flow pattern (all applied to the channel's highly erodible sandy substrate) were combining to create channel stability problems. These effects had been offset somewhat in the past by an undersized culvert bridge at Havana Road (downstream end of the project reach), which was creating a large backwater area and effectively slowing upstream velocities. The replacement of the culverts with a clear span bridge caused a substantial increase in velocities through the reach. The result was vertical streambanks that were over 12 feet high and channel downcutting up to 4 feet in some areas. Additionally, a large on-site meander was cut off as headcutting occurred through the reach. The unstable bed and banks were

threatening a treated wastewater reuse pipeline crossing and several structures situated along the streambank. The continual erosion had left virtually no streamside vegetation. Additionally, the degrading channel bed was causing an associated drop in the local water table, resulting in adverse impacts to streamside vegetation and related riparian and wetland habitats. Native plants along the corridor, especially the mature cottonwoods, which are an important park feature both for their age and because they provide nesting habitat for species such as Swainson's hawk and the great-horned owl, were showing signs of stress and losing ground to invasive species, including salt cedar tamarisk and Russian olive. Understory grasses and forbs also showed signs of takeover by invaders such as knapweed and leafy spurge.

Aquatic and Wetland Company (AWC) and Camp Dresser and McKee, Inc. (CDM) provided design and construction services for the City and County of Denver for the improvements to the Sand Creek corridor. The option of restoring the historically wide floodplain was not possible due to the adjacent development. Additionally, restoration of a wide floodplain through such high cutbanks would not have been economically feasible due to the large amounts of excavation that would be required. Therefore, the project sought to stabilize the channel bed and banks. All work needed to be compatible with and contribute to the use of the area as a recreational and educational facility. To that end, bioengineering treatments were integrated with more traditional bank stabilization methods, and the additional goals of riparian, wetland, and upland habitat restoration were included. Traditional bank stabilization measures, such as riprap and boulders, were limited to eroded slopes that were too steep for bioengineering treatments and to the critical junction of the channel bank and channel bed. Boulders placed at this junction provide protection to allow sufficient time for vegetation to be established and, eventually, cover the rock. The boulders and vegetation jointly provide protective cover for both vertebrate and macroinvertebrate fauna.

5.1 Design

In addition to the integration of bioengineering techniques and traditional methods to provide the necessary stabilization, an important design concept was to create a meandering low-flow channel within the armored outer banks, or flood channel. A 25- to 40-foot-wide low-flow channel designed to convey a base flow of 20 to 50 cfs was left completely unconstrained to meander at will within the 40- to 140-foot-wide channel (conveying the more frequently

occurring smaller flood flows), in an imitation of the creek's natural condition.

The primary treatment for stabilization of the main channel banks was a double boulder terrace with brush layering. The treatment consisted of two rows of large boulders set on a deep, concrete rubble foundation. The foundation was constructed using recycled runway concrete blocks from the demolition of Stapleton Airport. The minimum cutoff achieved by the boulders and the rubble foundation was 3 feet below the low-flow channel invert. A continuous line of coyote willow (*Salix exigua*) cuttings (brush layering) was then installed behind the lower boulder toe. The provision of vegetation along the immediate channel edge was especially important to restoring biological function because the plants provide leaf litter to the stream system (i.e., base of the food chain), as well as providing overhead cover for fish, and performing shading/cooling functions for the system. A planting terrace with a maximum slope of 3H:1V was created between the rows of boulders. The terraces were planted and seeded with native riparian trees, shrubs, grasses, and forbs. Two unique plant communities were established along the terraces – cottonwood gallery and riparian scrub-shrub. The bank side slopes created by the combination of boulders and terracing were designed with a maximum effective slope of 2.5H:1V, which provided a substantial reduction from existing slopes. In most areas, the effective side slopes that were achieved were flatter than the design maximum. The slope protection in this treatment comes primarily from the two rows of boulders and secondarily from the root structure, which will be created as the vegetation matures.

In some areas, such as low-risk inside bends, hard protection was not needed. A willow log designed specifically for the project was used for toe material, in place of boulders, in these areas. The logs were manually constructed on-site using coir erosion control fabric, native fill material generated by the project, supplemental imported mulch, and willow cuttings. In addition to creating a stabilizing toe for less critical banks, the logs were used to create a check structure to control the minor inflow, consisting of treated wastewater effluent, routed from the neighboring Aurora Wastewater Reuse Plant. The intent of the specialized willow logs was to let the willows in the outer layer of the log produce stabilizing roots and overhead foliage, which will continually increase bank protection as well as riparian habitat. The problem of securing the logs into loose, sandy soil was solved through the use of Duckbill anchors. The Duckbills have anchors that rotate when pulled, locking themselves into place deep in the banks. They perform exceptionally well in sands where typical staking may be ineffective.

Stabilization of the channel bed was accomplished through the installation of two grouted boulder drop structures with sheetpile cutoffs. The drop heights (4-foot and 8-foot) were set to achieve an average 0.2 % slope for the reach. The drop structures were designed with a step-pool configuration, with a maximum drop of 1.5 feet between each step and a minimum pool depth of 3 feet. These specifications allow for fish passage and provide resting habitat for migrating fauna. The boulder crests were installed in curving alignments and pools were given uneven shapes and sizes to avoid an overly structured look. The larger drop included a planting terrace along its length to restore streamside vegetation and soften the look of the structure. The structure included the wastewater effluent pipeline crossing in its crest. The cascading step-pool design of the grade control structures makes a nice park amenity with its soothing sound and natural aesthetic quality. Additionally, propane testing has indicated that the structure is an excellent passive re-aerator, with the 8-foot drop exhibiting an overall efficiency of 60% and individual step efficiency of close to 19%.

5.2 Criteria

The use of drop structures to reduce the channel slope to 0.2 % follows the recommendations of the District's 1984 Sand Creek Major Drainageway Plan. The channel improvements were designed for general channel stability up to the 10-year flow of 9,000 cfs. The low-flow channel carries the channel's base flow of approximately 20 to 50 cfs. Bioengineering techniques were utilized to the maximum extent possible. Wherever conditions exceeded the expected stabilization potential of available bioengineering methods, vegetative treatments were added to the riprap, boulder, and concrete techniques.

5.3 Construction

AWC and CDM Engineers and Constructors constructed the Sand Creek channel improvements. The 3,600 feet of channel improvements included almost 50,000 cubic yards of cut/fill (largely due to realignment of the lower reach of the creek to avoid the new Colorado Department of Correction Women's Detention Facility), sheetpile installation, structural concrete and grout work, boulder placement, and comprehensive planting and seeding. Timing was the biggest construction challenge. Contract delays caused a late construction start, which pushed construction into the summer thunderstorm season. As a result, construction was interrupted several times by rapidly rising water levels.

5.4 Success

Many of Bluff Lake's patrons have praised the Sand Creek Channel Improvements Project for the natural look that was achieved and for the improved habitat along the creek corridor. The project has, to date, met its goals of stabilizing the channel bed and banks and enhancing biological function, while maintaining compatibility with the District's master plan recommendations and contributing to the use of the park as an education facility. Healthy growth has been observed in the willow brush layering (installed behind the boulder toes, on top of buried rubble as part of all double boulder terrace bank treatments). Combined with pre-existing willow stands located along the creek, the new treatments have created over 5,000 linear feet of solid willow coverage along the water's edge. Individual plant growth was noted at over 3 feet in one growing season in some sections. The planted willows are healthy and robust and appear to be continuing the strong growth pattern as they mature. Great blue herons, hawks, and families of ducks have been observed along the creek and among the willows since the project's completion. This project illustrates that in this reach of Sand Creek, a reach that has been impacted by upstream urbanization, the combination of structural elements with bioengineering techniques can produce an environmentally productive and stable urban stream.

