CHAPTER 1

INTRODUCTION

March 7, 2011

Chapter One - Introduction

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1.1 Purpose

Providing adequate drainage in urban areas has been proven as a necessary component in maintaining the overall health, welfare, and economic well being of a region. Drainage is a regional feature that affects multiple jurisdictions and all parcels of land. It is important to develop drainage policy that balances both public and private considerations (UDFCD, 1969).

Certain underlying principles should be applied when planning drainage facilities. These principles apply to both water quantity and water quality management. Policy statements and technical criteria serve as the implementation tools for the underlying drainage principles (UDFCD, 1969).

The purpose of this Drainage Criteria Manual, in conjunction with the development of overall master planning of the major watersheds throughout the Waverly, Nebraska area, is to provide drainage facilities in urban areas that avoid the disruption of the community while improving the overall health and welfare of the region in an economic way.

1.2 Contents

This Drainage Criteria Manual has been prepared for the City of Waverly, Nebraska to provide guidance to design engineers, hydrologists, water quality specialists, and others involved in the management of stormwater runoff. It is comprised of nine technical chapters that provide guidance on the major aspects of urban stormwater management and drainage facility design. The Manual is intended to be an effective and practical resource that provides users with proven engineering approaches along with illustrative examples. The Manual represents a compilation of a large amount of technical information in a single document, which should help minimize the need for multiple outside references.

It is assumed that the user has basic knowledge of hydraulics, hydrology, and stormwater management concepts. While some theory is presented in the Manual, the text is devoted more to the practical application of the theory, as it relates to drainage management and design.

1.3 Objectives

Drainage, flood control, and water quality protection in the City of Waverly and its surrounding areas are an integral part of the comprehensive planning process. Drainage represents only one component of a larger urban system. The objectives of the City of Waverly with respect to drainage, flood control, and water quality protection are to:

- To protect the general health, safety, and welfare of the residents of the City of Waverly.
- To minimize property damage from flooding; including minimization of localized neighborhood flooding.
- To ensure that new buildings and facilities are free of flood hazard from major and smaller storm runoff events.
- To minimize water quality degradation by limiting the amount of sediment generation and erosion of channels.
- To encourage the retention of open space, particularly along natural drainageways.
- To plan for large and small flooding events by providing both major and minor drainage systems.
- To implement reasonable, cost effective best management practices (BMPs) for sediment control and water quality enhancement.
- To manage stream and drainage channel corridors to promote environmental diversity and to protect buildings and facilities from damage by channel erosion.
- To stabilize channels to, among other things, minimize the disruption of existing infrastructure such as bridges and utility lines.
- To comply with the City of Waverly National Pollutant Discharge and Elimination System (NPDES) permit requirements.
- To develop equitable methods to adequately fund construction, operation and maintenance, and administration of an up-to-date stormwater management program.

- To minimize future operating and maintenance expenses.
- To educate the public on stormwater policies and administrative procedures.
- To build a stormwater program based on understanding and cooperation with builders and developers, providing for effective administrative authority for the City.

The strict application of this Manual in the overall planning of new development is practical and economical; however, there are many built-up areas within and around the City of Waverly which will not conform to the drainage standards proposed in this Manual. In fact, the problems associated with these areas provided some of the impetus for the development of this Manual. The upgrading of built-up areas to conform to the policy, criteria, and standards contained in this Manual may be difficult, and sometimes impractical. Therefore, in the planning of drainage improvements in built-up areas, it is recommended that the design approaches presented in this Manual be adjusted to optimize the benefit to cost ratio.

1.4 Planning Concepts

The following general principles apply when planning for and designing urban storm drainage systems (ASCE, 1992):

1.4.1 Drainage is a Regional Phenomenon

Drainage is a regional phenomenon that does not respect the boundaries between government jurisdictions or between public and private properties. Therefore, a successful plan must integrate regional jurisdictional cooperation, where applicable, to accomplish established goals. The City of Waverly will seek the cooperation of LPSNRD and Lancaster County to minimize the contribution of all storm drainage systems to flooding and in the preparation and implementation of master drainage plans.

1.4.2 Storm Drainage is a Sub-System of the Total Urban System

Drainage is a sub-system of all urbanization. The planning of drainage facilities must be included in the urbanization process. The first step is to include drainage planning with all regional and local urban master plans.

Stormwater management facilities, such as open channels and storm drains, serve both conveyance and storage functions. When a channel is planned as a conveyance feature, it requires an outlet as well as downstream storage space to adequately contain the design flows. The space requirements for adequate drainage may become a competing use for space with other land uses. If adequate provision is not made in the land use plan for the drainage requirements, stormwater runoff will conflict with other land uses, will result in water damages, and will impair or even disrupt the functioning of other urban systems (Tulsa, 1993).

1.4.3 Urban Areas Have Two Drainage Systems

Urban areas are comprised of two drainage systems. The first is the minor or primary system, which is designed to provide public convenience and to accommodate relatively moderate frequent flows. The other is the major system, which carries more water and operates when the rate or volume of runoff exceeds the capacity of the minor system.

1.4.4 Runoff Routing is a Space Allocation Problem

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

1.4.5 Stormwater Runoff as a Resource

Stormwater runoff and the facilities to accommodate the runoff can be an urban resource when properly included in the urban system. Drainageways can provide environments for various life forms such as aquatic life, mammals, birds, and vegetation. In many cases the drainage facilities can provide areas for active and passive recreation for citizens to enjoy. Although sometimes a liability to urbanization, stormwater runoff can be beneficial as an urban resource (Tulsa, 1993).

When stormwater runoff is treated as a resource, water quality aspects become important. As such, it is important to implement best management practices (both structural and nonstructural) and effective erosion and sediment control measures.

Due to the multi-purpose potential of stormwater runoff, natural drainage channels should be given priority consideration in the preparation of drainage system designs and should be included as an integral part of the landscape.

1.4.6 Utilize the Features and Functions of the Natural Drainage System

Every site contains natural features that may contribute to the management of stormwater under existing conditions. Each development plan should carefully map and identify the natural system. Natural engineering techniques can preserve and enhance the natural features and processes of a site and maximize post-development economic and environmental benefits. Good designs improve the effectiveness of natural systems, rather than negate, replace, or ignore them.

1.4.7 Post-Development Flow Rates Shall Not Exceed Pre-Development Conditions

In new developments, post-development flow rates shall not exceed pre-development conditions. This can be addressed by considering the following: (1) minimizing the amount of directly connected impervious area and (2) controlling the rate of runoff by implementing stormwater management systems which use practices that maintain vegetative and porous land cover, or use of storage facilities..

1.4.8 Design the Stormwater Management System from the Point of Outflow

The downstream conveyance system should be evaluated to ensure that it has sufficient capacity to accept design discharges without adverse backwater impacts on the proposed conveyance system, or downstream impacts such as flooding, streambank erosion, and sediment deposition. Starting tailwater conditions for the major and minor design storm flow should be determined

1.4.9 Provide Regular Maintenance

Failure to provide proper maintenance reduces both the hydraulic capacity and pollutant removal efficiency of the system. Effective maintenance relies on clear assignment of tasks and a regular inspection schedule.

1.4.10 Preventive and Corrective Actions

In existing urban settings, it may be necessary to develop a stormwater management strategy based upon both preventive and corrective measures. For example, structural corrective measures such as inlets, storm drains, interceptor lines, channelized stream sections and reservoirs affect and control storm runoff and floodwaters directly. Nonstructural corrective measures, such as floodproofing and land use adjustments, help limit activities in the path of neighborhood storm runoff or in river floodplains. Preventive actions available for reducing storm runoff and flood losses include: flood-prone land acquisition, floodplain regulations, and control of land uses within flood-prone areas.

1.5 Criteria Summary

1.5.1 Drainage Design and Technical Criteria

The design criteria presented in this Manual are based on national engineering state-of-the-practice for stormwater management, modified to suit the needs of Waverly specifically. The criteria are intended to establish guidelines, standards, and methods for effective planning and design. The criteria should be revised and updated as necessary to reflect advances in the fields of urban drainage engineering and urban water resources management.

1.5.2 Minor and Major Drainage Systems

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

1.5.2.1 Minor Drainage System

The minor drainage system is typically thought of as storm drains and related appurtenances, such as inlets, curbs and gutters. The minor system is normally designed for floods with return frequencies of 2-years to 10-years, depending upon the kind of land use. The minor system has also been termed the "convenience" drainage system.

For Waverly, the minor drainage system design will be based on the 5-year to 10-year return frequency storms, depending on the design application. For residential areas, the 5-year storm is appropriate, while for downtown areas and industrial/commercial areas, the 10-year storm is appropriate.

During design, the hydraulic grade line for all enclosed systems shall be determined to ensure that inlets act as inlets, not outlets. All easements for storm drain pipe should be a minimum of 30 feet wide. In situations where the engineer can clearly demonstrate that an easement less than 30 feet is adequate, the City may consider such a request. Easements for storm drain pipe and surface water flowage shall be used where a drainageway must be maintained to carry stormwater flow in excess of the storm drain pipe capacity. The easement cross-section shall accommodate the depth and width of flow from the 100-year storm. The width must also be designed to allow for access of maintenance equipment during the major storm.

An additional design storm equal to the 2-year frequency shall be used in the design of detention facilities.

1.5.2.2 Major Drainage System

The major drainage system is designed to convey runoff from, and to regulate encroachments for, large, infrequently occurring events. When development planning and design do not properly account for the major storm flow path, floodwaters will seek the path of least resistance, often through individual properties, thus causing damage. An assured route of passage for major storm floodwaters should always be provided such that public and private improvements are not damaged. For subdivisions in Waverly, this need is to be provided for both in watershed headwaters settings and along major drainageways.

The 100-year return frequency storm shall be the major drainage system design storm for all new developments. Runoff from major storms should pass through a development without flooding buildings or homes. Overland flow routes can be provided using streets, swales, and open space.

Open channels for transportation of major storm runoff are desirable in urban areas and use of such channels is encouraged. Open channel planning and design objectives are best met by using natural, or natural-type channels, which characteristically have slow velocities, and a large width to depth ratio. Optimum benefits from open channels can best be obtained by incorporating parks and greenbelts with the channel layout.

To the extent practicable, open channels should follow the natural channels and should not be filled or straightened significantly. Effort must be made to reduce flood peaks and control erosion so that the natural channel regime is maintained. Channel improvement or stabilization projects are encouraged which minimize use of visible concrete, riprap, or other hard stabilization materials to maintain the riparian characteristics.

1.5.3 Storm Runoff Computation

The calculation of the storm runoff peaks and volumes is important to the proper planning and design of drainage facilities. The calculation of runoff magnitude shall be by either the rational method, the Soil Conservation Service (SCS, now known as the Natural Resource Conservation Service) TR-55 method, or using the SCS method in the U.S. Army Corps of Engineers (USACE) HEC-HMS software.

1.5.4 Detention

Detention facilities shall have release rates which do not exceed the pre-development peak discharge rates for the 2-year, 10-year, and 100-year storms. Hydrologic conditions as of 1 August 1999 shall be used to determine peak release rates for pre-development conditions. Submittal of hydraulic design calculations is required to document that major and minor design storm peak flows are attenuated. On-site and regional facilities shall be designed with adequate access and sediment storage right-of-way (including sediment fore-bays) to facilitate maintenance.

On-site detention is required unless the master planning process or a regional analysis has shown that the detention requirement can be transferred to a regional detention cell. On-site detention, however, may still be necessary to provide for receiving stream channel stability maintenance. To the extent feasible, extended detention design shall be utilized to enhance stormwater quality benefits, including increased sediment removal.

Where feasible, strategically located regional detention cells shall be used to reduce flow peaks from major storm events. Funding mechanisms will be developed to allow joint investment by benefitted parties in regional facilities, where transfer of detention requirements proves to be feasible and beneficial.

On-site and regional detention facilities shall be designed with adequate access and sediment storage right-of-way (including sediment forebays) to facilitate maintenance. Unless private maintenance of on-site detention facilities is acceptably performed, necessary maintenance by government forces shall be provided. The cost of this government service shall be equitably allocated to responsible parties. The owner shall provide record drawings of the storage facility to the City.

1.5.5 Streets

The primary drainage functions of streets are to convey nuisance flows quickly and efficiently to the storm drain or open channel drainage with minimal interference to traffic movement and to provide an emergency passageway for the major flood flows with minimal damage to adjoining properties, while allowing for safe movement of emergency vehicles.

The allowable use of streets for new land development in Waverly for minor and major storms runoff in terms of pavement encroachment is presented in Chapter 3.

1.5.6 Floodplains and Flood Prone Areas

100-year existing conditions flood profiles for the Waverly tributaries were developed during the Watershed Master Planning process. New development along the channel areas shall be such that the lowest opening in new buildings is protected from the flood profile.

In watersheds where FIS floodplains have not been delineated and where flood prone areas have not yet been determined through the watershed master planning process, regulate new development so the lowest opening of adjacent new buildings is protected to one foot above the calculated 100-year flood profile. Flood corridors delineated during development of land shall be legally described and recorded.

In watersheds where shallow flooding areas have been delineated, regulate new development according to the regulations set forth under “Flood Fringe Overlay District – (Including AO and AH Zones),” Chapter 11.535 of Article 5 of the City of Waverly Zoning Ordinance.

1.5.7 NPDES Construction Site Activities

A NPDES “notice of intent” and a Stormwater Pollution Prevention Plan (SWPPP) shall be required before land disturbance or vegetation removal activities occur on any site greater than or equal to 1.0 acre in size. The structural and non-structural best management practices (BMPs) are recommended to address stormwater quality enhancement. The SWPPP shall be prepared by a designated erosion control designer. A designated erosion control designer shall be a; licensed professional engineer, architect or landscape architect; a professional in Erosion and Sediment Control, certified by the Soil and Water Conservation Society; or a person with similar erosion and sediment control training and knowledge certified by a nationally recognized erosion and sediment control association. As one condition of approval, a construction schedule shall be included which indicates installation of as many of the proposed BMPs as are feasible before any land disturbing activity is conducted, including site grubbing. The schedule will also indicate a plan to limit the duration of exposure of disturbed land.

Contractors and developers shall contact the City on the business day prior to performing land disturbance or vegetation removal on any site greater than or equal to 1.0 acre. Construction sites will be inspected periodically for compliance with submitted SWPPPs.

1.5.8 Water Quality

Both structural and nonstructural best management practices (BMPs) are recommended that address long-term stormwater quality enhancement. Effective, reasonable, and cost-effective BMPs should be selected for implementation on a site-specific basis and in a manner that is consistent with existing basin master plans. For water quality control purposes, it is recommended that the first 0.5 inches of runoff be captured and detained for a period of at least 24 hours, and preferably longer.

The following is a list of voluntary structural BMPs that should be considered:

- Create temporary ponding areas on parking lots and in landscaped or turfed open areas of building sites;
- Use porous pavement for remote parking areas;
- Reduce the amount of impervious area directly connected to the storm drain system;
- Intentionally create longer vegetated drainage paths for minor storm events;
- Encourage use of constructed wetlands;
- Develop multipurpose extended detention facilities;
- Use retention facilities (wet ponds) where feasible.

The following is a list of voluntary non-structural BMPs that should be encouraged:

- Use of appropriate vegetation to reduce the need for fertilizer and pesticides;
- Preservation of environmentally sensitive areas to protect them from development or other disruption;
- Set aside more open space;
- Preserve or re-establish riparian vegetation;
- Implement staged grading of developments to minimize the amount of land disturbed at one time.

Additional structural and nonstructural BMPs are presented in Chapter 9 of the Manual.

1.6 Interrelationship Between Stormwater Quantity and Quality Management

With urbanization, the hydrology of a watershed changes in three important ways: (1) the total runoff volume is greater, (2) the runoff occurs more rapidly, and (3) the peak discharge is greater. The increase in runoff volume results from the decrease in infiltration and depression storage. The shortened time base results from the greater flow velocities in the drainage system. The increase in peak discharge is the inevitable consequence of a larger runoff volume occurring over a shorter time. This increase in peak discharge for any storm means a related high discharge occurs more frequently (ASCE, 1992).

Receiving water impacts are caused by a combination of physical and chemical effects. Impacts associated with stormwater discharges can be discussed in terms of three general classes: (1) short-term changes in water quality; (2) long-term water quality impacts; and (3) physical impacts. Short-term changes in water quality occur during and shortly after storm events. Long-term impacts are caused by the cumulative effects associated with repeated stormwater discharges. Physical impacts include erosional effects of high stream velocities that occur after the natural hydraulic cycle has been altered. More frequent occurrences of high discharges may cause or intensify channel erosion problems, disrupting the riparian habitat both where the erosion occurs and where the additional sediment is deposited downstream (ASCE, 1992).

The City of Waverly has seen the consequences of rapid urbanization on the water quality of its receiving streams. Consequently, this Manual is part of an effort to more effectively manage both stormwater quantity and quality. By implementing well planned and designed engineering approaches, the necessary measures can be taken to minimize the cumulative water quality and water quantity impacts that result from urbanization.

1.7 Limitations

The interpretation and application of the provisions in this Manual shall be the minimum requirements for promotion of the health, safety, convenience, order and general welfare of the community. The standards, however, should not be construed as rigid criteria. Rather, the criteria are intended to establish guidelines, standards and methods for sound planning and design. The City may set aside these criteria in the interest of the health, safety, convenience, order and general welfare of the community.

The Manual is not intended to interfere with, abrogate, or annul any other regulation, statute, or other provision of the law. Where any provision of this Manual imposes restrictions different from those imposed by any other provision of this Manual or any other regulation or provision of law, that provision which is more restrictive or imposes higher standards shall govern.

1.8 Updating

The policies and criteria presented in this Manual may be amended as new technology is developed and/or experience is gained in the use of the criteria indicate a need for revision. Amendments and revisions to the Manual will be made by the City of Waverly when necessary to accomplish the goal of reasonable public protection from surface water runoff.

References

American Society of Civil Engineers, Manuals and Reports of Engineering Practice No. 77. Design and Construction of Urban Stormwater Management Systems. 1992.

City of Tulsa, Oklahoma. Stormwater Management Criteria Manual. 1993

Denver Urban Drainage and Flood Control District. Drainage Criteria Manual, Volume I. March 1969.

APPENDIX 1-A

STORMWATER MANAGEMENT SYSTEMS DESIGN PROCEDURE

Appendix 1-A
Stormwater Management Systems Design

Appendix 1-A
Stormwater Management Systems Design

Appendix 1-A
Stormwater Management Systems Design

Section

APPENDIX 1-B

POLICY FOR CITY OF WAVERLY STORMWATER MANAGEMENT ISSUES

Appendix 1-B
Policy for City of Waverly Stormwater Management Issues

A. Encourage voluntary implementation of Better Management Practices (BMPs) using structural and non-structural measures

1. Utilize both structural and nonstructural BMP's for City of Waverly's stormwater management program addressing long-term stormwater quality enhancement. Select BMPs for implementation in site-specific projects or basin master plans that are effective, reasonable, and cost-effective.
2. Encourage implementation of voluntary nonstructural BMPs: use of appropriate vegetation to reduce the need for fertilizer, pesticides, and irrigation; provision of incentives to set aside more open space; preservation of more riparian zone or other environmentally sensitive areas than required by local, state or federal regulation; preservation or reestablishment of riparian vegetation.
3. Encourage implementation of voluntary structural BMPs; intentional creation of temporary ponding areas on parking lots and in landscaped or turfed open areas of building sites during the design process; maximize use of pervious surfaces in the constructed environment; disconnection of impervious areas from the storm drain system; use of constructed wetlands; incorporation of permanent pools and other water quality enhancement features into storage facilities.
4. Provide explicit guidance documents on design, installation, and maintenance of BMPs and provide education so reviewers, developers, contractors, and inspectors understand the purpose and function of BMPs.
5. Develop a management program to control pollutants as required for Municipal Separate Storm Sewer System (MS4) National Pollutant Discharge Elimination System (NPDES) Permit compliance.

Appendix 1-B
Policy for City of Waverly Stormwater Management Issues

B. Minimize Localized Flooding by encouraging better design of subdivisions to create and protect overflow routes for run-off from major storm events.

1. Formalize the City monitoring of stormwater system performance in developed areas to identify problem areas. Analyze and develop retrofit solutions where possible and appropriate. Provide technical support to assist homeowners and neighborhood officials in recognizing and resolving local problems.
2. Continue minor system design based on 5-year to 10-year frequency storms depending on the design application. During design, determine the hydraulic grade line for all enclosed systems to confirm that inlets act as inlets, not outlets. Use the 100-year return frequency storms for design of the major system in all new developments.
3. Runoff from major storms (i.e., 1% chance of recurrence, a.k.a. 100-year storms) should pass through a development without flooding buildings or homes. Overland flow routes must be provided using streets, swales, and open space, etc. Current design standards provide guidance to design the minor system (inlets, pipes, and small channels) but need refinement to complement the design standards required to address major storms.
4. Require stormwater flowage and maintenance easements that prohibit placement of structures, fences, change of grade, elevation, or contour without written consent of the City. All easements for storm drain pipe should be a minimum of 30 feet wide. In situations where the Project Engineer can clearly demonstrate that an easement less than 30 feet is adequate, the City may consider such a request. Easements for storm drain pipe and surface water flowage shall be used where a drainageway must be maintained to carry stormwater flow in excess of the storm drain pipe capacity. The easement cross-section must accommodate the depth and width of flow from the 100-year storm. The width must also be designed to allow for access of maintenance equipment. Implement a program to assure proper construction and maintenance of easement flow capacity.
5. Subdivision grading plans should show setback lines for a proposed development. Grade lots to drain from setback lines to street or perform analysis to show that not doing so will not result in flooding from the major storm. Grade lots so buildable area will be outside the limits of the area flooded by the major storm, from the upper-most inlet location to the downstream project limits. Provide design nomographs for street right-of-ways and channel capacities in the Drainage Criteria Manual to facilitate determination of water surface limits.
6. Require that elevational difference between the finish floor and low opening of new buildings and the top of curb at the upper lot line (or lot line adjoining drainage easements) be indicated on the Building Permit application. Compare this information to available flood level information before approval. Add a statement to the Building Permit Application which indicates by signing the application the applicant has taken into account available information when setting finish floor and low opening levels.
7. Provide public right to inspect private stormwater facilities and perform necessary maintenance to assure proper operation. Determine how public costs incurred to accomplish necessary maintenance will be recovered.

Appendix 1-B
Policy for City of Waverly Stormwater Management Issues

C. Minimize Flooding Along Tributaries and Drainageways.

1. Regulate development in Federal Emergency Management Agency Flood Insurance Study (FIS) delineated floodplains in accordance with the current floodplain regulations.
2. Through the watershed master planning process, develop approximate 100-year projected future conditions flood profiles for mainstem and tributary channel corridors that are between the limits of detailed study by FIS and the boundary of the uppermost 150-acre sub-basin(s). Once the master plan flood profiles have been accepted by the City, regulate new development along the channel areas so the lowest opening in new buildings is protected from the flood profile.
3. In watersheds where FIS floodplains have not been delineated and where flood prone areas have not yet been determined through the watershed master planning process, regulate new development so the lowest opening of adjacent new buildings is protected to one foot above the calculated 100-year flood profile.
4. In all watersheds where a FIS floodway has not been delineated, development shall preserve a corridor with a minimum width equal to the channel bottom width, plus 60 feet, plus six times the channel depth. The corridor width will be centered on the channel and be delineated along all channels with a drainage areas exceeding 150 acres..
5. Regulate new development so it does not occur within minimum corridors. Riparian vegetation within the identified flood corridors shall be preserved to the maximum extent practicable, or acceptably mitigated, during the development planning and construction processes. Encroachments of the riparian vegetation will be permitted to provide for OM&R, channel improvements, stormwater storage facilities, public parks, pedestrian/bike trails, other recreational uses, utility crossings, streets and driveways, and other public purposes.
6. Flood corridors delineated during development of land shall be legally described and recorded.
7. To preserve riparian characteristics of channels, design channel improvement or stabilization projects to minimize use of visible concrete, riprap, or other hard stabilization materials.
8. Design culvert or bridge structures which cross the channel to convey a 50-year frequency peak flow rate (for projected future conditions) without over-topping the roadway and without increasing the 100-year flood level (even under overtopping conditions) unless a flood storage easement upstream of roadway can be obtained.

Appendix 1-B
Policy for City of Waverly Stormwater Management Issues

D. Improve the design and construction of stormwater storage (detention/retention) facilities and, when appropriate, use regional storage facilities.

1. Where feasible, use strategically-located regional stormwater storage facilities to reduce flow peaks from major storm events. Evaluate downstream impacts of regional facilities. Select regional facility sites and reserve required land before development occurs, when possible. Develop funding mechanisms to allow joint investment by benefitted parties in regional facilities. These facilities will be constructed, operated and maintained by the City or LPSNRD. Require on-site detention storage unless the master planning process or a regional analysis has shown that the detention requirement can be transferred to a regional stormwater storage facility, which is determined to be of regional benefit to the storm drainage system by the City and NRD. Due to the fact that stream channel degradation is a major cause of stormwater-related water quality problems in Waverly, on-site detention facilities may still be necessary to provide maintenance of receiving stream channel stability, maintenance and water quality enhancement.
2. Design on-site and regional stormwater storage facilities with adequate access and sediment storage right-of-way to facilitate maintenance.
3. Design on-site and regional stormwater storage facilities with adequate access and sediment storage right-of-way to facilitate maintenance. Include sediment-trapping forebays and maintenance-friendly measures in stormwater storage designs.
4. Use the actual hydrologic conditions encountered on the site to determine peak release rates for existing conditions.
5. Require submittal of hydraulic design calculations of outlet works to document that major and minor storm peak flows will be attenuated to existing conditions. To promote channel stability, require on-site stormwater storage facilities to also attenuate 2-year peak flow rates to existing conditions.
6. Require that record drawings of stormwater storage facilities be provided to the City.
7. Unless private maintenance of on-site stormwater storage facilities is acceptably performed, provide necessary maintenance with government forces. Equitably allocate costs incurred to responsible parties.

APPENDIX 1-C

GLOSSARY OF KEY TERMS

Appendix 1-C
Glossary of Key Terms

1-year Flood: The flood typically occurring or being exceeded in any given year.

2-Year Flood: The flood having a fifty percent chance of being equaled or exceeded in any given year.

5-Year Flood: The flood having a twenty percent chance of being equaled or exceeded in any given year.

10-Year Flood: The flood having a ten percent chance of being equaled or exceeded in any given year.

Base Flood or 100-Year Flood: The flood having a one percent chance of being equaled or exceeded in any given year.

Compensatory Storage: Replacement of storage volume that is hydrologically equivalent to lost storage when encroachment occurs in the floodplain or floodprone area.

Conveyance structure: A pipe, open channel, or other facility that transports runoff from one location to another.

Drainage criteria: Specific guidance provided to the engineer/designer to carry out drainage policies. An example might be the specification of local design hydrology (“design storm”).

Existing Urban Area: Those areas inside the one mile extraterritorial jurisdiction of the City of Waverly having a zoning designation other than AG Agricultural District, as defined by the Waverly Zoning Regulations.

FEMA: The Federal Emergency Management Agency.

Flood Fringe: That portion of the floodplain which is outside of the floodway

Flood Insurance Rate Map (FIRM): Flood Insurance Rate Map (FIRM) shall mean the February 18, 2011 Flood Insurance Rate Map and any revisions thereto, on which FEMA has delineated both the areas of special flood hazards and the risk premium zones applicable to the community.

Floodplain: Those lands which are subject to a one percent or greater chance of flooding in any given year, as shown on the Flood Insurance Rate Map (FIRM) issued by FEMA for Lancaster County, Nebraska and incorporated areas, as amended.

Floodplain planning/floodplain management: Technical and nontechnical studies, policies, management strategies, statutes and ordinances that collectively manage floodplains along rivers, streams, major drainageways, outfalls, or other conveyances. The federal government normally plays a major role in floodplain planning and management, whereas in urban stormwater management and design, local governments dominate the decision-making process.

Floodprone Area: Those lands which are subject to a one percent or greater chance of flooding in any given year, as determined by hydrologic and hydraulic studies completed by the City or other government agency, or other acceptable source as approved by the City where this is the best available information.

Floodway: The channel of a river or other watercourses and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than one foot.

Appendix 1-C
Glossary of Key Terms

Major drainageway: A readily recognizable natural or improved channel that conveys runoff that exceeds the capacity of the minor drainage system, including emergency overflow facilities.

Major system: The portion of the total drainage system that collects, stores, and conveys runoff that exceeds the capacity of the minor system. The major system is usually less controlled than the minor system, and will function regardless of whether or not it has been deliberately designed and/or protected against encroachment, including when the minor system is blocked or otherwise inoperable. It may be collinear with, or separate from, the minor system. It should be noted that there are those who object to the use of the terms “major” and “minor” to describe portions of the drainage system, perhaps because these terms imply that the minor system is less important. Other terms (primary system, convenience or basic system, overflow system, major/primary drainage ways, subordinate system, etc.), have been suggested. Major/minor are used in this Manual because they seem to be the most widely used terms.

Master drainage plan: The plan that an engineer/designer formulates to manage urban stormwater runoff for a particular project or drainage area. It typically addresses such subjects as characterization of site development; grading plan; peak rates of runoff and volumes of various return frequencies; locations; criteria and sizes of detention ponds and conveyances; measures to enhance runoff quality; salient regulations and how the plan addresses them; and consistency with secondary objectives such as public recreation, aesthetics, protection of public safety, and groundwater recharge. It is usually submitted to regulatory officials for their review.

Minor or primary system: The portion of the total drainage system that collects, stores and conveys frequently occurring runoff, and provides relief from nuisance and inconvenience. This system has traditionally been carefully planned and constructed, and normally represents the major portion of the urban drainage infrastructure investment. The degree of inconvenience the public is willing to accept, balanced against the price it is willing to pay, typically establishes the discharge capacity or design recurrence frequency of a minor system. Minor systems include roof gutters and on-site drainage swales, curbed or side-swaled streets, stormwater inlets, underground system sewers, open channels and street culverts.

Multiple-purpose facility: An urban stormwater facility that fulfills multiple functions such as enhancement of runoff quality, erosion control, wildlife habitat, or public recreation, in addition to its primary goal of conveying or controlling runoff.

New Growth Areas: Those areas within the one mile extraterritorial jurisdiction of the City of Waverly and zoned AG Agricultural District, as defined by the Waverly Zoning Regulations.

Outfall facility: Any channel, storm drain, or other conveyance receiving water into which a storm drain or storm drainage system discharges.

“Risk-based” design: Design of urban stormwater management facilities not only on the basis of local standards, but also on the basis of the risk (cost) of the flow exceeding a selected design. Virtually all stormwater management projects have some component of risk which is inherent in selection of a design return frequency. Risk may also account for special upstream or downstream hazards that would be posed by adherence to some recommended standard. For example, the designer of culverts in a subdivision might choose to upsize particular storm drains from a 10-year to a 50-year basis to protect properties, or to make other provisions to secure emergency discharge capacity.

Appendix 1-C
Glossary of Key Terms

Special structures: Those components of urban drainage systems that can be thought of as “features” or “appurtenances” such as manholes, inlets, energy dissipators, transitions, channel slope protection, detention ponds and dams, and outlet works.

“Standard-based” design: Design of urban stormwater management facilities based on some specified set of regulatory standards. An example is the stipulation in local drainage policies that culverts for a given subdivision all be designed to pass the 100-year flood before road overtopping.

Storm drain: Often buried pipe or conduit, typically referred to as storm sewer that conveys storm drainage, also includes, curb & gutter, grate & curb inlets, swales, open channels, and culverts.

Stormwater detention: The temporary storage of stormwater runoff in ponds, parking lots, depressed grassy areas, rooftops, buried underground tanks, etc., for future release. Used to delay and attenuate flow, normally drained between storms.

Stormwater retention: Similar to detention except the facility may have a permanent pool of water or wet land that does not drain between storms.

Stream Crossing Structures: Structures used to convey pedestrians, motor vehicles, and/or utilities across drainageways. Stream crossing structures are composed of the structure, abutments, guard rails, fill, and other structural appurtenances that are generally perpendicular to the conveyance of flow within the floodplain or floodprone area.

Urban area: Land associated with, or part of, a defined city or town. This Manual generally applies to urban or urbanizing, rather than rural, areas.

Watershed Master Plan: A plan generated by the City or by the City in cooperation with other agencies, which includes hydrologic and hydraulic modeling for the base flood event, including floodplain elevation and limits.

CHAPTER 2

HYDROLOGY

March 7, 2011

Chapter Two - Hydrology

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APPENDICES

2.1 Overview

2.1.1 Introduction

Estimation of the peak rate of runoff, volume of runoff, and time distribution of flow is fundamental to the design of drainage facilities. Errors in the estimation will result in a structure that is either undersized and causes drainage problems (e.g., flooding, safety, nuisance, etc.) or oversized and costs more than necessary. On the other hand, it must be realized that any hydrologic analysis is only an approximation. The relationship between the amount of precipitation on a drainage basin and the amount of runoff from the basin is complex. Too few data are available on the factors influencing the rural and urban rainfall-runoff relationship to expect exact solutions.

2.1.2 Factors Affecting Floods

In the hydrologic analysis for a drainage structure, there are many factors that affect floods. Some of the factors which need to be recognized and considered on a site-by-site basis are:

<u>Drainage Basin Characteristics</u>

Size and Shape
Slope
Ground Cover and Land Use
Geology
Soil Types
Surface Infiltration
Ponding and Storage
Watershed Development Potential

<u>Stream Channel Characteristics</u>

Geometry and Configuration
Natural Controls
Artificial Controls
Channel Modifications
Agradation - Degradation
Debris
Manning's "n"
Slope

<u>Floodplain Characteristics</u>

Slope
Vegetation
Alignment
Storage
Location of Structures
Obstructions to Flow

<u>Meteorological Characteristics</u>

Time Rate and Amounts of Precipitation
Historical Flood Heights

2.1.3 Hydrologic Method Selection

Many hydrologic methods have been developed and used in urban watersheds. Table 2-1 lists two recommended methods. Other methods may be used if they received prior approval from the City and if they are calibrated to local conditions and tested for accuracy and reliability. In addition, complete source documentation must be submitted for approval.

Methods listed in Table 2-1 have been selected for use in Waverly, Nebraska based on several considerations, including the following:

- Verification of their accuracy in duplicating local hydrologic estimates of a range of design storms.
- Availability of equations, nomographs, and computer programs.
- Use and familiarity with the methods used by local municipalities and consulting engineers.

Table 2-1 Recommended Hydrologic Methods

Method	Size Limitations[1]	Comments
Rational	0 - 150 Acres	• Method can be used for estimating peak flows • Method used for design of storm sewer systems • DO NOT use for design of storage facilities
SCS[2] Curve Number	0 - 2,000[3] Acres	• Method can be used for estimating peak flows • Method used to develop hydrographs • Method can be used for design of culverts or bridges • Method MUST be used for design of storage facilities • Method MUST be used for areas over 150 acres

[1] Size limitation refers to the subwatershed size to the point where the stormwater management facility (i.e., culvert, inlet) is located.
[2] SCS is the Soil Conservation Service Method. Although the SCS is now called the Natural Resources Conservation Service, the hydrologic method is still called SCS.
[3] Will likely be less than 2000 acres in urban areas due to the need for homogeneous subwatersheds.

2.2 Symbols And Definitions

To provide consistency within this chapter, as well as throughout this manual, the following symbols will be used. These symbols were selected because of their wide use in hydrologic publications.

Table 2-2 Symbols And Definitions

Symbol	Definition	Units
A	Drainage area	acres or mi^2
C	Runoff coefficient	-
C_f	Frequency factor	-
CN	SCS-runoff curve number	-
d	Time interval	hours
F	Pond and swamp adjustment factor	-
I	Rainfall intensity	in./hr
IA	Percentage of impervious area	%
I_a	Initial abstraction from total rainfall	in.
NRCS	Natural Resources Conservation Service	-
n	Manning's roughness coefficient	-
P	Accumulated rainfall	in.
Q	Rate of runoff	cfs
q	Storm runoff during a time interval	in.
R	Hydraulic radius	ft
S or Y	Ground slope	ft/ft or %
S	Potential maximum retention storage	in.
SCS	Soil Conservation Service	-
SL	Main channel slope	ft/ft
S_L	Standard deviation of the logarithms of the peak annual floods	-
T_B	Time base of unit hydrograph	hours
t_c or T_c	Time of concentration	min or hours
T_L	Lag time	hours
V	Velocity	ft/s

2.3 Concept Definitions

A good understanding of the following concepts will be important in any hydrologic analysis. These concepts will be used throughout the remainder of this chapter in dealing with different aspects of hydrologic studies.

Antecedent Moisture Conditions

Antecedent moisture conditions are the soil moisture conditions of the watershed at the beginning of a storm. These conditions affect the volume of runoff generated by a particular storm event. Notably they affect the peak discharge in the lower range of flood magnitudes — say below about the 15-year event threshold. As floods become more rare, antecedent moisture has a rapidly decreasing influence on runoff.

Depression Storage

Depression storage is the water stored in natural depressions within a watershed. Generally, after the depression storage is filled, runoff will commence.

Frequency

The frequency with which a given flood can be expected to occur is the reciprocal of the probability or chance that the flood will be equaled or exceeded in a given year. If a flood has a 20 percent chance of being equaled or exceeded each year, over a long period of time, the flood will be equaled or exceeded on an average of once every five years. This is also referred to as the recurrence interval or return period.

Hydraulic Roughness

Hydraulic roughness is a measure of the physical characteristics which impede the flow of water across the earth's surface, whether natural or channelized. It affects both the time response of a watershed and drainage channel as well as the channel storage characteristics.

Hydrograph

A hydrograph is a graph of the time distribution of runoff (expressed as a flow rate) from a watershed.

Hyetographs

The hyetograph is a graph of the time distribution of rainfall (usually expressed as an intensity) over a watershed.

Infiltration

Infiltration is the complex process whereby water penetrates the ground surface and is either stored in the soil pore spaces or flows to lower layers. An infiltration curve is a graph of the time distribution at which this occurs.

Interception

Storage of rainfall on foliage and other intercepting surfaces during a rainfall event is called interception storage.

Lag Time

Lag time is defined as the time from the centroid of the excess rainfall to the peak of the hydrograph.

Peak Discharge

The peak discharge, sometimes called peak flow, is the maximum rate of flow of water passing a given point during or after a rainfall event or snowmelt.

Rainfall Excess

The rainfall excess is the water available to runoff after interception, depression storage and infiltration are satisfied.

Recurrence Interval

The time interval in which an event will occur once on the average. (i.e. a 10-year storm is expected to occur once every 10 years, on the average)

Stage

The stage of a river or other water body is the elevation of the water surface above some elevation datum.

Time Of Concentration

The time of concentration is the time it takes a drop of water falling on the hydraulically most remote point in the watershed to travel through the watershed to the outlet or design point.

<u>Unit Hydrograph</u>

A unit hydrograph is the storm hydrograph resulting from a rainfall event which has a specific temporal and spatial distribution, which lasts for a specific duration and has unit volume (or results from a unit depth of runoff). The ordinates of the unit hydrograph are such that the volume of runoff represented by the area under the hydrograph is equal to one inch of runoff from the drainage area. When a unit hydrograph is shown with units of cubic feet per second, it is implied that the ordinates are cubic feet per second per inch of direct runoff.

2.4 Design Frequency

2.4.1 Overview

Since it is not economically feasible to design a structure for the maximum runoff a watershed is capable of producing, a design frequency must be established. The designer should note that the 5-year flood is not one that will necessarily be equaled or exceeded every five years. There is a 20 percent chance that the flood will be equaled or exceeded in any year; therefore, the 5-year flood could conceivably occur in several consecutive years. The same reasoning applies to floods with other return periods.

2.4.2 Frequency Design Criteria

<u>Cross Drainage</u>: Cross drainage facilities transport storm runoff under roadways. The cross drainage facilities shall be designed to convey (at a minimum) the 50-year runoff event without overtopping the roadway. The flow rate shall be based on upstream ultimate buildout land-use conditions. In addition, the 100-year frequency storm shall be routed through all culverts to be sure structures are not flooded or increased damage does not occur to the roadway or adjacent property for this design event.

<u>Storm drains</u>: A storm drain shall be designed to accommodate a 5-year storm in residential areas and a 10-year storm in commercial developments, downtown areas and in industrial developments. The design shall be such that the storm runoff does not: increase the flood hazard significantly on adjacent property; encroach onto the street or highway so as to cause a safety hazard by impeding traffic, emerging vehicles, or pedestrian movements to an unreasonable extent.

Based on these criteria, a design involving temporary street or road inundation is acceptable practice for flood events greater than the design event but not for floods that are equal to or less than the design event. Thus, if a storm drainage system crosses under a roadway, the design flood must be routed through the system to show that the roadway will not be overtopped by this event. The excess storm runoff from events larger than the design storm may be allowed to inundate the roadway or may be stored in areas other than on the roadway until the drainage system can accommodate the additional runoff.

<u>Inlets</u>: Inlets shall be designed for a 5-year storm in residential areas and small commercial developments and a 10-year storm in downtown areas industrial developments, and arterial roads.

<u>Detention and retention storage facilities</u>: All storage facilities shall be designed to provide sufficient storage and release rates to accommodate the 2-, 10-, and 100-year design storm events such that the post-development peak discharges do not exceed the pre-development rates. The design shall be such that the storm runoff does not increase the flood hazard significantly for adjacent, upstream, or downstream property or cause safety hazards associated with the facility. An emergency spillway shall be provided. For storage facilities, outlet designs that provide some control for flood events below the 2-year storm (e.g., v-notch weirs) are preferred over outlets that do not provide this control (e.g., pipes). In addition, the final design shall be checked to ensure that flood peaks at the downstream property line have not increased.

Table 2-3 presents the design storm to be used for each type of facility.