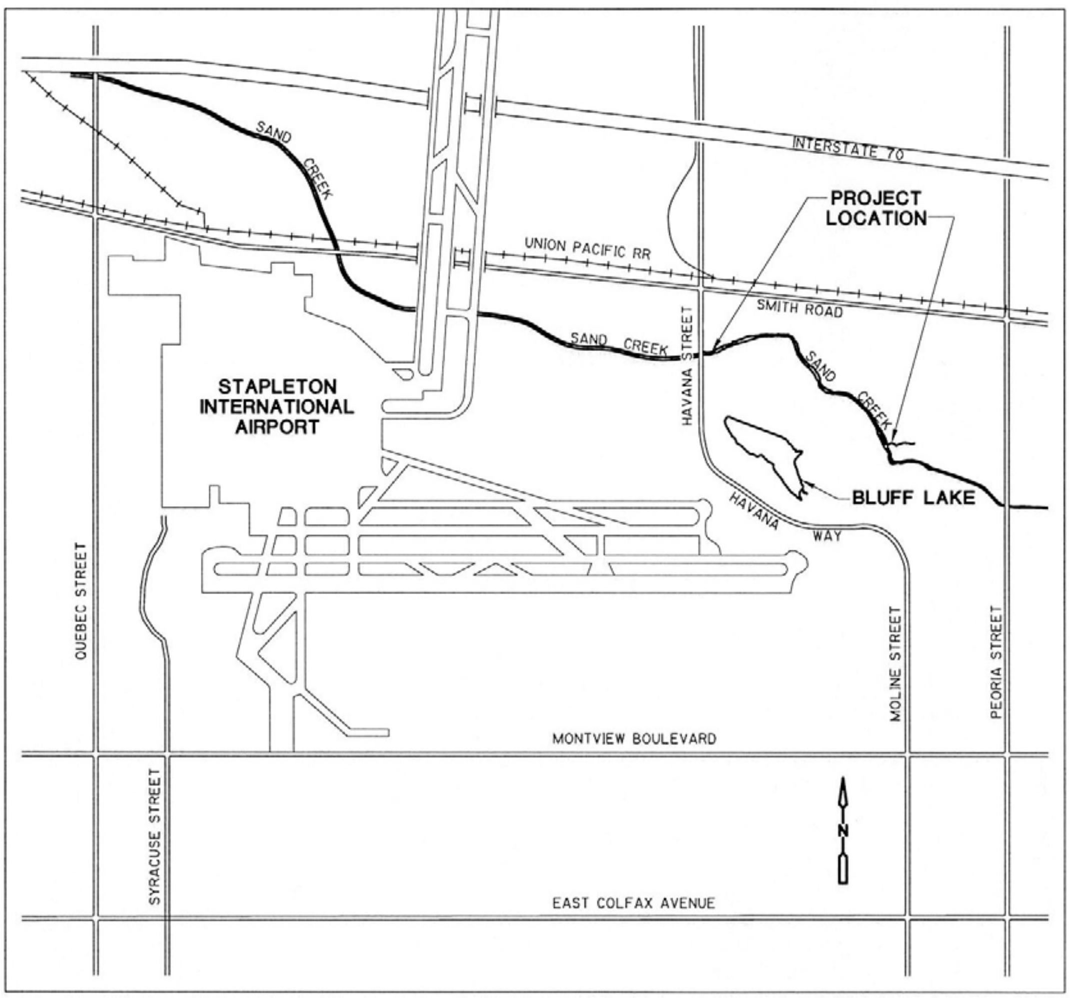

Figure 1—Location Map

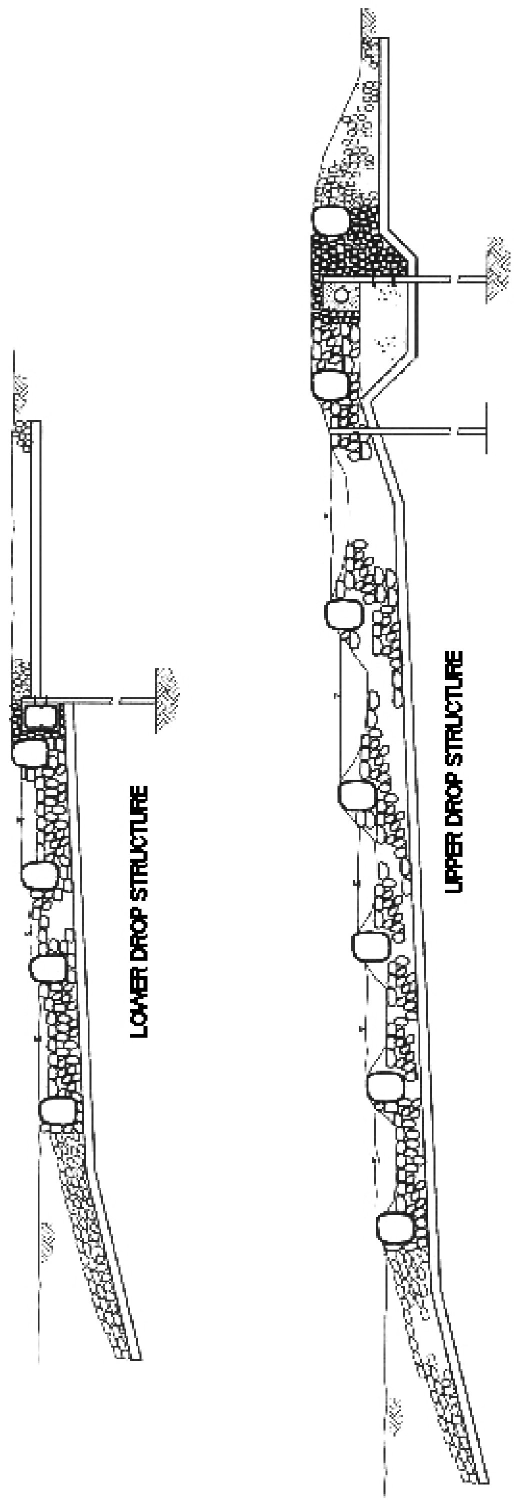

Figure 2—Drop Structures

Sand Creek Channel Improvements at Bluff Lake Park
Denver, Colorado

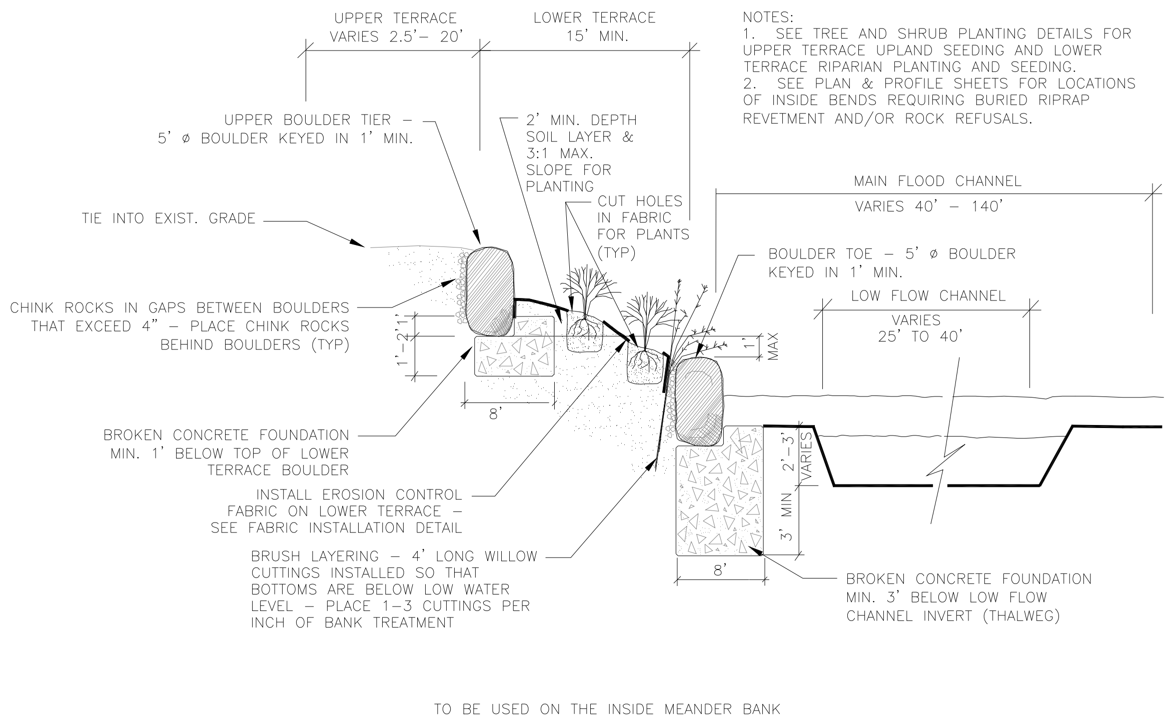

Figure 3—Double Boulder Terrace

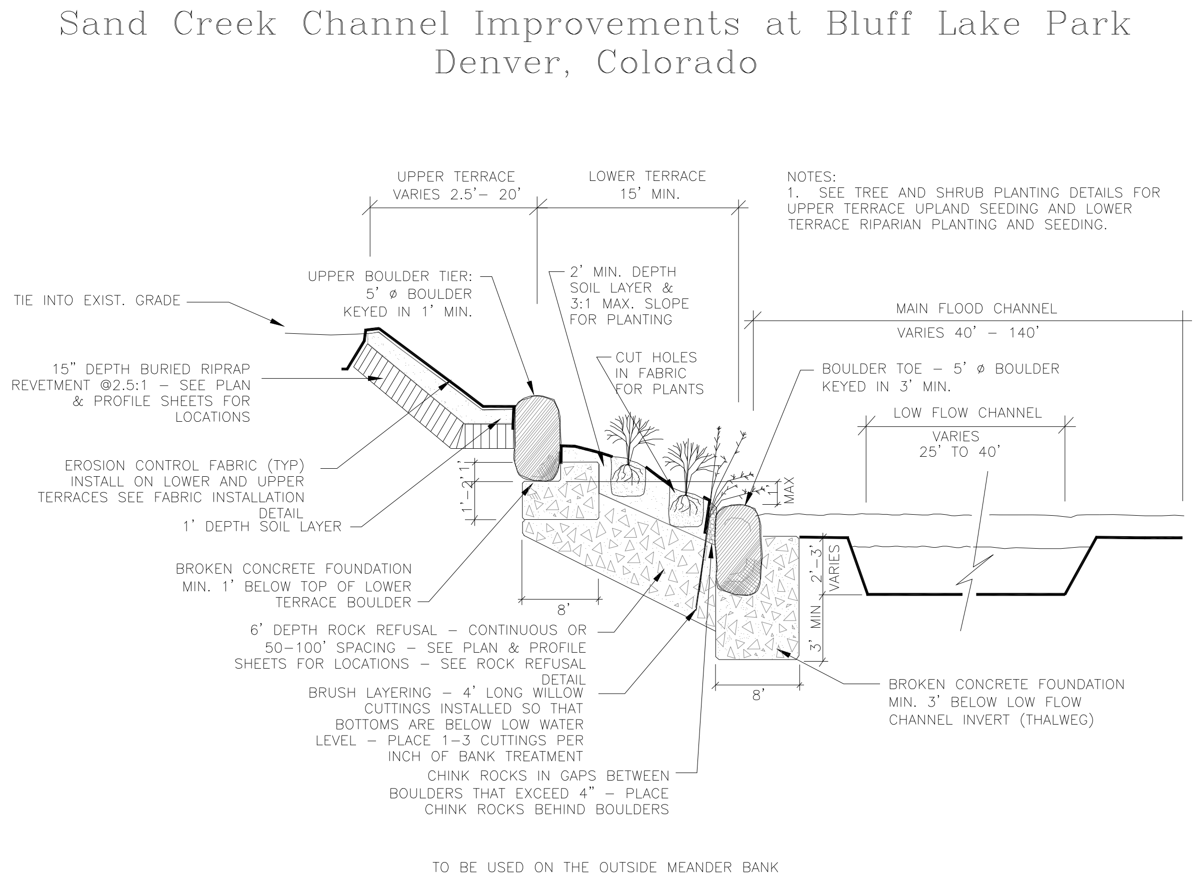

Figure 4—Double Boulder Terrace with Buried Riprap Revetment

Sand Creek Channel Improvements at Bluff Lake Park
Denver, Colorado

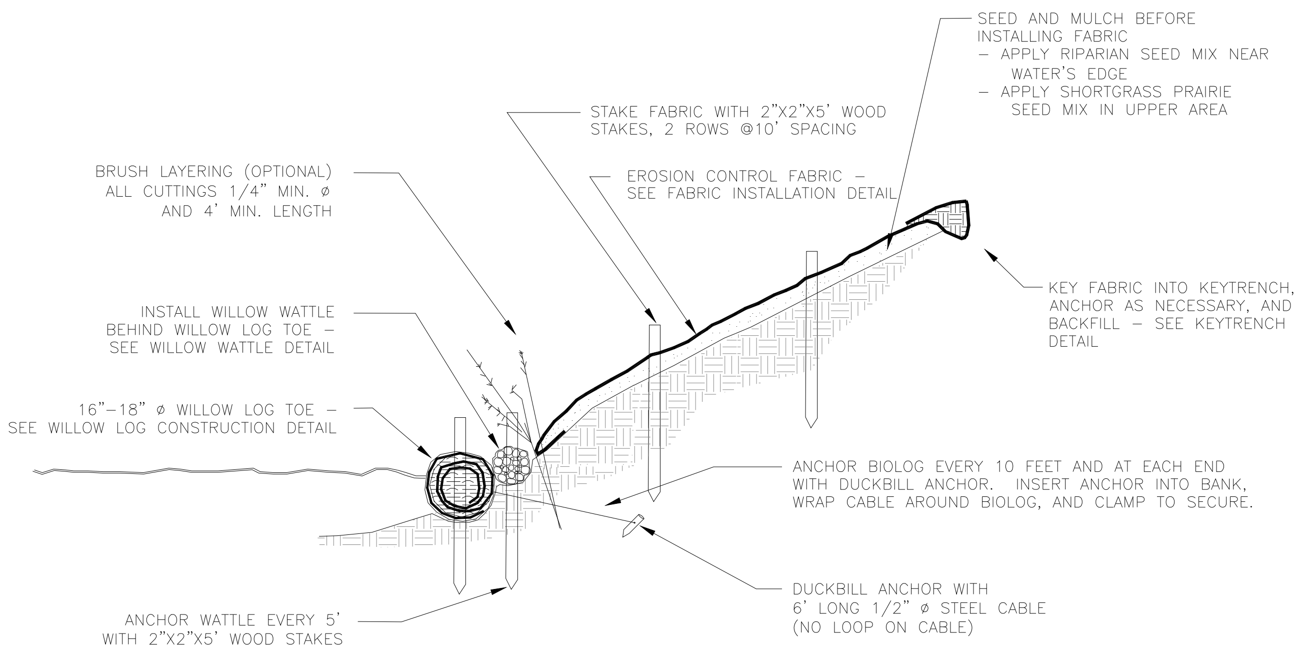

Figure 5—Willow Log Toe with Willow Wattle

Sand Creek Channel Improvements at Bluff Lake Park
Denver, Colorado

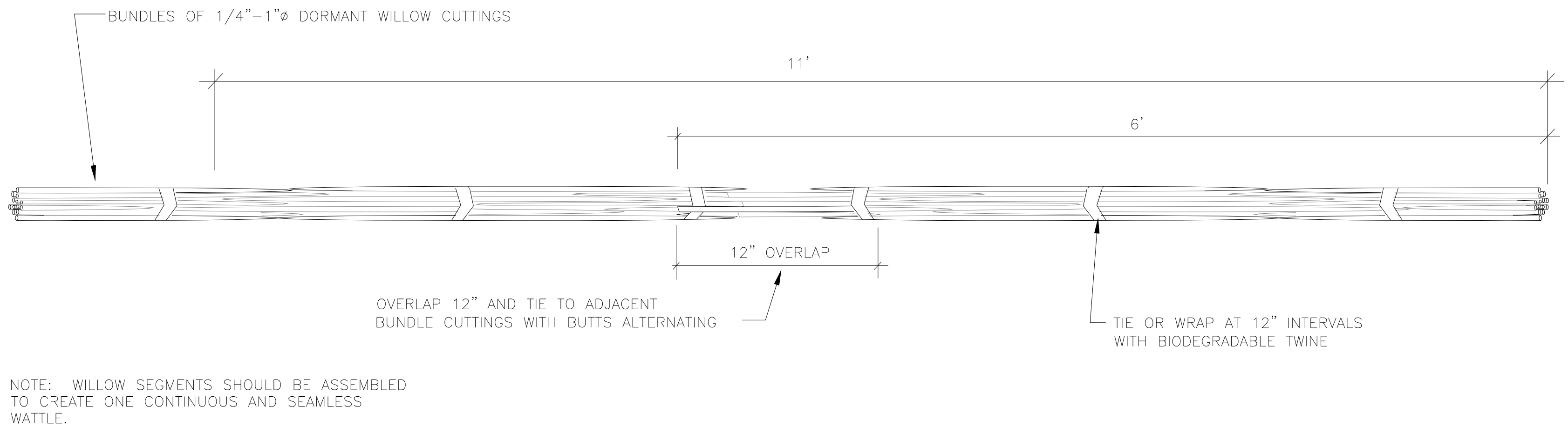

Figure 6—Willow Wattle

Sand Creek Channel Improvements at Bluff Lake Park
Denver, Colorado

NOTES:
1. USE 12' WIDE EROSION CONTROL FABRIC
2. CUT 20' LENGTH OF FABRIC
3. ROLL LOG IN THE DIRECTION OF FABRIC WIDTH
* OR SUBSTITUTE 18"ø BIOLOGS

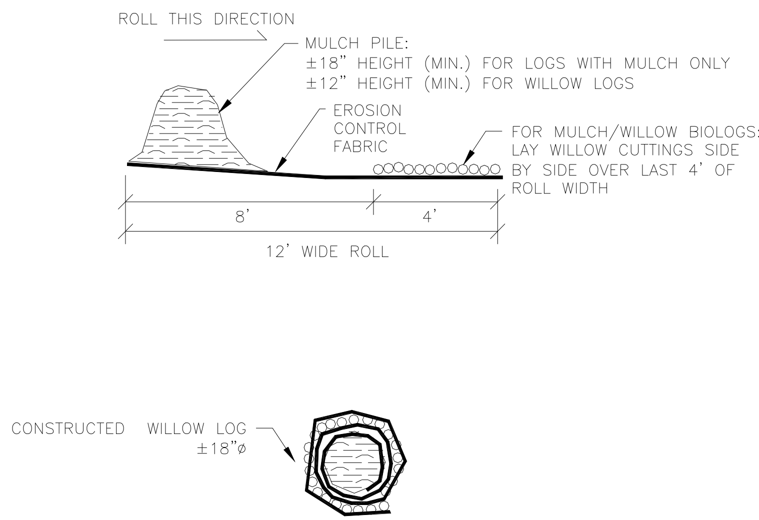

Figure 7—Willow Log Construction

DESIGN EXAMPLES—SECTION 6

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6.0 CASE STUDY— GOLDSMITH GULCH

Goldsmith Gulch flows through Bible Park in southeast Denver. Bible Park is located between Monaco Parkway and Quebec Street and south of Yale Avenue. The Highline Canal flows around the perimeter of the park. Bible Park has active recreational areas that include ball fields, tennis courts and playgrounds. The park has a significant trail system that connects the active recreational components of the park and allows for enjoyment of the passive areas. The channel in Bible Park had become deeply incised and very linear. The average slope of the existing channel in Bible Park prior to the drainageway maintenance project was approximately 0.5 percent. The channel bottom elevation at the upstream and downstream ends of the park was controlled but the channel in the park had become very incised with sloughing banks. The incised channel and unstable banks greatly reduced the potential for enjoyment of the channel by park users and presented a definite safety issue. The Urban Drainage and Flood Control District, the Denver Wastewater Management Division and the Denver Parks Department undertook a rehabilitative maintenance project for the Goldsmith Gulch channel in Bible Park in 1996. Sellards & Grigg, Inc. and Wenk Associates performed the design.

Dames & Moore provided environmental consultation. The rehabilitative maintenance of the Goldsmith Gulch channel was undertaken with the primary goals of stabilizing the channel in a manner that was environmentally sensitive and that enhanced the wildlife habitat in the park. A secondary goal was to enhance the passive and active enjoyment of the park. Bioengineering techniques were combined with traditional methods of channel stabilization to accomplish the project goals.

6.1 Design

Inherent in all of the alternatives that were considered for channel stabilization was the concept of reducing the channel slope by the construction of drop structures. As a result of the extensive public involvement process that was undertaken during the design phase of the project, it was decided that there would be two drop structures that would divide Bible Park into three distinctly different channel reaches.

6.1.1 Channel Reaches

The lower reach of the channel was constructed with boulder walls to protect a large area of trees that was adjacent to the channel. An island was created around the area of trees. The development of the island resulted in a new channel adjacent to the trail that connects to the below-grade crossing under Yale Avenue. The island in the lower channel reach provides for a more interesting and aesthetic experience for the park trail user. The reduced velocity and constant inundation in the widened low-flow channel upstream and downstream of the island has resulted in flourishing wetland vegetation.

The middle channel reach was the most deeply incised and linear. The middle channel reach is located in the passive area of the park. The width of the park in the middle reach was sufficient to allow for the redevelopment of a new meandering channel. The new meandering channel was designed with sweeping oxbows that would be frequently flooded to sustain wetland vegetation. Sand blankets were installed in the low-lying overbank areas on the inside of bends in an attempt to provide a direct hydraulic connection to the wetland vegetation during low-flow conditions. The low-flow channel banks for the middle reach of the channel were protected with soil riprap that was vegetated with wetland species. Over the course of time, the vegetation in the low-flow channel bank has obscured the soil riprap. A foot path constructed with crusher fines follows close to the constructed meandering channel in the middle portion of the park. The footpath allows for passive enjoyment of the tranquil meandering channel and the enhanced wildlife habitat.