Table 2-3 Design Storm Event by Facility Type

Facility Type	Design Storm Event
New Storm Sewer (Residential)	5 year
New Storm Sewer (Commercial/Arterial Street)	10 year
Redevelopment of Existing Urban Area	Coordinate with the City
Culverts (Driveway)	10 year
Culverts (Roadway)	50 year
Overland Flow Routes	100 year
Bridges	100 year
Open Channels	100 year
Detention / Retention Basins	100 year

2.5 Rational Method

2.5.1 Introduction

The rational method can be used to estimate the design peak discharge for areas as large as 150 acres. This method, while first introduced in 1889, is still used in many engineering offices in the United States. Even though it has frequently come under criticism for its simplistic approach, no other drainage design method has received such widespread use.

2.5.2 Concept and Equation

The rational formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration (the time required for water to flow from the hydraulically most remote point of the basin to the location being analyzed). The rational formula is expressed as follows:

$$Q = CIA \tag{2.1}$$

where:
- Q = peak rate of runoff, cfs
- C = runoff coefficient representing a ratio of runoff to rainfall for future land-use conditions
- I = average rainfall intensity for a duration equal to the time of concentration, for a selected return period, in./hr (see Figure 2-3)
- A = drainage area tributary to the design location, acres

2.5.3 Application

Peak discharges estimated using the rational formula are very sensitive to the parameters that are used. The designer must use good engineering judgment in assigning values to these parameters. Each of the parameters used in the rational method is discussed below.

2.5.3.1 Time Of Concentration

The time of concentration (t_c) is the time required for water to flow from the hydraulically most remote point of the drainage area to the point under investigation. Use of the rational formula requires the time of concentration (t_c) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the rainfall intensity (I). For a storm drain system, the time of concentration consists of an inlet time plus the time of flow in a closed conduit or open channel to the design point. Inlet time is the time required for runoff to flow over the surface to the nearest inlet and is primarily a function of the length of overland flow, the slope of the land and surface cover. Pipe or open channel flow time can be estimated from the hydraulic properties of the conduit or channel. One way to estimate overland flow time is to use Figure 2-1 to estimate overland flow velocity and divide the velocity into the overland travel distance.

For design situations that do not involve complex drainage conditions, Figure 2-2 can be used to estimate inlet time. For each drainage area, the distance is determined from the inlet to the most remote point in the tributary area. From a topographic map, the average slope is determined for the same distance. The Coefficient of Runoff, C is determined by the procedure described in a subsequent section of this chapter.

To obtain the total time of concentration, the pipe or open channel flow time must be calculated and added to the inlet time. After first determining the average flow velocity in the pipe or channel, the travel time is obtained by dividing velocity into the pipe or channel length. Manning's equation can be used to determine velocity. See Chapter 5 - Open Channel Hydraulics - for a discussion of Manning's equation.

Time of concentration is an important variable in most hydrologic methods. Several methods are available for estimating t_c. Appendix 2-C (Travel Time Estimation) at the end of this chapter describes the method from the SCS Technical Release No. 55 (2nd Edition). Figure 2-2 shows the velocities used for estimating time of concentration for various land use conditions. For inlet design the minimum t_c recommended should not be less than 8 minutes.

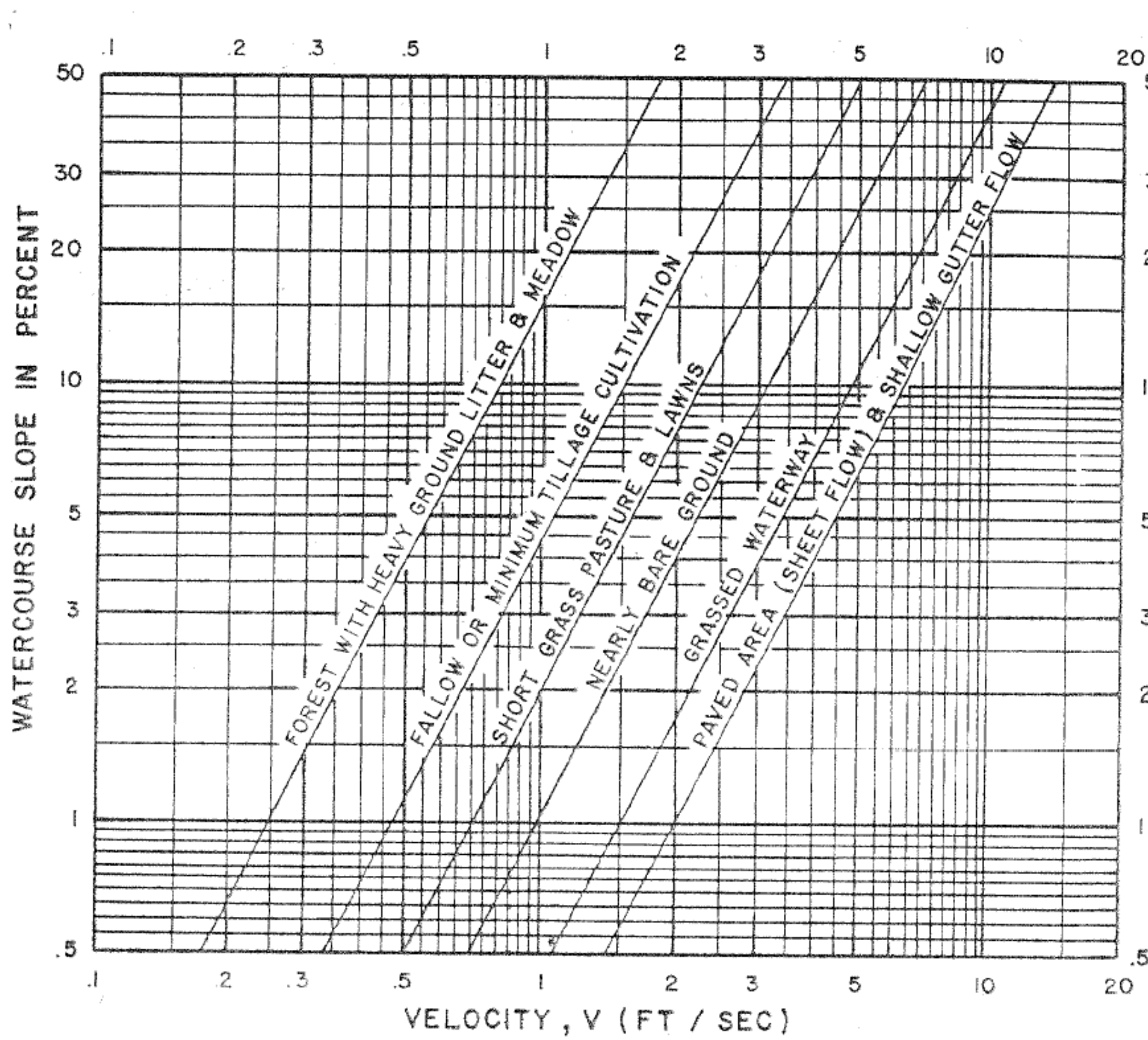

Figure 2-1 Velocities For Estimating Time Of Concentration

Source: HEC No. 19, FHWA

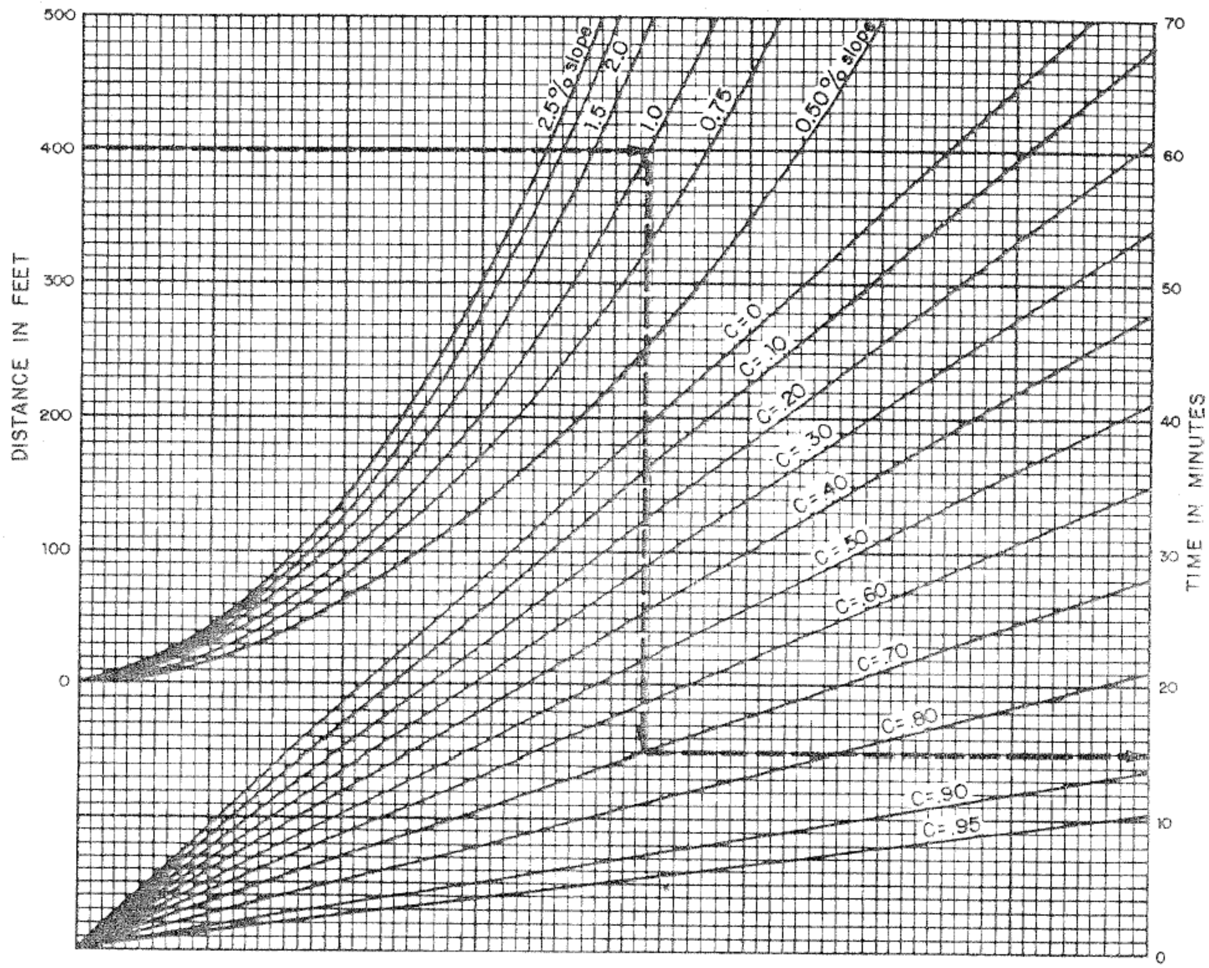

Figure 2-2 Overland Time Of Flow

Source: Airport Drainage, Federal Aviation Administration, 1965

2.5.3.1.1 Common Errors

Two common errors should be avoided when calculating t_c. First, in some cases runoff from a portion of the drainage area which is highly impervious may result in a greater peak discharge than would occur if the entire area were considered. In these cases, adjustments can be made to the drainage area by disregarding those areas where flow time is too slow to add to the peak discharge. Sometimes it is necessary to estimate several different times of concentration to determine the design flow that is critical for a particular application.

Second, when designing a drainage system, the overland flow path is not necessarily perpendicular to the contours shown on available mapping. Often the land will be graded and swales will intercept the natural contour and conduct the water to the streets, which reduces the time of concentration. Care should be exercised in selecting sheet flow paths in excess of 100 ft in urban areas and 300 ft in rural areas. Sheet flow conditions are not likely to be sustained for greater lengths and the estimated t_c will be too large.

2.5.3.2 Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate (in./hr) for a duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from Intensity-Duration-Frequency (IDF) curves. The data from the IDF curve for the City of Waverly are given in Figure 2-3.

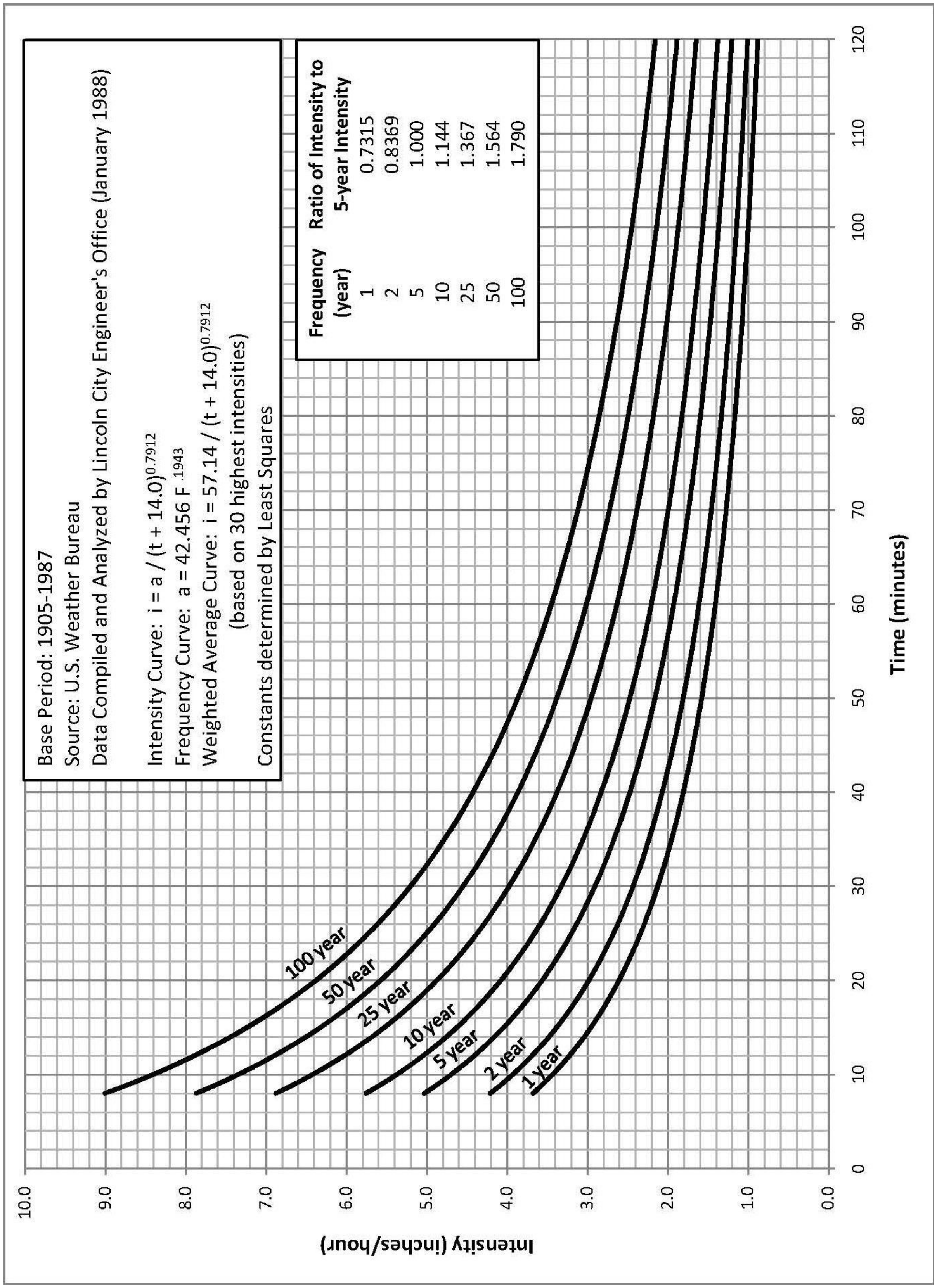

Figure 2-3 IDF Curves

2.5.3.3 Runoff Coefficient

The runoff coefficient (C) is the variable of the rational method least susceptible to precise determination and requires judgment and understanding on the part of the designer. Engineering judgment will always be required in the selection of runoff coefficients since a typical coefficient represents the integrated effects of many drainage basin parameters. The following discussion considers only the effects of soil groups, land use and average land slope.

The method for determining the runoff coefficient (C) is based on land use, soil groups and land slope. Table 2-4 in Manual gives the recommended coefficient C of runoff for pervious surfaces by selected hydrologic soil groupings and slope ranges. The value of C shall be based on fully built-out land use conditions. The minimum runoff coefficient shall be 0.4 , unless owner can clearly demonstrate that the value less then 0.4 is adequate.

Table 2-4 gives the recommended coefficient C of runoff for pervious surfaces by selected hydrologic soil groupings and slope ranges. From this table the C values for non-urban areas such as forest land, agricultural land, and open space can be determined. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Infiltration is the movement of water through the soil surface into the soil. Based on infiltration rates, the Soil Conservation Service (SCS) has divided soils into four hydrologic soil groups as follows:

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

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

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

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

A list of soils for the City of Waverly and their hydrologic classification is presented in the Lancaster County Soil Survey.

As the slope of the drainage basin increases, the selected C value should also increase. This is caused by the fact that as the slope of the drainage area increases, the velocity of overland and channel flow will increase, allowing less opportunity for water to infiltrate. Thus, more of the rainfall will become runoff from the drainage area.

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surface in the drainage area. Composites can be made with Tables 2-4 and 2-5. The composite procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide to selection of reasonable values of the coefficient for an entire area.

Table 2-4 Recommended Coefficient Of Runoff Values For Various Selected Land Uses

Description of Area	Runoff Coefficients
Business: Downtown areas	0.70-0.95
Neighborhood areas	0.50-0.70
Residential:	
Single-family areas	0.30-0.50
Multi units, detached	0.40-0.60
Multi units, attached	0.60-0.75
Suburban	0.25-0.40
Residential (1 acre lots or larger)	0.30-0.45
Apartment dwelling areas	0.50-0.70
Industrial:	
Light areas	0.50-0.80
Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25
Playgrounds	0.20-0.40
Railroad yard areas	0.20-0.40
Unimproved areas	0.04-0.38 (see Table 2-5)

Source: Hydrology, Federal Highway Administration, HEC No. 19, 1984

Table 2-5 Recommended Coefficient Of Runoff For Pervious Surfaces (Unimproved Areas) By Selected Hydrologic Soil Groupings And Slope Ranges

Slope	A	B	C	D
Flat (0 - 1%)	0.04-0.09	0.07-0.12	0.11-0.16	0.15-0.20
Average (2 - 6%)	0.09-0.14	0.12-0.17	0.16-0.21	0.20-0.25
Steep (Over 6%)	0.13-0.18	0.18-0.24	0.23-0.31	0.28-0.38

Source: Storm Drainage Design Manual, Erie and Niagara Counties Regional Planning Board.

2.5.3.3.1 Infrequent Storm

The coefficients given in Tables 2-4 and 2-5 are applicable for storms of 5-year to 10-year frequencies. Less frequent, higher intensity storms will require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff (Wright-McLaughlin, 1969). The adjustment of the rational method for use with major storms can be made by multiplying the right side of the rational formula by a frequency factor C_f. The rational formula now becomes:

$$Q = C_f CIA \qquad (2.1)$$

C_f values are listed in Table 2-6. The product of C_f times C shall not exceed 1.0.

Table 2-6 Frequency Factors For Rational Formula

Recurrence Interval (years)	C_f
up to 10	1.0
25	1.1
50	1.2
100	1.25

2.5.4 Limitations

Some precautions should be considered when applying the rational method.

- The first step in applying the rational method is to obtain a good topographic map and define the boundaries of the drainage area in question. A field inspection of the area should also be made to determine if the natural drainage divides have been altered.
- In determining the runoff coefficient (C) value for the drainage area, thought should be given to future changes in land use that might occur during the service life of the proposed facility that could result in an inadequate drainage system. Also, the effects of permanent upstream detention facilities may be taken into account.
- Restrictions to the natural flow such as highway crossings and dams that exist in the drainage area should be investigated to see how they affect the design flows.
- The charts, graphs and tables included in this section are not intended to replace reasonable and prudent engineering judgment which should permeate each step in the design process.

Characteristics of the rational method which limit its use to 150 acres include:

(1) The rate of runoff resulting from any rainfall intensity is a maximum when the rainfall intensity lasts as long or longer than the time of concentration. That is, the entire drainage area does not contribute to the peak discharge until the time of concentration has elapsed.

This assumption limits the size of the drainage basin that can be evaluated by the rational method. For large drainage areas, the time of concentration can be so large that constant rainfall intensities for such long periods do not occur and shorter, more intense rainfalls can produce larger peak flows.

(2) The frequency of peak discharges is the same as that of the rainfall intensity for the given time of concentration.

Frequencies of peak discharges depend on rainfall frequencies, antecedent moisture conditions in the watershed, and the response characteristics of the drainage system. For small and largely impervious areas, rainfall frequency is the dominant factor. For larger drainage basins and undeveloped drainage basins, the response characteristics control the frequencies of peak discharges. For drainage areas with few impervious surfaces (less urban development), antecedent moisture conditions usually govern, especially for rainfall events with a return period of 10 years or less.

(3) The fraction of rainfall that becomes runoff (C) is independent of rainfall intensity or volume.

This assumption is reasonable for impervious areas, such as streets, rooftops and parking lots. For pervious areas, the fraction of runoff varies with rainfall intensity and the accumulated volume of rainfall. Thus, the "art" necessary for application of the rational method involves the selection of a coefficient that is appropriate for the storm, soil and land use conditions. Many guidelines and tables have been established, but seldom, if ever, have they been supported with empirical evidence.

(4) The rational method provides estimates of only peak discharge rates of runoff. It does not provide information on the volume of runoff.

Modern drainage practice often includes detention of urban storm runoff to reduce the peak rate of runoff downstream. With only the peak rate of runoff, the rational method severely limits the evaluation of design alternatives available in urban and in some instances, rural drainage design.

Thus, the rational formula is best suited for small, highly impervious areas and least suitable for large drainage areas or drainage areas in natural or undeveloped conditions.

2.5.5 Example Problem - Rational Method

The following example problem illustrates the application of the rational method to estimate peak discharges. Preliminary estimates of the maximum rate of runoff are needed at the inlet to a culvert for a 10-year and 100-year return period.

<u>Site Data</u>

From a topographic map and field survey, the area of the drainage basin upstream from the culvert found to be 18 acres. In addition the following data were measured:

Length of overland flow = 50 ft
Average overland slope = 2.0%
Length of main basin channel = 1300 ft
Slope of channel = 0.018 ft/ft = 1.8%
Hydraulic radius = 1.97 ft
Estimated roughness coefficient (n) of channel = 0.090

Land Use And Soil Data

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

Residential (single family)	80%
Undeveloped (2% slope)	20%

For the undeveloped area, the soil group was determined from a SCS map to be:

Group B	100%

From existing land use maps, the land use for the overland flow area at the head of the basin was estimated to be:

Undeveloped (Soil Group B, 2.0% slope)	100%

Overland Flow

A runoff coefficient (C) for the overland flow area was determined to be 0.12 from Table 2-5.

Time Of Concentration

From Figure 2-2, with an overland flow length of 50 ft, slope of 2.0%, and a C of 0.12, the inlet time is 10 min. Channel flow velocity is determined from Manning's formula to be 3.5 ft/s (n = 0.090, R = 1.97 ft and S = 0.018 ft/ft). Therefore,

Flow Time = (1300 ft)/(3.5 ft/s)(60 s/min) = 6.2 min
and t_c = 10 + 6.2 = 16.2 min - say 16 min

Rainfall Intensity

From Figure 2-3 with duration equal to 16 min,

I_{10} (10-year return period) = 4.50 in./hr

I_{100} (100-year return period) = 7.05 in./hr

Runoff Coefficient

A weighted runoff coefficient C for the total drainage area is determined in Table 2-7 by utilizing the values from Tables 2-4 and 2-5.

Table 2-7 Weighted Runoff Coefficient, C

Land Use	(1) Percent of Total Land Area	(2) Weighted Runoff Coefficient	(3) Runoff Coefficient*
Residential (single family)	0.80	0.40	0.32
Undeveloped (Soil Group B)	0.20	0.12	0.02
Total Weighted Runoff Coefficient			0.34

* Column 3 equals column 1 multiplied by column 2.

Peak Runoff

From the rational equation:

$Q_{10} = CIA = 0.34 \times 4.50 \times 18 = 28$ cfs

$Q_{100} = C_fCIA = 1.25 \times 0.34 \times 7.05 \times 18 = 54$ cfs From Table 2-6

These are the estimates of peak runoff for a 10-year and 100-year design storm for the given basin.

2.6 SCS Unit Hydrograph Method

2.6.1 Introduction

Techniques developed by the U. S. Soil Conservation Service for calculating rates of runoff require the same basic data as the rational method: drainage area, a runoff factor, time of concentration and rainfall. The SCS approach, however, is more sophisticated in that it considers also the time distribution of the rainfall, the initial rainfall losses to interception and depression, storage and an infiltration rate that decreases during the course of a storm. With the SCS method, the direct runoff can be calculated for any storm, either real or fabricated, by subtracting infiltration and other losses from the rainfall to obtain the precipitation excess (runoff volume). Details of the methodology can be found in the SCS National Engineering Handbook, Section 4.

Two types of hydrographs are used in the SCS procedure, unit hydrographs and dimensionless hydrographs. A unit hydrograph represents the time distribution of flow resulting from one inch of direct runoff occurring over the watershed in a specified time. A dimensionless hydrograph represents the composite of many unit hydrographs. The dimensionless unit hydrograph is plotted in nondimensional units of time divided by time to peak and discharge divided by peak discharge.

Characteristics of the dimensionless hydrograph vary with the size, shape and slope of the tributary drainage area. The most significant characteristics affecting the dimensionless hydrograph shape are the basin lag and the peak discharge for a given rainfall. Basin lag is the time from the center of mass of rainfall excess to the hydrograph peak. Steep slopes, compact shape and an efficient drainage network tend to make lag time short and peaks high; flat slopes, elongated shape and an inefficient drainage network tend to make lag time long and peaks low.

2.6.2 Concepts and Equations

The following discussion outlines the basic concepts and equations utilized in the SCS method.

2.6.2.1 Rainfall-Runoff

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

$$\mathbf{Q = (P - I_a)^2 \quad / \quad (P - I_a) + S} \tag{2.2}$$

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

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

$$I_a = 0.2S \tag{2.3}$$

By substituting 0.2S for Ia in equation 2.3, the SCS rainfall-runoff equation becomes:

$$Q = (P - 0.2S)2 / (P + 0.8S) \tag{2.4}$$

S is related to the soil and cover conditions of the watershed through the curve number (CN) or runoff factor (See Section 2.6.3.1). CN has a range of 0 to 100, and S is related to CN by:

$$S = (1000 / CN) - 10 \tag{2.5}$$

Figure 2-4 is a graphical solution of equation 2.4 which enables the precipitation excess (runoff depth) from a storm to be obtained if the total rainfall and watershed curve number are known.

Drainage Area - The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas it might be necessary to divide the area into sub-drainage areas to account for major land use changes, to obtain analysis results at different points within the drainage area, or to locate stormwater drainage facilities and assess their effects on the flood flows. Also a field inspection of existing or proposed drainage systems should be made to determine if the natural drainage divides have been altered. These alterations could make significant changes in the size and slope of the subdrainage areas.

Rainfall - The SCS method is based on a 24-hr storm event with various time distributions, depending on the watershed location. The Type II storm distribution is a "typical" time distribution which the SCS has prepared from rainfall records and can be used in Waverly, Nebraska. Figure 2-5 shows this distribution. To use this distribution it is necessary for the user to obtain the 24-hr duration rainfall value for the frequency of the design storm desired from the Table 2-8.

Table 2-8 City Of Waverly 24-Hour Design Rainfall

Frequency	24-hour Rainfall	Frequency	24-hour Rainfall
2-year	3.00 in.	25-year	5.37 in.
5-year	3.93 in.	50-year	6.00 in.
10-year	4.69 in.	100-year	6.68 in.

Source: National Weather Service, Tech. Paper 40, “Rainfall Frequency Atlas of the U.S.”, May 1961.

2.6.2.2 Time Of Concentration

The average slope within the watershed together with the overall length and retardance of overland flow are the major factors affecting the runoff rate through the watershed. In the SCS method, time of concentration (t_c) is defined to be the time required for water to travel from the most hydraulically distant point in a watershed to its outlet. Lag (L) can be considered as a weighted time of concentration and is related to the physical properties of a watershed, such as area, length and slope. The SCS derived the following empirical relationship between lag and time of concentration:

$$L = 0.6\, t_c \tag{2.6}$$

See Appendix 2-C for information on the derivation of t_c.

In small urban areas (less than 2000 acres), a curve number method can be used to estimate the time of concentration from watershed lag. In this method the lag for the runoff from an increment of excess rainfall can be considered as the time between the center of mass of the excess rainfall increment and the peak of its incremental outflow hydrograph. The equation developed by SCS to estimate lag is:

$$\mathbf{L = (l^{0.8}\,(S + 1)^{0.7}) \,/\, (1900\ Y^{0.5})} \qquad \mathbf{(2.7)}$$

Where:
- L = lag, hrs
- l = length of mainstream to farthest divide, ft
- Y = average slope of watershed, %
- S = (1000/CN) - 10
- CN = SCS curve number

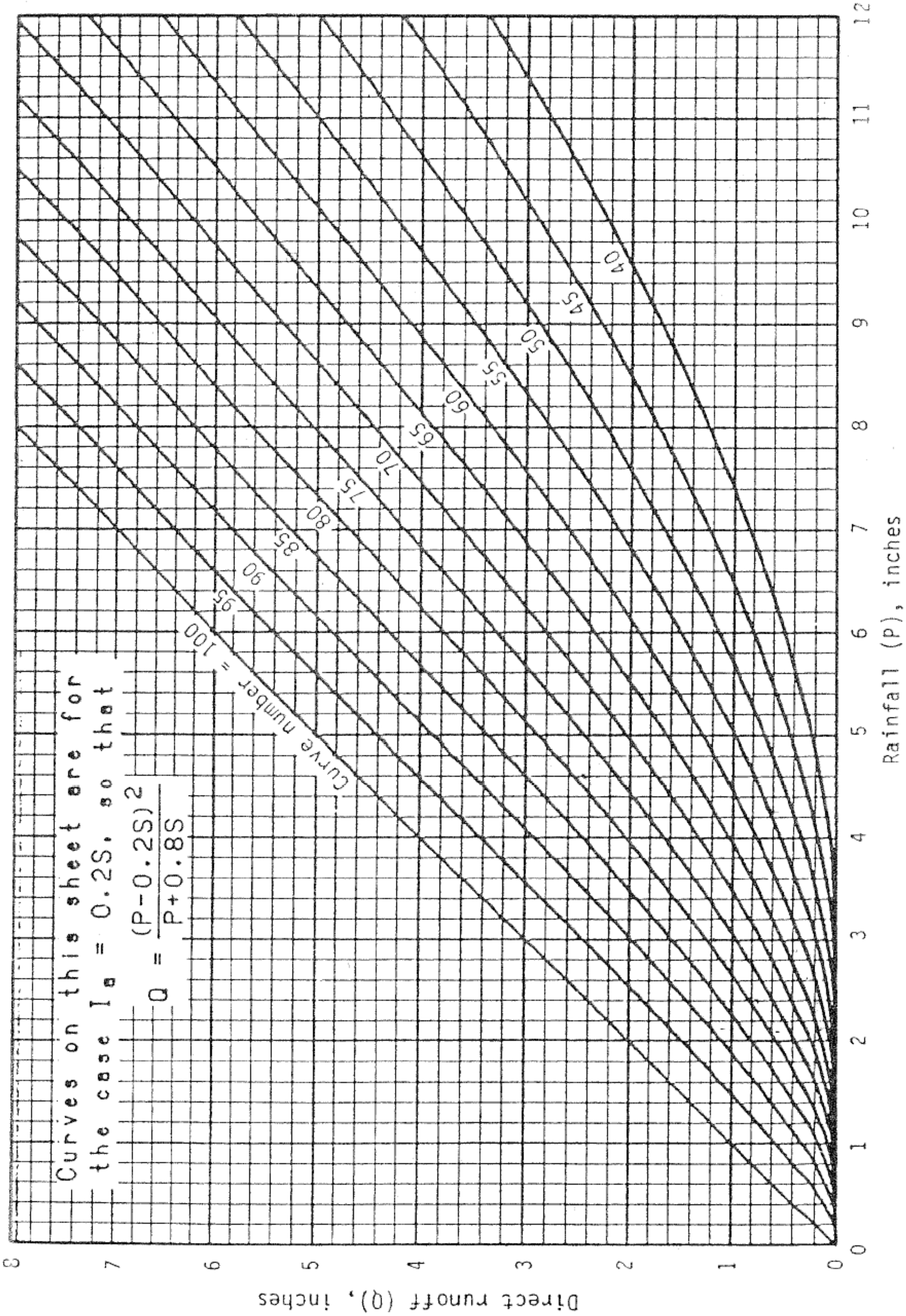

Figure 2-4 SCS Relation Between Direct Runoff, Curve Number And Precipitation

Source: HEC 19

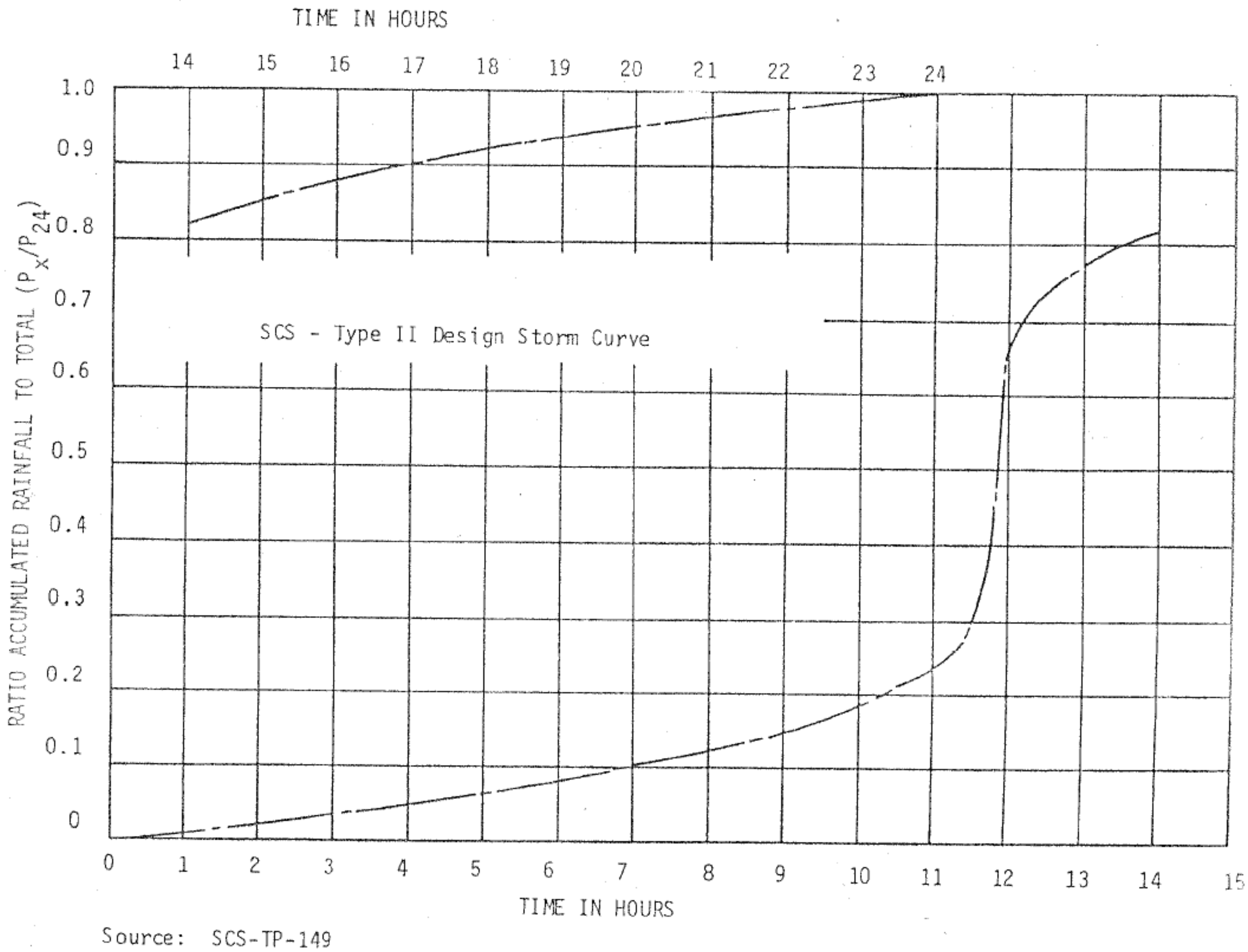

Figure 2-5 Type II Design Storm Curve

The lag time can be corrected for the effects of urbanization by using Figures 2-6 and 2-7. The amount of modifications to the hydraulic flow length usually must be determined from topographic maps or aerial photographs following a field inspection of the area. The modification to the hydraulic flow length not only includes pipes and channels but also the length of flow in streets and driveways.

After the lag time is adjusted for the effects of urbanization, the above equation that relates lag time and time of concentration can be used to calculate the time of concentration for use in the SCS method. Appendix 2-c presents an alternate procedure for travel time and time of concentration estimation.

2.6.2.3 Triangular Hydrograph Equation

The triangular hydrograph is a practical representation of excess runoff with only one rise, one peak and one recession. Its geometric makeup can be easily described mathematically, which makes it very useful in the processes of estimating discharge rates. The SCS developed the following equation to estimate the peak rate of discharge for an increment of runoff:

$$\mathbf{q_p = (484\ A\ (q\ /\ (d/2 + L)))} \qquad \mathbf{(2.8)}$$

Where:
- q_p = peak rate of discharge, cfs
- A = area, mi^2
- q = storm runoff during time interval, in.
- d = time interval, hrs
- L = watershed lag, hrs

This equation can be used to estimate the peak discharge for the unit hydrograph which can then be used to estimate the peak discharge and hydrograph from the entire watershed.

The constant 484, or peak rate factor, is valid for the SCS dimensionless unit hydrograph. Any change in the dimensionless unit hydrograph reflecting a change in the percent of volume under the rising side would cause a corresponding change in the shape factor associated with the triangular hydrograph and therefore a change in the constant 484. This constant has been known to vary from about 600 in steep terrain to 300 in very flat, swampy country.

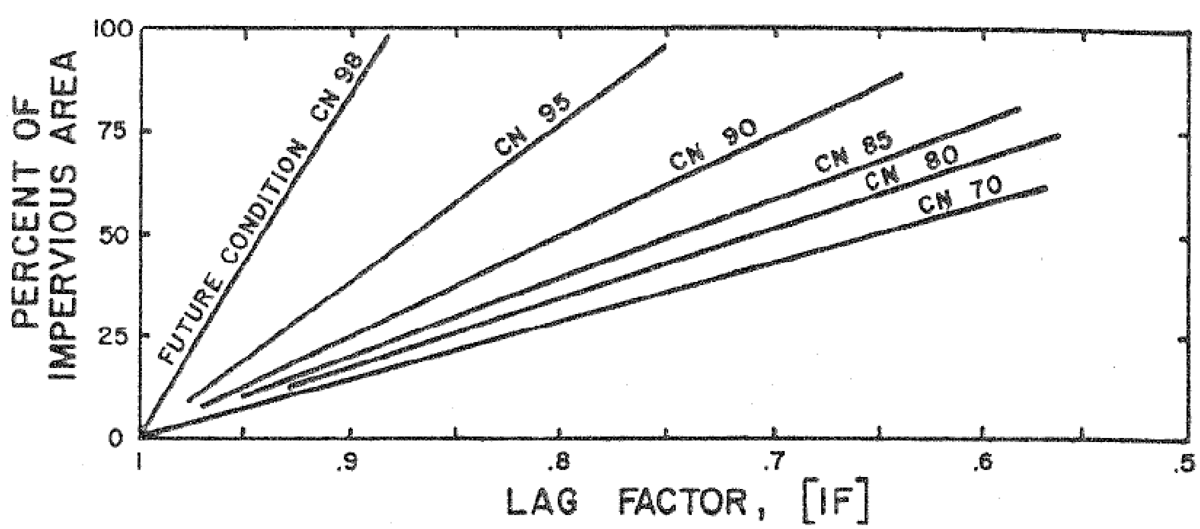

Figure 2-6 Factors For Adjusting Lag When Impervious Areas Occur In Watershed

Source: HEC-19

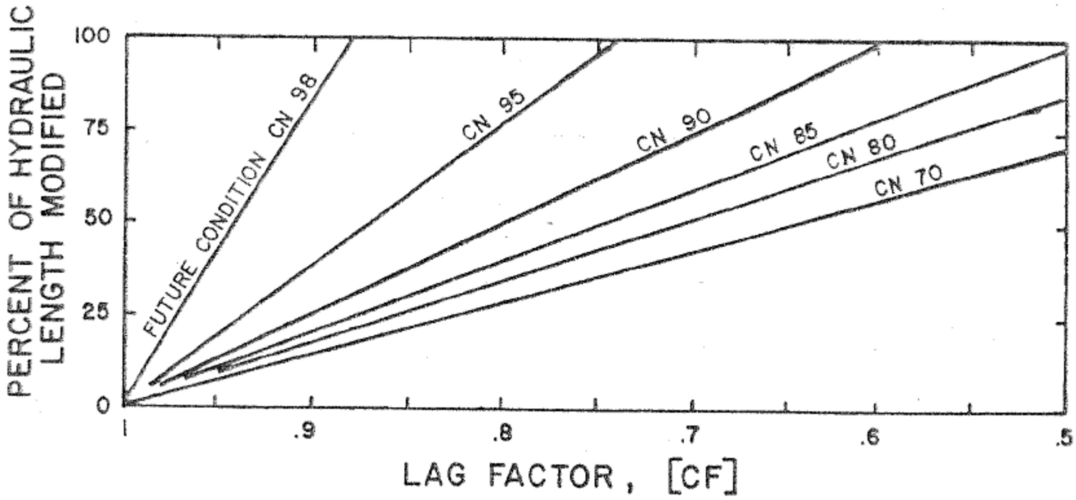

Figure 2-7 Factors For Adjusting Lag When The Main Channel Has Been Hydraulically Improved

Source: HEC 19

2.6.3 Application

The following discussion describes the procedures used in the SCS unit hydrograph method along with recommended design aids.

2.6.3.1 Runoff Factor

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

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

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

Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. The SCS has divided soils into four hydrologic soil groups based on infiltration rates (Groups A, B, C and D). These groups were previously described for the rational method. Refer to Lancaster County Soil Survey.

Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. Also, runoff curve numbers vary with the antecedent soil moisture conditions, defined as the amount of rainfall occurring in a selected period preceding a given storm. In general, the greater the antecedent rainfall, the more direct runoff there is from a given storm. A 5-day period is used as the minimum for estimating antecedent moisture conditions.

The following pages give a series of tables related to runoff factors. The first tables (Tables 2-9 - 2-11) give curve numbers for various land uses. These tables are based on an average antecedent moisture condition, i.e., soils that are neither very wet nor very dry when the design storm begins. Curve numbers should be selected only after a field inspection of the watershed and a review of zoning and soil maps. Table 2-12 gives conversion factors to convert average curve numbers to wet and dry curve numbers. Table 2-13 gives the antecedent conditions for the three classifications.

Table 2-9 Runoff Curve Numbers - Urban Areas[1]

Cover Type and Hydrologic Condition	Average Percent Impervious Area[2]	Curve numbers for hydrologic soil groups A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]					
Poor condition (grass cover <50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Urban districts:					
Commercial and business	85%	89	92	94	95
Industrial	72%	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65%	77	85	90	92
1/4 acre	38%	61	75	83	87
1/3 acre	30%	57	72	81	86
1/2 acre	25%	54	70	80	85
1 acre	20%	51	68	79	84
2 acres	12%	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94
Idle lands (CNs are determined using cover types similar to those in Table 2-11).					

[1] Average runoff condition, and $I_a = 0.2S$

[2] The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the SCS method has an adjustment to reduce the effect.

[3] CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

Source: TR-55

Table 2-10 Cultivated Agricultural Land[1]

Cover Type	Treatment	Hydrologic Condition	Curve numbers for hydrologic soil groups A	B	C	D
Fallow	Bare soil	-	77	86	91	94
	Crop residue	Poor	76	85	90	93
	cover (CR)	Good	74	83	88	90
Row	Straight row (SR)	Poor	72	81	88	91
Crops		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured &	Poor	66	74	80	82
	terraced (C&T)	Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
	Small grain (SR)	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
	Close-seeded (SR)	Poor	66	77	85	89
	or broadcast	Good	58	72	81	85
	Legumes or C	Poor	64	75	83	85
	Rotation	Good	55	69	78	83
	Meadow C&T	Poor	63	73	80	83
		Good	51	67	76	80

[1] Average runoff condition, and $I_a = 0.2S$.

[2] Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.

[3]Hydrologic condition is based on a combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or closed-seeded legumes in rotations, (d) percent of residue cover on the land surface (good > 20%) and (e) degree of roughness.

Poor: Factors impair infiltration and tend to increase runoff.

Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

Source: TR-55

Table 2-11 Other Agricultural Lands[1]

Cover Type	Hydrologic Condition	Curve numbers for hydrologic soil groups A	B	C	D
Pasture, grassland, or range-continuous forage for grazing[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay	—	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	[4]30	48	65	73
Woods—grass combination (orchard or tree farm)[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods[6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	[4]30	55	70	77
Farmsteads—buildings, lanes, driveways and surrounding lots	—	59	74	82	86

[1] Average runoff condition, and $I_a = 0.2S$

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

[3] Poor: < 50% ground cover
Fair: 50 to 75% ground cover
Good: > 75% ground cover

[4] Actual curve number is less than 30; use CN = 30 for runoff computations.

[5] CNs shown were computed for areas with 50% grass (pasture) cover. Other combinations of conditions may be computed from CNs for woods and pasture.

[6] Poor: Forest litter, small trees and brush are destroyed by heavy grazing or regular burning.
Fair: Woods grazed but not burned, and some forest litter covers the soil.
Good: Woods protected from grazing, litter and brush adequately cover soil.

Source: TR-55

Table 2-12 Conversion From Average Antecedent Moisture Conditions To Dry And Wet Conditions

CN For Average Conditions	Corresponding CNs For Dry	Corresponding CNs For Wet
100	100	100
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
60	40	78
55	35	74
50	31	70
45	26	65
40	22	60
35	18	55
30	15	50
25	12	43
15	6	30
5	2	13

Source: USDA Soil Conservation Service TP-149 (SCS-TP-149), "A Method for Estimating Volume and Rate of Runoff in Small Watersheds," revised April 1973.

Table 2-13 Rainfall Groups For Antecedent Soil Moisture Conditions During Growing And Dormant Seasons

Antecedent Condition	Conditions Description	Growing Season 5-day Antecedent Rainfall	Dormant Season 5-day Antecedent Rainfall
Dry	An optimum condition of watershed soils, where soils are dry but not to the wilting point and when satisfactory plowing or cultivation takes place	Less than 1.4 in.	Less than 0.5 in.
Average	The average case for annual floods	1.4 - 2.1 in.	0.5 - 1.1 in.
Wet	When a heavy rainfall, or light rainfall and low temperatures, have occurred during the five days previous to a given storm	Over 2.1 in.	Over 1.1 in.

Source: Soil Conservation Service

2.6.4 Limitations

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for urban areas. For example, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

The curve number values given in Table 2-9 are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over a pervious area and then into a drainage system. It is possible that curve number values from urban areas could be reduced by not directly connecting impervious surfaces to the drainage system. For a discussion of impervious areas and their effect on curve number values, see Appendix 2-B at the end of this chapter.

2.7 Simplified SCS Method

2.7.1 Introduction

The following SCS procedures were taken from the SCS Technical Release 55 (TR-55) which presents simplified procedures to calculate storm runoff volume, peak rate of discharges and hydrographs. These procedures allow manual calculation of hydrologic parameters. HEC-HMS performs the same calculations when the SCS methodology is selected within the software package. These procedures are applicable to small drainage areas and include provisions to account for urbanization. The following procedures outline the use of the SCS-TR 55 method.

2.7.2 Concepts and Equations - Peak Discharge Method

The SCS peak discharge method is applicable for estimating the peak run-off rate from watersheds with homogeneous land uses. The following method is based on the results of computer analyses performed using TR-20, "Computer Program for Project Formulation - Hydrology" (SCS 1983).

The peak discharge equation is:

$$\mathbf{Q_p = q_u A Q F_p} \qquad \textbf{(2.9)}$$

Where:
- Q_p = peak discharge (cfs)
- q_u = unit peak discharge (cfs/mi^2/in.)
- A = drainage area (mi^2)
- Q = runoff (in.)
- F_p = pond and swamp adjustment factor

The input requirements for this method are as follows:

1. Time of concentration, T_c (hours)
2. Drainage area (mi^2)
3. Type II rainfall distribution
4. 24-hour design rainfall
5. CN value
6. Pond and swamp adjustment factor (If pond and swamp areas are spread throughout the watershed and are not considered in the T_c computation, an adjustment is needed.)

Computations for the peak discharge method proceed as follows:

1. The 24-hour rainfall depth is determined from Table 2-8.

2. The runoff curve number, CN, is estimated from Table 2-9 through 2-11 and direct runoff, Q, is calculated using equation 2.4.

3. The CN value is used to determine the initial abstraction, I_a, from Table 2-14, and the ratio I_a/P is then computed. (P = accumulated rainfall or potential maximum runoff.)

4. The watershed time of concentration is computed using the procedures in Section 2.6.2.2 and is used with the ratio I_a/P to obtain the unit peak discharge, qu, from Figure 2-8 or the method given in Appendix 2-C. If the ratio I_a/P lies outside the range shown in Figure 2-8, either the limiting values or another peak discharge method should be used.

5. The pond and swamp adjustment factor, F_p, is estimated from the following information:

Pond & Swamp Areas (%)	F_p	Pond & Swamp Areas (%)	F_p
0	1.00	3.0	0.75
0.2	0.97	5.0	0.72
1.0	0.87		

6. The peak runoff rate is computed using equation 2.9.

Table 2-14 I_a Values For Runoff Curve Numbers

Curve Number	I_a (in)	Curve Number	I_a (in)
40	3.000	70	.857
41	2.878	71	.817
42	2.762	72	.778
43	2.651	73	.740
44	2.545	74	.703
45	2.444	75	.667
46	2.348	76	.632
47	2.255	77	.597
48	2.167	78	.564
49	2.082	79	.532
50	2.000	80	.500
51	1.922	81	.469
52	1.846	82	.439
53	1.774	83	.410
54	1.704	84	.381
55	1.636	85	.353
56	1.571	86	.326
57	1.509	87	.299
58	1.448	88	.273
59	1.390	89	.247
60	1.333	90	.222
61	1.279	91	.198
62	1.226	92	.174
63	1.175	93	.151
64	1.125	94	.128
65	1.077	95	.105
66	1.030	96	.083
67	.985	97	.062
68	.941	98	.041
69	.899		

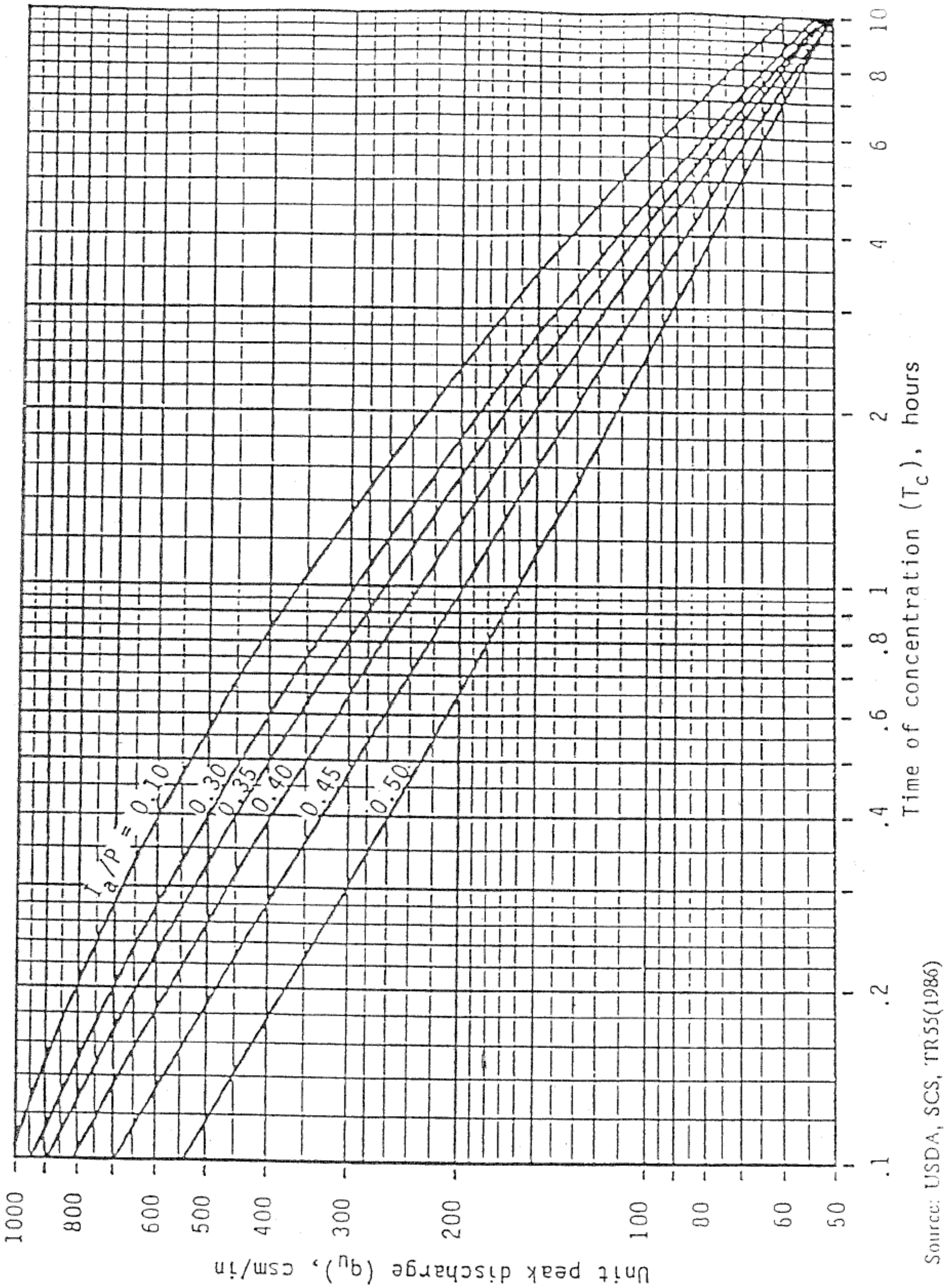

Figure 2-8 SCS Type II Unit Peak Discharge Graph

2.7.3 Limitations

The accuracy of the peak discharge method is subject to specific limitations, including the following.

1. The watershed must be hydrologically homogeneous and describable by a single/composite CN value.

2. The watershed may have only one main stream, or if more than one, the individual branches must have nearly equal time of concentrations.

3. Hydrologic routing cannot be considered.

4. The pond and swamp adjustment factor, Fp, applies only to areas located away from the main flow path.

5. Accuracy is reduced if the ratio Ia/P is outside the range given in Figure 2-7.

6. The weighted CN value must be greater than or equal to 40 and less than or equal to 98.

7. The same procedure should be used to estimate pre- and post-development time of concentration when computing pre- and post-development peak discharge.

8. The watershed time of concentration must be between 0.1 and 10 hours.

2.7.4 Example Problem

Compute the 25-year peak discharge for a 50-acre wooded watershed which will be developed as follows:

1. Forest land - good cover (hydrologic soil group B) = 10 ac.
2. Forest land - good cover (hydrologic soil group C) = 10 ac.
3. Town house residential (hydrologic soil group B) = 20 ac.
4. Industrial development (hydrological soil group C) = 10 ac.

Other data include:

Percentage of pond and swamp area = 0.

The hydrologic flow path for this watershed = 1,920 ft.

Segment	Type of Flow	Length	Slope (%)
1	Overland (n = .45)	70 ft.	2.0 %
2	Shallow channel	750 ft.	1.7 %
3	Main channel*	1100 ft.	0.20 %

* For the main channel, n = .025, width = 10 feet, depth = 2 feet, rectangular channel.

<u>Computations</u>

1. Calculate rainfall excess:

The 25-year, 24-hour rainfall for Waverly, Nebraska is 5.37 inches (see Table 2-8).

Composite weighted runoff coefficient is:

Dev. #	Area	% Total	CN	Composite CN
1	10 ac.	.20	55	11.0
2	10 ac.	.20	70	14.0
3	20 ac.	.40	85	34.0
4	10 ac.	.20	91	18.2
Total	50 ac.	1.00		77.2, use 77

2. Calculate time of concentration (Note: use the method outlined in Appendix 2-C.)

Segment 1 - Travel time from equation 2.C.3 with P2 = 3.00 in.

$T_t = [0.42 (0.45 \times 70)0.8] / [(3.00)0.5 (.02)0.4]$
$T_t = 18.3$ minutes

Segment 2 - Travel time from equation 2.C.5 and equation 2.C.1

V = 2.7 ft/sec (equation 2.C.5)

$T_t = 750 / 60 (2.7) = 4.6$ minutes

Segment 3 - Using equation 2.C.6 and equation 2.C.1

V = (1.49/.025) (1.43)0.67 (.002)0.5 = 3.4 ft/sec
$T_t = 1100 / 60 (3.4) = 5.4$ minutes

$T_c = 18.3 + 4.6 + 5.4 = 28.3$ minutes (.47 hours)

3. Calculate I_a/P

For CN = 77, $I_a = .597$ (Table 2-14)

$I_a/P = (.597 /5.37) = .111$
(Note: Use $I_a/P = .10$ to facilitate use of Figure 2-8.

4. Estimate unit discharge q_u from Figure 2-8 = 550 cfs/mi^2/in

5. Calculate peak discharge with $F_p = 1$ using equation 2.9

From Figure 2-4 (or equation 2.4), Q = 2.9 inches
$Q_{25} = 550 (50/640) (2.9) (1) = 125$ cfs.

2.7.5 Hydrograph Generation

In addition to estimating the peak discharge, the SCS method can be used to estimate the entire hydrograph. The Soil Conservation Service has developed a tabular hydrograph procedure which can be used to generate the hydrograph for small drainage areas. The tabular hydrograph procedure uses unit discharge hydrographs which have been generated for a series of times of concentrations.

The tables in Appendix 2-A at the end of this chapter give the unit discharges (csm/in) for different times of concentration which are applicable to the City of Waverly. The values that should be used are those with a travel time equal to zero. The other travel times indicate the unit hydrographs which would result if the hydrographs were routed through a channel system for a length of time equal to the travel time. Thus, using these unit hydrographs would account for the effects of channel routing. Straight line interpolation can be used for time of concentrations and travel times between the values given in the appendix.

2.7.6 Composite Hydrograph

The procedures given in this chapter are for generation of a hydrograph from a homogeneous developed drainage area. For drainage areas which are not homogeneous, hydrographs need to be generated from sub-areas and then routed and combined at a point downstream. To accomplish this, engineers should refer to the procedures outlined by the SCS in the 1986 version of TR-55 available from the National Technical Information Service in Springfield, Virginia or www.usda.nrcs.gov. The catalog number for TR-55, "Urban Hydrology for Small Watersheds," is PB87-101580.

2.7.7 Hydrograph Computation

For the example problem in 2.7.4, calculate the entire hydrograph from the 50 acre development.

Using the chart in Appendix 2-A with a time of concentration of 0.47 hours and $I_a/P = 0.10$, the following hydrograph can be generated (using straight line interpolation between time of concentration of .4 and .5 hours).

The values given in the charts are in csm/in or cubic feet per second per square mile per inch of runoff. Thus, for this example all values from the chart must be multiplied by 0.078 (50 acres/640 acres per square mile), 2.9 inches of runoff, and 1 for the ponding factor - (50/640)(2.9)(1) = 0.23

As an example, from the chart in Appendix 2-A with $T_c = 0.47$ hours and $I_a/P = 0.10$, the unit discharge at time 12.1 hours is 200 csm/in. Thus, the ordinate on the hydrograph for this example would be 200(.23) = 46 cfs. This calculation must be done for all hydrograph values. The results for selected time values are given in Table 2-15.

Table 2-15 Hydrograph Calculation Results for Selected Time Values

*Hydrograph Time (hours)	Unit Discharge (csm/in)	Hydrograph (cfs)
11.0	17	4
11.3	23	5
11.6	33	8
11.9	63	14
12.0	108	25
12.1	200	46
12.2	359	83
12.3	505	116
12.4	544	125
12.5	484	111
12.6	371	85
12.7	273	63
12.8	207	48
13.0	129	30
13.2	91	21
13.4	71	16
13.6	59	14
13.8	52	12
14.0	46	11
14.3	40	9
14.6	36	8
15.0	32	7
15.5	29	7
16.0	26	6

* Note skips in time increments.

2.8 Hydrologic Computer Modeling

2.8.1 Introduction

Hydrologic computer models are in widespread use. They are becoming more “user-friendly”, more capable and flexible, and usually provide “report-ready” output. However, a model’s real utility is in monitoring changes in the watershed or asking “what if” questions. For example, what happens to the 10-year peak discharge as a portion of the watershed becomes urbanized? Or, alternatively, can the peak discharge be reduced substantially with a strategically placed detention pond? Many hydrologic models will allow one to:

- quantify urban runoff (peaks, volumes, and in some cases, water quality),
- obtain design information (channels, pipes, reservoirs, etc.),
- determine the effects of control options (infiltration devices, retention ponds, etc.),
- perform frequency analysis, and
- provide input to economic models.

HEC-HMS (a nonproprietary model written by the U.S. Army Corps of Engineers) has been selected for use in Waverly.

As you begin to use hydrologic computer models, keep in mind the memorable cliché: “Computers are fast, accurate, and stupid. People are slow, inaccurate, and brilliant. The combination is an opportunity beyond imagination.” However, one needs to remain “brilliant” by studying the underlying algorithms these models use. If one knows their limitations, he or she can use computer models wisely.

2.8.2 Concepts and Equations

Modern hydrologic models generally require the user to assemble watershed elements on the computer screen in a link-node structure. That is, nodes represent sub-basins (sub-watersheds), confluences (junctions, manholes, etc.), channels/pipes, and reservoirs. These nodes are "linked" together in an arrangement that depicts how runoff passes through the watershed.

Mathematical algorithms are associated with each node. For example, a sub-basin node will require certain information from the user in order to generate a runoff hydrograph. Rainfall is a necessary input. The user will also be required to input items like area, curve number, slope, etc. With this information, the model uses internal algorithms to compute a runoff hydrograph and sends it to the next downstream element. If this element is a channel/pipe node, other data will be required to route the hydrograph to the next element. Reservoir nodes also perform routing computations. A confluence node combines two or more hydrographs from upstream sub-basins, channels/pipes, and/or reservoirs. The hydrograph(s) continue to move downstream through all of the watershed elements.

SCS procedures are embedded in most hydrologic models. HEC-HMS allow the user to model watersheds with SCS methodology. Therefore, the concepts and equations mentioned previously in this chapter are still appropriate. These include the 24-hour storm, SCS rainfall distributions (like the Type II appropriate for Waverly), the curve number method for allocating rainfall losses, and the SCS unit hydrograph procedure.

2.8.3 Application

The application of a good hydrologic model is not complicated, particularly if you have a good background in hydrology and a basic understanding of the underlying algorithms used by the model. The step-by-step modeling procedure listed below is typical of most modern hydrologic models. Of course, the sequence of steps taken and the particular data requested are dependent upon the model used and the solution methodology (algorithms) chosen.

The step-by-step modeling procedure is likely to progress as follows:

- Launch the model and name your new file.
- Choose a system of units, give the project a title, and insert project comments.
- Build a watershed schematic (link/node) using the elements provided on the “tool palette.”
- Choose a solution methodology (e.g., SCS) for individual watershed elements.
- Input requested data (e.g., rainfall, curve number, etc.) for each watershed element.

- Add any remaining general data (e.g., time step) and run the model.
- Interrogate individual elements from the watershed schematic for output (e.g., hydrographs).
- Evaluate the output data based on sound engineering judgment.
- Use the conclusions to determine estimates to the model for reliable output.

2.8.4 Limitations

Hydrologic models are subject to the same limitations as their underlying algorithms. For example, if SCS modeling procedures are utilized, the precautions and limitations mentioned in section 2.6.4 still apply. The major limitations of the SCS methodology are listed below.

- Curve numbers describe average conditions, particularly with regard to antecedent moisture conditions. Since a watershed or sub-watershed is described by one CN value, it should be delineated (to the extent feasible) such that it is hydrologically homogeneous. (See section 2.7.4 on weighted curve numbers.)
- Initial abstractions are assumed to be 20% of a basin's potential losses.
- Runoff from snowfall or frozen ground cannot be accounted for using SCS procedures.
- SCS procedures account for surface runoff only, not interflow or groundwater contribution.

Since many hydrologic procedures contain empirical parameters, the processes of calibration and verification can be very useful in improving model accuracy. These processes require measured rainfall and runoff data from historical events. Calibration requires that a watershed be modeled using rainfall information from a number of historical storms. Certain empirical parameters are adjusted in the process so that the modeled output matches the measured output. Verification follows calibration. Using completely different historical rainfall information (not the same storms used for calibration), the model is run again with the adjusted empirical parameters to determine the accuracy of the results. If the modeled runoff from these new storms closely matches the measured runoff, the model is assumed to be "verified." The process of calibration and verification is highly desirable and increases confidence in the results of a hydrologic model.

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APPENDIX 2-A

SCS UNIT DISCHARGE HYDROGRAPHS

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

HYDROGRAPH TIME (HOURS)

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

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
0.0	24	34	53	334	647	1010	623	217	147	123	104	86	76	66	57	51	46	42	38	34	32	29	26	23	21	20	19	18	15	13	12	0
.10	21	29	43	134	267	520	847	701	378	224	157	122	98	75	64	56	50	45	41	36	33	30	27	24	21	20	19	18	16	13	12	0
.20	18	25	35	61	110	215	418	704	702	486	312	209	151	94	73	62	54	49	44	33	34	31	28	25	22	21	19	18	16	14	12	0
.30	17	23	33	56	92	174	337	582	662	545	389	269	190	109	79	65	56	50	45	39	35	32	29	25	22	21	20	19	16	14	12	0
.40	15	20	28	41	51	78	142	272	478	601	563	447	328	172	104	76	63	55	49	42	37	33	29	26	23	21	20	19	17	14	12	0
.50	14	19	26	39	47	68	117	220	392	531	553	482	380	209	121	84	67	57	51	43	38	33	30	27	23	21	20	19	17	14	12	0
.75	12	15	21	29	33	33	49	73	126	224	343	432	464	385	252	156	103	76	62	50	43	36	31	28	25	22	21	19	17	15	12	0
1.0	9	12	15	21	23	26	29	33	40	55	86	148	238	406	434	317	205	130	89	62	50	41	34	30	27	24	22	20	18	16	12	0
1.5	7	8	10	14	15	16	18	20	22	25	29	34	45	101	220	339	373	720	234	131	80	53	40	34	30	27	24	21	19	17	12	2
2.0	4	6	7	9	9	10	11	12	13	15	16	18	23	25	37	72	150	252	336	312	216	109	58	42	34	30	27	24	20	18	13	8
2.5	3	4	5	6	7	7	8	8	9	10	11	12	13	16	19	25	39	75	142	262	308	229	108	58	41	34	30	27	22	19	14	11
3.0	1	2	3	4	4	5	5	6	6	7	7	8	8	10	12	14	17	22	31	76	169	288	236	122	64	43	35	30	24	20	16	11

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

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
0.0	0	0	0	154	568	536	924	217	172	149	126	107	97	85	76	69	63	58	53	48	46	42	38	34	31	30	28	27	24	20	19	0
.10	0	0	0	19	109	415	762	603	346	230	176	143	119	95	84	74	6d	62	57	50	47	44	40	35	32	30	29	27	24	21	19	0
.20	0	0	0	0	13	77	302	609	605	432	297	217	167	115	94	81	73	66	60	53	48	45	41	37	33	31	29	28	25	21	19	0
.30	0	0	0	0	9	54	219	479	563	476	357	263	199	129	99	85	75	68	62	54	49	45	41	37	33	31	29	23	25	21	19	0
.40	0	0	0	0	0	6	38	159	372	500	484	399	309	183	123	96	82	73	66	58	51	46	42	38	34	31	30	28	25	22	19	0
.50	0	0	0	0	0	4	27	115	287	429	465	421	346	213	138	103	86	76	68	59	52	47	43	39	34	32	30	29	25	22	19	0
.75	0	0	0	0	0	0	1	10	46	132	246	338	3e1	341	243	165	119	94	80	67	58	50	45	41	37	33	31	29	26	23	19	0
1.0	0	0	0	0	0	0	0	1	4	22	69	149	241	357	331	246	170	122	96	76	64	54	47	42	38	34	32	30	27	24	19	0
1.5	0	0	0	0	0	0	0	0	0	0	0	1	4	41	142	258	310	285	224	142	97	71	55	47	43	39	35	32	29	25	20	4
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	10	49	130	221	279	255	182	108	70	55	47	42	38	34	30	27	20	11
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	14	52	119	224	256	193	107	70	55	47	42	38	32	28	22	17
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	52	141	240	199	117	74	56	48	43	35	30	24	18

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

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
0.0	0	0	0	0	70	539	377	196	171	154	134	117	108	99	89	83	77	72	67	61	59	56	51	46	43	42	40	38	34	30	28	0
.10	0	0	0	0	47	375	376	256	199	169	146	126	114	102	92	85	79	73	68	62	59	56	52	47	43	42	40	38	34	30	28	0
.20	0	0	0	0	0	31	260	338	283	227	189	160	178	112	99	90	83	77	72	64	60	57	53	48	44	42	41	39	35	30	28	0
.30	0	0	0	0	0	0	21	180	285	284	246	208	176	131	110	97	88	82	76	68	62	59	54	50	45	43	41	39	36	31	28	0
.40	0	0	0	0	0	0	14	125	232	266	253	223	192	142	115	100	91	83	77	69	63	59	55	50	45	43	41	40	36	31	28	0
.50	0	0	0	0	0	0	9	86	183	239	248	231	205	154	122	104	93	85	79	71	64	59	55	51	46	43	41	40	36	32	28	0
.75	0	0	0	0	0	0	3	31	87	147	190	211	213	184	147	121	103	92	84	75	67	61	57	52	47	44	42	40	37	32	28	0
1.0	0	0	0	0	0	0	0	0	1	13	45	92	141	205	197	165	134	112	98	84	75	65	59	55	50	46	43	41	38	34	28	0
1.5	0	0	0	0	0	0	0	0	0	0	0	2	9	51	118	170	183	167	143	111	92	77	65	59	54	50	45	43	39	35	28	2
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	15	51	103	148	168	156	127	96	76	65	58	54	49	45	41	37	29	12
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	31	69	131	159	140	101	78	66	59	54	50	43	39	31	24
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	46	101	151	134	99	77	65	59	54	45	41	33	26

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

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

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.C	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
IA/P = 0.10														* * * TC = 0.2 HR * * *														IA/P = 0.10				
0.0	23	31	47	209	403	739	800	481	250	166	128	102	86	70	61	54	49	44	40	35	33	30	27	24	21	20	19	18	16	13	12	0
.10	19	26	39	86	168	325	601	733	565	355	229	161	122	83	69	59	53	47	43	37	34	31	28	25	22	21	19	18	16	14	12	0
.20	17	23	32	49	74	136	262	488	652	594	435	298	207	115	81	67	58	51	46	40	35	32	29	26	23	21	20	19	16	14	12	0
.30	16	22	30	46	64	112	212	396	566	585	485	360	258	139	90	71	60	53	48	41	36	32	29	26	23	21	20	19	16	14	12	0
.40	14	19	25	37	43	57	94	173	322	485	551	507	409	227	129	87	68	58	52	44	38	33	30	27	24	21	20	19	17	14	12	0
.50	13	18	24	35	40	52	80	142	262	410	504	506	441	269	153	98	73	61	53	45	39	34	30	27	24	22	20	19	17	15	12	0
.75	10	13	17	23	26	30	34	40	55	86	150	247	349	438	360	240	151	101	75	57	47	39	33	29	26	23	21	20	18	15	12	0
1.0	9	11	14	19	21	24	26	30	35	44	62	101	167	337	413	353	245	157	104	68	53	42	35	31	28	24	22	20	18	16	12	0
1.5	6	8	10	13	14	15	17	19	21	23	26	30	37	73	166	288	356	337	264	154	91	57	42	35	30	27	24	22	19	17	13	3
2.0	4	5	7	8	9	10	10	11	12	14	15	16	18	23	31	55	114	206	291	324	239	125	63	44	35	31	28	24	20	18	14	9
2.5	3	4	5	6	6	7	7	8	9	9	10	11	12	15	18	22	32	58	111	227	258	246	122	63	43	35	31	27	22	19	15	11
3.0	1	2	3	4	4	4	5	5	6	6	7	7	8	9	11	13	16	19	27	59	138	280	248	137	70	46	36	31	25	21	16	11
IA/P = 0.30														* * * TC = 0.2 HR * * *														IA/P = 0.30				
0.0	0	0	0	39	180	545	697	497	276	198	158	130	110	93	81	73	67	61	56	49	46	43	39	35	32	30	29	27	24	21	19	0
.10	0	0	0	2	27	129	407	600	532	361	252	190	150	108	90	79	71	65	59	52	48	44	41	36	32	31	29	28	25	21	19	0
.20	0	0	0	2	19	92	302	501	521	415	306	228	176	119	95	82	73	67	61	53	48	45	41	37	33	31	29	28	25	21	19	0
.30	0	0	0	0	1	13	66	223	408	484	438	350	269	163	114	93	80	72	65	57	51	46	42	38	34	31	30	28	25	22	19	0
.40	0	0	0	0	1	9	47	164	327	431	436	379	306	189	127	98	83	74	67	58	52	47	43	38	34	31	30	28	25	22	19	0
.50	0	0	0	0	0	0	6	33	120	258	374	415	391	271	173	121	95	81	72	62	55	48	44	40	35	32	30	29	26	22	19	0
.75	0	0	0	0	0	0	2	13	50	126	221	302	348	323	240	167	121	96	81	68	59	50	45	41	37	33	31	29	26	23	19	0
1.0	0	0	0	0	0	0	0	0	1	6	24	69	139	285	331	280	204	145	109	82	68	56	48	43	39	35	32	30	27	24	19	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	1	16	79	186	271	288	247	165	110	76	58	49	44	40	35	32	29	26	20	5
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	24	80	163	235	262	202	123	76	58	49	43	39	35	30	27	21	13
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	28	77	179	242	207	120	75	57	48	43	39	32	29	22	17
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	30	101	207	227	130	80	59	49	44	35	30	24	18
IA/P = 0.50														* * * TC = 0.2 HR * * *														IA/P = 0.50				
0.0	0	0	0	0	7	98	371	322	221	182	158	137	120	104	94	86	80	74	69	62	60	57	52	47	44	42	40	39	35	30	28	0
.10	0	0	0	0	4	67	270	305	249	204	174	149	130	108	97	88	82	76	71	64	60	57	53	48	44	42	41	39	35	30	28	0
.20	0	0	0	0	0	3	45	195	268	255	221	189	163	125	106	95	87	80	75	67	62	58	54	49	45	43	41	39	35	31	28	0
.30	0	0	0	0	0	2	31	140	226	245	229	203	176	134	111	98	89	82	76	68	62	59	55	50	45	43	41	39	36	31	28	0
.40	0	0	0	0	0	1	21	101	184	225	228	211	188	144	117	101	91	84	78	69	63	59	55	50	45	43	41	40	36	31	28	0
.50	0	0	0	0	0	0	1	14	72	146	199	218	213	175	137	113	99	89	82	73	66	60	56	52	47	43	42	40	36	32	28	0
.75	0	0	0	0	0	0	0	5	28	71	121	162	186	193	161	133	112	98	88	78	70	62	57	53	48	44	42	41	37	33	28	0
1.0	0	0	0	0	0	0	0	0	0	2	13	38	77	154	186	174	147	122	105	89	78	68	60	56	51	46	43	42	38	34	28	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	2	22	71	129	163	168	150	120	98	80	67	60	55	51	46	43	40	36	28	4
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5	25	65	112	146	157	134	103	79	67	60	55	50	46	41	38	29	14
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	26	60	117	148	136	101	79	66	59	54	50	43	39	31	24
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	40	90	142	130	99	78	66	59	54	45	41	33	26

RAINFALL TYPE = II * * * TC = 0.2 HR * * * SHEET 2 OF 10

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

HYDROGRAPH TIME (HOURS)

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
IA/P = 0.10														* * * TC = 0.3 HR * * *															IA/P = 0.10			
0.0	20	28	41	118	235	447	676	676	459	283	196	146	114	80	66	57	51	46	42	37	33	31	28	24	22	20	19	18	16	13	12	0
.10	19	26	39	99	189	361	571	641	520	362	251	181	136	89	70	60	53	48	43	37	34	31	28	25	22	21	19	18	16	14	12	0
.20	17	23	32	53	83	154	292	478	587	542	422	308	223	127	86	68	58	52	46	40	35	32	29	26	23	21	20	19	16	14	12	0
.30	16	22	30	49	72	127	237	398	524	536	460	359	268	151	97	73	61	53	48	41	36	32	29	26	23	21	20	19	16	14	12	0
.40	14	19	25	37	45	63	105	193	330	459	510	477	398	237	139	92	70	59	52	44	38	34	30	27	24	21	20	19	17	14	12	0
.50	13	18	24	35	42	56	89	158	272	397	472	475	424	274	163	104	76	62	54	46	39	34	30	27	24	22	20	19	17	15	12	0
.75	11	14	19	26	30	34	42	59	95	160	250	339	417	398	259	196	128	89	69	54	45	37	32	29	26	23	21	20	17	15	12	0
1.0	9	11	14	19	21	24	27	30	36	46	68	109	174	328	396	346	248	163	109	70	54	43	35	31	28	24	22	20	18	16	12	0
1.5	6	8	10	13	14	15	17	19	21	23	26	31	38	77	169	282	347	330	264	158	94	58	42	35	31	27	24	22	19	17	13	3
2.0	4	5	7	8	9	10	10	11	12	14	15	16	18	23	32	57	116	205	285	317	239	128	64	44	36	31	28	25	20	18	14	9
2.5	2	4	5	6	6	7	7	8	9	9	10	11	12	15	18	23	33	60	113	223	293	245	125	65	44	35	31	27	22	19	15	11
3.0	1	2	3	4	4	4	5	5	6	6	7	7	8	9	11	13	16	20	27	61	138	275	246	139	72	46	36	31	25	21	16	11
IA/P = 0.30														* * * TC = 0.3 HR * * *															IA/P = 0.30			
0.0	0	0	0	11	64	251	525	574	454	303	221	173	140	104	88	77	70	64	58	51	47	44	40	36	32	31	29	28	24	21	19	0
.10	0	0	0	0	7	45	183	411	520	476	360	268	205	133	101	85	76	69	62	55	49	45	41	37	33	31	30	28	25	21	19	0
.20	0	0	0	0	5	32	132	318	452	468	396	310	240	151	109	90	78	70	64	56	50	46	42	38	33	31	30	28	25	22	19	0
.30	0	0	0	0	0	3	22	96	244	383	440	411	344	217	142	105	87	76	69	60	53	47	43	39	35	32	30	29	26	22	19	0
.40	0	0	0	0	0	2	16	69	186	317	399	407	365	246	160	115	92	79	71	61	54	48	43	39	35	32	30	29	26	22	19	0
.50	0	0	0	0	0	0	2	11	50	140	258	352	389	327	223	149	110	89	77	66	57	50	45	41	36	33	31	29	26	23	19	0
.75	0	0	0	0	0	0	1	4	20	63	135	219	290	335	281	205	146	110	89	72	62	52	46	42	38	34	31	30	27	23	19	0
1.0	0	0	0	0	0	0	0	0	0	2	9	32	78	216	320	306	243	176	128	90	72	59	49	44	40	36	33	31	28	24	19	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	2	20	84	185	264	281	246	168	112	77	58	49	44	40	36	32	29	26	20	5
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	12	50	121	200	257	224	141	83	61	50	44	40	36	31	28	21	14
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	16	51	145	239	223	137	82	60	50	44	40	33	29	22	17
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	19	74	184	224	146	89	63	51	45	36	31	24	18
IA/P = 0.50														* * * TC = 0.3 HR * * *															IA/P = 0.50			
0.0	0	0	0	0	1	25	151	299	277	219	187	162	141	113	100	90	84	78	72	65	61	58	53	48	44	42	41	39	35	31	28	0
.10	0	0	0	0	1	17	106	235	263	234	202	175	152	120	104	93	85	79	73	66	61	58	54	49	44	42	41	39	35	31	28	0
.20	0	0	0	0	0	0	12	75	182	236	234	213	188	144	116	101	91	84	78	70	63	59	55	50	45	43	41	40	36	31	28	0
.30	0	0	0	0	0	0	8	52	138	203	224	217	197	154	123	105	94	86	79	71	64	59	55	51	46	43	42	40	36	32	28	0
.40	0	0	0	0	0	0	5	37	105	170	206	213	203	164	131	110	97	88	81	72	65	60	56	51	46	43	42	40	36	32	28	0
.50	0	0	0	0	0	0	0	4	26	78	140	184	203	191	155	126	107	95	86	76	69	62	57	53	48	44	42	41	37	33	28	0
.75	0	0	0	0	0	0	0	1	10	34	73	117	153	184	173	146	122	105	94	82	73	64	58	54	49	45	43	41	37	33	28	0
1.0	0	0	0	0	0	0	0	0	0	0	4	17	42	114	168	178	159	134	114	94	82	70	61	57	52	47	44	42	39	35	28	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	1	10	44	98	144	163	157	130	105	84	69	61	56	52	47	44	40	36	29	6
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	14	44	87	127	153	141	110	83	69	61	56	51	47	42	38	30	17
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	16	42	97	138	145	107	82	68	60	55	51	43	40	32	25
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	27	71	127	139	105	81	68	60	55	46	41	33	27