The upper channel reach was not as severely incised as the middle channel reach. There were a significant number of trees in close proximity to the channel. For the most part, the existing channel alignment was maintained in the upper portion of the park. The moderately degraded channel in the upper channel reach was stabilized by the design of the upper drop with a crest elevation somewhat above the existing channel bottom. The channel bottom in the upper third of the park was allowed to fill in by natural sedimentation processes in the pool area behind the drop. There is a very large five-cell box culvert at the upstream end of Bible Park. The channel immediately downstream of the five-cell box culvert was protected from erosion using bioengineering techniques. Soil riprap was planted with wetland species that have become very prolific in this area.

6.1.2 Drop Structures

The lower drop structure has significant drop and provides an interesting overlook for park users by combining the drop with a pedestrian crossing. There are significant structural elements to the lower drop structure. A concrete cutoff wall was integrated with the upstream edge of the trail crossing and the intermediate crests of the rock walls on the downstream face of the drop structure were stabilized with concrete walls that are hidden from view. The upper drop was constructed of boulders and was intended to provide a separation between the meandering channel portion of the park and the existing channel portion of the park. The drop structures have reduced the longitudinal slope of the channel to 0.21 percent.

6.2 Criteria

For the most part, the District criteria were followed for this project. The channel slope has been reduced to approximately 0.2 percent through the construction of grade control structures. The low-flow channel has been constructed for 100 cfs, which is approximately 3 percent of the 100-year flow of 3570 cfs. The lower drop structure is unique. There are three intermediate pools between the upper channel and the lower channel. The areas lateral to the low-flow throat of the lower drop structure have been armored with loose boulders. Subsequent to the completion of construction, the crevices between the loose boulders have become vegetated. The intermediate pools on the downstream side of the pedestrian crossing provide for an interesting sound effect that often captures the attention of the trail user. There is an interesting view of the island and the wetland area upstream of the island from the pedestrian crossing of the lower drop structure.

6.3 Construction

L&M Enterprises was the General Contractor for the rehabilitative maintenance in Bible Park. The project included substantial earthwork, structural concrete, placement of boulders and soil riprap, trail construction, and the establishment of vegetation. Getting the wetland species established was probably the biggest challenge of the project. The significant flood events that were experienced during construction and immediately after construction made it difficult to establish the wetland vegetation.

Replanting the wetland areas was necessary in the first growing season after the completion of the project.

6.4 Success

The rehabilitative maintenance project in Bible Park has been well received by the public and has attained the goals set by the sponsors and the design team. The experience of the design team and the project sponsors demonstrated that patience and perseverance are required when bioengineered solutions are employed for erosion protection in a drainageway that is subject to frequent flooding. It took approximately two years for the wetlands to become well established and provide for their intended erosion protection. Ultimately, the approach of combining armoring with bioengineered solutions resulted in a successful project. The project has stabilized the Goldsmith Gulch channel and has provided for enhanced enjoyment by the people who use the park.