RAINFALL TYPE = II * * * TC = 0.3 HR * * * SHEET 3 OF 10

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

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
IA/P = 0.10														* * * TC = 0.4 HR * * *														IA/P = 0.10				
0.0	18	25	36	77	141	271	468	592	574	431	298	216	163	104	77	63	55	49	44	38	34	31	28	25	22	21	20	18	16	14	12	0
.10	18	24	34	67	116	219	385	523	557	473	357	263	196	119	84	67	57	51	46	39	35	32	29	25	22	21	20	19	16	14	12	0
.20	15	20	28	44	59	97	179	316	454	523	489	401	309	178	112	81	65	56	49	42	37	33	30	26	23	21	20	19	17	14	12	0
.30	15	20	27	41	53	82	147	260	389	478	486	429	349	210	129	89	69	58	51	43	38	33	30	27	24	21	20	19	17	14	12	0
.40	13	17	23	33	38	48	71	121	214	331	429	467	442	308	189	120	85	66	56	47	41	35	31	28	24	22	20	19	17	15	12	0
.50	12	16	22	31	36	44	62	102	176	279	379	438	440	339	218	137	94	71	59	49	42	35	31	28	25	22	21	19	17	15	12	0
.75	10	13	17	24	26	30	35	45	65	106	170	251	326	393	341	245	164	112	81	59	48	39	33	30	26	23	21	20	18	15	12	0
1.0	8	10	13	17	19	21	24	27	31	37	50	75	118	251	360	375	292	205	138	83	60	45	36	32	28	25	22	21	18	16	12	1
1.5	6	7	9	12	13	14	15	17	19	21	23	26	31	56	121	224	311	333	293	192	115	66	45	36	31	28	25	22	19	17	13	4
2.0	4	5	6	8	8	9	10	10	11	12	14	15	16	20	27	43	85	159	243	306	264	154	74	47	37	32	28	25	21	18	14	9
2.5	2	3	4	5	6	6	7	7	8	9	9	10	11	13	16	20	27	46	85	184	285	262	147	74	47	37	32	28	22	19	15	11
3.0	1	2	2	3	4	4	4	5	5	6	6	7	7	8	10	12	14	17	23	47	109	227	268	160	83	50	38	32	25	21	16	11
IA/P = 0.30														* * * TC = 0.4 HR * * *														IA/P = 0.30				
0.0	0	0	0	4	26	113	296	480	495	413	306	234	186	127	100	84	74	67	61	54	49	45	41	37	33	31	29	28	25	21	19	0
.10	0	0	0	0	2	18	81	224	395	462	430	347	272	172	121	95	82	73	66	57	51	46	42	38	34	31	30	28	25	22	19	0
.20	0	0	0	0	2	13	59	169	320	414	424	373	305	195	134	103	85	75	67	59	52	47	43	39	34	32	30	29	25	22	19	0
.30	0	0	0	0	0	1	9	42	127	255	361	403	383	274	181	127	99	83	73	63	55	48	44	40	36	32	30	29	26	23	19	0
.40	0	0	0	0	0	1	6	30	94	202	308	372	379	293	203	141	106	87	76	65	56	49	44	40	36	32	31	29	26	23	19	0
.50	0	0	0	0	0	0	0	4	21	70	158	258	334	364	270	187	133	102	85	70	60	51	46	41	37	33	31	30	26	23	19	0
.75	0	0	0	0	0	0	0	2	8	30	76	145	229	321	305	241	177	130	102	78	65	55	47	43	38	34	32	30	27	24	19	0
1.0	0	0	0	0	0	0	0	0	0	1	4	15	42	150	267	308	272	209	154	103	79	62	51	45	41	37	33	31	28	25	19	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	1	10	51	135	226	274	263	195	131	85	62	51	45	41	36	33	29	26	20	6
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	31	86	162	252	239	162	93	64	52	45	41	37	31	28	21	15
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	33	112	202	235	155	92	64	52	45	41	33	29	23	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	21	76	182	221	148	90	63	51	45	36	31	24	18
IA/P = 0.50														* * * TC = 0.4 HR * * *														IA/P = 0.50				
0.0	0	0	0	0	0	7	59	168	245	257	213	186	163	128	109	96	88	81	75	67	62	58	54	50	45	43	41	39	35	31	28	0
.10	0	0	0	0	0	0	5	41	125	205	240	222	198	154	123	105	94	86	79	71	64	60	56	51	46	43	42	40	36	32	28	0
.20	0	0	0	0	0	0	3	28	93	168	216	220	205	164	131	110	97	88	81	72	65	60	56	51	46	43	42	40	36	32	28	0
.30	0	0	0	0	0	0	0	2	20	69	135	189	209	192	155	125	107	95	86	77	69	62	57	53	48	44	42	41	37	33	28	0
.40	0	0	0	0	0	0	0	1	14	50	106	161	193	202	163	133	112	98	89	78	70	62	58	53	48	44	42	41	37	33	25	0
.50	0	0	0	0	0	0	0	1	9	37	83	135	174	194	171	140	117	102	91	80	71	63	58	54	49	45	43	41	37	33	28	0
.75	0	0	0	0	0	0	0	0	0	3	15	40	76	147	177	169	146	124	107	90	79	68	60	56	51	47	43	42	38	34	28	0
1.0	0	0	0	0	0	0	0	0	0	0	1	7	21	73	141	173	167	146	125	101	86	73	63	58	53	48	45	42	39	35	28	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	5	26	71	121	153	159	139	113	89	72	63	57	53	48	44	40	37	29	7
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	16	45	86	138	150	125	93	74	64	58	53	48	42	39	31	20
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	17	59	112	143	121	91	73	63	57	53	45	40	32	26
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	11	40	101	138	117	90	73	53	57	48	42	34	27

RAINFALL TYPE = II * * * TC = 0.4 HR * * * SHEET 4 OF 10

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

HYDROGRAPH TIME (HOURS)

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
IA/P = 0.10														* * * TC = 0.5 HR * * *																IA/P = 0.10		
0.0	17	23	32	57	94	170	308	467	529	507	402	297	226	140	96	74	61	53	47	41	36	32	29	26	23	21	20	19	16	14	12	0
.10	16	22	30	51	80	140	252	395	484	499	434	343	265	162	108	80	65	55	49	42	36	33	29	26	23	21	20	19	16	14	12	0
.20	14	19	25	38	47	69	116	207	332	434	477	449	378	238	149	101	77	62	53	45	39	34	30	27	24	22	20	19	17	14	12	0
.30	13	18	24	35	43	60	97	170	278	382	446	448	401	270	171	114	83	66	56	46	40	34	31	27	24	22	20	19	17	15	12	0
.40	12	15	21	29	33	40	53	83	141	233	332	408	434	361	243	157	107	79	64	51	43	36	32	28	25	22	21	20	17	15	12	0
.50	11	15	20	28	31	37	48	71	118	194	286	367	412	378	271	178	119	86	68	53	44	37	32	29	25	23	21	20	17	15	12	0
.75	9	11	14	19	21	24	27	31	37	49	74	118	182	319	374	328	244	169	117	76	56	43	35	31	28	25	22	21	18	16	12	1
1.0	7	9	12	16	17	19	21	24	27	32	40	55	83	188	309	359	322	245	172	102	68	49	38	32	29	26	23	21	19	16	12	1
1.5	5	7	8	11	12	13	14	15	17	19	21	23	27	43	89	175	269	322	309	225	140	77	49	38	32	29	25	23	20	17	13	5
2.0	3	4	6	7	8	8	9	10	10	11	12	14	15	18	23	35	55	123	202	297	280	181	88	52	39	33	29	26	21	19	14	10
2.5	2	3	4	5	5	6	6	7	7	8	9	9	10	12	15	18	24	36	66	150	244	278	171	87	52	39	33	29	25	20	15	11
3.0	1	1	2	3	3	4	4	4	5	5	6	6	7	8	9	11	13	16	20	37	85	198	263	182	96	56	40	33	26	21	16	11
IA/P = 0.30														* * * TC = 0.5 HR * * *																IA/P = 0.30		
0.0	0	0	0	1	9	53	157	314	433	439	379	299	237	159	118	95	81	71	65	56	50	46	42	38	34	31	30	28	25	22	19	0
.10	0	0	0	0	1	6	37	117	248	372	416	391	330	218	150	113	92	79	70	60	53	47	43	39	35	32	30	29	26	22	19	0
.20	0	0	0	0	1	4	26	97	194	313	382	388	349	244	167	122	97	82	72	62	54	48	43	39	35	32	30	29	26	22	19	0
.30	0	0	0	0	0	0	3	19	64	151	259	341	372	316	223	156	117	94	80	67	58	50	45	41	36	33	31	29	26	23	19	0
.40	0	0	0	0	0	0	2	13	47	116	211	298	354	328	245	172	127	100	83	69	59	51	45	41	37	33	31	29	26	2'	19	0
.50	0	0	0	0	0	0	0	1	9	34	89	170	255	341	303	225	161	120	96	76	64	54	47	42	38	34	31	30	27	24	19	0
.75	0	0	0	0	0	0	0	1	4	14	41	89	152	270	305	268	207	155	118	87	70	57	48	44	39	35	32	30	27	24	19	0
1.0	0	0	0	0	0	0	0	0	0	0	2	7	22	98	212	295	285	237	181	120	88	67	53	46	42	38	34	31	28	25	19	2
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	5	30	95	183	249	265	217	152	96	66	53	46	41	37	34	30	26	20	8
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	18	59	125	221	245	152	105	69	54	47	42	38	32	28	22	16
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	21	84	174	230	172	103	69	54	46	42	34	30	23	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	13	56	157	217	163	101	68	53	46	37	31	25	18
IA/P = 0.50														* * * TC = 0.5 HR * * *																IA/P = 0.50		
0.0	0	0	0	0	0	2	26	89	170	217	229	200	179	144	119	104	93	85	78	70	64	59	55	51	46	43	41	40	36	32	28	0
.10	0	0	0	0	0	0	1	18	65	135	190	216	205	170	137	115	101	91	83	74	67	61	56	52	47	44	42	40	36	32	28	0
.20	0	0	0	0	0	0	1	12	47	106	162	198	203	178	145	121	105	94	85	76	68	61	57	52	48	44	42	40	37	32	28	0
.30	0	0	0	0	0	0	0	1	8	34	82	135	177	194	163	139	117	102	92	80	71	63	58	54	49	45	43	41	37	33	28	0
.40	0	0	0	0	0	0	0	0	6	25	63	111	155	189	174	146	122	105	94	82	73	64	58	54	50	45	43	41	37	33	28	0
.50	0	0	0	0	0	0	0	0	4	18	48	90	133	184	177	152	128	110	97	84	74	65	59	55	50	45	43	41	38	33	28	0
.75	0	0	0	0	0	0	0	0	1	7	22	47	180	142	169	164	144	124	108	91	79	68	61	56	51	47	44	42	38	34	28	0
1.0	0	0	0	0	0	0	0	0	0	0	1	3	11	51	112	155	166	154	134	109	91	76	65	59	54	49	45	43	39	35	28	2
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	2	15	50	97	135	154	145	121	95	75	64	58	54	49	45	41	37	29	10
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	18	47	86	134	146	125	94	75	64	58	53	49	42	39	31	21
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	11	44	95	140	127	97	77	65	58	54	45	41	33	26
3.3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	29	86	135	122	95	76	65	58	49	43	35	27

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

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

HYDROGRAPH TIME(HOURS)

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
IA/P = 0.10												* * * TC = 0.75 HR * * *																				
0.0	13	18	24	36	46	68	115	194	294	380	424	410	369	252	172	123	93	74	61	49	41	35	31	27	24	22	20	19	17	15	12	0
.10	13	17	23	34	42	59	97	162	250	337	395	405	381	279	191	135	100	79	65	51	42	36	31	28	25	22	21	19	17	15	12	0
.20	11	15	20	28	32	39	52	82	135	211	295	362	391	351	255	178	127	95	75	57	46	38	32	29	26	23	21	20	17	15	12	0
.30	11	14	19	26	30	36	47	70	113	179	256	326	379	360	277	196	140	103	80	60	48	38	33	29	26	23	21	20	18	15	12	0
.40	10	12	16	22	25	28	33	42	61	96	151	221	291	367	336	255	182	131	98	69	54	42	34	30	27	24	22	20	18	16	12	0
.50	9	12	16	21	24	27	31	39	53	82	128	190	258	358	343	274	200	144	106	74	56	43	35	30	27	24	22	20	18	16	12	0
.75	8	10	13	17	18	21	23	26	31	39	55	82	122	230	314	329	281	217	161	104	72	51	38	33	29	26	23	21	19	16	12	1
1.0	6	8	10	13	14	15	17	19	21	23	27	32	42	89	177	272	319	303	249	163	105	66	45	36	31	27	24	22	19	17	13	3
1.5	4	6	7	9	10	10	11	12	14	15	16	18	20	27	46	90	163	241	295	275	204	119	66	45	35	31	27	24	20	18	13	7
2.0	3	4	5	6	7	7	8	9	9	10	11	12	13	16	20	28	48	89	151	245	274	213	115	65	44	35	30	27	22	19	14	10
2.5	1	2	3	4	4	5	5	6	6	7	7	8	8	10	12	14	17	24	37	B6	170	260	219	127	71	47	36	31	24	20	16	11
3.0	1	1	2	3	3	3	4	4	4	5	5	5	6	7	8	10	11	14	17	30	64	157	247	205	122	70	46	36	27	22	17	12
IA/P = 0.30												* * * TC = 0.75 HR * * *																				
0.0	0	0	0	0	1	6	30	86	174	266	326	348	328	246	181	138	110	92	79	66	57	49	44	40	36	32	31	29	26	23	19	0
.10	0	0	0	0	0	1	4	22	65	137	223	292	329	303	228	170	131	106	89	73	61	52	46	41	37	33	31	29	26	23	19	0
.20	0	0	0	0	0	0	3	15	48	108	185	256	305	321	245	184	141	112	93	75	63	53	46	42	37	34	31	30	27	23	19	0
.30	0	0	0	0	0	0	2	11	36	84	151	221	277	308	260	199	152	120	98	78	65	54	47	42	38	34	31	30	27	23	19	0
.40	0	0	0	0	0	0	0	1	8	27	65	122	188	286	301	243	187	144	114	87	71	57	48	43	39	35	32	30	27	24	19	1
.50	0	0	0	0	0	0	0	1	6	20	50	98	158	263	292	254	200	155	122	91	74	59	49	44	40	35	32	30	27	24	19	1
.75	0	0	0	0	0	0	0	0	0	2	8	23	51	140	231	269	253	211	167	119	90	68	53	46	42	37	34	31	28	25	19	2
1.0	0	0	0	0	0	0	0	0	0	0	0	1	4	29	96	186	249	261	231	169	120	84	61	50	44	40	36	33	29	26	20	5
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	34	91	163	220	241	197	131	83	61	50	44	40	35	31	27	21	12
2.0	0	0	0	0	0	0	3	0	0	0	0	0	0	0	0	2	11	36	85	174	226	200	127	82	60	49	44	39	32	29	22	17
2.5	o	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	37	105	196	214	135	87	62	51	44	36	31	24	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	24	96	205	189	130	85	62	50	39	32	26	18
IA/P = 0.50												* * * TC = 0.75 HR * * *																				
0.0	0	0	0	0	0	0	2	16	45	92	137	166	185	170	146	125	110	98	89	79	70	63	58	53	48	44	42	41	37	33	28	0
.10	0	0	0	0	0	0	0	1	11	34	73	115	149	180	163	141	122	107	96	84	74	65	59	54	50	45	43	41	38	33	28	0
.20	0	0	0	0	0	0	0	1	8	25	57	96	131	173	166	146	126	111	99	86	76	66	59	55	50	46	43	41	38	34	28	0
.30	0	0	0	0	0	0	0	0	1	5	18	44	79	143	170	160	141	122	108	92	81	69	61	56	52	47	44	42	38	34	28	1
.40	0	0	0	0	0	0	0	0	0	4	14	34	64	127	166	162	145	127	111	95	82	70	62	57	52	47	44	42	38	34	28	1
.50	0	0	0	0	0	0	0	0	0	0	2	10	26	82	133	162	157	140	123	103	88	75	64	58	53	49	45	43	39	35	28	2
.75	0	0	0	0	0	0	0	0	0	0	1	4	12	47	98	139	154	148	135	113	96	80	67	60	55	50	46	43	39	36	29	3
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	6	30	73	119	146	151	134	113	91	74	63	58	53	48	45	41	37	29	7
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	30	66	105	143	143	117	90	73	63	57	52	48	42	39	30	18
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	30	77	121	137	114	88	72	63	57	52	44	40	32	25
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	19	55	111	132	111	87	71	62	56	47	42	34	27
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	12	51	112	128	108	86	71	62	51	44	36	27

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

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

HYDROGRAPH TIME (HOURS)

TRVL TIME (hr)	11.0	11.3	11.6	11.9	11.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	13.0	14.3	14.6	15.0	13.5	15.0	15.5	16.0	17.5	16.0	19.0	20.0	22.0	26.0
IA/P = 0.10														* * * TC = 1.0 HR * * *															IA/P = 0.10			
0.0	11	15	20	29	35	47	72	112	168	231	289	329	357	313	239	175	133	103	83	63	50	40	33	29	26	23	21	20	17	15	12	0
.10	10	13	17	24	27	33	42	62	95	144	202	260	306	340	293	222	165	126	98	72	56	43	35	30	27	24	22	20	18	15	12	0
.20	10	13	17	23	26	30	38	54	82	123	176	232	281	332	303	238	179	136	105	76	59	45	35	30	27	24	22	20	18	16	12	1
.30	9	12	16	22	24	28	35	48	70	105	152	205	256	323	310	254	193	146	113	81	61	46	36	31	27	24	22	20	18	16	12	1
.40	8	11	14	19	21	23	27	32	42	61	91	132	181	276	318	294	237	181	138	95	70	51	39	32	28	25	23	21	18	16	12	1
.50	8	10	13	18	20	22	25	30	38	53	78	114	159	253	311	300	251	195	149	102	74	53	40	33	29	25	23	21	18	16	12	1
.75	7	8	11	14	16	17	19	21	25	30	38	53	76	146	228	284	293	256	208	143	99	65	46	36	31	27	24	22	19	17	13	2
1.0	5	7	8	11	12	13	14	16	17	19	22	25	31	57	111	188	256	286	272	208	144	90	56	41	33	29	26	23	20	17	13	4
1.5	4	5	6	8	8	9	10	11	12	13	14	15	17	22	33	59	107	171	231	268	235	157	88	56	41	33	29	25	21	18	14	8
2.0	2	3	4	5	5	6	6	7	7	8	9	9	10	12	15	19	27	44	78	157	231	252	167	96	59	42	34	29	23	20	15	11
2.5	1	2	2	3	4	4	4	5	5	6	6	7	7	8	10	12	15	19	27	58	120	214	241	159	94	59	42	34	26	21	16	11
3.0	0	1	1	2	2	3	3	3	4	4	4	5	5	6	7	8	10	12	14	22	44	113	214	231	152	91	58	42	29	23	17	12
IA/P = 0.30														* * * TC = 1.0 HR * * *															IA/P = 0.30			
0.0	0	0	0	0	1	4	16	42	83	137	195	243	271	292	227	178	143	117	98	79	66	55	47	42	38	34	31	30	27	23	19	0
.10	0	0	0	0	0	0	3	12	32	66	113	168	218	279	260	213	169	136	113	88	72	59	49	43	39	35	32	30	27	24	19	1
.20	0	0	0	0	0	0	2	9	24	52	93	143	193	271	271	225	180	145	119	92	75	60	50	44	39	35	32	30	27	24	19	1
.30	0	0	0	0	0	0	1	6	18	41	75	120	169	246	264	234	191	153	125	96	78	62	51	44	40	36	33	31	27	24	19	1
.40	0	0	0	0	0	0	0	1	4	14	32	61	100	190	251	259	222	181	146	109	86	67	53	46	41	37	33	31	28	25	19	2
.50	0	0	0	0	0	0	0	1	3	10	24	49	83	168	237	254	230	191	155	115	90	69	54	47	42	37	34	31	28	25	19	2
.75	0	0	0	0	0	0	0	0	0	1	4	12	25	76	150	213	239	228	198	149	112	82	61	50	44	39	35	32	29	26	20	4
1.0	0	0	0	0	0	0	0	0	0	0	0	1	2	15	51	113	182	226	234	197	150	104	72	56	47	42	38	34	30	27	20	7
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	18	51	104	162	220	210	158	102	71	56	47	42	37	31	28	22	13
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	20	49	121	187	209	152	100	70	55	47	41	34	29	23	17
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	32	87	171	199	146	98	69	54	46	37	31	24	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	13	62	158	192	151	103	73	56	41	34	26	18
IA/P = 0.50														* * * TC = 1.0 HR * * *															IA/P = 0.50			
0.0	0	0	0	0	0	0	1	7	21	42	71	101	126	160	154	138	123	110	100	87	77	67	60	55	50	46	43	41	38	34	28	1
.10	0	0	0	0	0	0	0	1	5	15	33	58	87	134	156	149	134	120	108	93	82	71	62	57	52	47	44	42	38	34	28	1
.20	0	0	0	0	0	0	0	1	4	12	26	48	74	123	153	153	137	123	111	95	84	72	63	57	52	47	44	42	38	34	28	1
.30	0	0	0	0	0	0	0	0	3	9	20	38	62	111	143	150	140	127	114	98	86	73	63	58	53	48	45	42	39	35	28	1
.40	0	0	0	0	0	0	0	0	0	2	6	16	31	75	120	145	148	137	123	106	91	77	66	59	54	49	45	43	39	35	29	2
.50	0	0	0	0	0	0	0	0	0	1	5	12	25	64	109	139	146	139	127	108	94	79	67	60	55	50	46	43	39	36	29	3
.75	0	0	0	0	0	0	0	0	0	0	2	5	12	39	78	115	136	140	134	117	101	84	70	62	56	51	47	44	40	36	29	4
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	26	59	96	125	139	133	117	97	78	66	59	54	49	46	41	37	29	8
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	26	54	86	123	133	119	95	77	66	59	54	49	43	39	31	17
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	25	64	104	129	116	93	76	65	58	53	45	41	33	24
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	10	34	84	125	117	96	78	66	59	49	43	35	27
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	32	89	122	114	94	77	66	53	45	37	27

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

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

HYDROGRAPH TIME (HOURS)

TRVL TIME (hr)	11.0	9.3	9.6	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.3	12.6	13.0	13.5	14.0	14.5	15.0	15.5	16.0	17.0	18.0	20.0	24.0
IA/P = 0.10														* * * TC = 1.25 HR * * *															IA/P = 0.10			
0.0	10	13	18	25	29	38	54	81	118	163	213	256	284	311	266	212	163	129	104	78	61	47	37	31	27	24	22	20	18	16	12	1
.10	10	13	17	23	27	34	47	69	102	143	189	234	267	297	274	226	175	138	111	82	64	48	38	31	27	24	22	20	18	16	12	1
.20	9	11	15	20	22	26	31	42	60	88	124	168	212	280	292	261	212	166	131	95	72	53	40	33	28	25	23	21	18	16	12	1
.30	8	11	14	19	21	24	29	38	53	76	108	148	190	263	288	268	224	177	140	101	76	55	41	34	29	25	23	21	18	16	12	2
.40	8	10	13	18	20	23	27	34	46	66	94	130	170	245	282	273	235	188	149	107	80	58	42	34	29	26	23	21	19	16	12	2
.50	7	9	12	16	17	19	22	25	31	41	58	82	114	190	256	279	262	222	178	127	93	65	46	36	31	27	24	22	19	17	13	2
.75	6	8	10	14	15	17	19	21	25	31	41	56	78	139	207	254	265	245	208	152	110	75	51	39	32	28	25	22	19	17	13	3
1.0	5	6	8	10	11	13	14	15	17	19	22	26	33	60	109	173	230	261	255	208	153	100	64	46	36	30	26	24	20	18	13	5
1.5	3	4	5	7	7	8	9	9	10	11	12	13	15	19	27	45	79	130	186	247	239	180	108	68	48	37	31	27	22	19	14	10
2.0	2	3	4	5	6	6	7	7	8	8	9	10	11	13	16	22	35	59	98	171	236	236	156	95	62	44	35	30	23	20	15	11
2.5	1	2	2	3	4	4	4	5	5	5	6	6	7	8	10	12	14	19	28	58	414	197	226	163	102	65	46	36	26	21	16	11
3.0	0	1	1	2	2	2	2	3	3	3	4	4	4	5	6	7	9	10	13	19	35	88	184	218	169	109	70	49	31	24	18	12
IA/P = 0.30														* * * TC = 1.25 HR * * *															IA/P = 0.30			
0.0	0	0	0	0	0	2	9	25	50	86	130	174	208	253	235	201	164	136	115	92	76	61	51	44	39	35	32	30	27	24	19	1
.10	0	0	0	0	0	0	1	6	19	40	71	110	153	217	247	227	191	157	131	103	84	66	53	46	41	36	33	31	28	24	19	2
.20	0	0	0	0	0	0	1	4	14	31	58	93	133	202	239	231	199	165	138	108	87	68	55	47	41	37	33	31	28	25	19	2
.30	0	0	0	0	0	0	0	1	3	10	24	46	77	152	210	236	222	190	158	122	97	74	58	49	43	38	34	32	28	25	20	3
.40	0	0	0	0	0	0	0	0	2	8	19	37	64	134	196	232	225	198	166	127	101	77	59	50	43	38	35	32	28	25	20	3
.50	0	0	0	0	0	0	0	0	0	2	6	14	30	82	151	206	228	217	189	146	113	85	64	52	45	40	36	33	29	26	20	5
.75	0	0	0	0	0	0	0	0	0	1	2	7	15	49	105	164	205	218	205	166	129	95	69	55	47	41	37	33	29	26	20	6
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	32	77	134	185	214	203	166	120	83	63	52	45	39	35	30	27	21	10
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	33	72	121	184	203	171	117	82	62	51	44	39	32	29	22	15
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	21	67	132	194	174	123	86	64	52	45	35	31	24	13
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	13	46	121	187	166	119	84	63	52	39	32	25	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	44	129	180	160	116	83	63	44	35	27	18
IA/P = 0.50														* * * TC = 1.25 HR * * *															IA/P = 0.50			
0.0	0	0	0	0	0	0	1	5	13	26	44	68	91	125	142	142	128	117	107	94	83	72	63	57	52	47	44	42	38	34	28	2
.10	0	0	0	0	0	0	0	0	3	10	20	36	57	100	129	140	136	125	114	100	88	76	65	59	54	49	45	43	39	35	29	3
.20	0	0	0	0	0	0	0	0	2	7	16	30	48	90	122	139	139	127	117	102	90	77	66	60	54	49	45	43	39	35	29	3
.30	0	0	0	0	0	0	0	0	0	2	5	12	24	59	98	126	137	134	125	109	96	82	69	61	56	51	46	44	40	36	29	4
.40	0	0	0	0	0	0	0	0	0	1	4	10	19	51	89	119	134	136	127	112	98	83	70	62	56	51	47	44	40	36	29	5
.50	0	0	0	0	0	0	0	0	0	1	3	7	15	43	79	112	131	135	129	114	100	35	71	63	57	52	47	44	40	36	29	6
.75	0	0	0	0	0	0	0	0	0	0	0	1	3	15	39	71	102	123	130	125	112	94	78	67	60	54	49	46	41	37	29	9
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	17	40	71	101	121	129	121	103	84	71	62	56	51	47	42	38	30	13
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	26	51	92	119	125	105	86	72	63	57	52	44	40	32	23
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	11	35	72	112	122	103	85	71	63	56	47	42	34	26
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	24	66	111	119	101	83	71	62	51	44	36	27
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	23	71	110	116	99	82	70	55	46	37	27

RAINFALL TYPE = II * * * TC = 1.25 HR * * * SHEET 8 OF 10

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

HYDROGRAPH TIME (HOURS)

TRVL TIME (hr)	11.0	9.3	9.6	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.3	12.6	13.0	13.5	14.0	14.5	15.0	15.5	16.0	17.0	18.0	20.0	24.0
IA/P = 0.10														* * * TC = 1.5 HR * * *														IA/P = 0.10				
0.0	9	11	15	21	25	31	41	58	82	112	147	184	216	255	275	236	198	159	129	98	76	57	43	35	30	25	23	21	18	16	12	1
.15	9	10	13	18	20	23	28	37	51	72	98	131	166	226	265	254	226	187	151	113	86	63	46	37	31	26	23	21	19	16	13	2
.20	8	10	13	17	19	22	26	33	45	63	87	116	149	212	259	259	233	197	160	119	90	66	48	38	32	27	24	22	19	16	13	2
.30	7	9	12	16	18	21	24	30	40	55	76	103	134	197	244	255	238	206	169	125	95	68	49	38	32	27	24	22	19	17	13	2
.40	7	8	11	14	15	17	19	23	28	36	49	67	91	151	208	247	252	230	196	146	109	77	54	41	34	29	25	22	19	17	13	3
.50	6	8	10	13	15	16	18	21	26	33	43	59	80	136	194	238	249	235	204	154	115	81	56	42	34	29	25	23	20	17	13	3
.75	5	7	8	11	12	13	14	16	18	21	25	32	42	76	125	179	222	240	233	193	148	102	67	48	38	32	27	24	20	18	13	5
1.0	4	5	7	8	9	10	11	12	13	14	16	18	22	34	59	101	152	201	236	230	193	135	86	59	44	35	30	26	21	18	14	7
1.5	3	4	5	6	6	7	8	8	9	10	11	12	13	16	22	34	58	95	141	203	226	197	131	84	58	43	35	29	23	20	15	10
2.0	1	2	3	4	4	5	5	6	6	7	7	8	9	10	12	16	22	34	56	110	172	218	187	126	82	57	43	34	25	21	16	11
2.5	1	1	2	2	3	3	3	4	4	4	5	5	6	7	8	9	11	14	18	34	69	141	210	190	133	37	60	44	30	23	17	12
3.0	0	0	1	1	2	2	2	2	3	3	3	3	4	5	5	6	8	9	11	16	27	66	149	204	181	128	85	58	35	25	18	12
IA/P = 0.30														* * * TC = 1.5 HR * * *														IA/P = 0.30				
0.0	0	0	0	0	0	1	6	15	31	53	80	112	144	193	225	208	136	157	134	108	89	70	56	48	42	37	34	31	28	25	20	2
.10	0	0	0	0	0	0	1	4	12	25	43	68	97	157	158	219	203	178	151	120	98	77	60	50	44	38	35	32	28	25	20	3
.20	0	0	0	0	0	0	0	1	3	9	19	35	57	114	168	201	213	196	171	135	108	84	64	53	46	40	36	33	29	26	20	4
.30	0	0	0	0	0	0	0	1	2	7	15	29	48	100	155	193	210	200	177	140	113	87	66	54	46	41	36	33	29	26	20	5
.40	0	0	0	0	0	0	0	0	2	5	12	23	39	87	141	184	207	202	182	146	117	89	68	55	47	41	36	33	29	26	20	5
.50	0	0	0	0	0	0	0	0	0	1	4	9	18	51	101	153	190	205	197	164	131	99	73	58	49	43	38	34	30	26	20	7
.75	0	0	0	0	0	0	0	0	0	0	2	4	9	30	68	116	160	189	197	179	147	110	80	62	52	45	39	35	30	27	21	8
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	20	49	92	138	175	195	178	137	97	72	57	48	42	37	31	28	21	12
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	21	47	95	145	187	178	133	95	71	57	48	42	34	29	23	16
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	13	45	97	162	180	138	99	74	58	49	38	32	25	18
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	31	89	161	174	133	97	72	58	42	34	26	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	29	98	160	169	129	95	71	48	37	28	19
IA/P = 0.50														* * * TC = 1.5 HR * * *														IA/P = 0.50				
.0	0	0	0	0	0	0	0	3	8	16	27	42	59	92	115	128	130	121	112	100	90	78	67	60	55	50	46	43	39	35	29	4
.10	0	0	0	0	0	0	0	2	6	12	22	35	51	84	110	125	128	123	114	102	91	79	68	61	55	50	46	43	39	35	29	4
.20	0	0	0	0	0	0	0	0	1	4	10	18	29	60	91	114	126	128	120	108	97	83	71	63	57	52	47	44	40	36	29	5
.30	0	0	0	0	0	0	0	0	1	3	8	14	24	52	83	108	123	126	122	110	98	85	72	63	57	52	48	44	40	36	29	6
.40	0	0	0	0	0	0	0	0	0	1	2	6	12	31	60	90	112	124	126	116	104	90	75	66	59	54	49	45	41	37	29	8
.50	0	0	0	0	0	0	0	0	0	0	2	4	9	26	53	83	106	121	125	118	106	91	77	67	60	54	49	46	41	37	29	8
.75	0	0	0	0	0	0	0	0	0	0	1	2	5	16	36	b2	88	108	119	122	112	97	81	69	62	56	51	47	42	38	30	11
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	26	49	75	98	118	121	108	90	76	66	59	54	49	43	39	31	16
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	11	25	45	80	107	118	106	89	75	65	59	53	45	41	32	23
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	11	32	63	100	115	104	87	74	65	58	48	42	34	26
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	16	48	94	113	105	89	76	66	53	45	36	27
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	15	54	96	111	103	88	75	58	48	38	28

RAINFALL TYPE = II　　　　* * * TC = 1.5 HR * * *　　　　SHEET 9 OF 10

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

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.
IA/P = 0.10														* * * TC =2.0 HR * * *														IA/P = 0.10				
0.0	7	9	12	16	18	21	27	36	49	64	82	104	127	171	201	226	208	193	171	132	105	79	58	45	36	30	26	23	20	17	13	3
.10	6	8	10	14	15	17	20	25	33	43	57	74	94	139	179	204	218	205	188	150	118	88	63	48	38	32	27	24	20	17	13	4
.20	6	8	10	13	14	16	19	23	29	39	51	66	84	128	169	198	213	207	192	157	123	91	65	49	39	33	28	24	20	17	13	4
.30	6	7	9	12	14	15	18	21	27	35	45	59	76	117	159	191	211	208	196	163	128	95	68	51	40	33	28	25	20	18	13	4
.40	5	6	8	11	12	13	15	17	20	24	31	41	53	87	128	167	197	209	205	180	145	106	75	55	43	35	30	26	21	18	14	5
.50	5	6	8	10	11	13	14	16	18	22	28	37	48	78	118	158	190	208	208	185	151	111	77	57	44	36	30	26	21	18	14	5
.75	4	6	7	9	10	11	12	13	15	18	22	27	35	58	91	129	164	191	202	194	167	125	87	63	48	38	32	27	22	18	14	6
1.0	3	4	6	7	8	8	9	10	11	12	14	16	18	28	46	74	110	147	178	201	193	156	108	76	56	43	35	30	23	19	14	8
1.5	2	3	3	5	5	5	6	6	7	8	8	9	10	12	16	23	36	57	86	137	178	195	160	113	79	58	45	36	26	21	16	11
2.0	1	2	2	3	3	4	4	4	5	5	6	6	7	8	10	12	16	23	35	67	112	169	190	154	110	78	57	44	30	23	17	11
2.5	0	1	1	2	2	2	3	3	3	4	4	4	5	6	7	8	9	12	16	28	52	105	170	185	149	107	76	56	35	26	18	12
3.0	0	0	1	1	1	1	1	2	2	2	2	3	3	3	4	5	6	7	8	12	18	41	99	161	180	152	112	80	45	30	19	12
IA/P = 0.30														* * * TC =2.0 HR * * *														IA/P = 0.30				
0.0	0	0	0	0	0	1	3	8	15	25	38	54	74	115	148	168	185	170	159	131	110	89	70	57	49	42	38	34	29	26	20	5
.10	0	0	0	0	0	0	0	2	6	12	21	32	47	85	124	153	169	180	168	145	120	96	75	60	51	44	39	35	30	26	20	6
.20	0	0	0	0	0	0	0	2	4	10	17	27	41	75	114	146	165	175	170	149	124	99	76	62	52	45	39	35	30	27	21	6
.30	0	0	0	0	0	0	0	0	1	3	7	14	23	49	86	122	151	170	174	160	136	107	82	66	54	47	41	37	31	27	21	8
.40	0	0	0	0	0	0	0	0	1	2	6	11	19	43	77	113	144	165	173	163	140	111	85	67	55	47	41	37	31	27	21	8
.50	0	0	0	0	0	0	0	0	1	2	4	9	16	37	68	104	136	160	171	165	144	114	87	69	56	48	42	37	31	27	21	9
.75	0	0	0	0	0	0	0	0	0	0	1	2	5	15	34	62	96	127	152	167	160	132	100	77	62	52	45	40	32	28	22	11
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	24	48	79	111	150	166	153	118	90	71	58	49	43	34	29	23	14
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	10	24	45	98	130	161	148	115	88	70	57	48	37	31	24	17
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	10	32	68	122	157	143	113	87	68	56	42	34	26	18
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	16	51	114	153	144	116	89	70	49	38	27	19	
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	15	59	118	150	140	113	88	57	42	29	19	
IA/P = 0.50														* * * TC =2.0 HR * * *														IA/P = 0.50				
0.0	0	0	0	0	0	0	0	1	4	8	13	20	28	51	73	92	104	111	112	106	97	86	75	66	60	54	49	46	41	37	30	7
.10	0	0	0	0	0	0	0	1	3	6	11	17	24	45	68	87	101	109	112	107	98	88	76	67	60	55	50	46	41	37	30	8
.20	0	0	0	0	0	0	0	1	2	5	9	14	21	40	62	82	98	107	111	108	100	89	77	68	61	55	50	47	41	37	30	8
.30	0	0	0	0	0	0	0	0	0	2	4	7	12	26	46	67	86	100	108	111	104	93	80	70	63	57	52	48	42	38	30	10
.40	0	0	0	0	0	0	0	0	0	1	3	6	10	22	41	62	81	96	106	110	105	94	81	71	63	57	52	48	42	38	30	11
.50	0	0	0	0	0	0	0	0	0	0	1	2	4	13	27	46	67	85	99	110	108	98	85	74	66	59	54	49	43	39	31	13
.75	0	0	0	0	0	0	0	0	0	0	0	1	2	7	18	33	52	71	88	104	108	102	89	77	68	61	55	50	44	39	31	15
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	13	25	43	62	87	103	108	97	84	73	65	59	53	45	41	32	20
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	12	24	48	74	99	106	95	83	72	64	58	48	43	34	25
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	17	37	69	99	104	94	82	72	64	52	45	36	27
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	27	65	95	102	95	83	73	58	49	38	28
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	32	68	95	101	93	82	64	52	40	28

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

APPENDIX 2-B

IMPERVIOUS AREA CALCULATIONS

Appendix 2-B
Impervious Area Calculations

2.B.1 Urban Modifications

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing the CN for urban areas. For example, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

The curve number values given in Table 2-8 are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over pervious areas and then into a drainage system. It is possible that curve number values from urban areas could be reduced by not directly connecting impervious surfaces to the drainage system. The following discussion will give some guidance for adjusting curve numbers for different types of impervious areas.

<u>Connected Impervious Areas</u>

Urban CNs given in Table 2-8 were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that:

(a) pervious urban areas are equivalent to pasture in good hydrologic condition, and

(b) impervious areas have a CN of 98 and are directly connected to the drainage system.

Some assumed percentages of impervious area are shown in Table 2-8.

If all of the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in Table 2-8 are not applicable, use Figure 2-B-1 to compute a composite CN. For example, Table 2-8 gives a CN of 70 for a ½-acre lot in hydrologic soil group B, with an assumed impervious area of 25 percent. However, if the lot has 20 percent impervious area and a pervious area CN of 61, the composite CN obtained from Figure 2-B-1 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

<u>Unconnected Impervious Areas</u>

Runoff from these areas is spread over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system, (1) use Figure 2-B-2 if total impervious area is less then 30 percent or (2) use Figure 2-B-1 if the total impervious area is equal to or greater than 30 percent, because the absorptive capacity of the remaining pervious areas will not significantly affect runoff.

When impervious area is less than 30 percent, obtain the composite CN by entering the right half of Figure 2-B-2 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move left to the appropriate pervious CN and read down to find the composite CN. For example, for a ½-acre lot with 20 percent total impervious area (75 percent of which is unconnected) and pervious CN of 61, the composite CN from Figure 2-B-2 is 66. If all of the impervious area is connected, the resulting CN (from Figure 2-B-1) would be 68.