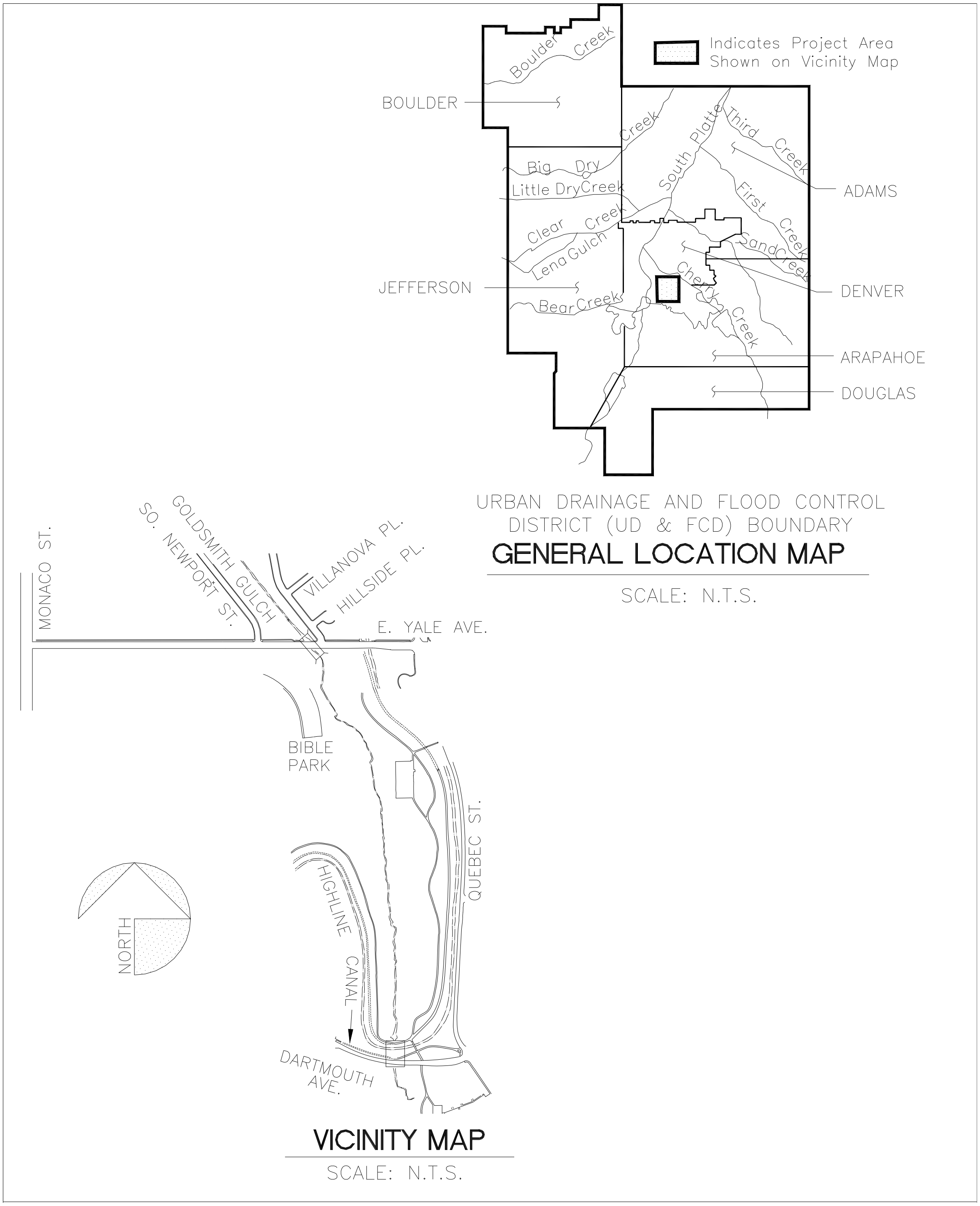

Figure 1—Location and Vicinity Map

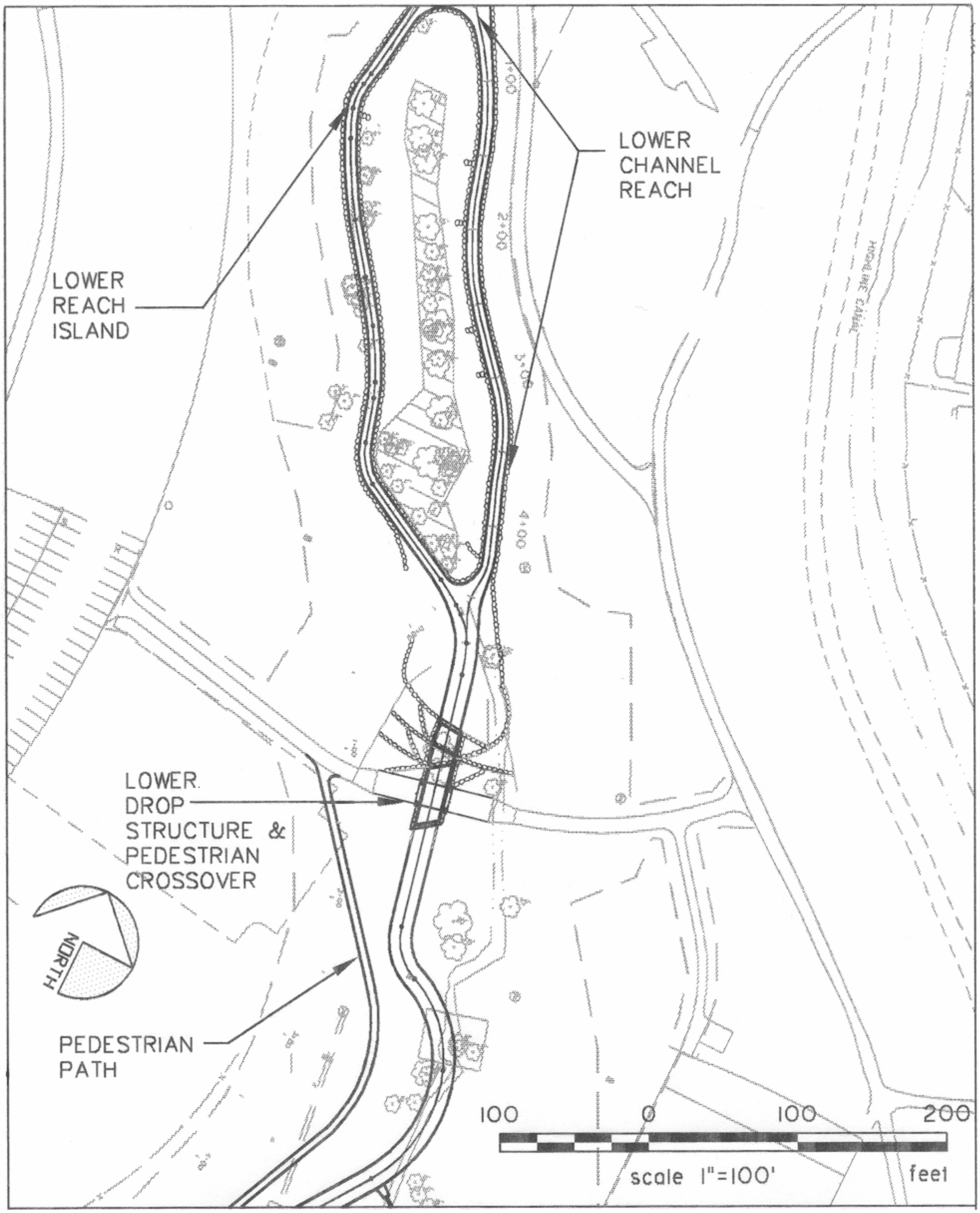

Figure 2—Lower Channel Reach

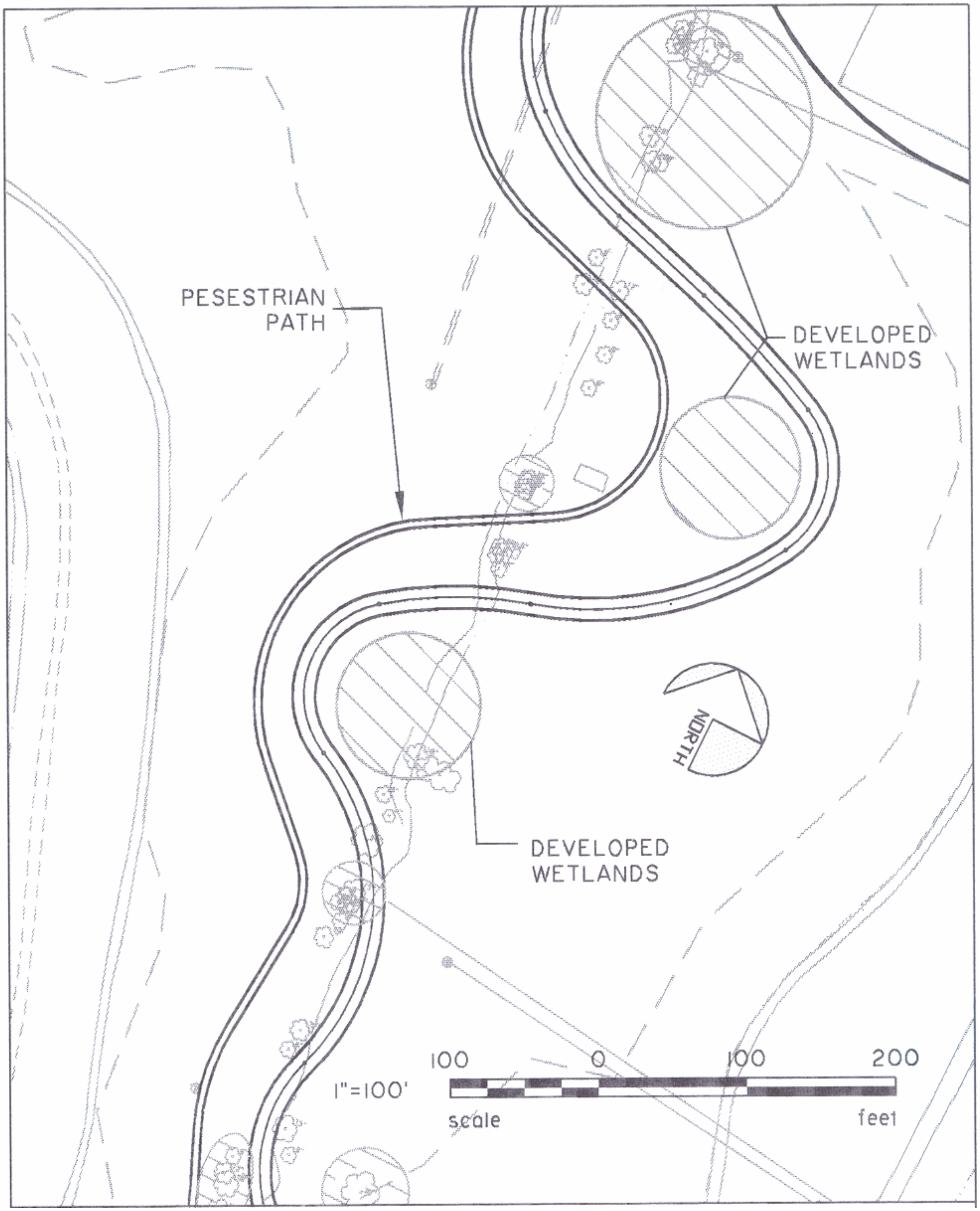

Figure 3—Middle Channel Reach

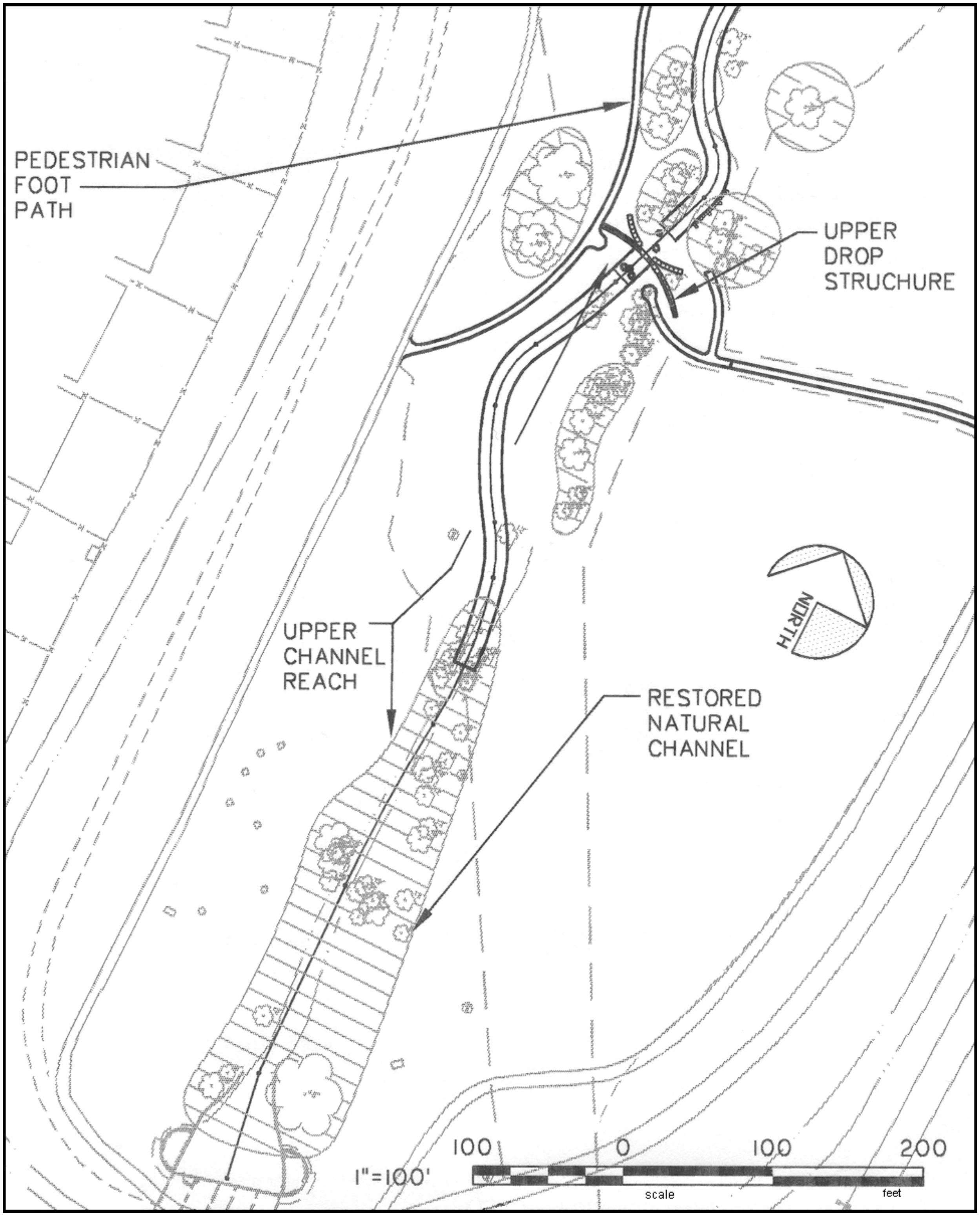

Figure 4—Upper Channel Reach

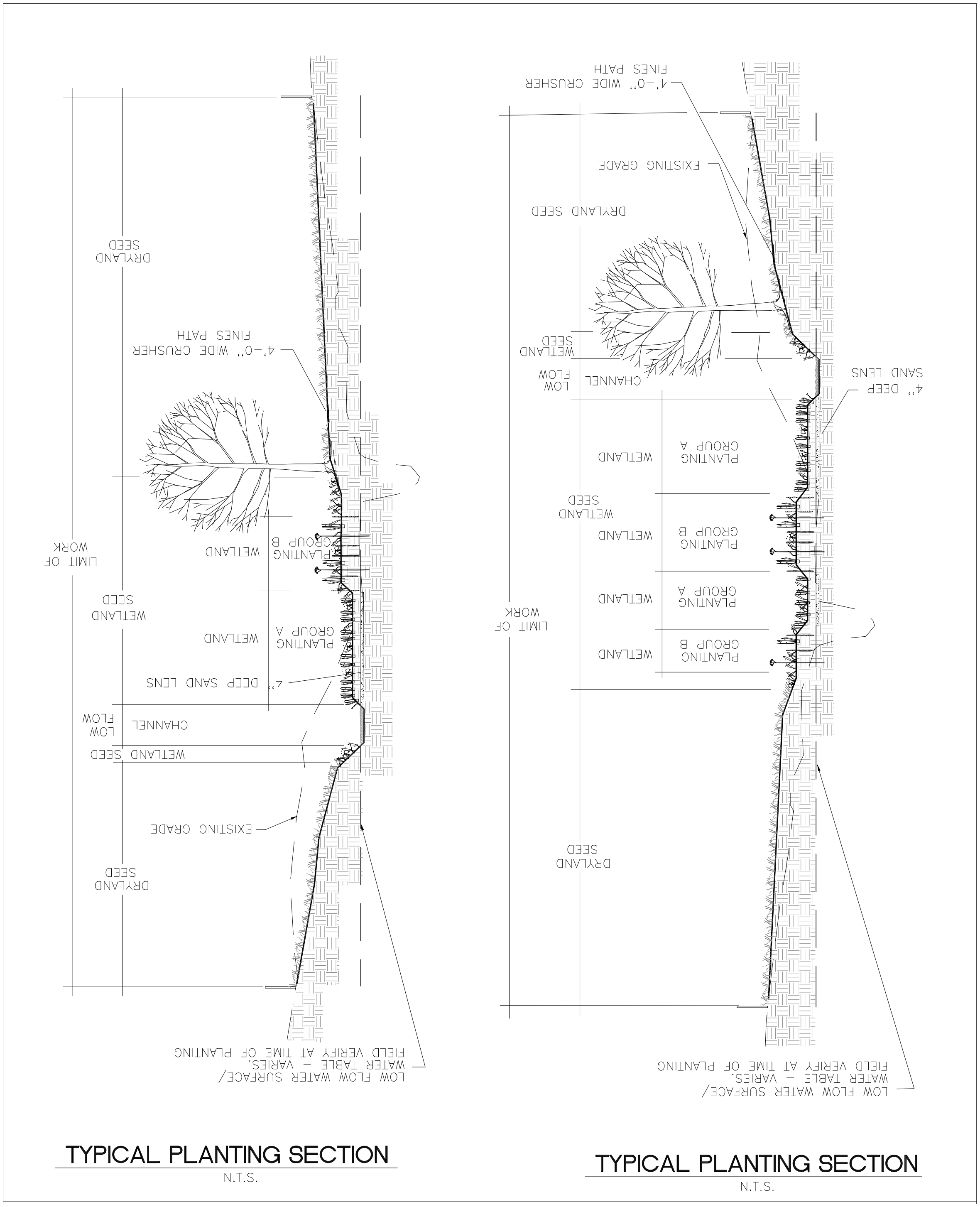

Figure 5—Typical Section for Areas of Wetland Development

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7.0 CASE STUDY—GREENWOOD GULCH

Greenwood Gulch, a tributary of Little Dry Creek, flows in a northwesterly direction through Greenwood Village (Figures 1 and 2). The headwater area of Greenwood Gulch is dominated by high density office park developments, the central area by single family residential development and the lower area by a regional park, rural residential lots and a residential golf course development. The Highline Canal transverses the basin near the center of the watershed and intercepts the entire base flow of Greenwood Gulch. The watershed is virtually built-out with little potential for additional infill development.

Table 1—Greenwood Gulch Hydrology

Condition	Flow at Holly Street
Base Flow	
winter	2 cfs
summer	5 cfs
2-year Storm	830 cfs
10-year Storm	1200 cfs
50-year Storm	1620 cfs
100-year Storm	1750 cfs

The urbanization of the watershed has changed Greenwood Gulch from an intermittent stream to a perennial stream with an average wintertime base flow of approximately 2 cfs and an average summertime base flow of approximately 5 cfs. Stormwater flows have also increased substantially over predevelopment conditions. The new flow regime has caused significant erosion of the stream channel in the central parts of the watershed.

Photo 1. Erosion of Residential Properties

The increased erosion, in combination with some residential encroachment of the natural floodplain, threatened some private properties between Orchard Avenue and Holly Street (Photo 1). Informal attempts at erosion control by the property owners along Greenwood Gulch proved to be ineffective. The eroded materials tended to be deposited downstream in the vicinity of the Holly Street bridge. The aggradation of the channel and over bank areas at the Holly Street bridge reduced the flood conveyance capacity of the bridge and increased the flood risks for neighboring properties.

The new flow regime initially caused the growth of wetlands in the Greenwood Gulch floodplain between Holly Street and the Highline Canal. A new residential development in this area in the 1990s perceived the wetlands as a valuable asset, avoided encroachment in the floodplain, included wetland symbols in its logo and adopted “The Preserve” as its name. Homes were constructed and occupied alongside the riparian corridor of the 100-year floodplain beginning in the early 1990s. The Greenwood Gulch corridor also contained a heavily used regional trail connecting to the Highline Canal Trail and Greenwood Village’s Perry Preserve Regional Park.

The changing flow and channel erosion regimes, however, were dynamic and eventually the channel became incised in some places to a depth of approximately 10 feet (Photo 2). This further changed the hydrologic regime by lowering the water table in the floodplain, drying up the riparian wetlands and allowing for the encroachment of noxious weeds. The public voiced significant concern with the erosion damage to the trail and the loss of the wetland habitat.

Photo 2. Loss of Wetland Habitat

7.1 Design

The District, in cooperation with Greenwood Village, initially identified four options in 1996 for controlling erosion in the 1,400-foot reach of Greenwood Gulch from Orchard Avenue to approximately 700 feet upstream of the Holly Street bridge. The local community requested an expansion of the study to control erosion for the entire 2,100-foot reach between Orchard Avenue and Holly Street, restore the lost flood conveyance capacity of the Holly Street bridge, and control the ongoing erosion and loss of wetland habitat in the 2,900-foot reach between Holly Street and the Highline Canal.

Pre-design studies evaluated excavation of aggraded materials to restore the conveyance capacity of the Holly Street Bridge, relocation of the trail beneath the bridge alongside the improved stream channel, placement of six additional low-head drop structures in the floodplain downstream of the Holly Street bridge and placement of one moderate head drop structure (8 feet) in the channel immediately upstream of the Highline Canal. The low-head drop structures downstream of the Holly Street bridge would be designed to span the entire 100-year floodplain (60 to 100 feet wide) to eliminate channel erosion and spread the base flows to restore the wetland hydrology throughout the width of the floodplain. Hydraulic studies were also completed using HEC-RAS computer modeling methods to ensure that the flattened channel grades between drop structures would not increase flood elevations during the 100-year storm event.

The District, after consideration of all the alternatives, decided to participate in the costs for the final design, construction, and maintenance of the Greenwood Village proposal. The District retained the design team of Sellards and Grigg, Inc., Water & Waste Engineering, Inc., and Design Concepts, Inc. to prepare the final design and construction documents.

7.2 Criteria

The design followed the District criteria that were applicable to the aesthetic, recreation and wetland restoration goals of the community.

The final design for the reach between Orchard Avenue and the Holly Street bridge included one 4-foot large boulder drop structure immediately downstream of the Orchard Road bridge and six large boulder 1.5-foot drop structures (Photo 3 and Figures 3 and 4). The inclusion of these drop structures flattened the channel bottom slope to an average of 0.30%. The channel side slopes were regraded to slopes ranging from 2:1 to 3.7:1 and were protected with Type M riprap soil.

Photo 3 Large Boulder Drop Structure

The large boulders (5 to 6 feet diameter) presented the opportunity to minimize the depth of grout required to stabilize the boulders. This improved the design aesthetics without any apparent increase in the costs of construction. The locations and alignments for the drop structures were chosen carefully to encourage the formation of some sinuosity in the alignment of the channel. The placement of the boulders during construction was also carefully managed to bring a natural appearance to the construction. The side slopes were planted with a mixture of native grasses, shrubs and trees to control side slope erosion and riparian wildlife habitat (Figure 5).

Photo 4 Two-Tier Large Boulder Drop Structure

Photo 5 Holly Street Bridge and Riparian Trail

One two-tier large boulder 4.0-foot drop structure was added upstream of the Holly Street bridge to lower the channel bottom to restore the conveyance capacity of the Holly Street bridge (Photo 4 and Figures 3 and 6).

The bridge abutments and an 18-inch gas main crossing the stream channel complicated the relocation of the trail below the Holly Street bridge (Figure 6). The bridge abutments required structural shoring with a 12-inch-thick by 5.2-foot-high concrete wall. The trail was separated from the stream channel by means of a 6-

foot-high curved wall (Photo 5). In one location, the top of the trail was approximately 2 feet below the channel bottom. A sump pump dewaters the foundation for the trail. The trail is protected with a Type H riprap slope against the trail wall with the opposite protected by Type M riprap soil.

The design for the restoration of the wetland habitat downstream of Holly Street was based on analyses of 1948 to 1995 aerial photographs to document the changing wetland habitat, soil borings, four groundwater monitoring wells, and detailed vegetation surveys. The goal of the design was the restoration and maintenance of approximately 8 acres of wetland habitat between Holly Street and the Highline Canal.

The construction included the excavation of approximately 9,000 cubic yards of sediment deposits (Photo 6). The floodplain was then graded to maintain a "channel" slope of 0.38% to 0.40% between three drop structures constructed with 36-inch minimum dimension boulders (Figure 7). The boulders were carefully placed with strict tolerances (+/-2 inches) for top edge elevations to create a wide (80 to 170 feet) flat-bottomed channel (Figure 8). The drop structures were installed in a curvilinear configuration to minimize their potential visual impact.

Photo 6 Excavation of Accumulated Sediment

This wide and level configuration for the drop structures encouraged surface flows to spread throughout most of the width of the floodplain shortly following construction (Photos 7 and 8). The flat channel slopes control channel erosion and the wide flow path encourages infiltration of base flows and stormwater. In addition, the cutoff walls at each drop structure impede the longitudinal flow of groundwater, causing it to rise closer to the surface. These higher groundwater elevations, combined with the shallow surface

Photo 7 Upstream View toward Holly Street with Lower Drop No.1 in Foreground

Photo 8 Downstream View from Holly Street toward Lower Drops No. 2 and No. 3

flows, combine to create wetland conditions throughout much of the floodplain. The trail was moved to the edge of the floodplain into an upland area (above the 10-year flood elevation wherever possible). This made the trail more usable and reduced the risk of further erosion damage.

Transplanted root pads (minimum 6 square feet by 6 inches deep) were placed in the channel bottom to encourage rapid restoration of the wetland areas. Upland shrubs and trees were planted along the edge of the channel bottom to provide shading and a variety of wildlife habitat (Figure 9). The wetland vegetation spread very quickly, and within the first growing season, a healthy community of wetland plants was established in the designated areas (Photos 9 and 10).

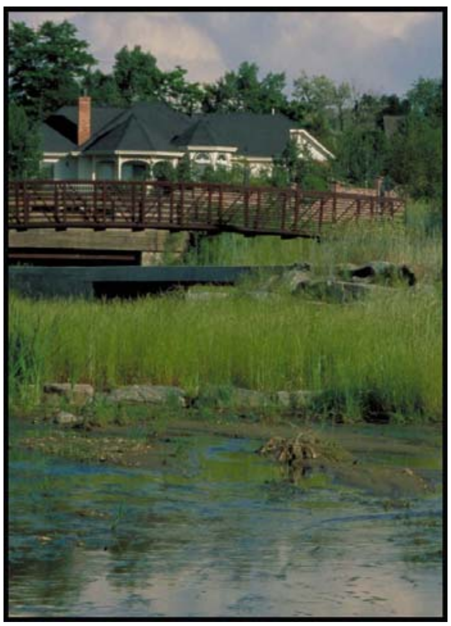

Photo 9 View toward Holly Street and Wetland Area and Lower Drop No. 2

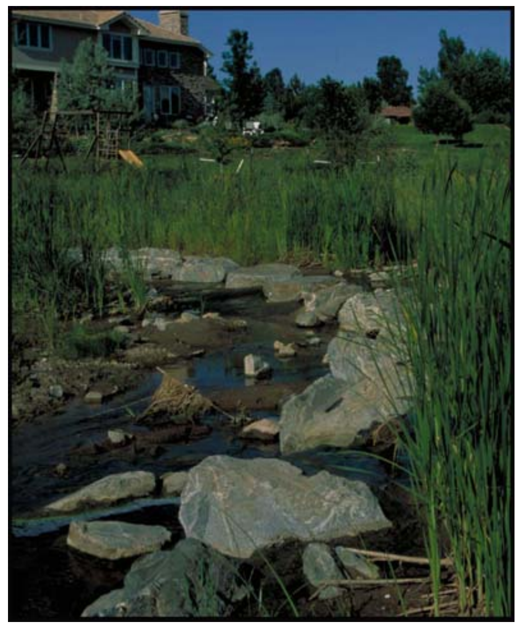

Photo 10 Base Flow over Lower Drop

The design of the lowermost drop structures, immediately upstream of the Highline Canal, presented different challenges. Greenwood Gulch had split into two distinct flow channels. The slopes of the channels were less than 0.5% and a healthy wetland habitat dominated the last 1,100 feet of the Greenwood Gulch floodplain before it discharged into the Highline Canal. Two 8-foot-deep erosion channels, however, had worked their way about 150 feet back from the Highline Canal. If left alone, these erosion channels would likely continue to work their way back upstream and ultimately threaten the nearby wetland areas.

Two large boulder drop structures were constructed approximately 150 feet upstream of the Highline Canal on the two channels (Figure 10 and Photo 11). The same large boulder design concepts used upstream of Holly Street were applied to these lowermost 4-foot-high two-tiered drop structures. Both included bridges for pedestrian trail crossings over the split Greenwood Gulch channels.

7.3 Construction

The District awarded the construction contract to Randall & Blake, Inc. in the spring of 1998. The District administered the contract via an intergovernmental agreement with Greenwood Village. The contract was awarded in two phases to accommodate right of way negotiations with homeowners adjacent to the upstream portion of the project. Some delays were encountered during construction due to thunderstorm activity and unforeseen conditions at the Holly Street bridge. The construction sequence was adjusted in the fall of 1998 to accommodate the critical fall planting of vegetation.

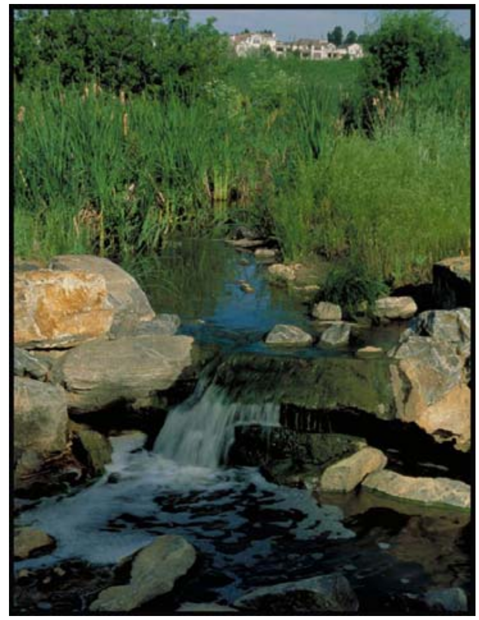

Photo 11 Upstream View of Drop Structure No. 2 from Pedestrian Crossing

7.4 Success

The Greenwood Gulch Channel Improvement Project is a success. The revegetation has been successful and the erosion has been controlled. The damage to private properties from Orchard Road to Holly Street has been stopped and approximately 8 acres of wetland habitat have been restored from Holly Street to the Highline Canal. The trail from Orchard Road to the Highline Canal is one of the most heavily used trails in the Greenwood Village trail system. The large boulder drop structures are visual amenities and the riffle/pool flow patterns in the narrow channel upstream of Holly have improved the wildlife habitat of the riparian corridor. The wetlands below Holly Street also improve the urban wildlife habitat and are an amenity for enjoyment by the users of the trail. The entire project has enhanced the property values for the area and has received ongoing support from the local community.