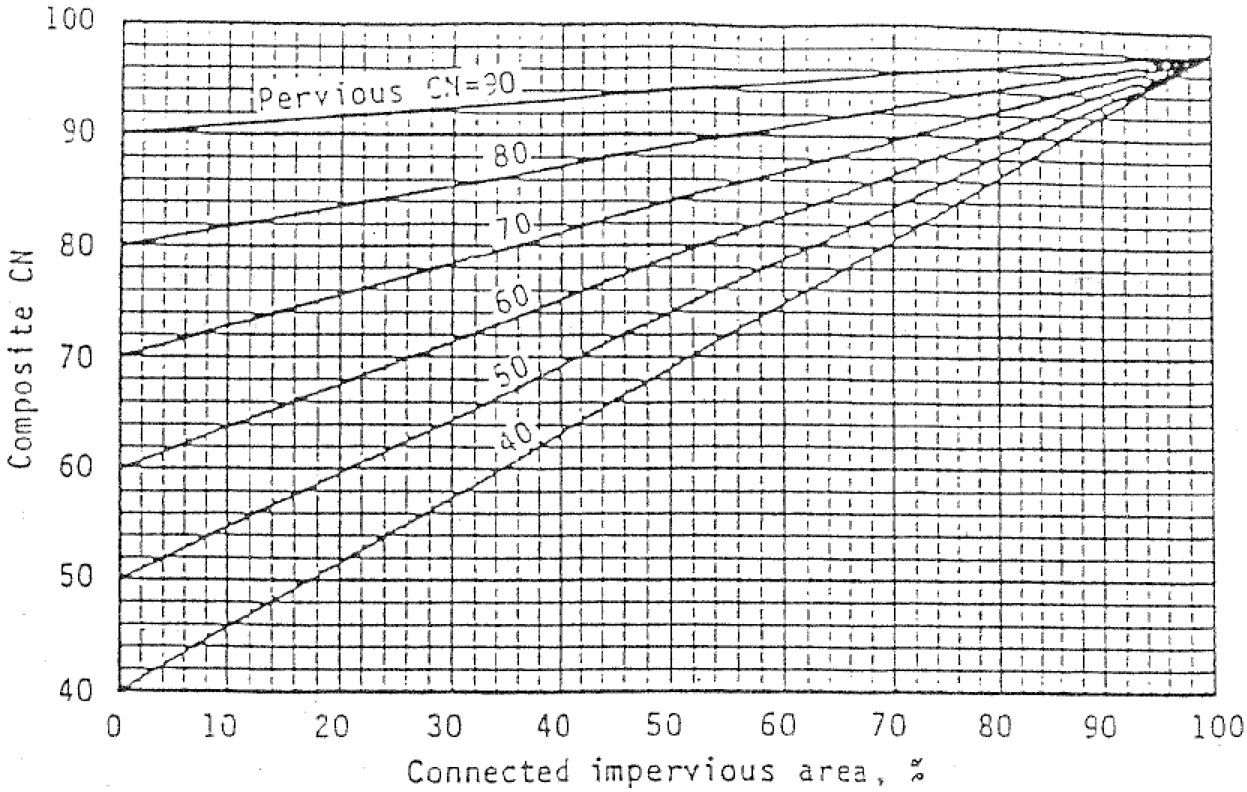


Figure 2-B-1 Composite CN With Connected Impervious Areas

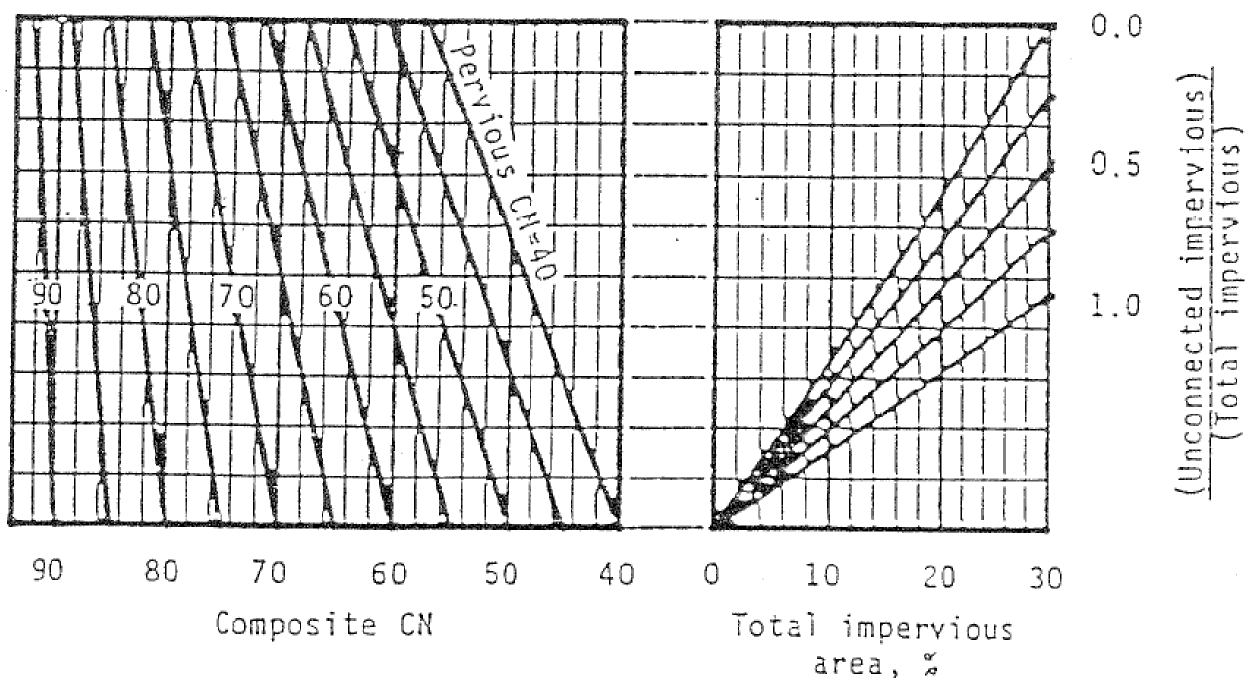


Figure 2-B-2 Composite CN With Unconnected Impervious Areas
(Total Impervious Area less than 30%)

2.B.2 Composite Curve Numbers

When a drainage area has more than one land use, a composite curve number can be calculated and used in the analysis. It should be noted that when composite curve numbers are used, the analysis does not take into account the location of the specific land uses but sees the drainage area as a uniform land use represented by the composite curve number.

Composite curve numbers for a drainage area can be calculated by entering the required data into a table such as Table 2-B-1.

Table 2-B-1 Composite Curve Number Calculations

(1) Land Use	(2) Curve Number	(3) Area	(4) % of Total Area	(5) Composite Curve No. (Col 2 X Col 4)

The composite curve number for the total drainage area is then the sum of the composite curve numbers from column 5.

The different land uses within the basin should represent a uniform hydrologic group represented by a single curve number. Any number of land uses can be included, but if their spatial distribution is important to the hydrologic analysis, then sub-basins should be developed and separate hydrographs developed and routed to the study point.

APPENDIX 2-C

TRAVEL TIME ESTIMATION

Appendix 2-C
Travel Time Estimation

2.C.1 Introduction

Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. T_t is a component of time of concentration (T_c), which is the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system.

Procedures and equations for calculating travel time and time of concentration are discussed in the following sections.

2.C.2 Travel Time

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time is the ratio of flow length to flow velocity:

$$\mathbf{T_t = L/(3600V)} \qquad \textbf{(2.C.1)}$$

Where: T_t = travel time, hr
L = flow length, ft
V = average velocity, ft/s
3600 = conversion factor from sec to hrs

2.C.3 Time Of Concentration

The time of concentration is the sum of Tt values for the various consecutive flow segments:

$$\mathbf{T_c = T_{t1} + T_{t2} + ... T_{tm}} \qquad \textbf{(2.C.2)}$$

Where: T_c = time of concentration, hr
m = number of flow segments

2.C.4 Sheet Flow

Sheet flow is flow over plane surfaces. It usually occurs in the headwater of watersheds. With sheet flow, the friction value (Manning's n) is an effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges and rocks; and erosion and transportation of sediment. These n values are for very shallow flow depths of about 0.1 ft or so. Table 2-C-1 gives Manning's n values for sheet flow for various surface conditions.

Sheet flow conditions are unlikely for length in excess of 300 ft. In urban residential development, sheet flow conditions may occur in rear yards and other open areas but generally ease when flow occurs between buildings. For sheet flow use Manning's kinematic solution (Overton and Meadows 1976) to compute T_t:

$$\mathbf{T_t = [0.42\ (nL)^{0.8} / (P_2)^{0.5} s^{0.4}]} \qquad \textbf{(2.C.3)}$$

Where: T_t = travel time, min
n = Manning's roughness coefficient (Table 2-C-1)
L = flow length, ft
P_2 = 2-year, 24-hr rainfall, in. (3.0 inches in Lincoln)
s = slope of hydraulic grade line (land slope), ft/ft

T able 2-C-1 Roughness Coefficients (Manning's n) for Sheet Flow

Surface Description	n[1]
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover < 20%	0.06
Residue cover > 20%	0.17
Grasses:	
Short grass prairie	0.15
Dense grasses[2]	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods:[3]	
Light underbrush	0.40
Dense underbrush	0.80

[1] The n values are a composite of information complied by Engman (1986).

[2] Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass and native grass mixtures.

[3] When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

This simplified form of the Manning's kinematic solution is based on the following:

1. shallow steady uniform flow,
2. constant intensity of rainfall excess (rain available for runoff),
3. rainfall duration of 24 hrs, and
4. minor effect of infiltration on travel time.

Another approach is to use the kinematic wave equation. For details on using this equation consult the publication by R. M. Regan, “A Nomograph Based on Kinematic Wave Theory for Determining Time of Concentration for Overland Flow,” Report Number 44, Civil Engineering Department, University of Maryland at College Park, 1971.

2.C.5 Shallow Concentrated Flow

After a maximum of 300 ft, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from equations 2.C.4 and 2.C.5, in which average velocity is a function of watercourse slope and type of channel.

Unpaved V = 16.1345(s)0.5 **(2.C.4)**

Paved V = 20.3282(s)0.5 **(2.C.5)**

Where: V = average velocity, ft/s
s = slope of hydraulic grade line (watercourse slope), ft/ft

These two equations are based on the solution of Manning's equation with different assumptions for n (Manning's roughness coefficient) and r (hydraulic radius, feet). For unpaved areas, n is 0.05 and r is 0.4 ft; for paved areas, n is 0.025 and r is 0.2 ft.

After determining average velocity, use equation 2.C.1 to estimate travel time for the shallow concentrated flow segment.

2.C.6 Open Channels

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

Manning's equation is:

$$\mathbf{V = (1.49\, r^{2/3}\, s^{1/2})/n} \qquad \textbf{(2.C.6)}$$

where: V = average velocity, ft/s
r = hydraulic radius, ft (equal to a/pw)
a = cross sectional flow area, ft^2
p_w = wetted perimeter, ft
s = slope of the hydraulic grade line, ft/ft
n = Manning's roughness coefficient

After average velocity is computed using equation 2.C.6, Tt for the channel segment can be estimated using equation 2.C.1.

2.C.7 Reservoir or Lake

Sometimes it is necessary to compute a T_c for a watershed which has a relatively large body of water in the flow path. This travel time is normally very small and can be assumed as zero.

One must not overlook the fact that this does not account for the travel time involved with the passage of the inflow hydrograph through spillway storage and the reservoir or lake outlet. This time is generally much longer and is added to the travel time across the lake. The travel time through lake storage and its outlet can be determined by the storage routing procedures in Chapter 6.

2.C.8 Limitations

- Manning's kinematic solution should not be used for sheet flow longer than 300 ft. Equation 2.C.3 was developed for use with the four standard SCS rainfall intensity-duration relationships. (i.e., Type II)
- In watersheds with storm drains, carefully identify the appropriate hydraulic flow path to estimate T_c. Storm drains generally handle only a small portion of a large event. The rest of the peak flow travels by streets, lawns, and so on, to the outlet. Consult Chapter 3 to determine average velocity in pipes for either pressure or nonpressure flow.
- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. Detailed storage routing procedures should be used to determine the outflow through the culvert.

CHAPTER 3

STORM DRAINAGE SYSTEM

March 7, 2011

Chapter Three - Storm Drainage System

Table Of Contents

3.1 Overview

3.1.1 Introduction

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

In this chapter, guidelines are given for evaluating and designing storm drainage of the minor system. The minor drainage system is typically thought of as storm drains and related appurtenances, such as inlets, curbs and gutters. The minor system is normally designed for floods with return frequencies of 5-years to 10-years, depending upon the kind of land use. The minor system has also been termed the "convenience" drainage system. If downstream drainage facilities are undersized for the design flow, a detention structure may be needed to reduce the possibility of flooding. Storm sewer systems shall be designed using the City of Waverly Stormwater Design Standards.

3.1.2 Symbols and Definitions

To provide consistency within this chapter as well as throughout this manual, the following symbols will be used. These symbols were selected because of their wide use in storm drainage publications. In some cases the same symbol is used in existing publications for more than one definition. Where this occurs in this chapter, the symbol will be defined where it occurs in the text or equations.

Table 3-1 Symbols, Definitions And Units

Symbol	Definition	Units
a	Gutter depression	in
A	Area of cross section	ft^2
d or D	Depth of gutter flow at the curb line	ft
D	Diameter of pipe	ft
E_o	Ratio of frontal flow to total gutter flow Q_w/Q	-
g	Acceleration due to gravity (32.2 ft/s^2)	ft/s^2
h	Height of curb opening inlet	ft
H	Head loss	ft
K	Loss coefficient	-
L	Length of curb opening inlet	ft
L_T	Length of curb opening inlet required for total interception of gutter flow	ft
P	Pipe length	ft
n	Roughness coefficient in the modified Manning formula for triangular gutter flow	-
P	Perimeter of grate opening, neglecting bars and side against curb	ft
Q	Rate of discharge in gutter	cfs
Q_i	Intercepted flow	cfs
Q_s	Gutter capacity above the depressed section	cfs
R	Hydraulic radius	ft
S or S_x	Cross slope - Traverse slope	ft/ft
S or S_L	Longitudinal slope of pavement	ft/ft
S_f	Friction slope	ft/ft
S'_w	Depression section slope	ft/ft
T	Top width of water surface (spread on pavement)	ft
T_s	Spread above depressed section	ft
V	Velocity of flow	ft/s
W	Width of depression for curb opening inlets	ft
Z	T/d, reciprocal of the cross slope	-

3.1.3 Concept Definitions

Definitions of concepts important in storm drain analysis and design used in this chapter are presented below.

Bypass

Flow which bypasses an inlet on grade and is carried in the street or channel to the next inlet downgrade. Inlets may be designed to allow a certain amount of bypass for one design storm and larger or smaller amounts for other design storms. The spread for lower catch basins must consider a reasonable calculated bypass flow from upper facilities.

Curb-Opening Inlet

A drainage inlet consisting of an opening in the roadway curb.

Drop Inlet

A drainage inlet with a horizontal or nearly horizontal opening.

Equivalent Cross Slope

An imaginary continuous cross slope having conveyance capacity equal to that of the given compound cross slope.

Flanking Inlets

Inlets placed upstream and on either side of an inlet at the low point in a sag vertical curve. The purpose of these inlets are to intercept debris as the slope decreases and to act in relief of the inlet at the low point.

Frontal Flow

The portion of the flow which passes over the upstream side of a grate.

Grate Inlet

A drainage inlet composed of a grate in a parking lot, alley or area drain. Grated inlets are not allowed in standard roadway sections.

Gutter

That portion of the roadway section adjacent to the curb which is utilized to convey storm runoff water. It may include a portion or all of a traveled lane, shoulder or parking lane, and a limited width adjacent to the curb may be of different materials and have a different cross slope.

Hydraulic Grade Line

The hydraulic grade line is the locus of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a pipe run.

Inlet Efficiency

The ratio of flow intercepted by an inlet to total flow in the gutter.

Pressure Head

Pressure head is the height of a column of water that would exert a unit pressure equal to the pressure of the water.

Scupper

A vertical hole through a bridge deck for the purpose of deck drainage. Sometimes, a horizontal opening in the curb or barrier is called a scupper.

Side-Flow Interception

Flow which is intercepted along the side of a grate inlet, as opposed to frontal interception.

Slotted Drain Inlet

A drainage inlet composed of a continuous slot built into the top of a pipe which serves to intercept, collect and transport the flow.

Splash-Over

Portion of the frontal flow at a grate which skips or splashes over the grate and is not intercepted.

Spread

The width of flow measured laterally from the roadway curb.

Velocity Head

Velocity head is a quantity proportional to the kinetic energy of flowing water expressed as a height or head of water.

For a more complete discussion of these concepts and others related to storm drain design, the reader is referred to - Drainage of Highway Pavements, Federal Highway Administration, Hydraulic Engineering Circular No. 12, March 1984.

3.2 Pavement Drainage

3.2.1 Introduction

There are many details to consider in the design and specification of storm drain systems. ASCE Manuals of Engineering Practice (1960, 1982, 1983) as well as other trade and vendor publications provide construction and specification details beyond the scope of this text. During the design phase, the system drainage area is defined and preliminary drainage routes are identified based on hydrologic analyses. Integration of the system with environmental features and neighborhood amenities should be assessed, and the location of quantity and quality control structures is determined.

The hydrologic analyses should include defining drainage areas for each inlet or ditch start, developing flow estimates for design frequencies throughout the system, and development of flow and spread calculations to determine permissible maximum spread.

Typical design factors to be considered during gutter, inlet, and pavement drainage calculations include:

- Return period
- Spread
- Storm drain location
- Inlet types and spacing
- Longitudinal slope
- Cross slope
- Curb and gutter sections
- Roadside and median channels
- Bridge decks
- Shoulder
- Median/Median barriers

3.2.2 Return Period

The design storm return period for pavement drainage should be consistent with the frequency selected for other components of the drainage system. The major considerations for selecting a design frequency are roadway classification, roadway speed, hazards, and pedestrian traffic.

3.2.3 Spread

For multi-laned curb and gutter or guttered roadways with no parking, it is not practical to avoid travel lane flooding when grades are flat. Allowable maximum encroachment is provided in the following table.

Table 3-2 Allowable Maximum Encroachment for Minor Storms

Street Classification	Maximum Encroachment
Local & Collector	No curb overtopping.
Arterial	No curb overtopping. Flow spread must leave at least one lane free of water in each direction.

When these encroachments are met, the storm drain system shall commence.

For the major storm runoff, the following street inundation is allowable:

Table 3-3 Allowable Maximum Encroachment for Major Storms

Street Classification	Maximum Encroachment
Local & Collector	Flow spread shall not exceed the right-of-way width.
Arterial	The depth of water at the street crown shall not exceed 6 inches.

Table 3-4 provides recommendation for allowable cross street flow.

Table 3-4 Allowable Cross Street Flow

Street Classification	Minor Storm Design Runoff	Major Storm Design Runoff
Local	Flow equivalent to not greater than 5" allowable depth in upstream curb and gutter.	Flow spread shall not exceed the right-of-way width.
Collector	None	Flow spread shall not exceed the right-of-way width.
Arterial	None	6 inches or less over crown.

3.2.4 Longitudinal Slope

A minimum longitudinal gradient is important for a curbed pavement, since it is susceptible to stormwater spread. Flat gradients on uncurbed pavements can lead to a spread problem if vegetation is allowed to build up along the pavement edge.

Curb and gutter grades that are equal to pavement slopes shall not exceed 8 percent or fall below 0.5 percent without approval from the City.

3.2.5 Cross Slope

Roadway cross slopes are determined by the City of Waverly standard roadway sections. Drainage from median areas should not cross traveled lanes. Median shoulders should generally be sloped to drain away from the pavement. Narrow, raised medians are not subject to these provisions.

3.2.6 Curb And Gutter

Curb and gutter installation shall be designed in accordance with the most current City Standard Drawings and Specifications.

3.2.7 Roadside And Median Channels

Curbed highway sections are relatively inefficient at conveying water. The area tributary to the gutter section should be kept to a minimum to reduce the hazard from water on the pavement. Where practicable, the flow from major areas draining toward curbed highway pavements should be intercepted by channels and routed away from the highway pavement.

Large median areas and inside shoulders should be sloped to a center swale, preventing drainage from the median area from running across the pavement. This is particularly important for high-speed facilities, and for facilities with more than two lanes of traffic in each direction.

3.2.8 Bridge Decks

Drainage of bridge decks is similar to other curbed roadway sections. It is often less efficient, because cross slopes are flatter, parapets collect large amounts of debris, and small drainage inlets on scuppers have a higher potential for clogging by debris. Because of the difficulties in providing and maintaining adequate deck drainage systems, gutter flow from roadways should be intercepted before it reaches a bridge. In many cases, deck drainage must be carried several spans to the bridge end for disposal.

Scuppers are the recommended method of deck drainage because they can reduce the problems of transporting a relatively large concentration of runoff in an area of generally limited right-of-way. For situations where traffic under the bridge or environmental concerns prevent the use of scuppers, grated bridge drains should be used.

3.2.9 Median/Barriers

Weep holes are often used to prevent ponding of water against barriers (especially on superelevated curves). In order to minimize flow across traveled lanes, it is preferable to collect the water into a subsurface system connected to the main storm drain system.

3.3 Gutter Flow Calculations

3.3.1 General

Gutter flow capacities for City of Waverly standard street cross-sections are provided in Figure 3-1 for 2.5% pavement cross-slope and in Figure 3-2 for 3.0% pavement cross-slope. For non-standard applications, refer to Sections 3.3.2 through 3.3.7.

3.3.2 Formula

The following form of Manning's Equation should be used to evaluate gutter flow hydraulics:

$$\mathbf{Q = [0.56 / n] \quad S_x^{5/3} \quad S^{1/2} \quad T^{8/3}} \qquad \textbf{(3.1)}$$

Where:
- Q = gutter flow rate (cfs)
- n = Manning's roughness coefficient
- S_x = pavement cross slope (ft/ft)
- S = longitudinal slope (ft/ft)
- T = width of flow or spread (ft)

3.3.3 Nomograph

A nomograph for solving Equation 3.1 is presented Figure 3-3. Manning's n values for various pavement surfaces are presented in Table 3-2.

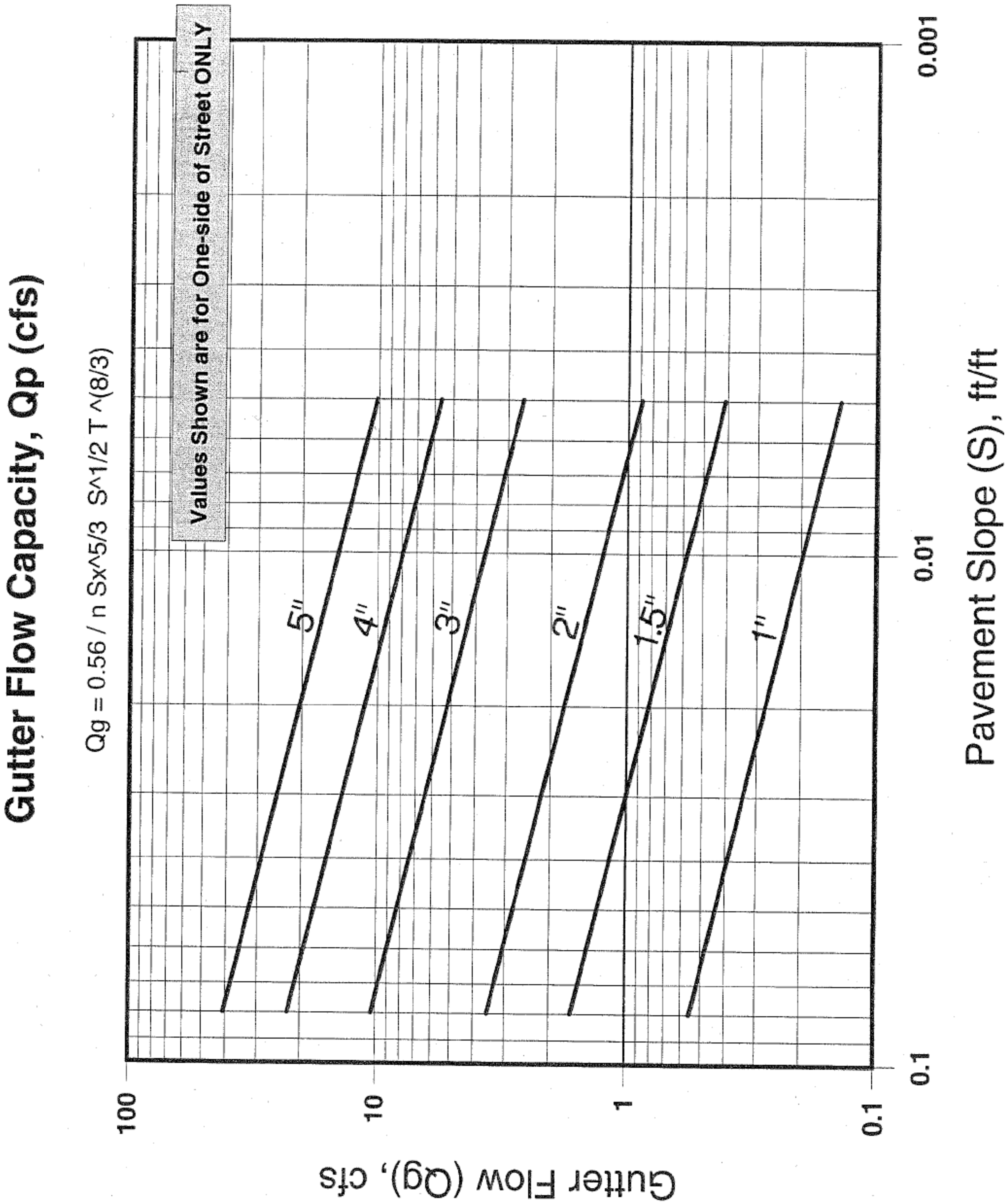


Figure 3-1 Gutter Flow Capacity for 2.5% Pavement Cross-Slope

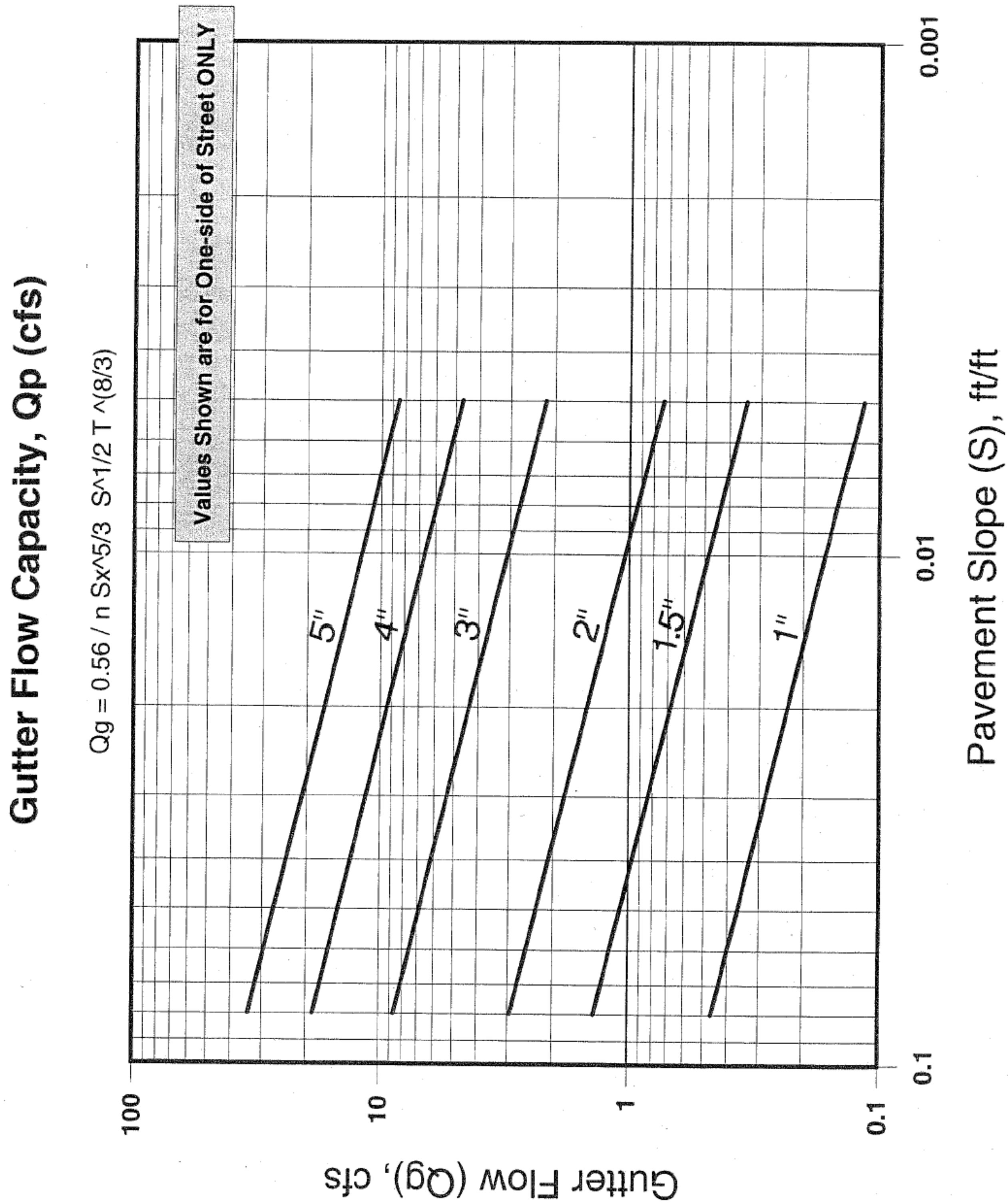


Figure 3-2 Gutter Flow Capacity for 3.0% Pavement Cross-Slope

3.3.4 Manning's n Table

Table 3-5 Manning's n Values For Street And Pavement Gutters

Type of Gutter or Pavement		Range of Manning's n
Concrete gutter, troweled finish		0.012
Asphalt pavement:	Smooth texture	0.013
	Rough texture	0.016
Concrete gutter with asphalt pavement:	Smooth	0.013
	Rough	0.015
Concrete pavement:	Float finish	0.014
	Broom finish	0.016

For gutters with small slopes, where sediment may accumulate, increase above values of n by 0.002

Note: Estimates are by the Federal Highway Administration
Reference: USDOT, FHWA, HDS-3 (1961).

3.3.5 Uniform Cross Slope

The nomograph in Figure 3-3 is used with the following procedures to find gutter capacity for uniform cross slopes:

Condition 1: Find spread, given gutter flow.

1. Determine input parameters, including longitudinal slope (S), cross slope (Sx), gutter flow (Q), and Manning's n.

2. Draw a line between the S and Sx scales and note where it intersects the turning line.

3. Draw a line between the turning line intersection point from Step 2 and the appropriate gutter flow value on the capacity scale. If Manning's n is 0.016, use Q from Step 1 and the right scale on the capacity line. If the Manning's n is not 0.016, multiply Q and n from Step 1 and use the left scale on the capacity scale.

4. Read the value of the spread (T) at the intersection of the line from Step 3 and the spread scale.

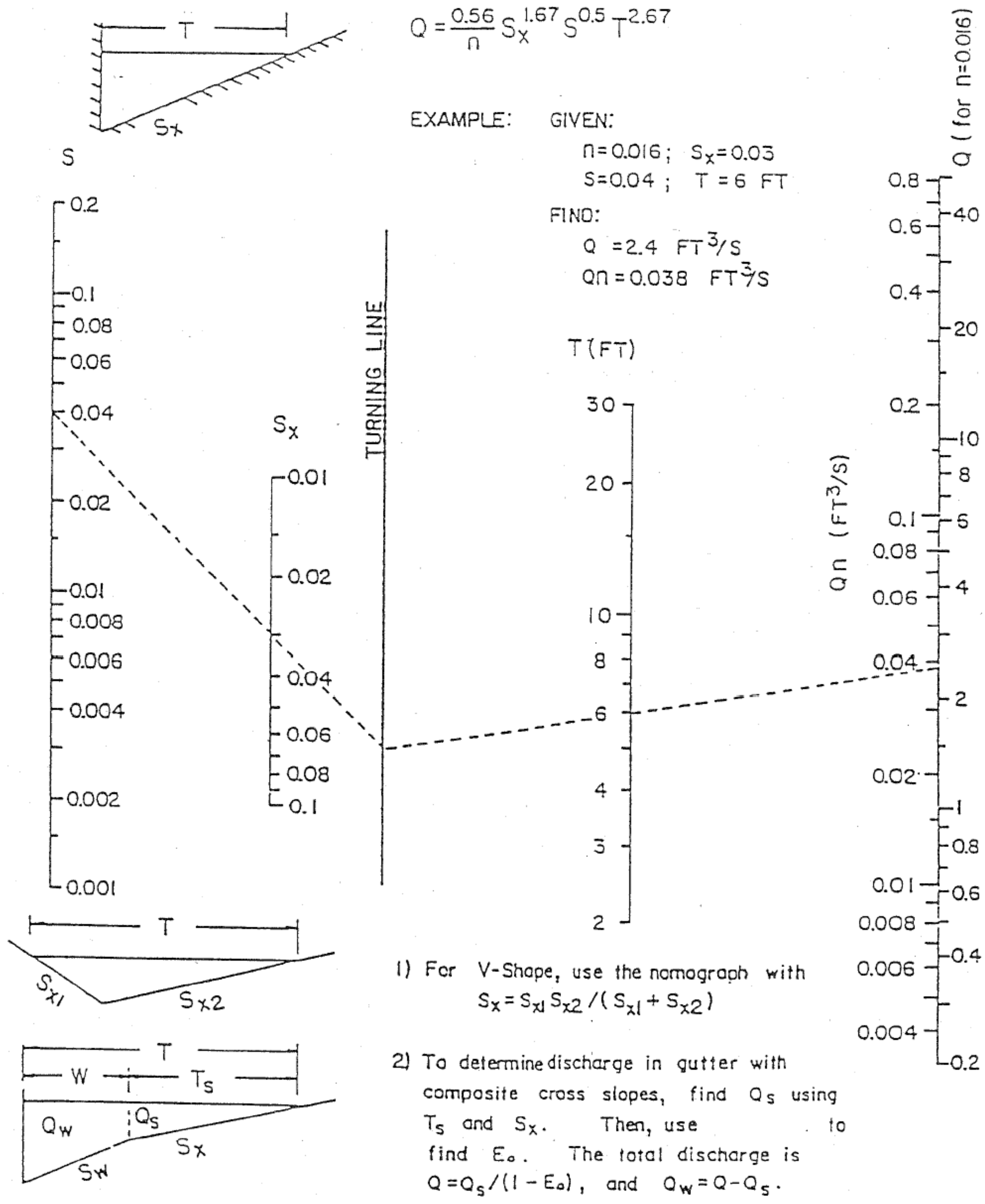


Figure 3-3 Flow In Triangular Gutter Sections

Source: AASHTO Model Drainage Manual, 1991

Condition 2: Find gutter flow, given spread.

1. Determine input parameters, including longitudinal slope (S), cross slope (S_x), spread (T), and Manning's n.

2. Draw a line between the S and S_x scales and note where it intersects the turning line.

3. Draw a line between the turning line intersection point from Step 2 and the appropriate value on the T scale. Read the value of Q (from the right side of the scale) or Qn (from the left side of the scale) from the intersection of that line on the capacity scale.

4. For Manning's n values of 0.016, the gutter capacity (Q) from Step 3 is selected. For other Manning's n values, the gutter capacity times Manning's n (Qn) is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.

3.3.6 Composite Gutter Sections

Figure 3-4 in combination with Figure 3-3 can be used to find the flow in a gutter with width (W) less than the total spread (T). Such calculations are generally used for evaluating composite gutter sections. Figure 3-4 provides a direct solution of gutter flow in a composite gutter section. The flow rate at a given spread or the spread at a known flow rate can be found from this figure. Typical of graphical solutions, extreme care in using the figure is necessary to obtain accurate results.

Condition 1: Find spread, given gutter flow.

1. Determine input parameters, including longitudinal slope (S), cross slope (S_x), depressed section slope (S_w), depressed section width (W), Manning's n, gutter flow (Q), and a trial value of the gutter capacity above the depressed section (Q_s).

2. Calculate the gutter flow in W (Qw), using the equation:

$$\mathbf{Q_w = Q - Q_s} \qquad \textbf{(3.2)}$$

3. Calculate the ratios Q_w/Q or E_o and S_w/S_x and use Figure 3-4 to find an appropriate value of W/T.

4. Calculate the spread (T) by dividing the depressed section width (W) by the value of W/T from Step 3.

5. Find the spread above the depressed section (T_s) by subtracting W from the value of T obtained in Step 4.

6. Use the value of T_s from Step 5 along with Manning's n, S, and S_x to find the actual value of Q_s from Figure 3-3.

7. Compare the value of Q_s from Step 6 to the trial value from Step 1. If values are not comparable, select a new value of Q_s and return to Step 1.

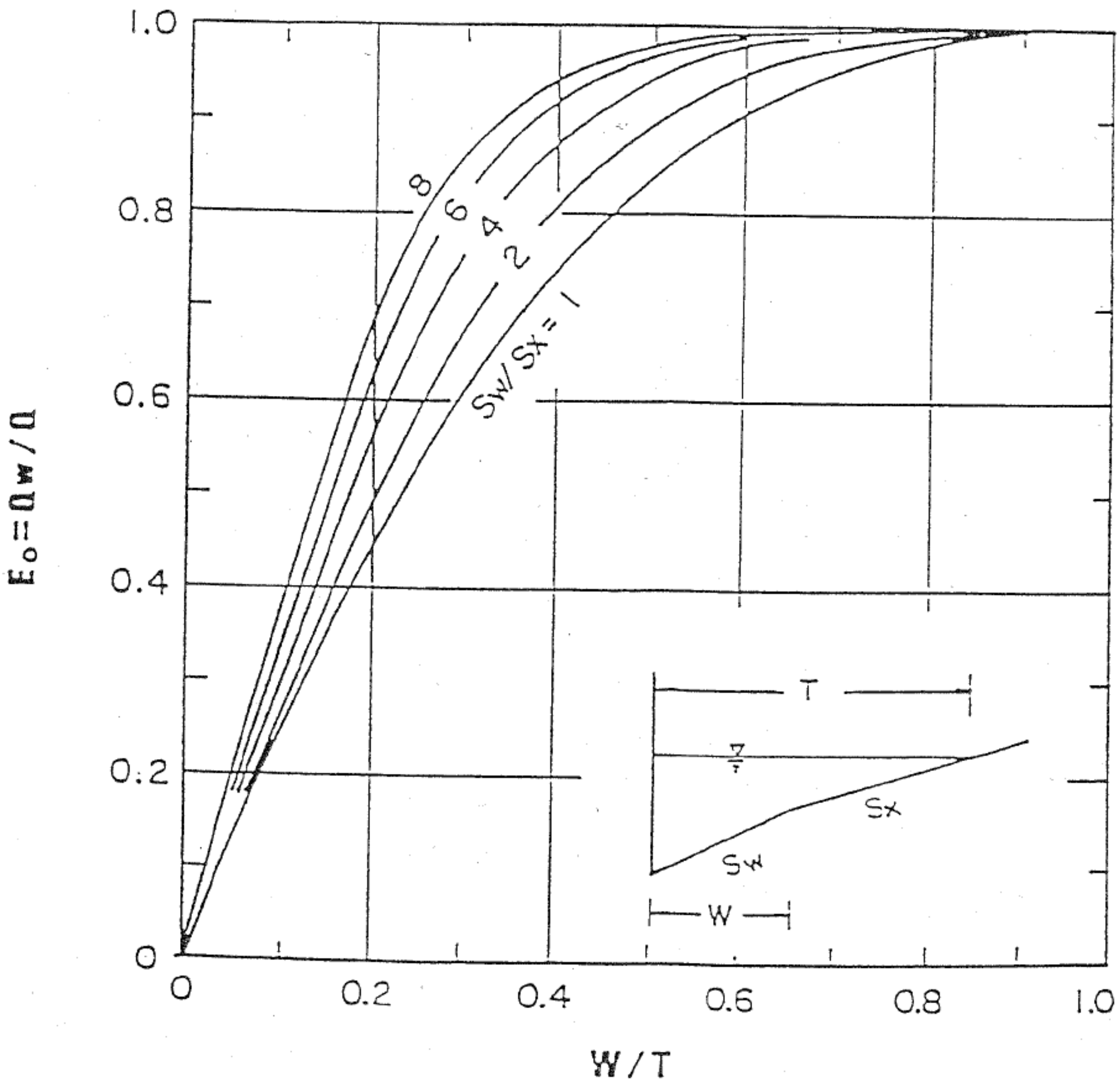


Figure 3-4 Ratio Of Frontal Flow To Total Gutter Flow

Source: AASHTO Model Drainage Manual, 1991

Condition 2: Find gutter flow, given spread.

1. Determine input parameters, including spread (T), spread above the depressed section (T_s), cross slope (S_x), longitudinal slope (S), depressed section slope (S_w), depressed section width (W), Manning's n, and depth of gutter flow (d).

2. Use Figure 3-2 to determine the capacity of the gutter section above the depressed section (Q_s). Use the procedure for uniform cross slopes (Condition 2), substituting Ts for T.

3. Calculate the ratios W/T and S_w/S_x, and, from Figure 3-4, find the appropriate value of E_o (the ratio of Q_w/Q).

4. Calculate the total gutter flow using the equation:

$$\mathbf{Q = Q_s / (1 - E_o)} \qquad \textbf{(3.3)}$$

Where: Q = gutter flow rate (cfs)
Q_s = flow capacity of the gutter section above the depressed section (cfs)
E_o = ratio of frontal flow to total gutter flow (Q_w/Q)

5. Calculate the gutter flow in width (W), using Equation 3.2.

3.3.7 Examples

Example 1

Given: T = 8 ft
S_x = 0.025 ft/ft
S = 0.01 ft/ft
n = 0.015

Find: (1) Flow in gutter at design spread
(2) Flow in width (W = 2 ft) adjacent to the curb

Solution: (1) From Figure 3-3, Qn = 0.03
Q = Qn/n = 0.03/0.015 = 2.0 cfs

(2) T_s = 8 - 2 = 6 ft
$(Qn)_2$ = 0.014 (Figure 3-1) (flow in 6 ft width outside of width W)

Q = 0.014/0.015 = 0.9 cfs

Q_w = 2.0 - 0.9 = 1.1 cfs

Flow in the first 2 ft adjacent to the curb is 1.1 cfs and 0.9 cfs in the remainder of the gutter.

Example 2

Given: $T = 6$ ft $S_w = 0.0833$ ft/ft $n = 0.014$
$T_s = 6 - 1.5 = 4.5$ ft $S = 0.04$ ft/ft
$S_x = 0.03$ ft/ft $W = 1.5$ ft

Find: Flow in the composite gutter

Solution: (1) Use Figure 3-3 to find the gutter section capacity above the depressed section.

$Q_s n = 0.017$

$Q_s = 0.017/0.014 = 1.2$ cfs

(2) Calculate $W/T = 1.5/6 = 0.25$ and

$S_w/S_x = 0.0833/0.03 = 2.78$

Use Figure 3-3 to find $E_o = 0.64$

(3) Calculate the gutter flow using Equation 3.3:

$Q = 1.2/(1 - 0.64) = 3.3$ cfs

(4) Calculate the gutter flow in width, W, using Equation 3.2:

$Q_w = 3.3 - 1.2 = 2.1$ cfs

3.4 Storm Water Inlets

3.4.1 Overview

The primary aim of drainage design is to limit the amount of water flowing along the gutters or ponding at the sags to quantities which will not interfere with the passage of traffic for the design frequency. This is accomplished by placing inlets at such points and at such intervals to intercept flows and control spread. In this section, guidelines are given for designing roadway features as they relate to gutter and inlet hydraulics and storm drain design. Procedures for performing gutter flow calculations are based on a modification of Manning's Equation. Inlet capacity calculations are based on information contained in HEC-12 (USDOT, FHWA, 1984). Storm drain design is based on the use of the rational formula.

Drainage inlets are located to limit the depth or spread on traffic lanes to allowable limits for the design storm. Grates should safely accommodate bicycle and pedestrian traffic where appropriate.

Inlets at vertical curve sags in the roadway grade should also be capable of limiting the spread to allowable limits. The width of water spread on the pavement should not be greater than the width of spread encountered on continuous grades. Inlets should be located so that concentrated flow and heavy sheet flow will not cross traffic lanes, and should be located just upgrade of pedestrian crossings and locations where the pavement slope reverses.