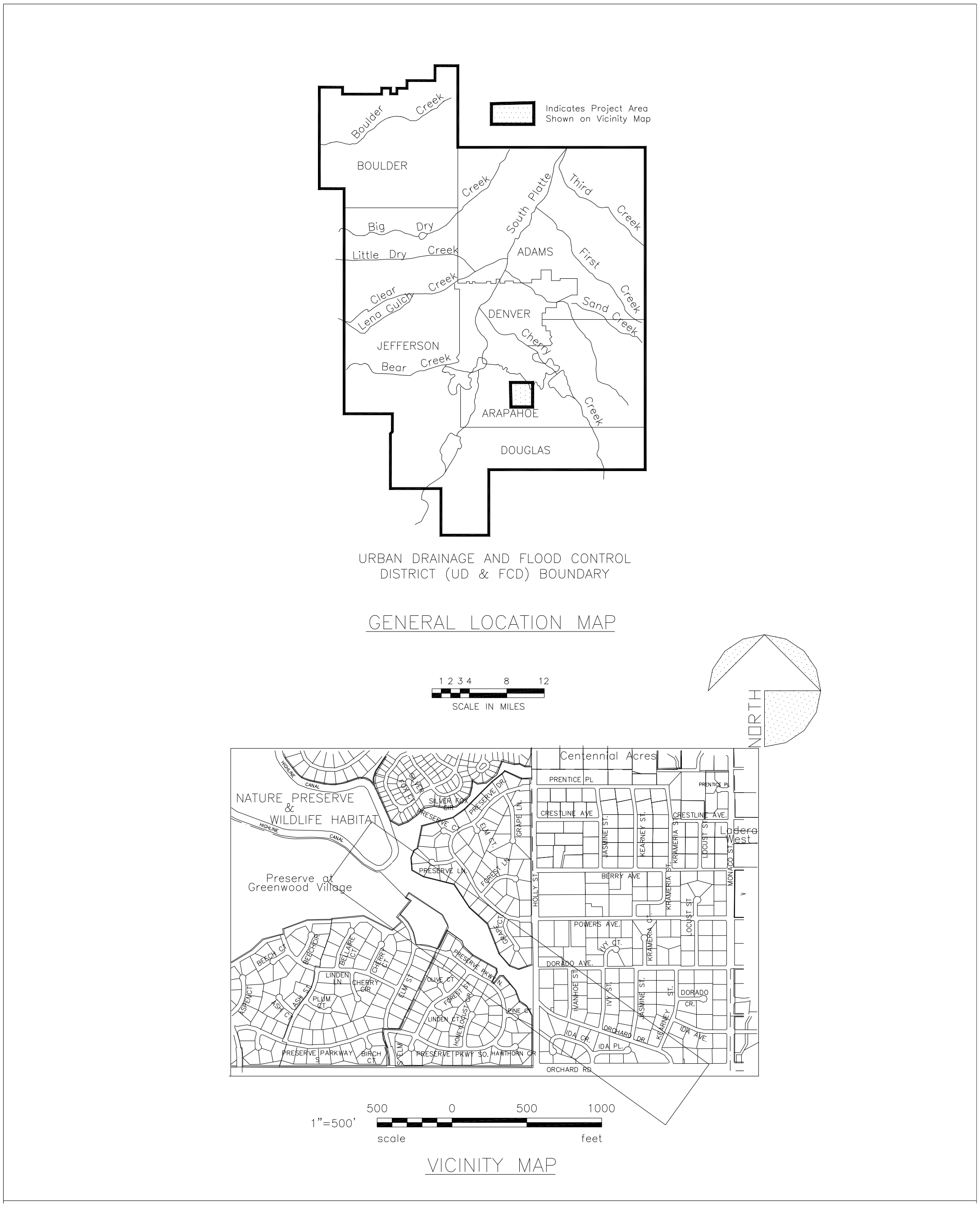

Figure 1—Location and Vicinity Maps

Figure 2—Urbanization of Greenwood Gulch

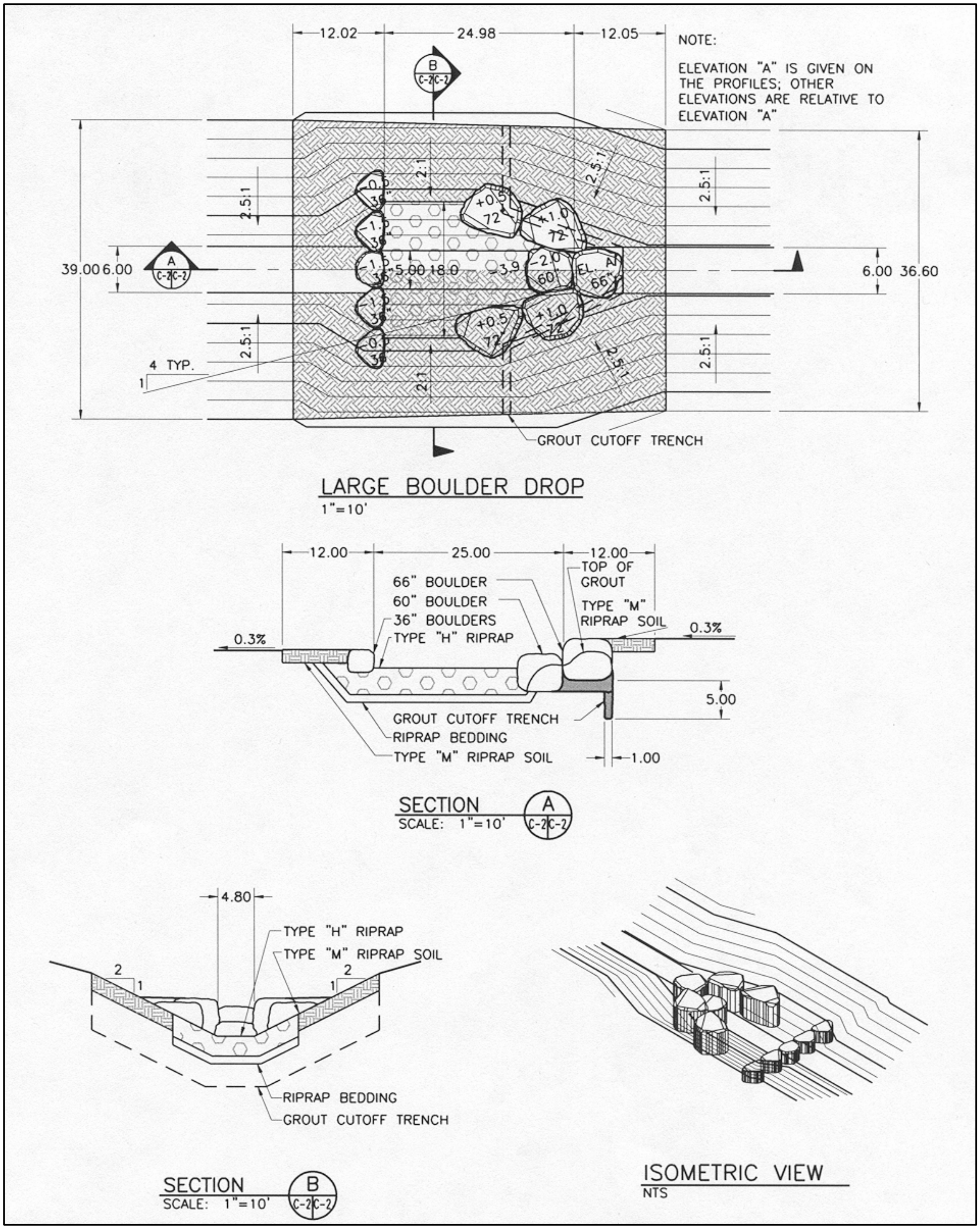

Figure 3—Large Boulder Drop Structure

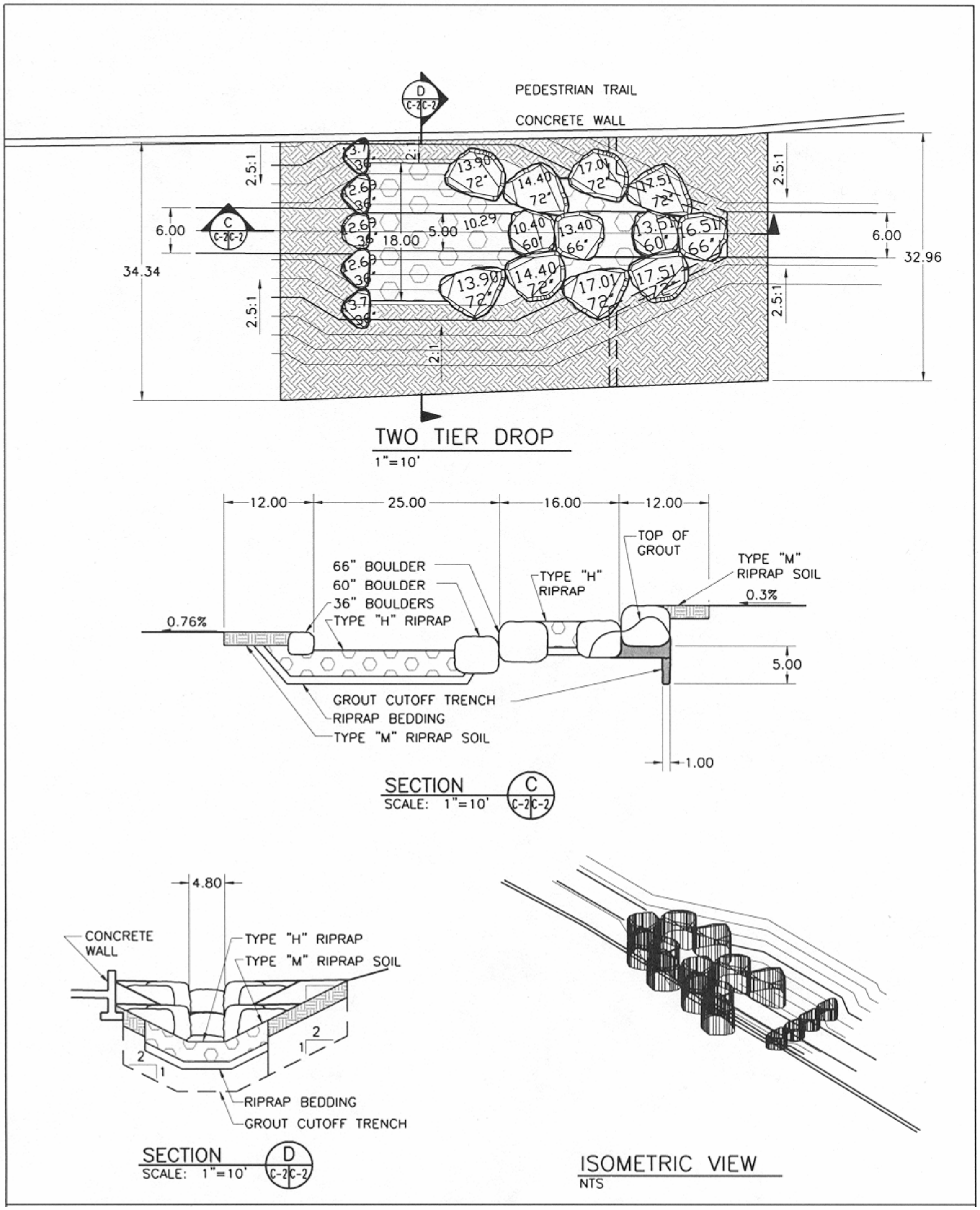

Figure 4—Large Boulder Drop Structure

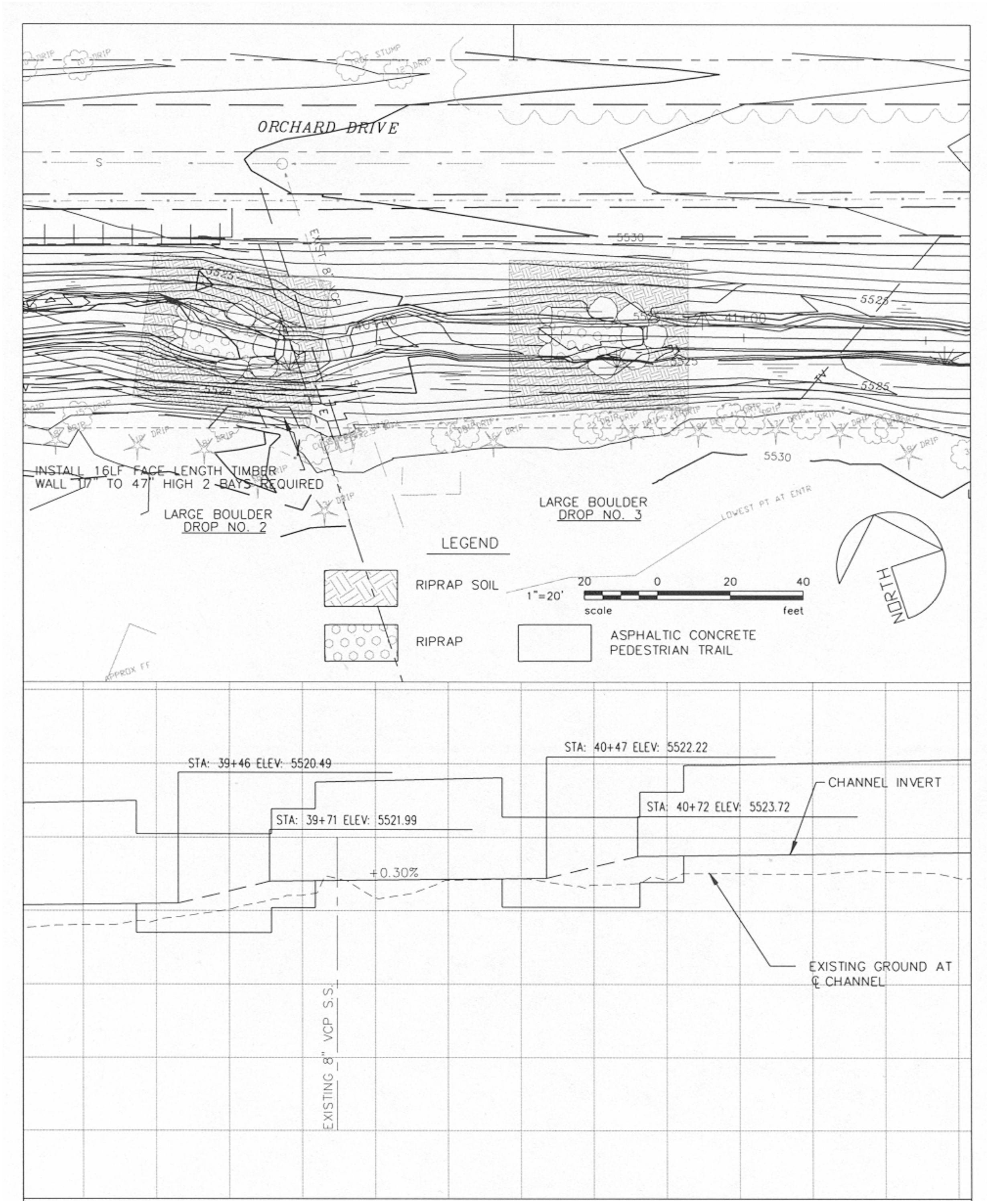

Figure 5—Plan and Profile Upstream of Holly Street

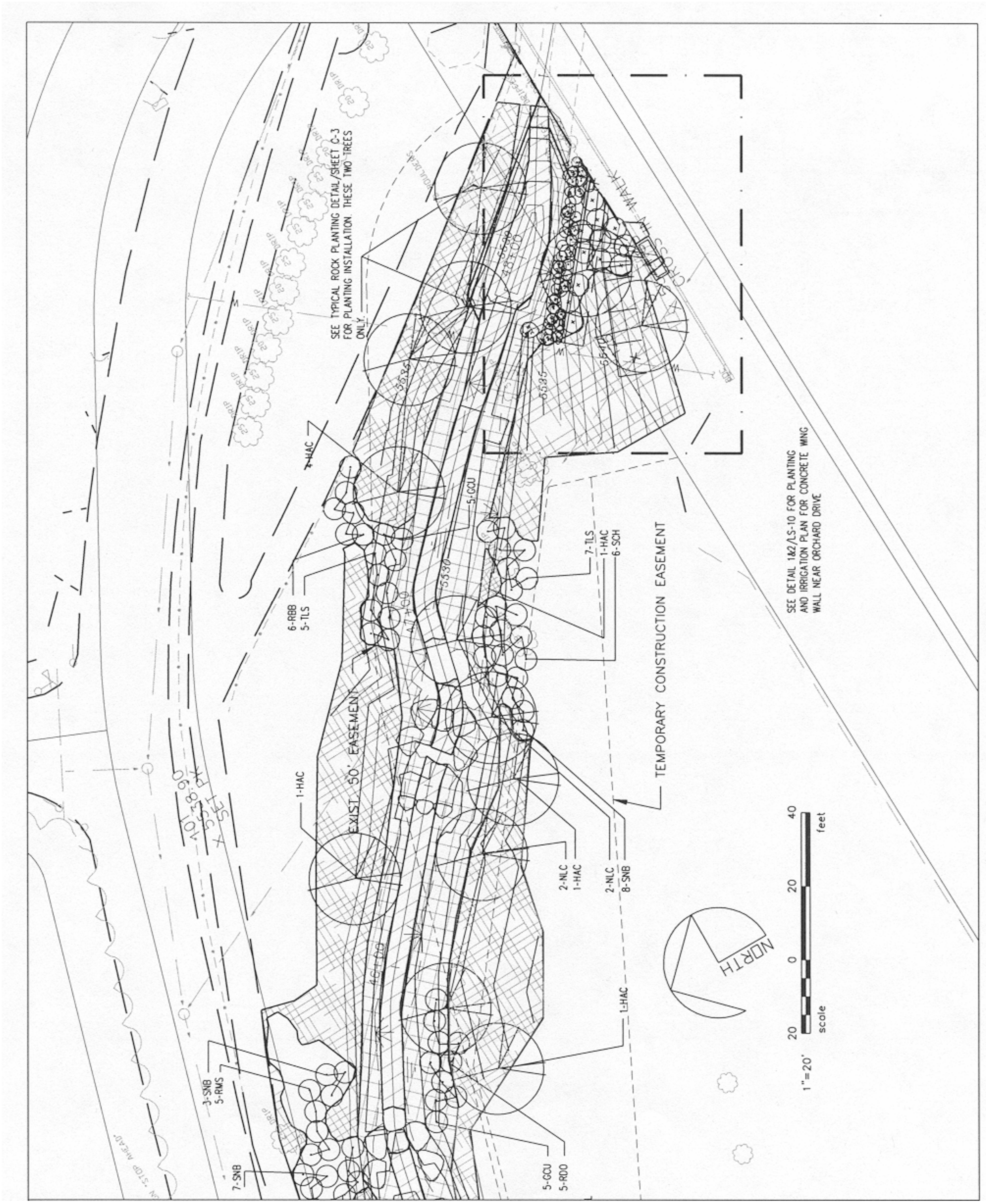

Figure 6—Landscape Plan Upstream of Holly Street

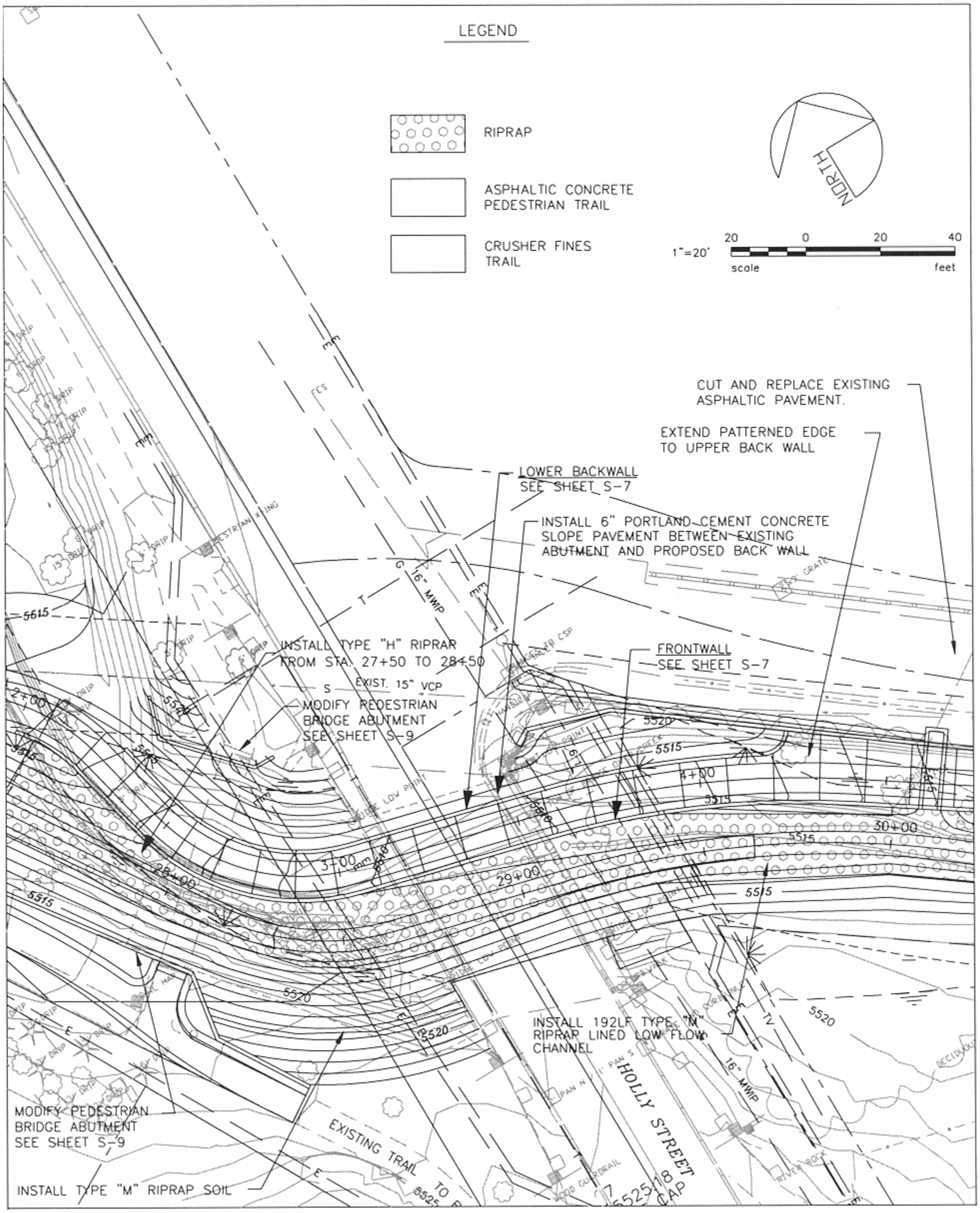

Figure 7—Holly Street Bridge

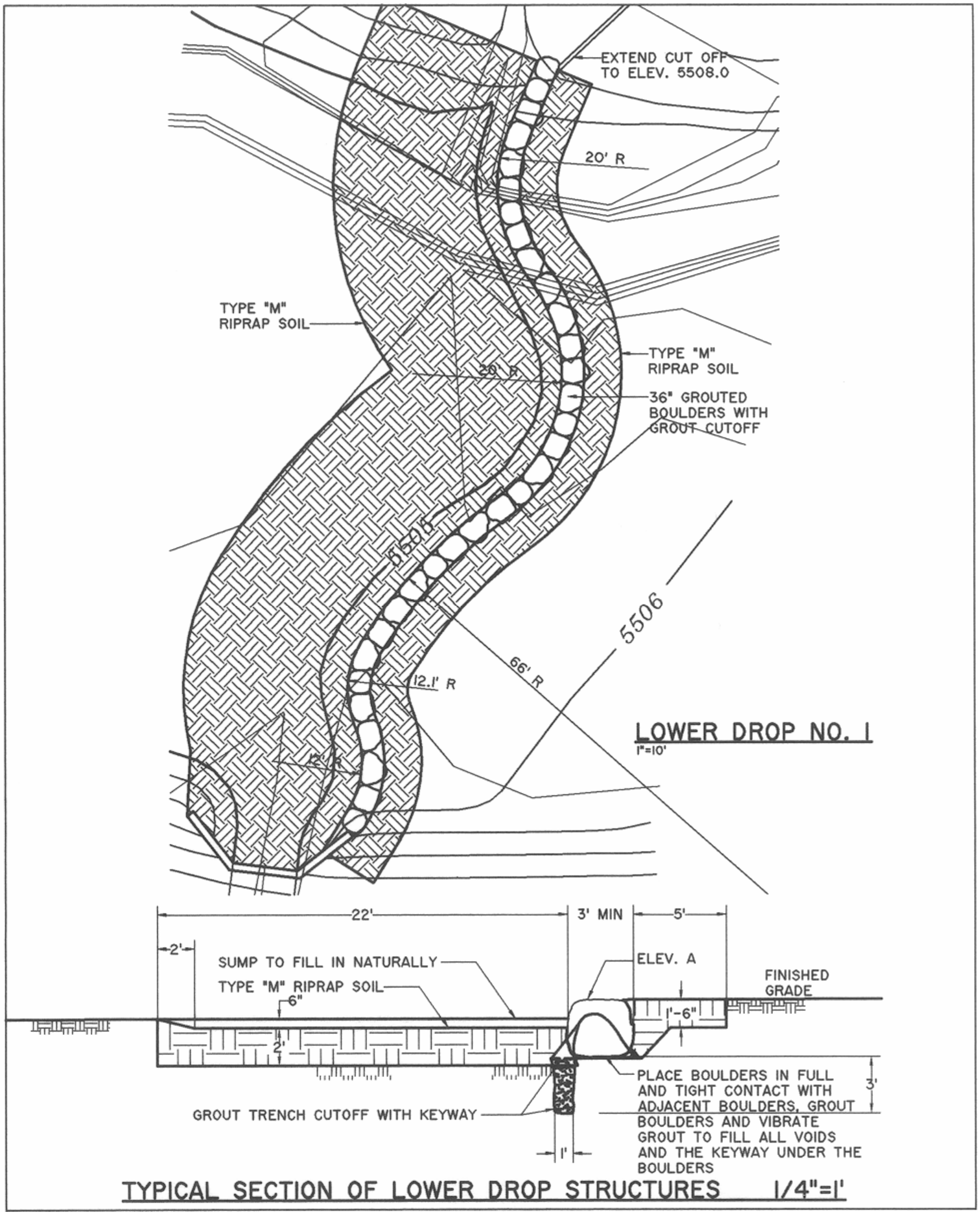

Figure 8—Lower Drop Structure Downstream of Holly Street

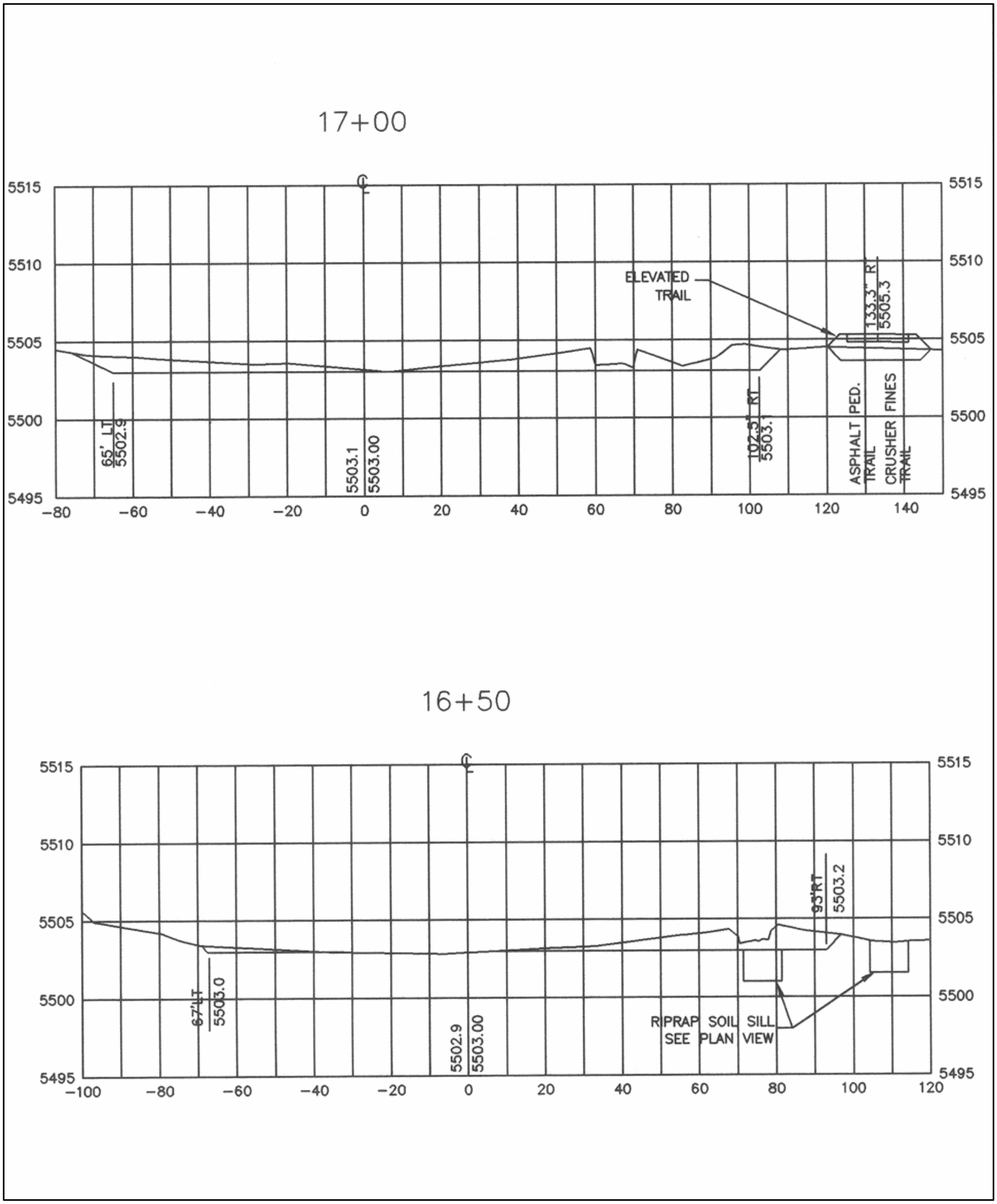

Figure 9—Downstream of Holly Street Channel Cross Sections

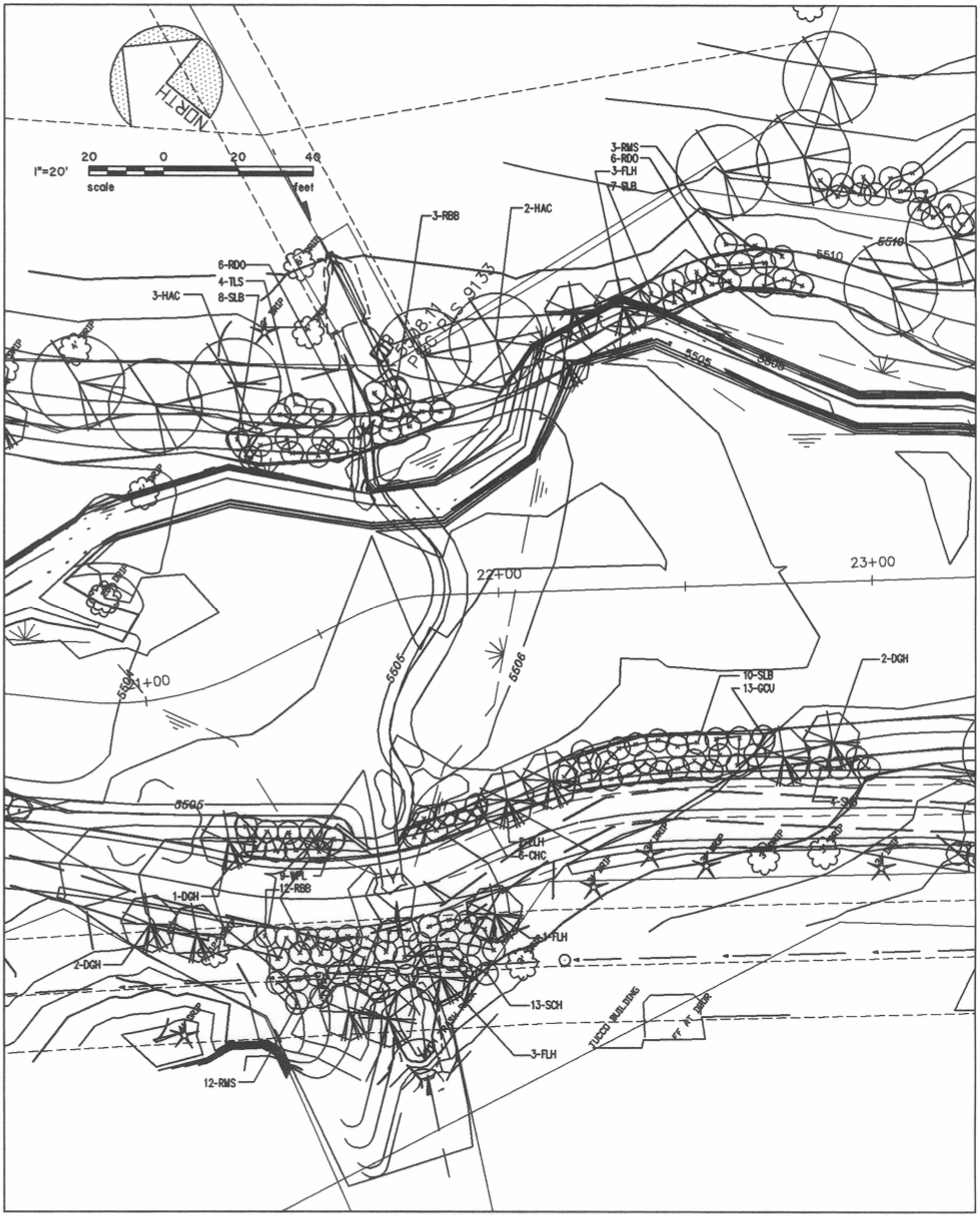

Figure 10—Landscape Plan Downstream of Holly Street

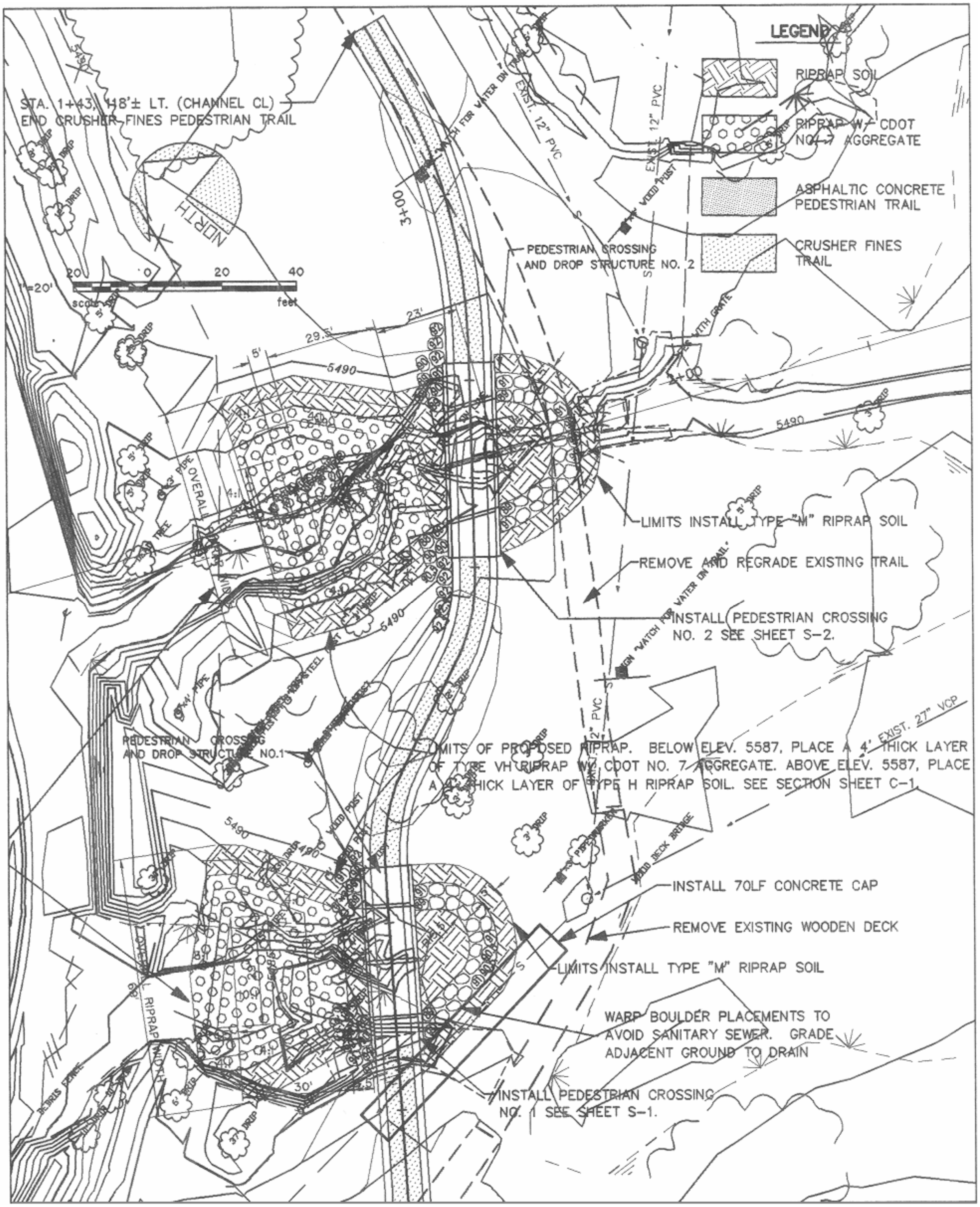

Figure 11—Large Boulder Drop Above Highline Canal

DESIGN EXAMPLES—SECTION 8

CONTENTS

Figures for Section 8

8.0 CASE STUDY—LENA GULCH DROP STRUCTURE

8.1 Background

Lena Gulch is a major drainageway that flows through Jefferson County in Colorado. The drainage basin area is approximately 13.9 square miles and is almost completely developed. At one point, Lena Gulch flows into and out of Maple Grove Reservoir, which serves as a water storage facility operated by the Consolidated Mutual Water Company. The water level of the reservoir is controlled by an inflatable fabridam. Downstream of the reservoir, Lena Gulch flows from a flat, wide channel into a steep, narrow dumped-concrete and sheet-pile drop structure, which was severely undercut and in danger of complete failure. Downstream of the drop structure, scour and bank erosion were endangering a home and a pedestrian bridge over Lena Gulch. Because of these safety and drainage concerns, the City of Wheat Ridge requested assistance from the District to replace this structure.

Photo 1.

The existing failing drop structure was situated on a jurisdictional boundary that required the involvement of three different local government sponsors in addition to the District, the City of Lakewood, the City of Wheat Ridge, and the Consolidated Mutual Water Company. The lower end of the drop structure and channel were situated on private property, which required the close involvement of the affected homeowner. The District needed both permanent and temporary construction easements to construct the project, so addressing their needs was critical. The project team interviewed several consultants and chose Taggart Engineering Associates to design the drop structure and channel improvements.