Inlets may be classified as being on a continuous grade or in a sump. The term "continuous grade" refers to an inlet located on the street with a continuous slope past the inlet with water entering from one direction. The "sump" condition exists when the inlet is located at a low point and water enters from both directions.

Inlets used for the drainage of paved or unpaved surfaces can be divided into two major classes. These classes are:

1. Grate Inlets - These inlets include grate inlets consisting of an opening covered by one or more grates, and slotted inlets consisting of a pipe cut along the longitudinal axis with a grate of spacer bar to form slot openings.

2. Curb-Opening Inlets - These inlets are vertical openings in the curb covered by a top slab.

3.4.2 Criteria

The following criteria shall be used for inlet design:

Land Use	Average Return Frequency (years)
Residential Areas	5
Commercial, Industrial, and Arterial Roads	10

Inlets

- 72-inch straight and canted curb inlets shall be used in the public street system where there is sufficient grade to construct a storm sewer system
- Grate inlets may be used for parking lot drains, area drains, etc.
- Flow in the gutter should not exceed five (5) inches.
- Inlets should be placed at the low points in the street grade.

Design charts for standard inlets are provided in Figures 3-5 through 3-8. The location of the first inlet shall be determined by a trial and error process based upon a point where the maximum depth of flow in the gutter is five inches. Subsequent inlets downstream from the initial inlets shall be located at or before points where the depth of flow in gutter is five inches. Usually inlets shall be placed at the ends of radii and/or before crosswalks at intersections. Inlets which the study shows are needed at locations other than at intersections shall generally be centered between lot lines. Inlets shall be installed at the upper end of all storm drain lines and at low points in the street grades. It may be necessary at some locations to use more than one inlet to pick up the contributing flow. Canted inlets shall not be placed along intersection radii.

Concrete valley gutters may be used across roadways at T-intersections of local roadways, if the calculated depth of flow for the minor system design flow in the curb and gutter section immediately upstream is less than 5 inches and if there is no existing or proposed storm drain conduit extended to the intersection. The pavement cross-slope on the "uphill" lane of the minor approach shall be reduced at a gradual rate from 3% to 1% to allow drainage of the "uphill" gutter flow line through the return. No valley gutters shall be used across collector or arterial roadways.

Curb and gutter grades that are equal to pavement slopes shall not exceed 8 percent or fall below 0.3 percent without approval from the City.

The detailed procedures and necessary charts to design inlets are described in Chapter 3 of the Manual.

3.4.3 Manholes

Manholes shall be installed at the upper end of all storm drain lines and at all changes in grade, size, or alignment. The recommended maximum spacing is 600 feet for storm drain lines, 36 inches and less in diameter. Greater spacings than this will require approval by the City. The crowns of all storm drain pipes entering and leaving a junction shall be at the same elevation. Laterals from a storm drain inlet to the main storm drain line may be tapped directly into the main storm drain line if the diameter of the lateral does not exceed one-half the diameter of the pipe being tapped. If the diameter of the lateral does exceed one-half the diameter of the pipe being tapped, a storm drain manhole or inlet will be required. The crown of the lateral pipe shall match the crown of the main storm drain pipe.

3.4.4 Grate Inlets

A design chart for the three standard grate inlets plus a 72" straight curb inlet in a sump condition is provided in Figure 3-5.

The capacity of an inlet depends upon its geometry and the cross slope, longitudinal slope, total flow, depth of flow and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs. For grates less than 2 feet long, intercepted flow is small. Inlet interception capacity has been investigated by agencies and manufacturers of grates. For inlet efficiency data for various sizes and shapes of grates, refer to Hydraulic Engineering Circular No. 12 Federal Highway Administration and inlet grate capacity charts prepared by grate manufacturers.

3.4.5 Curb Inlets

Capacities for the standard 72-inch straight and canted curb inlets are provided in Figures 3-6 and 3-7. Capacity of 48-inch straight inlets used for median drainage is provided in Figure 3-8. In extraordinary conditions, non-standard designs may be needed. Refer to “Drainage of Highway Pavements” Hydraulic Engineering Circular No. 12. A sample Inlet Design Computation Form is provided in Figure 3-9.

Formulas for inlet capacities

Inlet capacity in a sump is controlled by two flow conditions, weir flow and orifice flow. Figure 3-5 was developed using the following two equations:

Sump conditions

$$\mathbf{Q_w = 3.0\ Pd^{1.5} / FOS} \qquad \mathbf{(3.4a)}$$

and

$$\mathbf{Q_o = 0.67\ A\ (2gd)^{0.5} / FOS} \qquad \mathbf{(3.4b)}$$

Where,

Q_w = weir capacity (cfs) and,
P = perimeter (ft)
d = depth (ft)
FOS = Factor of Safety = 2

Q_o = orifice capacity (cfs)
A = clear opening area (sf)
g = acceleration of gravity = 32.2 ft/s^2
d = depth (ft)
FOS = Factor of Safety = 2

Curb inlet interception capacity is the flow intercepted by an inlet under a given set of conditions. The efficiency of an inlet changes with changes in cross slope, longitudinal slope, total gutter flow, and to a lesser extent, pavement roughness.

Interception capacity for curb inlets on grade shall be determined by the nomographs included in Figures 3-6 through 3-8.

3.4.6 Concrete Flumes

In some locations within the City of Waverly, there is insufficient depth for construction of a storm sewer system. In these locations, concrete flumes can be installed to direct flow out of the street section and into an adjacent ditch. The preferred location of these concrete flumes would match the placement of storm sewer inlets within the storm drainage system. In the event that concrete flumes cannot be installed at these locations, the City may grant a variance to allow construction of a concrete flume at the point where the entire street section is filled with water during the design storm event. The City shall only allow this variance during redevelopment of Existing Urban Areas where this would be an improvement over the existing storm drainage system.

3.4.7 Flared End Sections

Capacities for flared end sections shall be determined using the procedures provided in Chapter 4 - Design of Culverts.

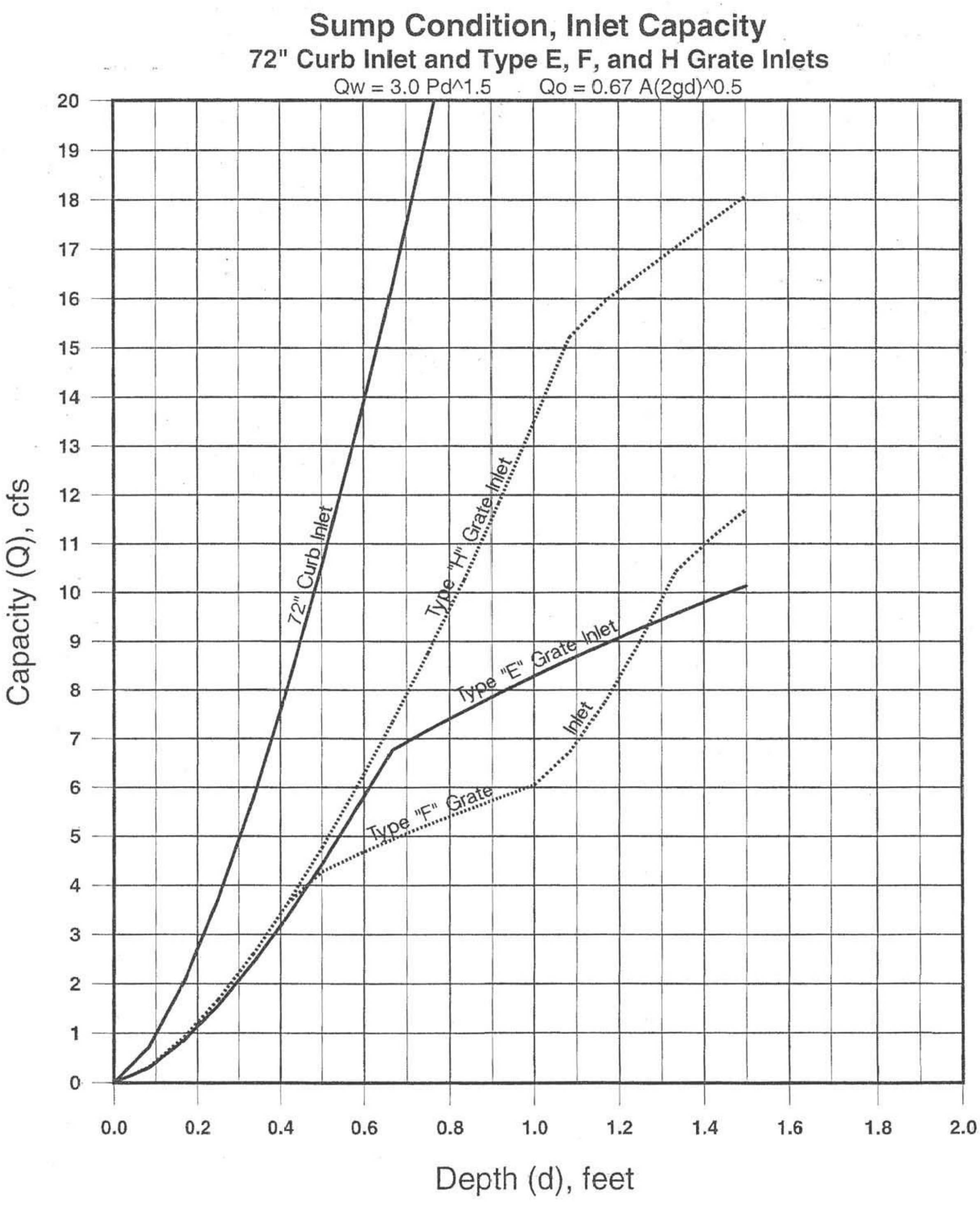


Figure 3-5 Capacity for Standard Grate Inlets

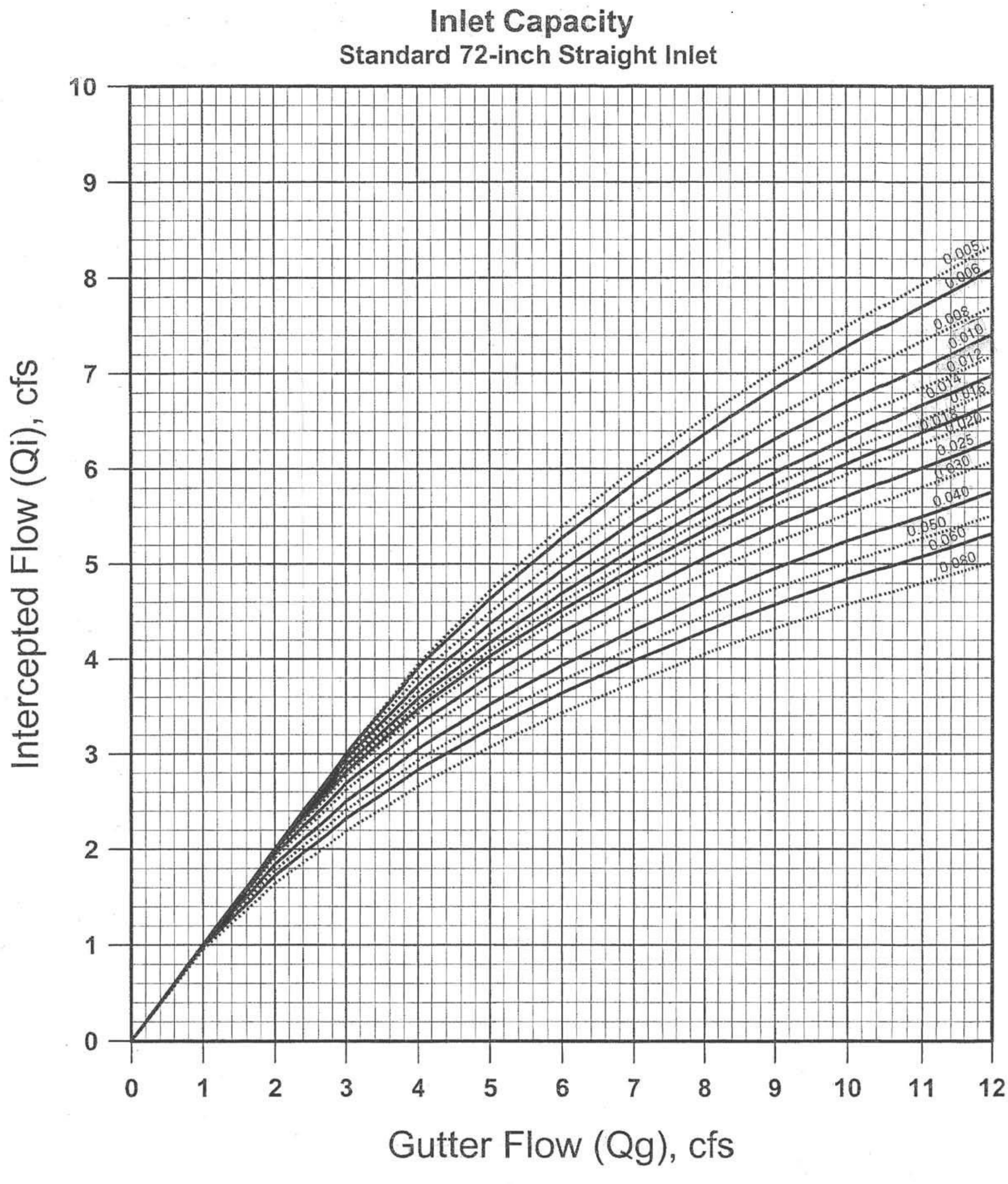


Figure 3-6 Inlet Capacity for Standard 72-Inch Straight Curb Inlet

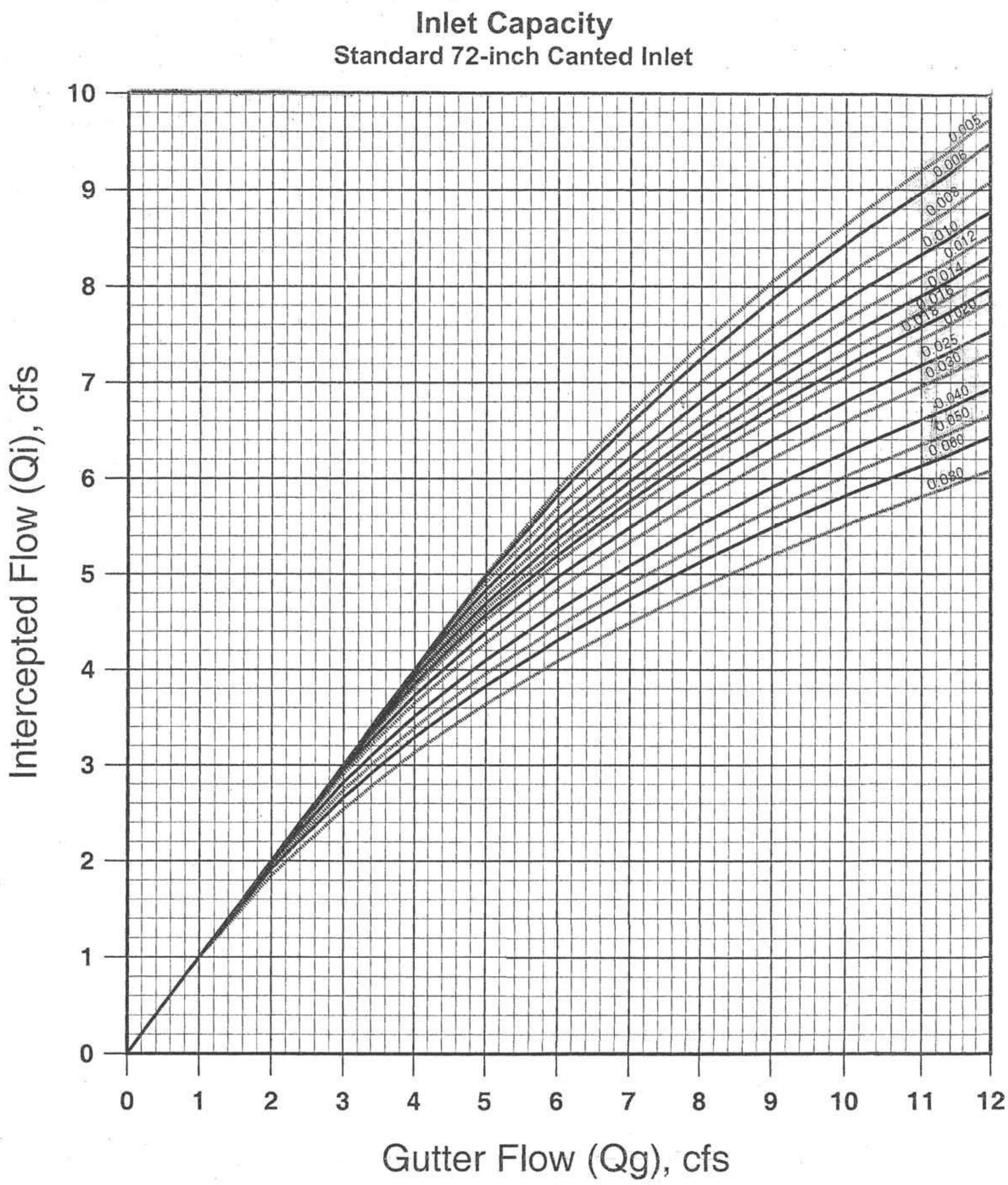


Figure 3-7 Inlet Capacity for Standard 72-Inch Canted Curb Inlet

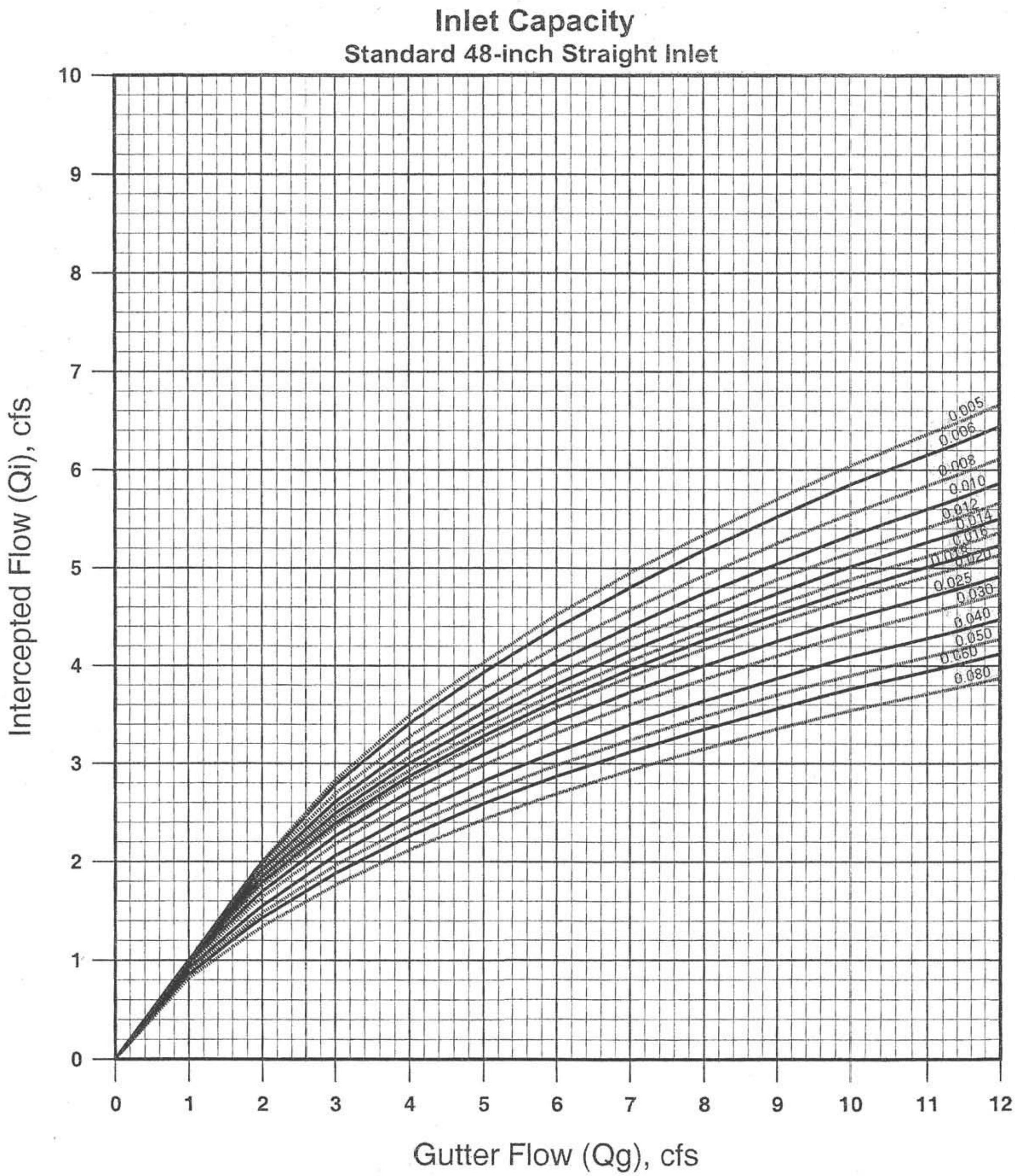


Figure 3-8 Inlet Capacity for Standard 48-Inch Straight Curb Inlet

Inlet Design

Q = CIA   Minor Storm Average Return Frequency ______

Location	Area, A	Runoff Coeficient, C	Time of Concen-tration	Intensity, I	Runoff, Q	By-Pass, Qb	Crossover, Qc	Total, Q	Paving Slope, S	Depth of Curb Flow	Inlet Type	Intercepted, Qi	Bypassed, Qb
	acres	constant	minutes	inch/hour	cfs	cfs	cfs	cfs	ft/ft	inches		cfs	cfs

Figure 3-9 Inlet Design Computation Form

3.5 Storm Drains

3.5.1 Introduction

After the tentative location of inlets has been determined and the inlets sized, the next logical step is the computation of the rate of discharge to be carried by each drain pipe and the determination of the size and gradient of pipe required to carry this discharge. The procedure is carried out for each section of pipe starting at the most upstream inlet and proceeding downstream. It should be recognized that the rate of discharge to be carried by any particular section of drain pipe is not necessarily the sum of the inlet design discharge rates of all inlets above that section of pipe, but as a general rule is somewhat less than this total. In other words, the inlets are designed to assure that the full pipe capacity is utilized. It is useful to understand that the time of concentration is most influential and as the time of concentration grows larger, the proper rainfall intensity to be used in the design grows smaller.

For ordinary conditions, drain pipes should be sized on the assumption that they will flow full or practically full under the design discharge but will not be placed under pressure head. The Manning Formula is recommended for capacity calculations.

3.5.2 Design Criteria

The standard recommended maximum and minimum slopes for storm drains shall conform to the following criteria:

1. The maximum hydraulic gradient shall not produce a velocity that exceeds 20 feet per second.

2. The minimum desirable physical slope shall be 0.5 percent or the slope which will produce a velocity of 3.0 feet per second when the storm drain is flowing full, whichever is greater.

In order to determine if design flows can be accommodated by the storm drains system without causing flooding, or causing flows to exit the system at unacceptable locations, the designer shall determine the hydraulic gradient. The following design criteria shall be followed when determining the elevation along the hydraulic grade line (HGL):

- The hydraulic grade line shall be 0.75 feet below the intake lip of any affected inlet, any manhole cover, or any entering nonpressurized system.
- The energy grade line shall not rise above the intake lip of any affected inlet, any manhole cover or any entering nonpressurized system.

All storm drains should be designed such that velocities of flow will not be less than 3.0 feet per second at design flow, with a minimum slope of 0.5 percent. For very flat flow lines the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system.

<u>Location and Alignment</u>

In new subdivisions the center of the street is reserved for storm drain system. When construction of a storm drain system is necessary in the older parts of the town, the location is determine by the City. No structures may be placed over a public storm drain system.

<u>Depth of Cover</u>

The desired depth of cover above a storm drain pipe shall be 2 to 3 feet, with 1.5 feet being the absolute minimum at an inlet location. Depth of cover greater than 3 feet shall be avoided due to the possibility of the storm drain blocking access of sanitary sewer service lines to the main sanitary sewer lines.

<u>Bar Grates on End Sections</u>

An open pipe inlet from an open channel (similar to a culvert inlet) into a closed pipe storm drain shall be designed and constructed with flared end sections with a bar grate. No bar grate is required on the end section of a pipe outlet into an open channel unless directed by the City.

3.5.3 Design Procedures

The design of storm drain systems is generally divided into the operations listed below. Supporting documentation shall be submitted with development plans for review:

1. The first step is the determination of inlet location and spacing as outlined earlier in this chapter.
2. The second step is the preparation of a plan layout of the storm drain system establishing the following design data:
   a. Location of storm drains.
   b. Direction of flow.
   c. Location of manholes.
   d. Location of existing facilities such as water, gas, or underground cables.
3. The design of the storm drain system is then accomplished by determining drainage areas, computing runoff by rational method, and computing the hydraulic capacity by Manning's equation.
4. The storm drain design computation sheet (Figure 3-12) shall be used to summarize the preliminary system design computations.
5. The hydraulic grade line computation from Figure 3-14 shall be used to determine the hydraulic gradient. The hydraulic grade line profile shall be provided on the storm drain system plans for the minor design storm.

3.5.4 Capacity

Storm drain capacity for reinforced concrete pipe can be determined using Figure 3-13. For non-standard applications, hydraulic capacity can be determined using the information provided below.

Formulas for Gravity and Pressure Flow

The most widely used formula for determining the hydraulic capacity of storm drain pipes for gravity and pressure flows is the Manning Formula and it is expressed by the following equation:

$$\mathbf{V = [1.486\ R^{2/3}S^{1/2}]/n} \qquad \mathbf{(3.6)}$$

Where: V = mean velocity of flow (ft/s)
R = the hydraulic radius (ft) - the area of flow divided by the wetted flow surface or wetted perimeter (A/WP)
S = the slope of hydraulic grade line (ft/ft)
n = Manning's roughness coefficient

In terms of discharge, the above formula becomes:

$$\mathbf{Q = [1.486\ AR^{2/3}S^{1/2}] / n} \qquad \mathbf{(3.7)}$$

Where: Q = rate of flow (cfs)
A = cross sectional area of flow (ft^2)

For pipes flowing full, the above equations become:

$$\mathbf{V = [0.590\ D^{2/3}S^{1/2}] / n} \qquad \mathbf{(3.8)}$$
$$\mathbf{Q = [0.463\ D^{8/3}S^{1/2}] / n} \qquad \mathbf{(3.9)}$$

Where: D = diameter of pipe (ft)

The Manning's equation can be written to determine friction losses for storm drain pipes as:

$$\mathbf{H_f = [2.87\ n^2V^2L] / [S^{4/3}]} \qquad \mathbf{(3.10)}$$
$$\mathbf{H_f = [29\ n^2LV^2] / [(R^{4/3})(2g)} \qquad \mathbf{(3.11)}$$

Where: H_f = total head loss due to friction (ft)
D = diameter of pipe (ft)
L = length of pipe (ft)
V = mean velocity (ft/s)
R = hydraulic radius (ft)
g = acceleration of gravity - 32.2 ft/s^2

3.5.4.1 Street Right-of-way and Overland Swale

Street right-of-ways convey the portion runoff in excess of pipe capacity, whether planned or not. Street right-of-way capacity is determined using Manning's equation for open channel flow conditions.

$$\mathbf{Q = \frac{1.486}{n} AR^{2/3}S^{1/2}} \qquad \textbf{(3.6)}$$

The City of Waverly uses standard street and right-of-way cross-sections for municipal streets, the formula can be simplified to:

$Q = K S^{1/2}$, where conveyance constant, $K = \frac{1.486}{n} AR^{2/3}$

Area, wetted perimeter, and roughness coefficient are constant, the only variable being the street slope.

i.e., For residential	For overland swales
A = 21.662 square feet,	A = 22.5 square feet,
R = 0.360 feet, and	R = 0.149 feet, and
$n_{(wtd)} = 0.026$	n = 0.032

The following table gives the conveyance constants for residential, commercial and major two-lane streets and a 30-foot wide swale with 10:1 side slopes.

Table 3-6 Conveyance Constants for Standard Street Right-of-Ways and 30' Swale

Residential	620
Business with parking	970
Business without parking	790
Major two-lane	1100
30-foot Swale	780

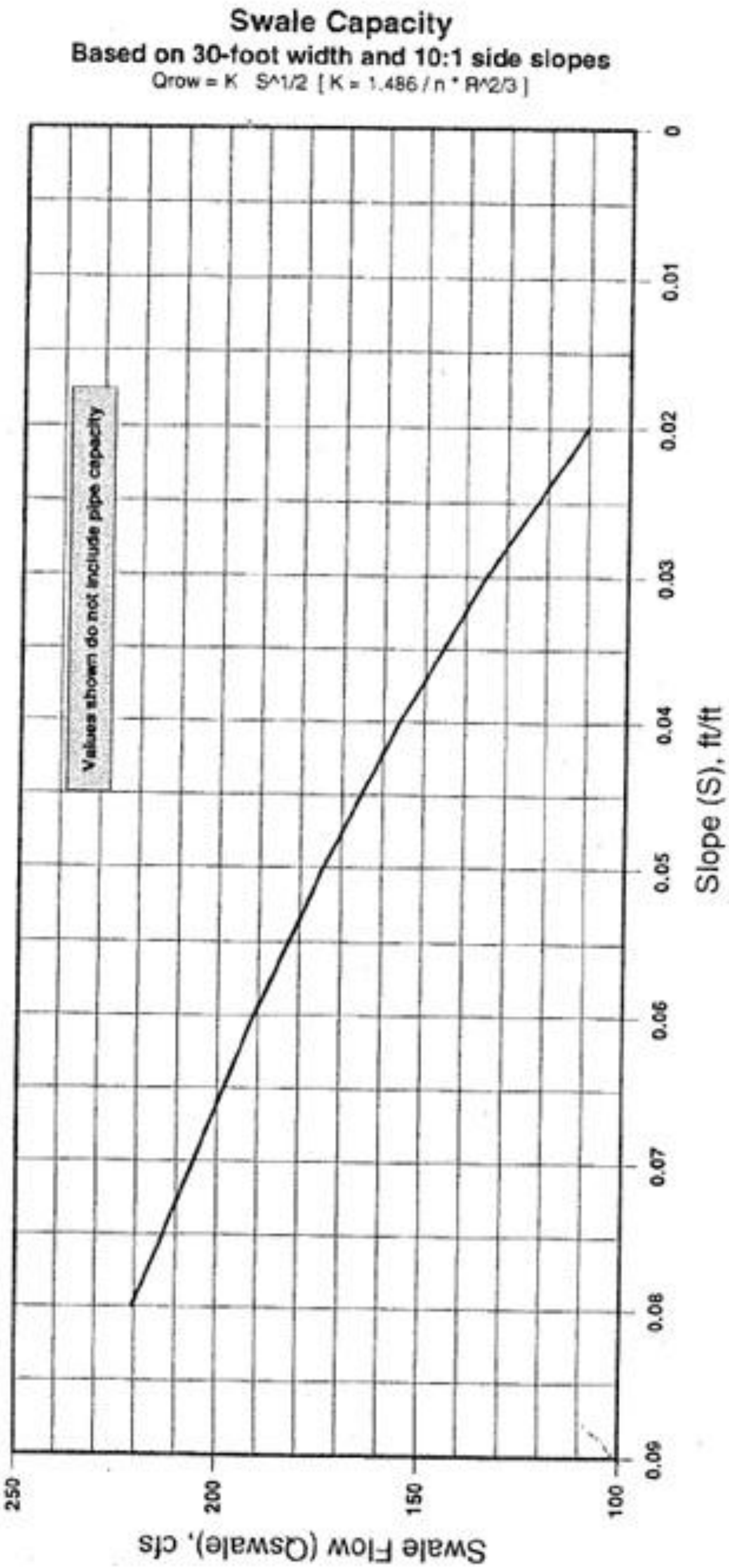


Figure 3-10 Swale Capacity Chart

TABLE III: Circular Concrete and Corrugated Metal Pipe								
Pipe Dia. (Inch)	A Area (Square Feet)	R Hydraulic Radius (Feet)	Values of $1.486/n \times A \times R^{2/3}$					
			Concrete Pipe			Corrugated Metal Pipe		
			n = 0.011	n = 0.012	n = 0.013	2-2/3" x 1/2" n = 0.024	3" x 1" n = 0.027	6" x 2" n = 0.033
8	0.349	0.167	14.3	13.1	12.1	6.5		
10	0.545	0.208	25.8	23.6	21.8	11.8		
12	0.785	0.250	42.1	38.6	35.7	19.3		
15	1.227	0.312	76.5	70.1	64.7	35.0		
18	1.767	0.375	124.2	113.8	105.1	56.9		
21	2.405	0.437	187.1	171.5	158.3	85.7		
24	3.142	0.500	267.4	245.1	226.2	122.5		
27	3.976	0.562	365.8	335.3	309.6	167.7		
30	4.909	0.625	484.7	444.3	410.1	222.2		
33	5.940	0.688	623.6	573.7	529.6	286.9		
36	7.069	0.750	788	722	666	361	321	
42	9.621	0.875	1189	1090	1006	545	484	
48	12.566	1.000	1698	1556	1436	778	692	
54	15.904	1.125	2325	2131	1967	1065	947	
60	19.635	1.250	3077	2821	2604	1410	1254	1026
66	23.758	1.375	3967	3636	3357	1818	1616	1323
72	28.274	1.500	5004	4587	4234	2293	2039	1668
78	33.183	1.625	6195	5679	5242	2839	2524	2065
84	38.485	1.750	7549	6920	6388	3460	3075	2517
90	44.179	1.875	9078	8321	7681		3698	3026
96	50.266	2.000	10776	9878	9119		4390	3592
102	56.745	2.125	12671	11615	10722			4224
108	63.617	2.250	14756	13526	12486			4919
114	70.882	2.375	17044	15624	14422			5682
120	78.540	2.500	19544	17915	16537			6515
126	86.590	2.625	22255	20397	18829			7417
132	95.030	2.750	25200	23104	21327			8401
138	103.870	2.875	28372	26009	24011			9459
144	113.100	3.000	31780	29133	26894			10594
150	122.720	3.125						11810
156	132.730	3.250						13115
162	143.140	3.375						14504
168	153.940	3.500						16160
174	165.130	3.625						17551
180	176.710	3.750						19212

Table 3-7 Values of 1.486/n x A x R^{-} for Circular Concrete and Corrugated Metal Pipe
Source: ACPA, Design Data 4, Hydraulic Capacity of Sewers, Table III

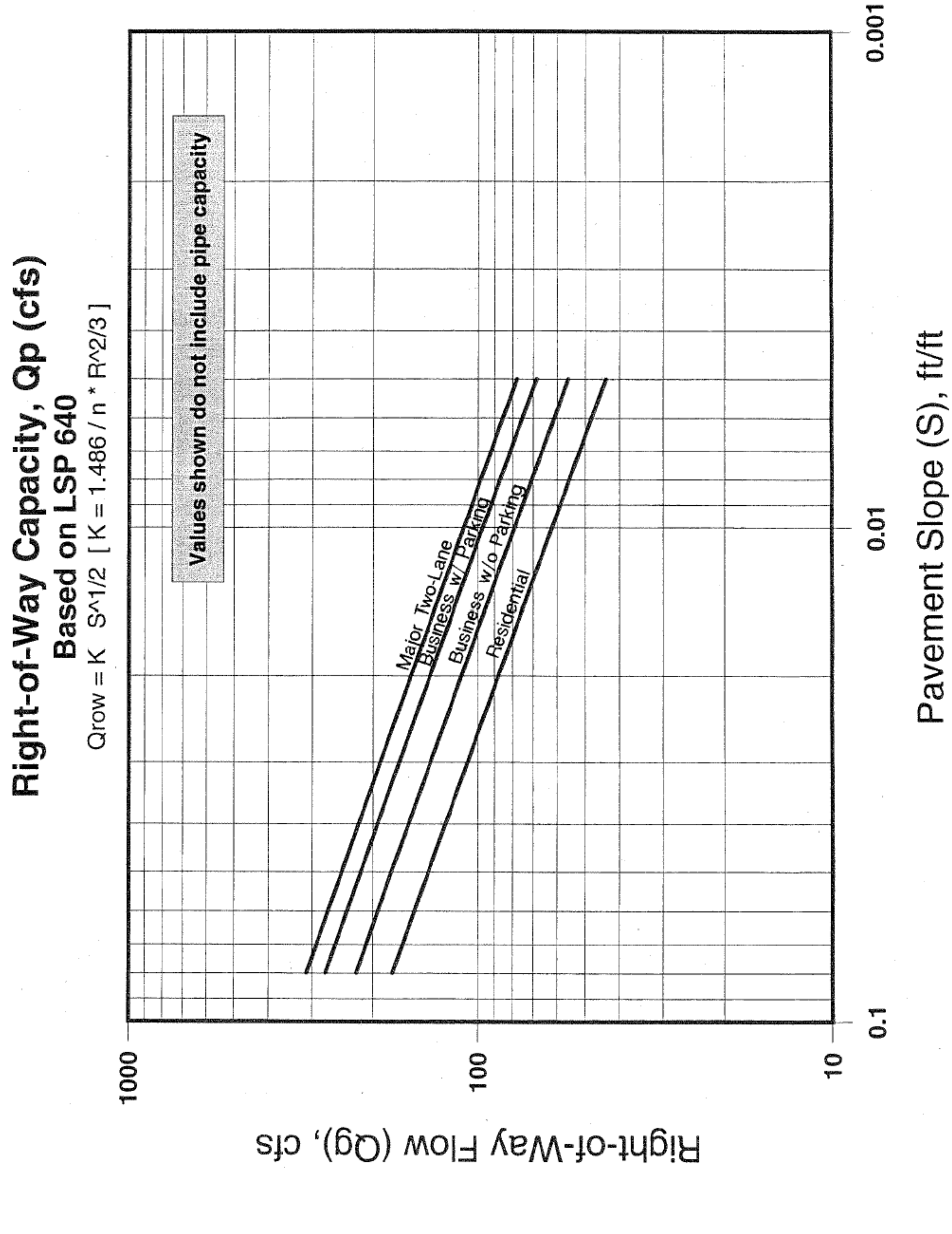


Figure 3-11 Right-of-Way Capacity Chart

Preliminary Pipe Sizing Calculations

Minor Storm System Conveyance Analysis

Minor Storm Average Return Frequency, years ________

Major Storm System Conveyance Analysis

Major Storm Average Return Frequency, 100 years

Location	Area, A acres	Coefficient, C	A * C	Sum A * C	Time of Concentration, Tc minutes	Intensity, I in/hour	Runoff, Qr cfs	Pipe Slope, Sp ft/ft	Pipe Length, L feet	Pipe Diameter, D inches	Pipe Capacity, Qp cfs	Pipe Velocity, Vp fps	Time in Section, Tp minutes	Intensity, I 100 in/hour	Flow, Q 100 cfs	Overflow Route Slope, Sov ft/ft	Street Width, Wst feet	Street Capacity, Qst cfs	Swale Width, W feet	Swale Capacity, Qsw cfs	Overflow plus Pipe Capacity, Qov cfs	Comments
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)

(3) = (1) x (2) (6) $I_F = 42.456\, F^{0.1943} / (T + 14)^{0.7912}$ (7) $Q_F = C \times I_F \times A$ (11) $Q_P = (1.486 / n) \times A \times R^{2/3} \times S^{1/2}$ (12) $V_P = Q_P / A_P = (11) / A_P$

(13) $T_P = L / 60 / V_P$ (14) $Q_{100} = C \times I_{100} \times A$ (18) $Q_{ST} = K_{ST} \times S_{OV}$ (20) $Q_{SW} = K_{SW} \times S_{OV}$ (21) Q_{OV} = (11) + (18) + (20)

Figure 3-12 Storm Drain Computation Form

Column (1) - Contributing area at the point-of-study
Column (2) - Coefficient of runoff for Rational Method, see Table 2-3 and Table 2-4
Column (3) - Product of area and coefficient of runoff, C x A or Col. (3) = Col. (1) x Col. (2)
Column (4) - Summation of Col. (3) for all contributing drainage basins to the point-of-study
Column (5) - Time of concentration to the point of study for the drainage basin or the accumulated travel time of the aggregate drainage basins, whichever is greater, T_c
Column (6) - Minor storm rainfall intensity, from Figure 2-3
or $I = 42.456 \; F^{0.1943} / (T_c+14.0)^{0.7912}$; F = Average Return Frequency
Column (7) - Peak rate of flow for minor storm runoff at the point-of-study, $Q_r = CIA$
or Col. (7) = Col. (4) x Col. (6)
Column (8) - Preliminary pipe slope
Column (9) - Pipe length segment from center to center of structures
Column (10) - Preliminary pipe size required to convey minor storm runoff. Indicate diameter or span x rise
Column (11) - Capacity of pipe for full flow conditions

$$Q = \frac{1.486}{0.013} A R^{-} S^{1/2} \quad \text{or Figure 3-13}$$

Column (12) - Velocity in the pipe for full-flow conditions, V = Q/A or Figure 3-13
Column (13) - Time of travel in pipe segment, $T_p = \frac{L}{60V}$ or Col. (13) = Col. (9) / Col. (12) / 60

Column (14) - 100-year storm rainfall intensity, from Figure 2-3
or $I_{100} = 103.882 / (T_c + 14)^{0.7912}$
Column (15) - Peak rate of flow for 100-year storm runoff at the point-of-study, $Q_{100} = CI_{100}A$
or Col. (15) = Col. (4) x Col. (14)
Column (16) - Slope of overland flow route for 100-year storm runoff
Column (17) - Street and right-of-way width
Column (18) - Street capacity for flow to the limits of right-of-way for LSP-640

$$Q = \frac{1.486}{n} A R^{-} S^{1/2}$$

$K = \frac{1.486}{n} A R^{-}$, is constant for full depth flow conditions.

K for each standard street and ROW width is provided below (e.g., K 26/60 for a 26' street with a 60-foot ROW)

Residential $K_{(26/60)}$ = 620   Business with parking $K_{(38/72)}$ = 970
Major two lane $K_{(32/80)}$ = 1100   Business without parking $K_{(33/66)}$ = 790
30-foot Swale K_{swale} = 780
or See Figure 3-10 or Figure 3-11

Column (19) - Combined capacity of the street and minor drainage systems must be equal to or greater than the peak rate of flow for the 100-year storm.
Column (20) - Swale width, where flow from major storms is not contained in the street system an overland flow route must be provided.
Column (21) - Combined capacity of the swale and minor systems must be equal to or greater than the peak rate of flow for the major storm.
Column (22) - Clarifying comments

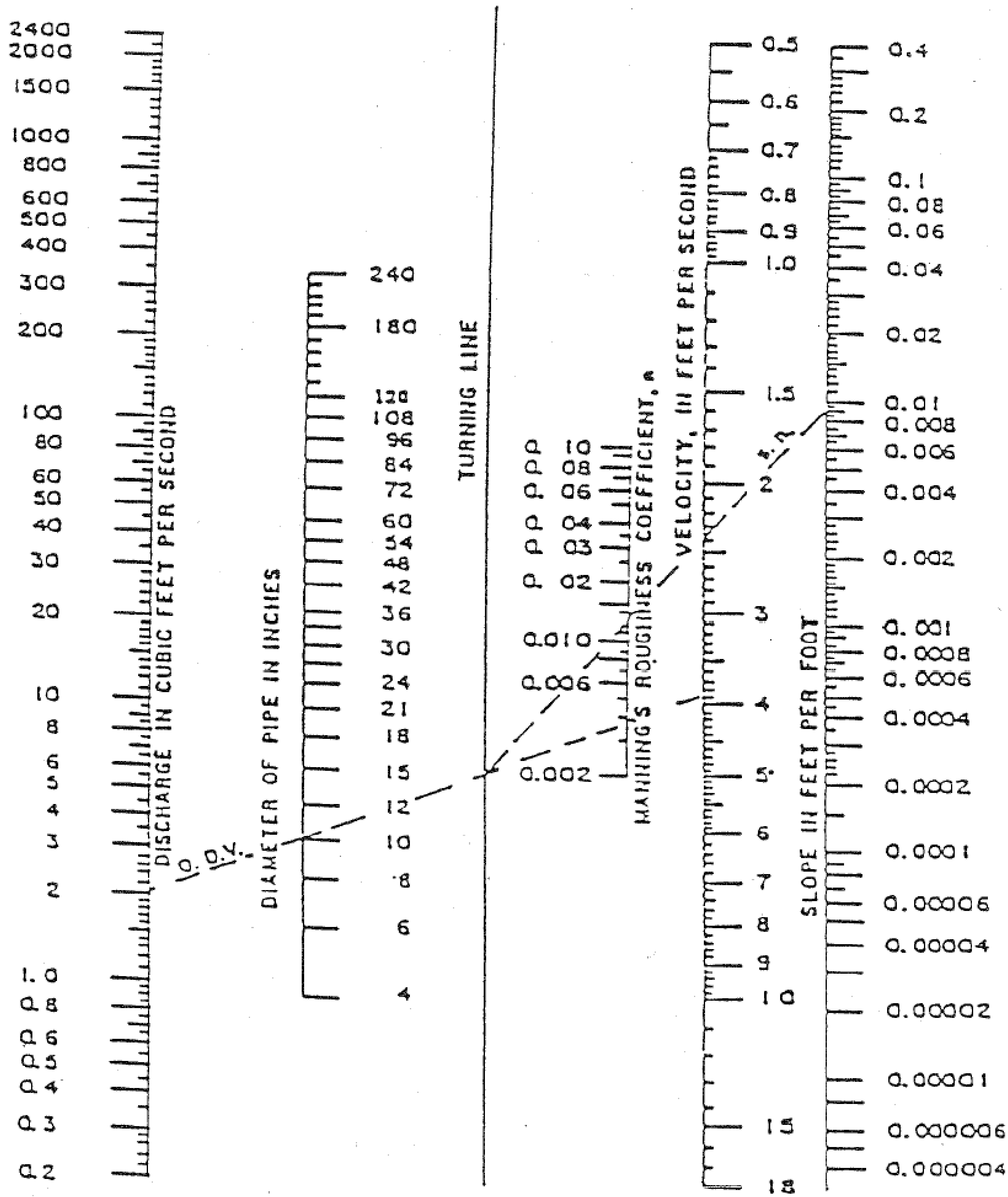


Figure 3-13 Nomograph For Solution of Manning's Formula In Storm Drains

3.5.5 Hydraulic Gradient

In order to determine if design flows can be accommodated by the storm drains system without causing flooding, or causing flows to exit the system at unacceptable locations, the designer shall determine the hydraulic gradient. Computing the hydraulic gradient will determine the elevation to which water will rise in inlets and manholes. The following sections provide the necessary procedures and equations to determine the hydraulic gradient.

3.5.5.1 Friction Losses

Energy losses from pipe friction may be determined by rewriting the Manning equation.

$$\mathbf{S_f = [Qn/1.486\ A(R^{2/3})]^2} \qquad \textbf{(3.12)}$$

Then the head losses due to friction may be determined by the formula:

$$\mathbf{H_f = S_f L} \qquad \textbf{(3.13)}$$

Where: H_f = friction head loss (ft)
S_f = friction slope (ft/ft)
L = length of outflow pipe (ft)

3.5.5.2 Velocity Head Losses

From the time storm water first enters the sewer system at the inlet until it discharges at the outlet, it will encounter a variety of hydraulic structures such as inlets, manholes, junctions, bends, contractions, enlargements and transitions, which will cause velocity head losses. Velocity losses may be expressed in a general form derived from the Bernoulli and Darcy-Weisback equations.

$$\mathbf{H = KV^2/2g} \qquad \textbf{(3.14)}$$

Where: H = velocity head loss (ft)
K = loss coefficient for the particular structure
V = velocity of flow (ft/s)
g = acceleration due to gravity (32.2 ft/s)

3.5.5.3 Entrance Losses

Following are the equations used for entrance losses.

$$\mathbf{H_{tm} = V^2/2g} \qquad \textbf{(3.15)}$$
$$\mathbf{H_e = KV^2/2g} \qquad \textbf{(3.16)}$$

Where: H_{tm} = terminal (beginning of run) loss (ft)
H_e = entrance loss for outlet structure (ft)
K = 0.5 (assuming square-edge)
(Other terms defined above.)

3.5.5.4 Junction Losses

Incoming Opposing Flows

The head loss at a junction, Hj1 for two almost equal and opposing flows meeting head on with the outlet direction perpendicular to both incoming directions, head loss is considered as the total velocity head of outgoing flow.

$$\mathbf{H_{j1} = (V^2)/2g} \qquad \mathbf{(3.17)}$$

Where: H_{j1} = junction losses (ft)
(Other terms are defined above.)

Changes in Direction of Flow

When main storm drain pipes or lateral lines meet in a junction, velocity is reduced within the chamber and specific head increases to develop the velocity needed in the outlet pipe. The sharper the bend (approaching 90o) the more severe this energy loss becomes. When the outlet conduit is sized, determine the velocity and compute head loss in the chamber by the formula:

$$\mathbf{H_b = K_b(V^2)/2g} \qquad \mathbf{(3.18)}$$

Where: H_b = bend head loss (ft)
K_b = junction loss coefficient

The following Table 3-8 lists the values of Kb for various changes in flow direction and junction angles.

Table 3-8 Values Of K_b For Change In Direction Of Flow In Lateral

K	Degree of Turn (In Junction)
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90 and greater

K values for other degree of turns can be obtained by interpolating between values.

Table 3-9 lists the values for the junction loss coefficient for various conditions at pipe junctions.

Table 3-9 Values Of K At Junctions

For no bends at junctions -	K = 0.20
For bends at junctions of 25 degrees -	K = 0.30
For bends at junctions of 45 degrees -	K = 0.40
For bends at junctions of 90 degrees -	K = 0.60
For junctions of three pipes -	K = 0.80
For junctions of four or more pipes -	K = 1.00

Several Entering Flows

The computation of losses in a junction with several entering flows utilizes the principle of conservation of energy. For a junction with several entering flows, the energy content of the inflows is equal to the energy content of outflows plus additional energy required by the collision and turbulence of flows passing through the junction. The total junction losses can be determined from equation 3-17. See also Figure 3-14.

$$\mathbf{H_{j2} = [(Q_4V_4^2)-(Q_1V_1^2)-(Q_2V_2^2)+(KQ_1V_1^2)]/(2gQ_4)} \quad \mathbf{(3.19)}$$

Where: H_{j2} = junction losses (ft)
Q = discharges (cfs)
V = horizontal velocities (ft/s) (V_3 is assumed to be zero)
g = acceleration due to gravity (32.2 ft/s^2)
K = bend loss factor

Where subscript nomenclature is as follows:

Q_1 = 90^o lateral (cfs)
Q_2 = straight through inflow (cfs)
Q_3 = vertical dropped-in flow from an inlet (cfs)
Q_4 = main outfall = total computed discharge (cfs)
V_1,V_2,V_3,V_4 are the horizontal velocities of foregoing flows, respectively, in feet per second
V_3 assumed to be = 0

Also assume:

- Hb = K(V12)/2g for change in direction.
- No velocity head of an incoming line is greater than the velocity head of the outgoing line.
- Water surface of inflow and outflow pipes in junction to be level.

When losses are computed for any junction condition for the same or a lesser number of inflows, the above equation will be used with zero quantities for those conditions not present. If more directions or quantities are at the junction, additional terms will be inserted with consideration given to the relative magnitudes of flow and the coefficient of velocity head for directions other than straight through.

3.5.5.5 Summary

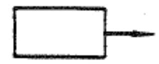
The final step in designing a storm drain system is to check the hydraulic grade line (HGL) as described in the next section of this chapter. Computing the HGL will determine the elevation, under design conditions, to which water will rise in various inlets, manholes, junctions, and etc. The following design criteria shall be followed when determining the elevation at the HGL:

- The hydraulic grade line shall be 0.75 feet below the intake lip of any affected inlet, any manhole cover, or any entering nonpressurized system.
- The energy grade line shall not rise above the intake lip of any affected inlet, any manhole cover or any entering nonpressurized system.

A summary of energy losses which shall be considered is presented in Figure 3-14.

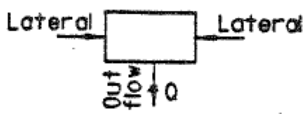
$$H_{tm} = \frac{V^2}{2g}$$

TERMINAL LOSSES
(at beginning of run)
Where g = gravitational constant, 32.2 feet per second per second.



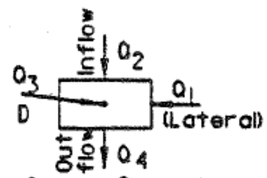
$$H_e = \frac{0.5\,V^2}{2g}$$

ENTRANCE LOSSES
(at end of run)
Assuming square - edge



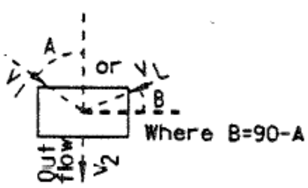
$$H_{J1} = \frac{V^2}{2g}\ \text{(Outflow)}$$

JUNCTION LOSSES
(Incoming-opposing Flow)
Use only where flows are identical to above, otherwise use H_{J2} Equation.



$$H_{J2} = \frac{Q_4V_4^2 - Q_1V_1^2 - Q_2V_2^2 + KQ_1V_1^2}{2gQ_4}$$

JUNCTION LOSSES
(Several Entering Flows)
Total losses to include H_{J2} plus losses for changes in direction of less than 90° (h_b).
Where K = Bend loss coefficient
Q_3 = Vertical dropped-in flow from an inlet
V_3 = Assumed to be zero



$$H_b = \frac{KV_1^2}{2g}$$

BEND LOSSES
(changes in direction of flow)

Where K	Degree of Turn (A) in Junction
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90

FRICTION LOSSES $(H)_f$

$H_f = S_f \times L$
Where H_f = friction head loss
S_f = friction slope
L = length of conduit

$$\text{Where } S = \left(\frac{Qn}{1.486AR^{\frac{2}{3}}}\right)^2$$

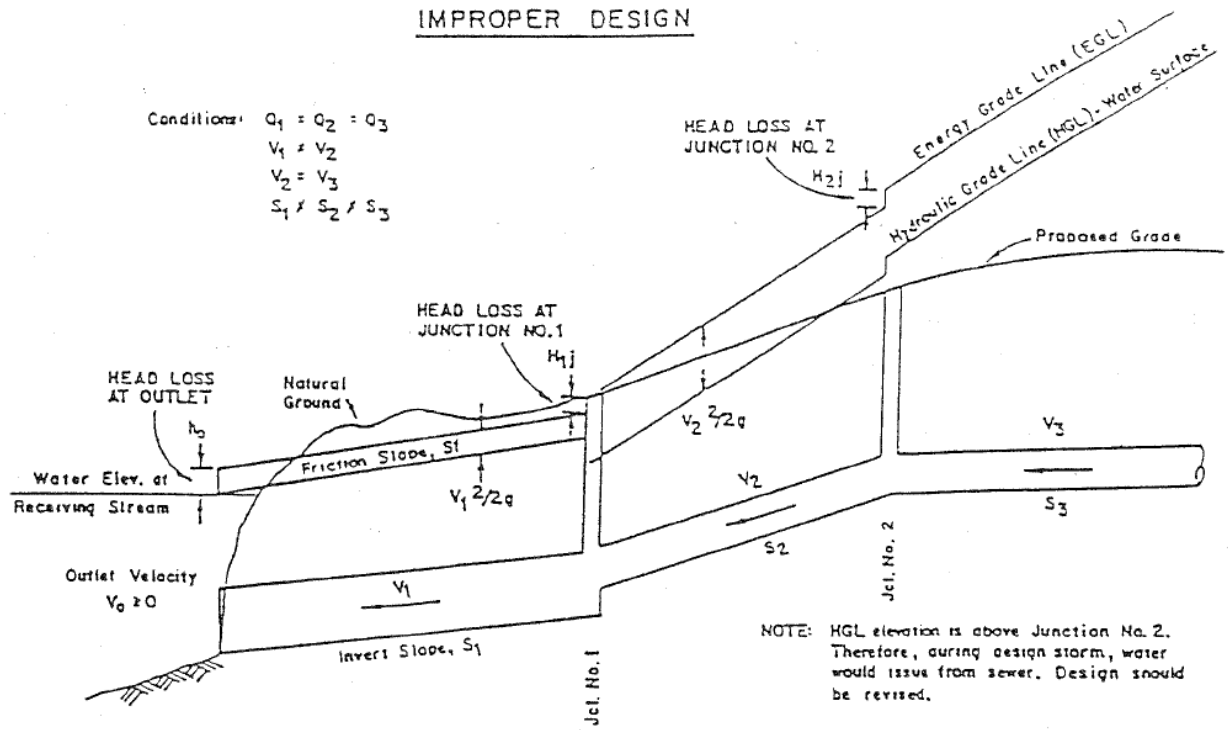
Q = Discharge of conduit
n = Mannings coefficient of roughness
A = area of conduit
R = hydraulic radius of conduit

TOTAL ENERGY LOSSES AT EACH JUNCTION

$$H_T = H_{tm} + H_e + (H_{J1} \text{ or } H_{J2}) + H_b + H_f$$

Figure 3-14 Summary Of Energy Losses

Source: AASHTO Model Drainage Manual, 1991



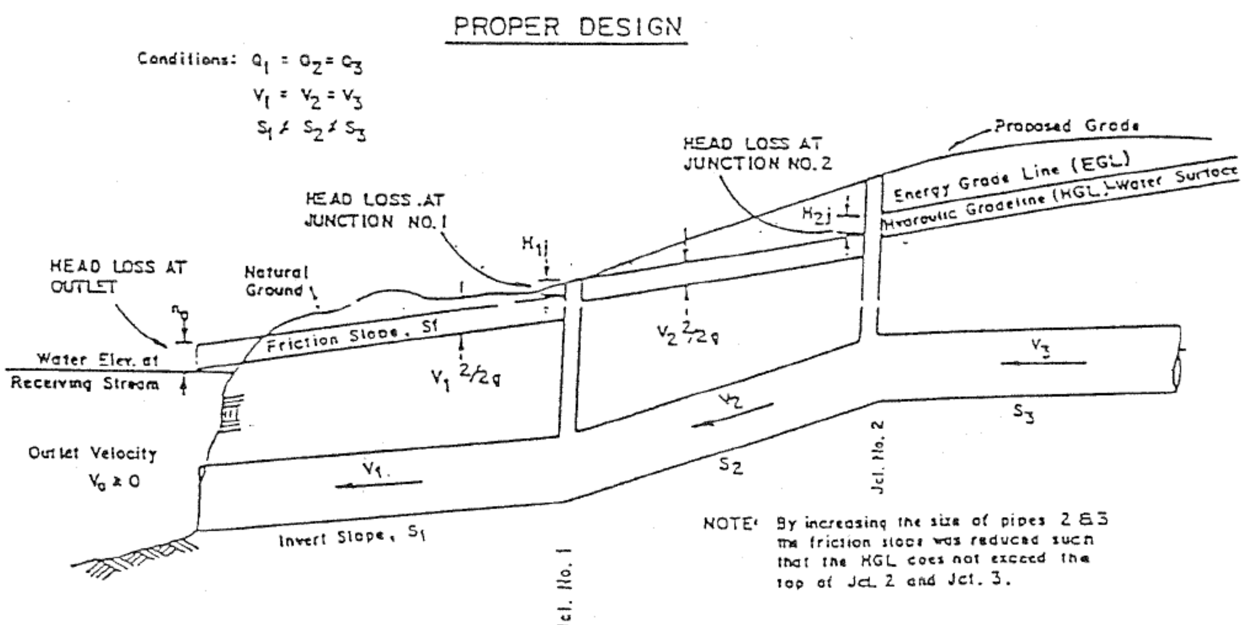


Figure 3-15 Energy And Hydraulic Grade Lines For Storm drain Under Constant Discharge

Source: AASHTO Model Drainage Manual, 1991

3.5.6 Hydraulic Grade Line Design Procedure

The hydraulic grade line is calculated beginning at the system outlet proceeding upstream. Conditions expected at the outlet for the minor design storm shall be used for the starting water surface elevation.

Hydraulic Grade Line Calculation Sheet

Minor Storm Ave. Return Frequency ______

Pipe Segment		Q (cfs)	Length (feet)	Pipe Size (in)	Kp	Outlet Elevation (feet)	Inlet Elevation (feet)	Barrel Area (sqft)	Barrel Velocity (fps)	Barrel Velocity Head (feet)	Tail Water Elevation (feet)	Friction Loss (feet)	Entrance HGL Elevation (feet)	Entrance Head Loss (feet)	Exit Head Loss (feet)	Outlet Control Elevation (feet)	Inlet Control Elevation (feet)	Appr. Vel. Head (feet)	Bend Head Loss (feet)	Junc. Head Loss (feet)	HW Elevation (feet)	Top Curb Elevation (feet)	Elevation Difference (ft)	Comment
from	to	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)

(8) = Q/A = (1)/(7) (9) = $h_v = V^2/2g$ = (8)/64.4 (11) = h_f = SL; S_f = Q/K; K from Table 3-7 (12) 10 + 11

(13) = $K_e(V^2/2g) = K_e(9)$ (14) = $1.0(V^2/2g) = 1.0(9)$ (15) 12+13+14 (16) Figure 4-2 (17) = (9)

(18) = $K_b(V^2/2g) = K_b(17)$ (19) = $K_j(V^2/2g) = K_j(17)$ (20) = (15 or 16)-(17)+(18)+(19) (22) Reduces Inlet Capacity if < 1.5 feet

Figure 3-16 Hydraulic Grade Line Computation Form

Column (1) - Design flow to be conveyed by pipe segment.
Column (2) - Length of pipe segment.
Column (3) - Pipe Size; Indicate pipe diameter or span x rise.
Column (4) - Constant K, from American Concrete Pipe Association <u>Design Data</u>:

$$Kp = \frac{1,486}{n} AR^{-} \text{; or from Table 3-7}$$

Column (5) - Flowline Outlet Elevation of pipe segment.
Column (6) - Flowline Inlet Elevation of pipe segment.
Column (7) - Barrel Area is the full cross sectional area of the pipe.
Column (8) - Barrel Velocity is the full velocity in the pipe as determined by:

$$V = Q/A \quad \text{or} \quad \text{Col. (8)} = \text{Col. (1)} \ / \ \text{Col. (7)}$$

Column (9) - Barrel Velocity Head = $V^2/2g$ or Col. $(8)^2 / 2g$
Where, g = 32.2 ft/sec^2 (acceleration due to gravity)

Column (10) - Tailwater (TW) Elevation; this is the water surface elevation at the outlet of the pipe segment. If the pipe's outlet is not submerged by the TW and the TW depth is less than $(D+d_c)/2$, set the TW elevation equal to $(D+d_c)/2$. This will keep the analysis simple yet still obtain reasonable results (D = pipe barrel height and d_c = critical depth, both in ft. See Appendix 4-B for determination of d_c).

Column (11) - Friction Loss = S_1 x L or S_1 x Col. (2)
Where, S_1 is the friction slope or head loss per lineal foot of pipe as determined by Manning's Equation expressed in the form:

$$S_1 = S_f = (Q / K_p)^2; \quad \text{K from Table 3-7}$$

Column (12) - Hydraulic Grade Line (HGL) Elevation just inside the entrance of the pipe barrel; this is determined by adding the friction loss to the TW elevation:

Col. (12) = Col. (11) + Col. (10)

If this elevation falls below the pipe's inlet crown, it no longer represents the true HGL when computed in this manner. The true HGL will fall somewhere between the pipe's crown and either normal flow depth or critical flow depth, whichever is greater. To keep the analysis simple and still obtain reasonable results (i.e., erring on the conservative side), set the HGL elevation equal to the crown elevation.

Column (13) - Entrance Head Loss = K_e x V^2 / 2g or K_e x Col. (9) Where, K_e = Entrance Loss Coefficient (0.5 assuming square-edge) This is the head lost due to flow contractions at the pipe entrance.

Column (14) - Exit Head Loss = 1.0 x V^2 / 2g or 1.0 x Col. (9)
This is the velocity head lost or transferred downstream.

Column (15) - Outlet Control Elevation = Col. (12) + Col. (13) + Col. (14)
This is the maximum headwater elevation assuming the pipe's barrel and inlet/outlet characteristics are controlling capacity. It does not include structure losses or approach velocity considerations.

Column (16) - Inlet Control Elevation (See Figure 4-2 for computation of inlet control on culverts). This is the maximum headwater elevation assuming the pipe's inlet is controlling capacity. It does not include structure losses or approach velocity considerations.

Column (17) - Approach Velocity Head; this is the head (energy) being supplied by the discharge form an upstream pipe or channel section, which serves to reduce the headwater elevation. If the discharge is from a pipe, the approach velocity head is equal to the barrel velocity head computed for the upstream pipe. If the upstream pipe outlet is significantly higher in elevation (as in a drop manhole) or lower in elevation such that its discharge energy would be dissipated, an approach velocity head of zero should be assumed.

Column (18) - Bend Head Loss = K_b x V^2 / 2g or k_b x Col. (17)
Where, K_b = Bend Loss Coefficient (from Table 3-7). This is the loss of head/energy required to change direction of flow in an access structure.

Column (19) - Junction Head Loss; this is the loss in head (energy) that results from the turbulence created when two or more streams are merged into one within the access structure. Table 3-8 can be used to determine junction loss coefficients for use in the following equations given in Figure 3-14.

Column (20) - Headwater (HW) Elevation; this is determined by combining the energy heads in Columns 17, 18, and 19 with the highest control elevation in either Column 15 or 16, as follows:

Col. (20) = Col. (15 or 16) - Col. (17) + Col. (18) + Col. (19)

Column (21) - Top of curb elevation at an inlet or rim elevation at a storm sewer manhole.

Column (22) - Inlet capacity is reduced if the hydraulic gradeline elevation interferes with the napping effect during weir or orifice flow conditions.

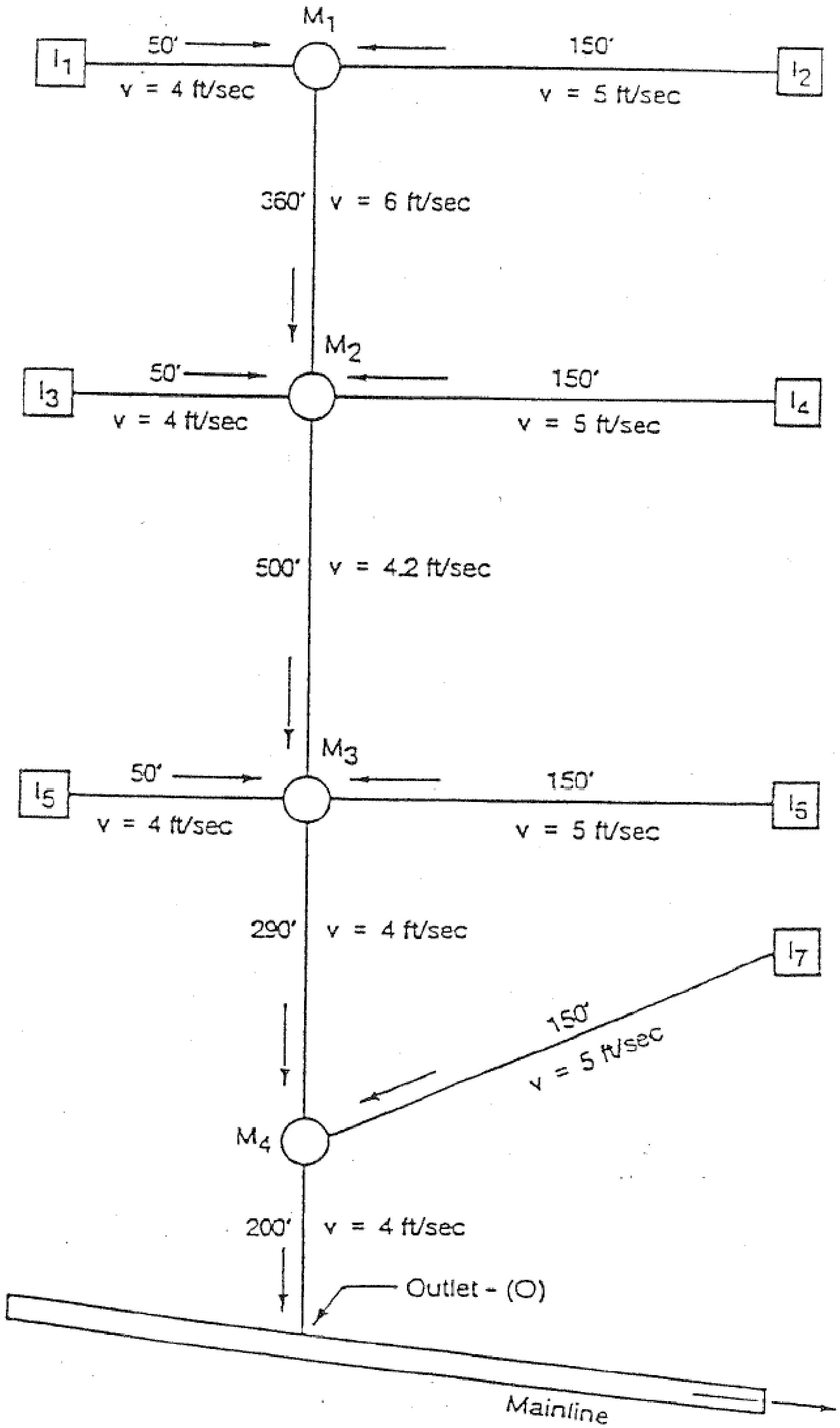


Figure 3-17 Hypothetical Storm Drain System Layout

3.6 Computer Programs

There are numerous proprietary and non-proprietary computer models that may be used to design components of the minor storm drainage system. The reader is referred to the user manual for any particular program to determine its suitability for solving storm drainage problems.

References

U. S. Department of Transportation, Federal Highway Administration, 1984. Drainage of Highway Pavements. Hydraulic Engineering Circular No. 12.

American Concrete Pipe Association, March 1968, Design Data.

CHAPTER 4

DESIGN OF CULVERTS

March 7, 2011

Chapter Four - Design of Culverts

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Chapter Four - Design of Culverts

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4.1 Overview

4.1.1 Introduction

The design of a culvert is influenced by cost, hydraulic efficiency, purpose, and the topography at the proposed culvert site. Thus physical data must be integrated with engineering and economic considerations. The information contained in this chapter should give the design engineer the ability to design culverts taking into account the factors that influence their design and selection.

4.1.2 Definition

Culverts are structures used to convey surface runoff through embankments. Culverts are usually covered with embankment and composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert. For economy and hydraulic efficiency, culverts should be designed to operate with the inlet submerged during flood flows, if conditions permit. Cross-drains are those culverts and pipes that are used to convey runoff from one side of a roadway to another.

4.1.3 Purpose

The primary purpose of a culvert is to convey surface water across or from the roadway right-of-way. In addition to the hydraulic function, a culvert must also support the embankment and roadway for traffic conveyance, and protect the traveling public and adjacent property owners from flood hazards to the extent practicable and in a reasonable and prudent manner.

4.1.4 Considerations

Primary considerations for the final selection of any drainage structure are that its design be based upon appropriate hydraulic principles, economy, and minimized effects on adjacent property by the resultant headwater depth and outlet velocity. In addition to sound hydraulic design, sound structural design, site design, and construction practices are necessary for a culvert to function properly. The allowable headwater elevation is that elevation above which damage may be caused to adjacent property and/or the roadway. It is this allowable headwater depth that is the primary basis for sizing a culvert.

To ensure safety during major flood events, access and egress routes to developed areas shall be checked for the 100-year flood to determine if these streets will provide safe access for emergency vehicles and local residents.

4.1.5 Bridge or Culvert Selection

At many sites, either a bridge or a culvert will fulfill the structural and hydraulic requirements. The structural choice should be based on:

- risk of property damage,
- construction and maintenance costs,
- traffic safety,
- environmental considerations,
- risk of failure, and
- aesthetic considerations.

4.2 Symbols And Definitions

To provide consistency within this chapter, as well as throughout this manual, the following symbols will be used. These symbols were selected because of their wide use in many culvert design publications.

Table 4-1 Symbols, Definitions And Units

Symbol	Definition	Units
A	Area of cross section of flow	sq. ft
B	Barrel width	ft
C_d	Overtopping discharge coefficient	-
D	Culvert diameter or barrel depth	in. or ft
d	Depth of flow	ft
d_c	Critical depth of flow	ft
d_u	Uniform depth of flow	ft
g	Acceleration of gravity	ft/s
H	Total energy loss	ft
H_e	Entrance head loss	ft
H_f	Friction head loss	ft
h_o	Height of hydraulic grade line above outlet invert	ft
HW	Headwater depth above invert of culvert (depth from inlet invert to upstream total energy grade line)	ft
K_e	Inlet loss coefficient	-
L	Length of culvert	ft
P	Empirical approximation of equivalent hydraulic grade line	ft
Q	Rate of discharge	cfs
S	Slope of culvert	ft/ft
TW	Tailwater depth above invert of culvert	ft
V	Mean velocity of flow	ft/s
V_c	Critical velocity	ft/s

4.3 Concept Definitions

Critical Depth

Critical depth can best be illustrated as the depth at which water flows over a weir, this depth being attained automatically where no other backwater forces are involved. For a given discharge and cross-section geometry there is only one critical depth. Appendix B at the end of this chapter gives a series of critical depth charts for the different culvert shapes.

Uniform Flow

Uniform flow is flow in a prismatic channel of constant cross section having a constant discharge, velocity and depth of flow throughout the reach. This type of flow will exist in a culvert operating on a steep slope provided the culvert is sufficiently long.

Free Outlets

Free outlets are outlets whose tailwater is equal to or lower than critical depth. For culverts having free outlets, lowering of the tailwater has no effect on the discharge or the backwater profile upstream of the tailwater.

Submerged Outlets

Partially submerged outlets are outlets whose tailwater is higher than critical depth and lower than the height of the culvert. Submerged outlets are outlets having a tailwater elevation higher than the soffit of the culvert.

Submerged Inlets

Submerged inlets are those inlets having a headwater greater than about one and one-half times the diameter of the culvert.

Improved Inlets

Flared, improved, or tapered inlets indicate a special entrance condition which decreases the amount of energy needed to pass the flow through the inlet and thus increases the capacity of culverts at the inlet.

Soffit

Soffit refers to the inside top of the culvert. The soffit is also referred to as the crown of the culvert.

Invert

Invert refers to the flowline of the culvert (inside bottom).

Steep and Mild Slope

A steep slope culvert operation is where the computed critical depth is greater than the computed uniform depth. A mild slope culvert operation is where critical depth is less than uniform depth.

4.4 Culvert Design Steps

Following are the recommended steps in the design of a culvert in order to ensure that all design aspects are taken into account.

Step 1 Determine And Analyze Site Characteristics - Site characteristics include the generalized shape of the roadway embankment, bottom elevations and cross sections along the stream bed, the approximate length of the culvert, and the allowable headwater elevation. In determining the allowable headwater elevation, roadway elevations and the elevation of upstream property should be considered. The consequences of exceeding the allowable headwater elevation should be evaluated and kept in mind throughout the design process.

Culvert design is actually a trial-and-error procedure because the length of the barrel cannot be accurately determined until the size is known, and the size cannot be precisely determined until the length is known. In most cases, however, a reasonable estimate of length will be accurate enough to determine the culvert size.

Step 2 Perform Hydrologic Analysis - Delineate the drainage area above the culvert site. Develop flow estimates for the design frequencies. Design frequencies are discussed in Section 4.5.2. The probable accuracy of the estimate should be kept in mind as the design proceeds.

Step 3 Perform Outlet Control Calculations And Select Culvert - These calculations are performed before inlet control calculations in order to select the smallest feasible barrel which can be used without the required headwater elevation in outlet control exceeding the allowable headwater elevation. The full flow outlet control performance curve for a given culvert (size, inlet edge, shape, material) defines its maximum performance. Therefore, the inlet improvements beyond the beveled edge or changes in inlet invert elevation will not reduce the required outlet control headwater elevation. This makes the outlet control performance curve an ideal limit for improved inlet design. The results of these calculations should be the outlet control performance curve. In addition to considering the allowable headwater elevation, the velocity of flow at the exit to the culvert should be checked to determine if downstream erosion problems will be created.

Step 4 Perform Inlet Control Calculations For Conventional And Beveled Edge Culvert Inlets - Perform the inlet control calculations to develop the inlet control performance curve to determine if the culvert design selected will be on inlet or outlet control for the design and check flood frequencies. A drop may be incorporated upstream of the culvert to increase the flow through the culvert.

Step 5 Perform Throat Control Calculations For Side- And Slope-Tapered Inlets - The same concepts are involved here as with conventional or beveled edge culvert design.

Step 6 Analyze The Effect of a Drop On Inlet Control Section Performance -The purpose of this step is to determine if having a drop before the inlet of the culvert would increase the capacity of the culvert and if a drop can be justified from a cost perspective and site characteristics.

Step 7 Design Side- And/Or Slope-Tapered Inlet - Side- and slope-tapered inlets can be used to significantly increase the capacity of many culvert designs. Develop performance curves based on side- and/or slope-tapered inlets and determine from a cost perspective and site characteristics if such a design would be justified.

Step 8 Complete File Documentation - Complete a documentation file for the final design selected.

4.5 Engineering Design Criteria

4.5.1 Criteria

The design of a culvert should take into account many different engineering and technical aspects at the culvert site and adjacent areas. The following design criteria should be considered for all culvert designs as applicable.

Engineering aspects
- Flood frequency
- Velocity limitation
- Buoyancy protection

Site criteria
- Length and slope
- Debris control

Design limitations
- Headwater
- Tailwater conditions
- Storage

Design options
- Culvert inlets
- Inlets with headwalls
- Wingwalls and aprons
- Improved inlets
- Material selection
- Culvert skews
- Culvert sizes

Related designs
- Weep holes
- Outlet protection
- Erosion and sediment control
- Environmental considerations
- Safety considerations
- Loading requirements

Some culvert designs are relatively simple, involving a straight-forward determination of culvert size and length. Other designs are more complex where structural, hydraulic, environmental, or other considerations must be evaluated and provided for in the final design. The design engineer must incorporate personal experience and judgment to determine which criteria must be evaluated and how to design the final culvert installation.

Following is a discussion of each of the above criteria as it relates to culvert siting and design.

4.5.2 Flood Frequency

Culverts shall be designed to convey (at a minimum) the 50-year runoff event without overtopping the roadway. The flow rate shall be based on upstream full-buildout land-use conditions from the City of Waverly comprehensive plan. Where roadside ditches convey the minor storm drainage in lieu of storm sewers, appurtenant culverts shall be designed to convey the 10-year storm event, but in no case shall be less than the minimum sizes specified in Section 4.5.16 of this chapter.

In addition, the 100-year frequency storm shall be routed through all culverts to be sure structures are not flooded or increased damage does not occur to the roadway or adjacent property for this design event.

An economic analysis may justify a design to pass floods greater than those noted above where potential damage to adjacent property, to human life, or heavy financial loss due to flooding is significant.

Also, in compliance with the National Flood Insurance Program, it is necessary to consider the 100-year frequency flood at locations identified as being special flood hazard areas. This does not necessitate that the culvert be sized to pass the 100-year flood, provided the capacity of the culvert plus flow by-passing the culvert, is sufficient to accommodate the 100-year flood without raising the associated water surface elevation more than floodplain regulations or adjacent property elevations allow for that location. In addition, stormwater management facilities cannot be installed which would result in a major lowering of the associated water surface elevation without a downstream evaluation. The design engineer should review the City floodway regulations for more information related to floodplain regulations.

4.5.3 Velocity Limitations

Both minimum and maximum velocities should be considered when designing a culvert. The maximum velocity should not exceed culvert manufacturer recommendations. The maximum velocity should be consistent with channel stability requirements at the culvert outlet. As outlet velocities increase, the need for channel stabilization at the culvert outlet increases. If velocities exceed permissible velocities for the various types of nonstructural outlet lining material available, the installation of structural energy dissipators is appropriate.

A minimum velocity of 3.0 ft/s when the culvert is flowing partially full is recommended to ensure a self-cleaning condition during partial depth flow. Energy dissipation may be required at the outlet of the culvert (see Chapter 7).

4.5.4 Buoyancy Protection

Headwalls, endwalls, slope paving or other means of anchoring to provide buoyancy protection should be considered for all flexible culverts. Buoyancy is more serious with steepness of the culvert slope, depth of the potential headwater (debris blockage may increase), flatness of the upstream fill slope, height of the fill, large culvert skews, or mitered ends.

4.5.5 Length and Slope

Since the capacity of culverts on outlet control will be affected by the length of the culvert, their length should be kept to a minimum and existing facilities shall not be extended without determining the decrease in capacity that will occur. In addition, the culvert length and slope should be chosen to approximate existing topography. To the degree practicable, the culvert invert should be aligned with the channel bottom and the skew angle of the stream, and the culvert entrance should match the geometry of the roadway embankment.

4.5.6 Debris Control

The need for bar grates should be considered for each culvert site, but in general, bar grates shall not be used on end sections for culverts (either inlets or outlets) unless approved by the City.

4.5.7 Headwater Limitations

The allowable headwater elevation is determined from an evaluation of land use upstream of the culvert and the proposed roadway elevation. Headwater is the depth of water above the culvert invert measured at the entrance end of the culvert.

The following criteria related to headwater should be considered:

- The allowable headwater for design frequency conditions should allow for the following upstream controls:
  - 12 inch freeboard.
  - Avoidance of upstream property damage.
  - Elevations established to delineate floodplain zoning.
  - Low point in the road grade either adjacent to or away from the culvert location.
  - Ditch elevation of the terrain that would permit flow to divert around culvert.
- The headwater shall be checked for the 100-year flood to ensure compliance with floodplain management criteria and to avoid flooding of building sites. For most facilities, the culvert should be sized to maintain flood-free conditions on major thoroughfares for one-half lane of two-lane facilities and one lane of multi-lane facilities.
- The maximum acceptable outlet velocity shall be identified. Either the headwater shall be set to produce acceptable velocities or stabilization measures shall be provided where these velocities are exceeded.
- Site-specific design considerations shall be addressed.
- In general the constraint which gives the lowest allowable headwater elevation establishes the criteria for the hydraulic calculations.

Invert elevations will be established after determining the allowable headwater elevation, tailwater elevation, and approximate length. Scour can be minimized if the culvert has the same slope as the channel. Thus, to reduce the chance of failure due to scour, invert elevations should correspond to the natural grade where feasible. In addition, the flow conditions and velocity in the channel upstream from the culvert should be investigated to determine if scour will occur.

If there is insufficient headwater elevation to convey the required discharge, it will be necessary to either use a larger culvert, lower the inlet invert, use an irregular cross section, use an improved inlet if in inlet control, use multiple barrels or a bridge, or use a combination of these measures. If the inlet invert is lowered, special consideration must be given to scour.

4.5.8 Tailwater Considerations

The hydraulic conditions downstream of the culvert site must be evaluated to determine a tailwater depth for a range of discharge. At times there may be a need for calculating backwater curves to establish the tailwater conditions. If the culvert outlet is operating with a free outfall, the critical depth and equivalent hydraulic grade line should be determined.

For culverts which discharge to an open channel, the stage-discharge curve for the channel must be determined. (See Chapter 5.)

If an upstream culvert outlet is located near a downstream culvert inlet, the headwater elevation of the downstream culvert may establish the design tailwater depth for the upstream culvert.