8.2 Design Considerations

Since there were five different participants on the project team, each with their own design considerations and concerns, the initial meetings were critical to the success of the design. Consolidated Mutual Water's concerns were the efficient transportation of water through their property and the removal of some existing ponding just upstream of the failing drop structure. The City of Lakewood, which is responsible for the trail in the area and bridge over the drop structure, was concerned about trail access during construction and placement of the bridge on a new alignment. The City of Wheat Ridge, which represents the homeowners downstream of the drop, was primarily concerned with reducing the flood hazard to their constituents.

The District had two primary issues that needed to be addressed with the new drop structure. First was the ability of the new drop structure to funnel the 100- year flood from a wide floodplain into a deep, narrow flow. The second was the possibility of failure of the inflatable fabridam upstream at Maple Grove Reservoir. If the fabridam stayed intact during the 100-year flood, the design flow at the drop structure was approximately 1725 cfs. If the fabridam failed in the flood event, the flow downstream increased to approximately 3800 cfs. The project team believed that it was imperative that the new drop structure be designed for the 1725 cfs flow, but be able to handle the 3800 cfs in the event of fabridam failure.

In addition to the local government concerns were the concerns of the homeowners immediately downstream of the failing drop structure. They would have to grant a significant permanent easement in their backyard where the pool of the new drop structure was to be constructed. Their property had been designated as a Backyard Wildlife Habitat, and they were concerned that the disturbance caused by the project would adversely affect this habitat. In order to keep the wildlife habitat designation, the final design would have to replace food-bearing bushes and trees lost during construction, provide habitat for aquatic and terrestrial life, and improve the creek aeration. The property owner was also concerned with the aesthetic aspects of the project since the project would severely impact most of their backyard.

After reviewing several different design alternatives, a final design was chosen that addressed all of the project requirements. A four-stage drop structure was designed which alternatively funnels the water and dissipates energy with an upstream curved, grouted, stacked boulder drop, a deep grouted boulder-lined transition pool, a lower cascade drop, and a lower stilling pool (Figures 1 and 2). The resulting drop structure looked natural, but the size and location of every drop and rock in the waterway and on the banks were strategically sized and placed for flood control and habitat. Below the curved entrance, a sheet pile cutoff wall was installed, and the joints were sealed with a water sensitive expansive product.

Adjacent to the drop structure, an overflow spillway was designed to handle the additional flow in the event of failure of the fabridam. This area was shaped to direct flow back into the main channel at the stilling basin. The spillway was lined with boulders and riprap to prevent scour and vegetated with trees and shrubs.

In addition to the structural components of the drop structure, a number of innovative planting techniques were used to soften the appearance of the rock and provide the required habitat. Adjacent to the main pools of the drop, planted grouted boulders were used (Figure 3). The boulders in these areas were only grouted halfway up the rock’s depth, and the remaining depth between the boulders was filled with soil and then planted with native material. Above the main pool areas along the bank, planted riprap was used to provide additional energy dissipation and help anchor the riprap. Below the stilling basin area, a variety of plants were selected and installed along the water's edge to provide a food source for the birds and cover for the fish. In addition, a number of trees were planted to provide additional habitat and screening for the affected property owner.

8.3 Construction

L & M Enterprises was awarded the contract for construction of this project. Because of the tight site constraints, coordination with the local governments and the affected homeowner was critical. The contractor was required to provide temporary trail access across the drainageway as much as possible during construction. This was accomplished by constructing a temporary channel crossing upstream of the project area and diverting users along the new alignment. Another challenge during construction was effectively handling the constant base flow of the gulch and the occasional storm event, which severely tested the water control. In addition, L & M worked closely with the homeowner to minimize the impacts during construction and allow as much use of the property as possible.

Construction began in the fall of 1997 and was completed in early 1998. The plant material was installed shortly after construction was completed, but before the wet spring season. The homeowners were happy with the appearance and function of the new drop structure. They took real ownership of the completed project and provided all irrigation and maintenance of the newly installed plants, shrubs, and trees. Since the project has been completed, they have installed additional landscaping and plantings to further enhance their backyard habitat.

Photo 2

8.4 Conclusion

The Lena Gulch Drop Structure Project is a real success story. The project started as a complicated design with multiple concerns to address, and finished as an award-winning project with which all project participants are very pleased. It has been several years since the project has been completed, and in that time, it has seen numerous storm events. The drop structure has functioned well, and the revegetation has been established and is thriving (Photos 1 & 2). The homeowner was able to keep the Backyard Wildlife Habitat designation and noted that several species of fish have moved into the pools below the drop structure.

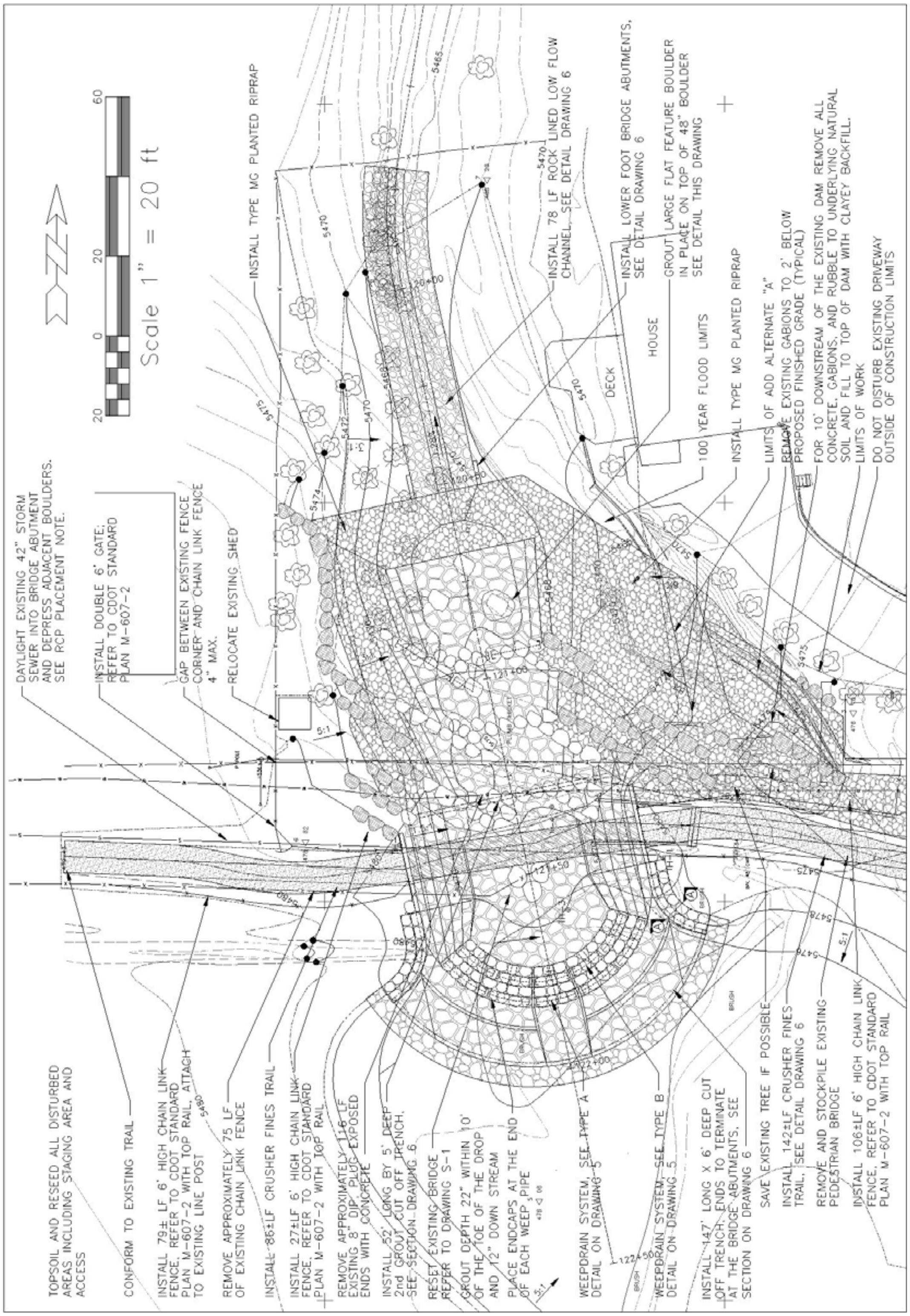

Figure 1—Plan

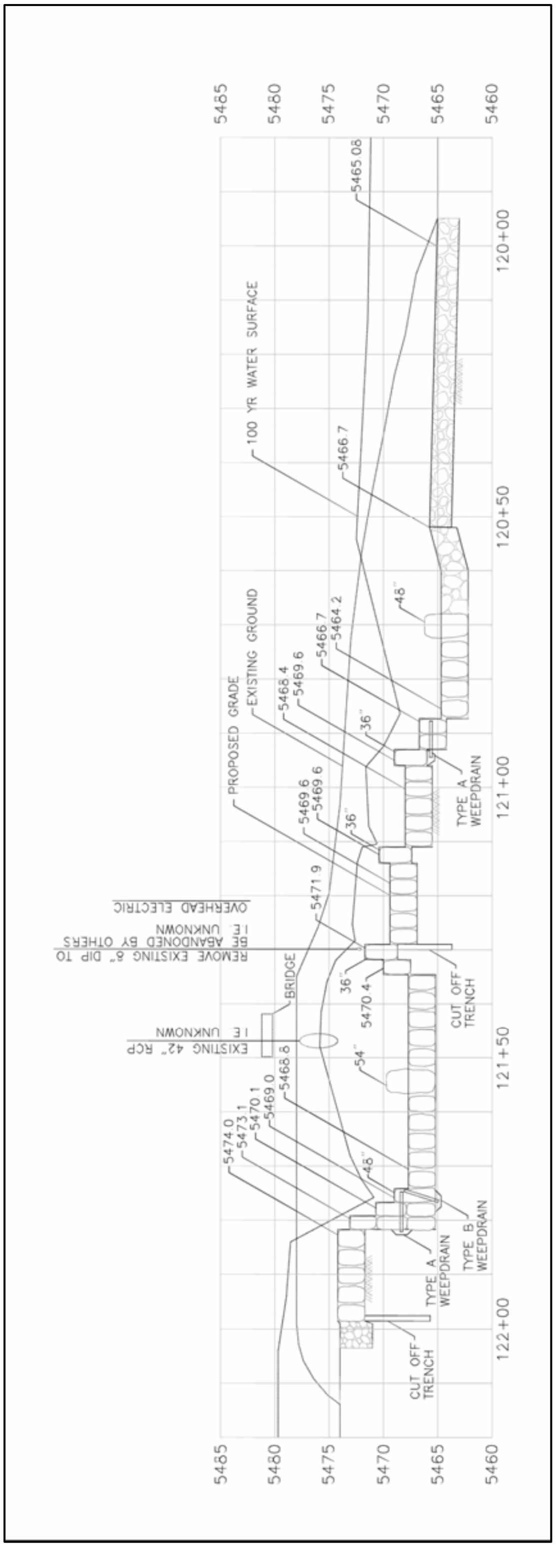

Figure 2—Profile

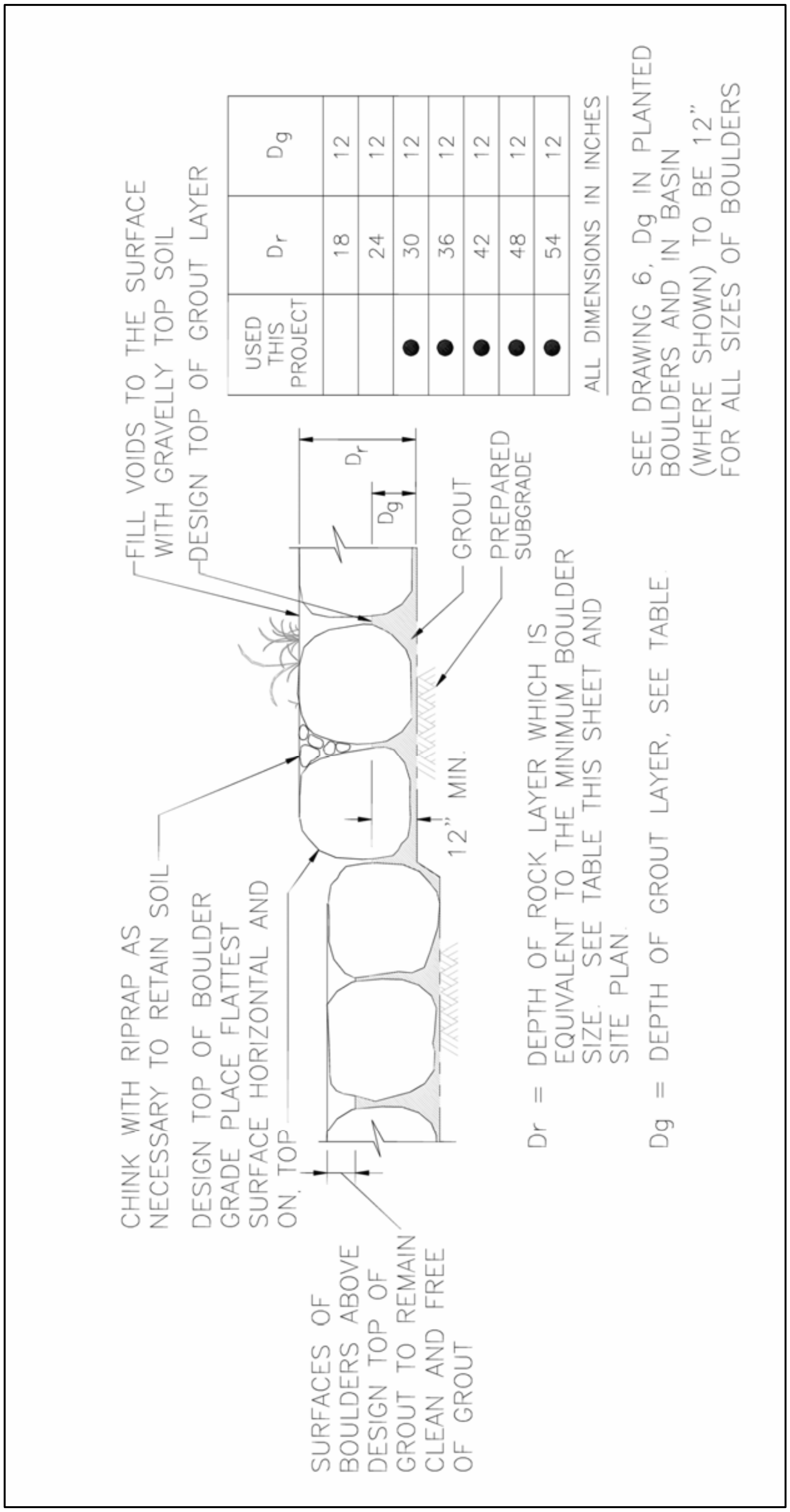

Figure 3—Planted Grouted Boulders