If the culvert discharges to a lake, pond, or other major water body, the expected high water elevation of the particular water body may establish the culvert tailwater.

4.5.9 Freeboard

In the design of cross drainage culverts, there shall be a minimum of a one-foot freeboard between the flood elevation and the roadway surface for all floods that are equal to or less than the design flood event. In addition, there shall be a minimum of one-foot freeboard between the headwater elevation for a culvert under 100-year storm event flow or by-pass conditions and the low opening of upstream or adjacent building sites.

4.5.10 Culvert Inlets

Selection of the type of inlet is an important part of culvert design, particularly with inlet control. Hydraulic efficiency and cost can be significantly affected by inlet conditions.

The inlet coefficient Ke, is a measure of the hydraulic efficiency of the inlet, with lower values indicating greater efficiency. All methods described in this chapter, directly or indirectly, use inlet coefficients. Recommended inlet coefficients are given in Table 4-2.

Table 4-2 Inlet Coefficients

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

* Note: End Sections conforming to fill slope, made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections incorporating a closed taper in their design have a superior hydraulic performance.

Source: HDS:5

4.5.11 Inlets With Headwalls

Headwalls may be used for a variety of reasons:

(1) increasing the efficiency of the inlet
(2) providing embankment stability
(3) providing embankment protection against erosion
(4) providing protection from buoyancy or
(5) to shorten the length of the required structure.

The primary reasons for using headwalls are for embankment protection, buoyancy control, and ease of maintenance. Figure 4-1 shows typical headwall and wingwall configurations. Culvert or storm sewer headwalls constructed in or adjacent to public right-of-way shall be designed to protect pedestrians. This protection shall include a pipe railing fence on the headwall and any wingwalls, unless the grading and size of the pipe precludes the need for the fence, as approved by the City.

4.5.12 Wingwalls And Aprons

Wingwalls are used where the side slopes of the channel adjacent to the entrance are unstable or 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 other reasons. Wingwalls can be used to increase hydraulic efficiency if designed as a side-tapered inlet (See Section 4.9.6.2 for more information on the design of side-tapered inlets.)

If high headwater depths are to be encountered, or the approach velocity in 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.

4.5.13 Improved Inlets

Where inlet conditions control the amount of flow that can pass through the culvert, improved inlets can greatly increase the hydraulic performance at the culvert. For these designs refer to the section 4.9 which describes the design of improved inlets.

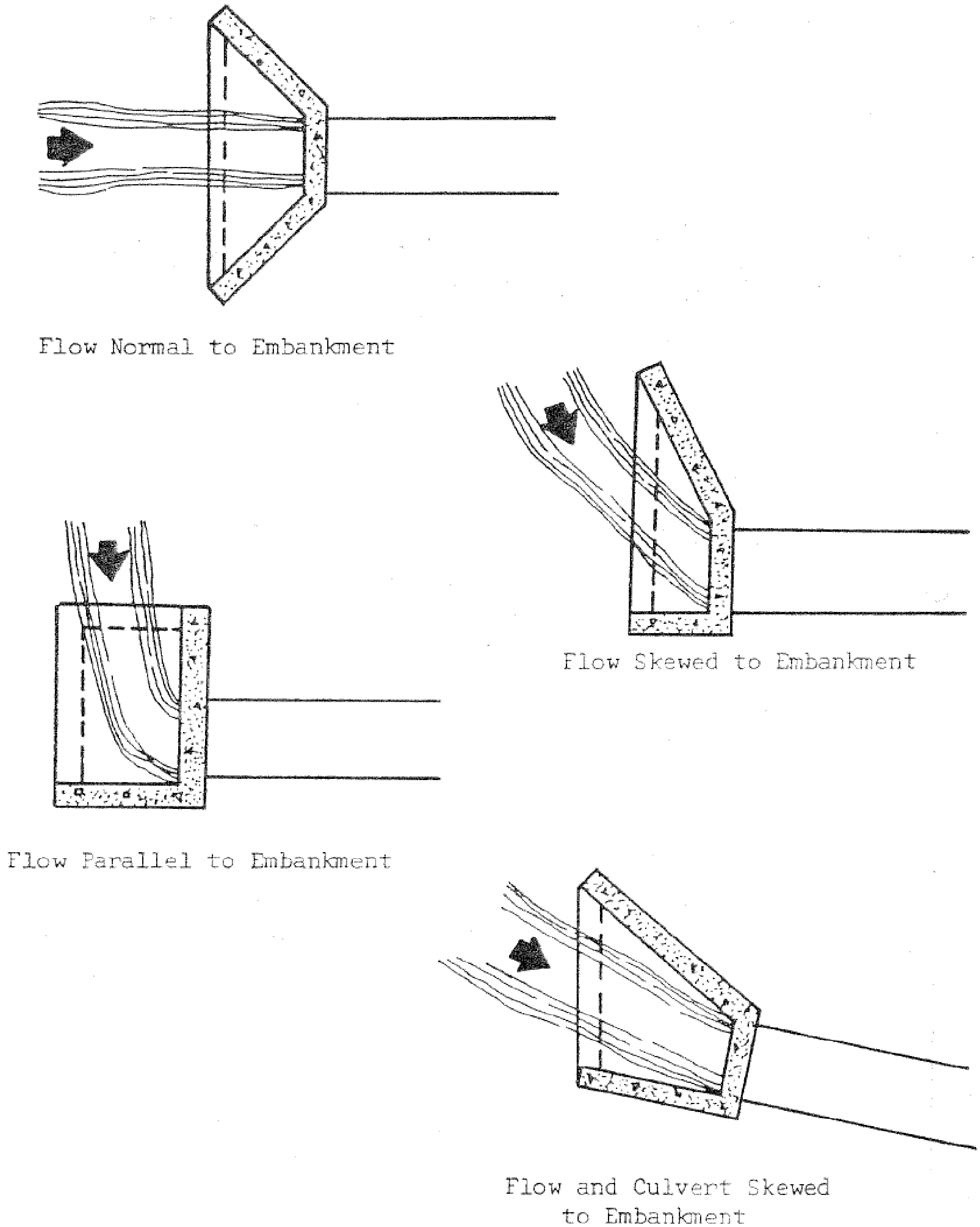


Figure 4-1 Typical Headwall and Wingwall Configurations

Source: Wright-McLaughlin Engineers

4.5.14 Manning’s n Values

For culvert selection, only reinforced concrete pipe is allowed within City street right-of-way except for driveway culverts. For culverts equal to or greater than 60 inches in diameter, corrugated metal pipe is allowed if it is bituminous coated with a concrete-poured invert. Table 4-3 gives recommended Manning's n values.

Table 4-3 Manning's n Values

Type of Conduit	Wall & Joint Description	Manning's n
Concrete Pipe	Good joints, smooth walls	0.011-0.013
	Good joints, rough walls	0.014-0.016
	Poor joints, rough walls	0.016-0.017
Concrete Box	Good joints, smooth finished walls	0.014-0.018
	Poor joints, rough, unfinished walls	0.014-0.018
Corrugated Metal Pipes and Boxes, Annular Corrugations	2 2/3 by 1/2-inch corrugations	0.027-0.022
	6 by 1-inch corrugations	0.025-0.022
	5 by 1-inch corrugations	0.026-0.025
	3 by 1-inch corrugations	0.028-0.027
	6 by 2-inch structural plate	0.035-0.033
	9 by 2 1/2-inch structural plate	0.037-0.033
Corrugated Metal Pipes	2 2/3 by 1/2-inch corrugated, 24-inch plate width	0.024-0.012
Helical Corrugations, Full Circular Flow Spiral Rib Metal Pipe	3/4 by 3/4-inch recesses at 12-inch spacing, good joints	0.012-0.013

Note: For further information concerning Manning n values for selected conduits, consult Hydraulic Design of Highway Culverts, Federal Highway Administration, HDS No. 5, page 163.

4.5.15 Culvert Skews

Culvert skews shall not exceed 45 degrees as measured from a line perpendicular to the roadway centerline without approval of the City.

4.5.16 Minimum Culvert Size

The minimum culvert size shall be 18 inches for roadways and 15 inches for driveways.

4.5.17 Outlet Protection

See Chapter 7 for information on the design of outlet protection. In general, scour holes at culvert outlets provide efficient energy dissipation. As such, outlet protection for the culvert should be provided where the outlet scour hole depth computations indicate:

- the scour hole will undermine the culvert outlet,
- the expected scour hole may cause costly property damage,
- the scour hole will cause a nuisance effect (most common in urban areas), or
- the scour hole will conflict with land use.

4.5.18 Permitting Considerations

There may be federal or state permitting implications that affect the culvert design. These could include wetlands, regulatory floodplains and preparation of a stormwater pollution prevention plan for construction activity.

4.5.19 Safety Considerations

Traffic should be protected from culvert ends as follows.

- Small culverts should use an end section or a sloped headwall.
- Large culverts should receive one of the following treatments:
  a. Be extended to the appropriate "clear zone" distance per AASHTO Roadside Design Guide.
  b. Shielded with a traffic barrier if the culvert is very large, cannot be extended, or has a channel which cannot be safely traversed by a vehicle.
- Routinely inspect each site to determine if safety problems exist for traffic or for the structural safety of the culvert and embankment.

4.5.20 Loading Requirements

Reinforced concrete box culverts, reinforced concrete pipe culverts, and corrugated metal pipe culverts shall all be designed for HS20 live load, with the appropriate impact factor, and dead load. Dead load (fill) shall be based on the depth of earth cover, plus pavement, above the top of the culvert.

4.6 Culvert Flow Controls And Equations

4.6.1 Introduction

Generally, the hydraulic control in a culvert will be at the culvert outlet if the culvert is operating on a mild slope. Entrance control usually occurs if the culvert is operating on a steep slope.

For outlet control, the head losses due to tailwater and barrel friction are predominant in controlling the headwater of the culvert. The entrance will allow the water to enter the culvert faster than the backwater effects of the tailwater and barrel friction will allow it to flow through the culvert.

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

The design procedures contained in this chapter are for the design of culverts for a constant discharge, considering inlet and outlet control.

4.6.2 Inlet And Outlet Control

Inlet Control - If the culvert is operating on a steep slope it is likely that the entrance geometry will control the headwater and the culvert will be on inlet control.

Outlet Control - If the culvert is operating on a mild slope, the outlet characteristics will probably control the flow and the culvert will be on outlet control.

Proper culvert design and analysis requires checking for both inlet and outlet control to determine which will govern particular culvert designs. For more information on inlet and outlet control see the Federal Highway Administration publication entitled Hydraulic Design Of Highway Culverts, HDS-5, 1985.

4.6.3 Equations

There are many combinations of conditions which classify a particular culvert's hydraulic operation. By consideration of a succession of parameters; the engineer may arrive at the appropriate calculation procedure. The most common types of culvert operations for any barrel type are classified as follows.

4.6.3.1 Mild Slope

Critical Depth - Outlet Control - The entrance is unsubmerged ($HW \leq 1.5D$), the critical depth is less than uniform depth at the design discharge ($d_c < d_u$) and the tailwater is less than or equal to critical depth ($TW \leq d_c$). This condition is a common occurrence where the natural channels are on flat grades and have wide, flat floodplains. The control is critical depth at the outlet.

$$\mathbf{HW = d_c + V_c^2/(2g) + H_e + H_f - SL} \qquad \mathbf{(4.1)}$$

Where: HW = headwater depth (ft)
d_c = critical depth (ft)
V_c = critical velocity
g = 32.2 (ft/sec^2)
H_e = entrance headloss (ft)
H_f = friction headloss (ft)
S = slope of culvert (ft/ft)
L = length of culvert (ft)

Tailwater Depth - Outlet Control - The entrance is unsubmerged ($HW \leq 1.5D$), the critical depth is less than uniform depth at design discharge ($d_c < d_u$) and TW is greater than critical depth ($TW > d_c$) and TW is less than D ($TW < D$). This condition is a common occurrence where the channel is deep, narrow, and well defined. The control is tailwater at the culvert outlet. The outlet velocity is the discharge divided by the area of flow in the culvert at tailwater depth.

$$\mathbf{HW = TW + V^2/(2g) + H_e + H_f - SL} \qquad \mathbf{(4.2)}$$

Where: HW = headwater depth (ft)
TW = tailwater at the outlet (ft)
V = velocity based on tailwater depth (ft)
g = 32.2 (ft/sec^2)
H_e = entrance headloss (ft)
H_f = friction headloss (ft)
S = slope of culvert (ft/ft)
L = length of culvert (ft)

Tailwater Depth > Barrel Depth - Outlet Control - This condition will exist if the critical depth is less than uniform depth at the design discharge ($d_c < d_u$) and TW depth is greater than D ($TW > D$), or; the critical depth is greater than the uniform depth at the design discharge ($d_c > d_u$) and TW is greater than (SL + D), [$TW > (SL + D)$]. The HW may or may not be greater than 1.5D, though often it is greater. If the critical depth of flow is determined to be greater than the barrel depth (only possible for rectangular culvert barrels), then this operation will govern. Outlet velocity is based on full flow at the outlet.

$$\mathbf{HW = H + TW - SL} \qquad \mathbf{(4.3)}$$

Where: HW = headwater depth (ft)
H = total head loss of discharge through culvert (ft)
TW = tailwater depth (ft)
SL = culvert slope times length of culvert (ft)

Tailwater Depth < Barrel Depth - Outlet Control - The entrance is submerged ($HW > 1.5D$) and the tailwater depth is less than D ($TW < D$). Normally, the engineer should arrive at this type of operation only after previous consideration of the operations depth covered when the critical depth, tailwater depth, or "slug" flow controls the flow in outlet control conditions. On occasion, it may be found that ($HW \geq 1.5D$) for the three previously outlined conditions but ($HW < 1.5D$) for equation 4.4. If so, the higher HW should be used. Outlet velocity is based on critical depth if TW depth is less than critical depth. If TW depth is greater than critical depth, outlet velocity is based on TW depth.

$$HW = H + P - SL \tag{4.4}$$

Where: HW = headwater depth (ft)
H = total head loss of discharge through culvert (ft)
P = empirical approximation of equivalent hydraulic grade line (ft)
P = $(d_c + D)/2$ if TW depth is less than critical depth at design discharge. If TW is greater than critical depth, then P = TW. (ft)
SL = culvert slope times length of culvert (ft)

4.6.3.2 Steep Slope

Tailwater Insignificant - Inlet Control - The entrance may be submerged or unsubmerged, the critical depth is greater than uniform depth at the design discharge ($d_c > d_u$), TW depth is less than SL (tailwater elevation is lower than the upstream flowline). Tailwater depth with respect to the diameter of the culvert is inconsequential as long as the above conditions are met. This condition is a common occurrence for culverts in rolling or hilly country. The control is critical depth at the entrance for HW values up to about 1.5D. Control is the entrance geometry for HW values over about 1.5D. HW is determined from empirical curves in the form of nomographs that are discussed later in this chapter. If TW is greater than D, outlet velocity is based on full flow at the outlet. If TW is less than D, outlet velocity is based on uniform depth for the culvert.

4.6.3.3 Slug Flow

Inlet or Outlet Control - For "slug" flow operation the entrance may be submerged or unsubmerged, critical depth is greater than uniform depth at the design discharge ($d_c > d_u$), TW depth is greater than ($SL + d_c$) (TW elevation is above the critical depth at the entrance), and TW depth is less than SL + D (TW elevation is below the upstream soffit). TW depth with respect to D alone is inconsequential as long as the above conditions are met. This condition is a common occurrence for culverts in rolling or hilly country. The control for this type of operation may be at the entrance or the outlet, or control may transfer itself back and forth between the two (commonly called "slug" flow). For this reason, it is recommended that HW be determined for both entrance control and outlet control and the higher of the two determinations be used. Entrance control HW is determined from the inlet control nomographs and outlet control HW is determined by equations 4.3, 4.4, or the outlet control nomographs.
If TW depth is less than D, outlet velocity should be based on TW depth. If TW depth is greater than D, outlet velocity should be based on full flow at the outlet.

4.7 Design Procedures

4.7.1 Procedures

There are two procedures for designing culverts described in this chapter: (1) the manual use of inlet and outlet control nomographs and (2) the use of a personal computer system HYDRAIN.

It is recommended that the HYDRAIN computer model be used for culvert design since it will allow the engineer to easily develop performance curves to examine more than one design situation. The personal computer system HYDRAIN uses the theoretical basis for the nomographs to size a culvert. In addition, this system can evaluate improved inlets, route hydrographs, consider road overtopping, and evaluate outlet streambed scour. By using water surface profiles, this procedure is more accurate in predicting backwater effects and outlet scour.

The following will outline the design procedures for use of the nomograph. The use of the computer model will follow the discussion on flood routing and culvert design. Other computer programs can be used if approved by the City.

4.7.2 Tailwater Elevations

In some cases, culverts fail to perform as intended because of tailwater elevations high enough to create backwater. The problem is more severe in areas where gradients are very flat, and in some cases in areas with moderate slopes. Thus, as part of the design process, the normal depth of flow in the downstream channel at discharges equal to those being considered should be computed.

If the tailwater computation leads to water surface elevations below the invert of the culvert exit, there are obviously no problems; if elevations above the culvert invert are computed, the culvert capacity will be somewhat less than assumed. The tailwater computation can be simple, and on steep slopes requires little more than the determination of a cross section downstream where normal flow can be assumed, and a Manning equation calculation. (See Chapter 5 for more information on open channel analysis.) Conversely, with sensitive flood hazard sites, if the slopes are flat, or natural and man-made obstructions exist downstream, a water surface profile analysis reaching beyond these obstructions may be required.

4.7.3 Culvert Design Nomographs

The use of culvert design nomographs requires a trial and error solution. The solution provides reliable designs for many applications. It should be remembered that velocity, hydrograph routing, roadway overtopping, and outlet scour require additional, separate computations beyond what can be obtained from the nomographs.

Figures 4-2 and 4-3 show examples of an inlet control and outlet control nomograph that can be used to design concrete pipe culverts. For culvert designs not covered by these nomographs, refer to the complete set of nomographs given in Appendix D at the end of this chapter.

4.7.4 Steps In The Design Procedure

The design procedure requires the use of inlet and outlet nomographs.

Step	Action
(1)	List design data:

Q = discharge (cfs)
L = culvert length (ft)
S = culvert slope (ft/ft)
HW = allowable headwater depth for the design storm (ft)
V = velocity for trial diameter (ft/s)
K_e = inlet loss coefficient
TW = tailwater depth (ft)

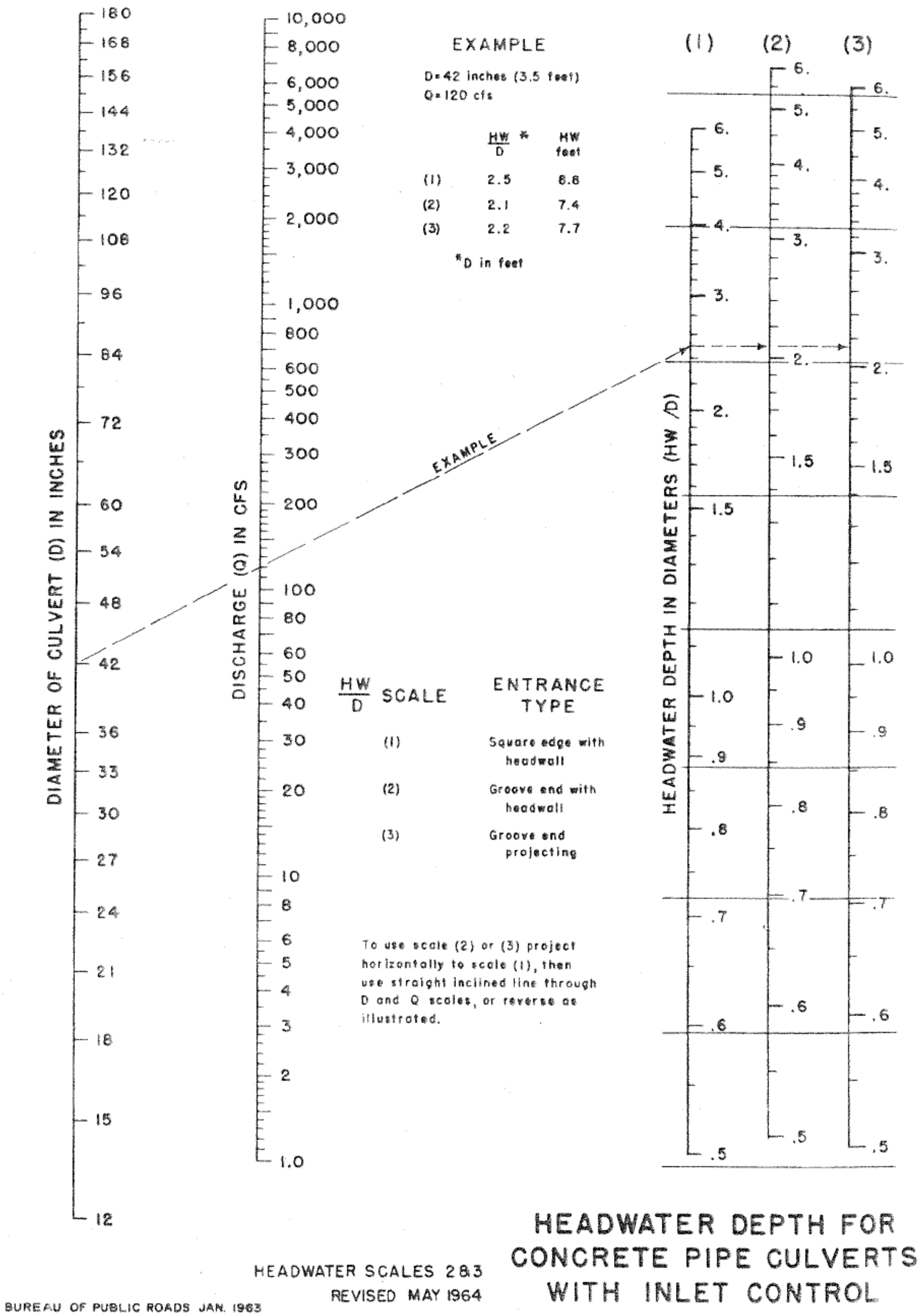
(2) Determine trial culvert size by assuming a trial velocity 3 to 5 ft/s and computing the culvert area, A = Q/V. Determine the culvert diameter (inches).

(3) Find the actual HW for the trial size culvert for both inlet and outlet control.

- For inlet control, enter inlet control nomograph with D and Q and find HW/D for the proper entrance type.
- Compute HW and, if too large or too small, try another culvert size before computing HW for outlet - control.
- For outlet control, enter the outlet control nomograph with the culvert length, entrance loss coefficient, and trial culvert diameter.
- To compute HW, connect the length scale for the type of entrance condition and culvert diameter scale with a straight line, pivot on the turning line, and draw a straight line from the design discharge through the turning point to the head loss scale H. Compute the headwater elevation HW from the equation:

$$\mathbf{HW = H + h_o - LS} \qquad \textbf{(4.5)}$$

Where: h_o = 1/2 (critical depth + D), or tailwater depth, whichever is greater.



DIAMETER OF CULVERT (D) IN INCHES
DISCHARGE (Q) IN CFS
HEADWATER DEPTH IN DIAMETERS (HW /D)
EXAMPLE
D=42 inches (3.5 feet)
Q=120 cfs
HW/D * | HW feet
(1) 2.5 8.8
(2) 2.1 7.4
(3) 2.2 7.7
*D in feet
EXAMPLE
HW/D SCALE
ENTRANCE TYPE
(1) Square edge with headwall
(2) Groove end with headwall
(3) Groove end projecting
To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated.
HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL
HEADWATER SCALES 2&3 REVISED MAY 1964
BUREAU OF PUBLIC ROADS JAN. 1963

Figure 4-2

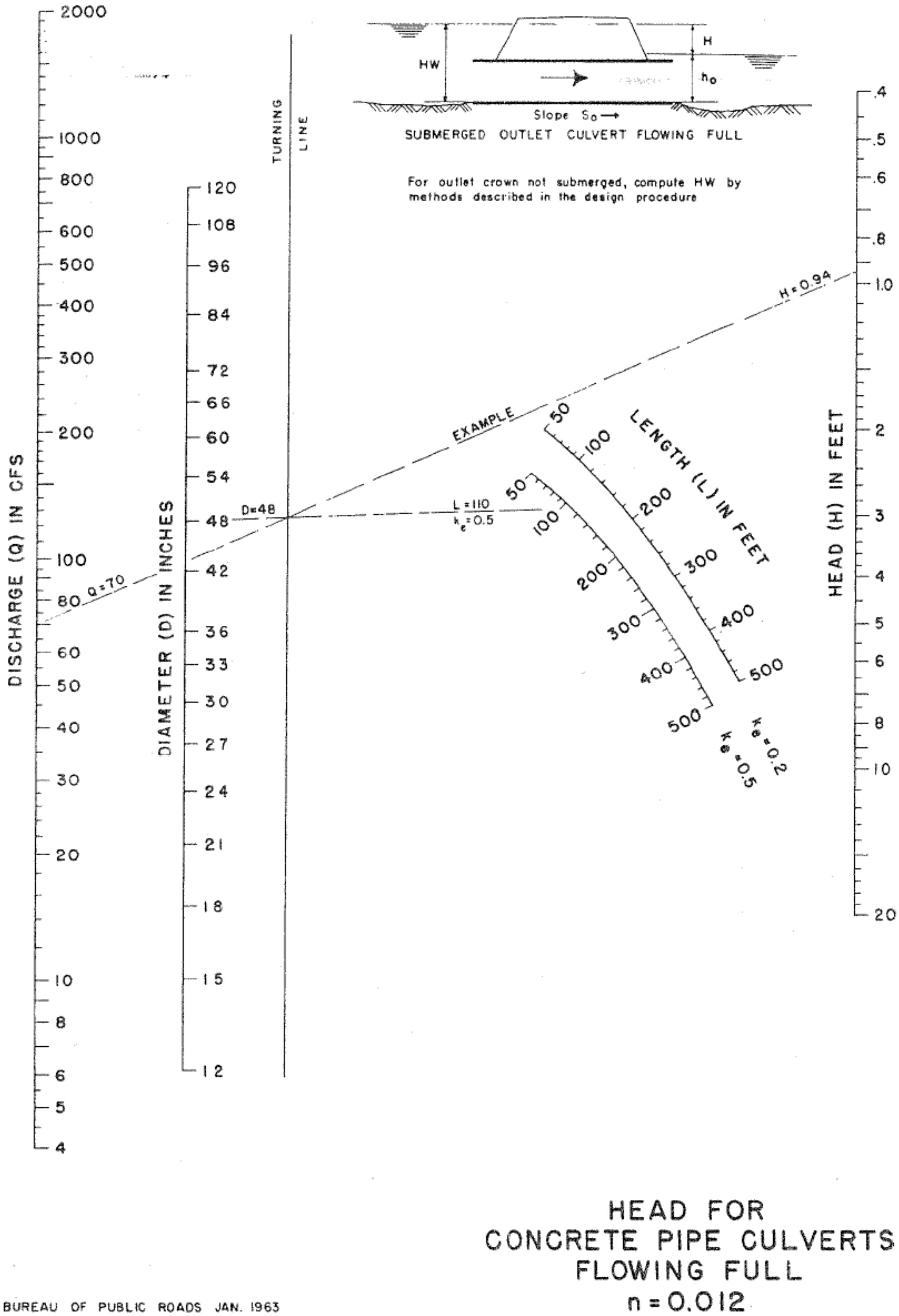

SUBMERGED OUTLET CULVERT FLOWING FULL
For outlet crown not submerged, compute HW by methods described in the design procedure
HW
H
h_o
Slope S_o
TURNING LINE
DISCHARGE (Q) IN CFS
DIAMETER (D) IN INCHES
LENGTH (L) IN FEET
HEAD (H) IN FEET
EXAMPLE
Q=70
D=48
L=110
k_e=0.5
H=0.94
k_e=0.2
k_e=0.5
HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012
BUREAU OF PUBLIC ROADS JAN. 1963

Figure 4-3

(4) 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.

(5) Calculate exit velocity and expected streambed scour to determine if an energy dissipator is needed.

4.7.5 Performance Curves

A performance curve for any culvert can be obtained from the nomographs by repeating the steps outlined above for a range of discharges that are of interest for that particular culvert design. A graph is then plotted of headwater vs. discharge with sufficient points so that a curve can be drawn through the range of interest. These curves are applicable through a range of headwater, velocities, and scour depths versus discharges for a length and type of culvert. Usually charts with length intervals of 25 to 50 feet are satisfactory for design purposes. Such computations are made much easier by the computer program discussed in section 4.11 of this manual.

4.7.6 Roadway Overtopping

To complete the culvert design, roadway overtopping should be analyzed. A performance curve showing the culvert flow as well as the flow across the roadway is a useful analysis tool. Rather than using a trial and error procedure to determine the flow division between the overtopping flow and the culvert flow, an overall performance curve can be developed. The overall performance curve can be determined as follows:

Step Action

(1) Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. The flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters should be calculated.

(2) Combine the inlet and outlet control performance curves to define a single performance curve for the culvert.

(3) When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and equation 4.6 to calculate flow rates across the roadway.

$$\mathbf{Q = C_d L (HW)^{1.5}} \qquad \mathbf{(4.6)}$$

Where: Q = overtopping flow rate (ft^3/s)
C_d = overtopping discharge coefficient
L = length of roadway (ft)
HW = upstream depth, measured from the roadway crest to the water surface upstream of the weir drawdown (ft)

Note: See Figure 4-4 for guidance in determining a value for C_d. For more information on calculating overtopping flow rates see pages 39 - 42 in HDS No. 5.

(4) Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve.

4.7.7 Storage Routing

A significant storage capacity behind a roadway embankment attenuates a flood hydrograph. Because of the reduction of the peak discharge associated with this attenuation, the required capacity of the culvert, and its size, may be reduced. If significant storage is anticipated behind a culvert, the design may be checked by routing the design hydrographs through the culvert to determine the discharge and stage behind the culvert. Routing procedures are outlined in Hydraulic Design of Highway Culverts, Section V - Storage Routing, HDS No. 5, Federal Highway Administration. If storage routing is performed for a culvert, the facility should be designed as a detention pond and the area inundated by floodwater should not be encroached upon.

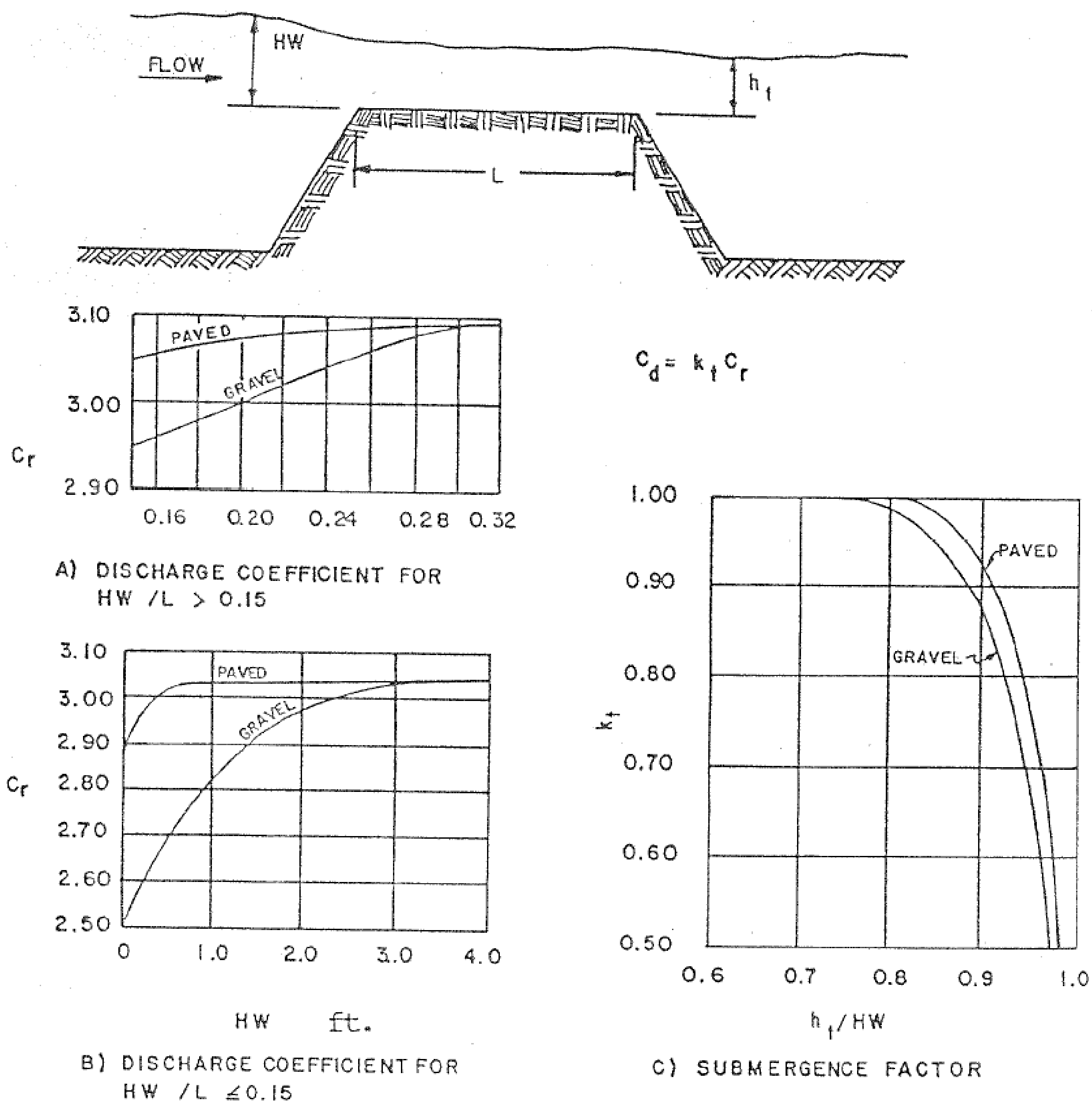


Figure 4-4 Discharge Coefficients For Roadway Overtopping

4.8 Culvert Design Example Using Nomographs

The following example problem illustrates the procedures to be used in designing culverts using the nomographs.

Size a culvert given the following design conditions which were determined by physical limitations at the culvert site and hydraulic procedures described elsewhere in this handbook.

<u>Input Data</u>

Discharge for 50-yr flood = 70 cfs
Discharge for 100-yr flood = 176 cfs
Allowable HW for 10-yr discharge = 4.5 ft
Allowable HW for 100-yr discharge = 7.0 ft
Length of culvert = 100 ft
Natural channel invert elevations: inlet = 15.50 ft
outlet = 15.35 ft
Culvert slope = 0.0015 ft/ft
Tailwater depth for 50-yr discharge = 3.0 ft
Tailwater depth for 100-yr discharge = 4.0 ft
Tailwater depth is the normal depth in downstream channel
Entrance type = Groove end with headwall
Culvert type = Reinforced concrete

<u>Computations</u>

1. Assume a culvert velocity of 5 ft/s.

   Required flow area = 70 cfs/5 ft/s = 14 sq ft
   (for the 50-yr recurrence flood).

2. The corresponding culvert diameter is about 48 in.

   This can be calculated by using the formula for area of a circle:

   Area = $(3.14D^2)/4$ or D = $(\text{Area x } 4/3.14)^{0.5}$

   Therefore: D = $((14 \text{ sq ft x } 4)/3.14)^{0.5}$ x 12 in./ft

   D = 50.7 in.

3. A grooved end culvert with a headwall is selected for the design. Using the inlet control nomograph (Figure 4-2), with a pipe diameter of 48 in. and a discharge of 70 cfs; read a HW/D value of 0.93.

4. The depth of headwater (HW) is (0.93) x (4) = 3.72 ft which is less than the allowable headwater of 4.5 ft.

5. The culvert is checked for outlet control by using Figure 4-3.

   With an entrance loss coefficient K_e of 0.20 (see Table 4-2), a culvert length of 100 ft, and a pipe diameter of 48 in., an H value of 0.77 ft is determined. The headwater for outlet control is computed by the equation:

   $HW = H + h_o - LS$

   For the tailwater depth lower than the top of culvert,

   h_o = TW or 1/2 (critical depth in culvert + D) whichever is greater.
   h_o = 3.0 ft or h_o = 1/2 (2.55 + 4.0) = 3.28 ft

The headwater depth for outlet control is:

$HW = H + h_o - LS = 0.77 + 3.28 - (100) \times (0.0015) = 3.90$ ft

6. Since HW for outlet control (3.90 ft) is greater than the HW for inlet control (3.72 ft), outlet control governs the culvert design.

   Thus, the maximum headwater expected for a 50-yr recurrence flood is 3.90 ft, which is less than the allowable headwater of 4.5 ft.

7. The performance of the culvert is checked for the 100-yr discharge.

   The allowable headwater for a 100-yr discharge is 7 ft; critical depth in the 48 in. diameter culvert for the 100-yr discharge is 3.96 ft.

   For outlet control, an H of 4.6 is read from the outlet control nomograph. The maximum headwater is:

   $HW = H + h_o - LS = 4.6 + 4.0 - (100) \times (0.0015) = 8.45$ ft

   This depth is greater than the allowable depth of 7 ft, thus a larger size culvert must be selected.

8. A 54 in. diameter culvert is tried and found to have a maximum headwater depth of 3.74 ft for the 10-yr discharge and a maximum headwater depth of 6.97 ft for the 100-yr discharge. These values are acceptable for the design conditions.

9. Estimate outlet exit velocity. Since this culvert is on outlet control and discharges into an open channel downstream, the culvert will be flowing full at the flow depth in the channel. Using the 100-year design peak discharge of 176 cfs and the area of a 54 in. or 4.5 ft diameter culvert, the exit velocity will be:

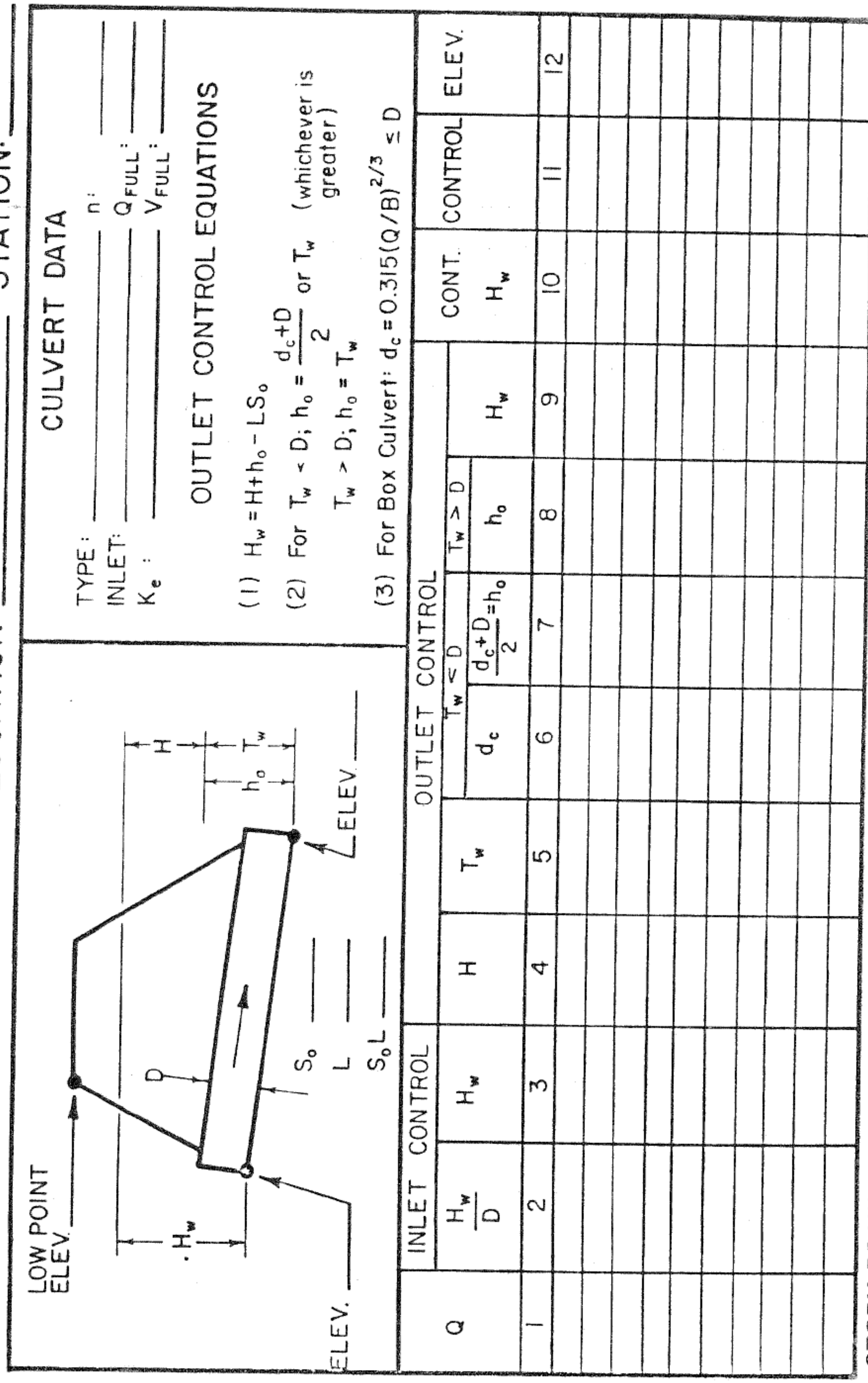
   $V = Q/A = 176 / (3.14 \quad (4.5)^2)/4 = 11.1$ ft/s

   With this high velocity, some energy dissipator is needed downstream from this culvert for stream-bank protection. It will first be necessary to compute a scour hole depth and then decide what protection is needed. See Chapter 7, Energy Dissipators for design procedures related to energy dissipators.

10. Design engineers should check minimum velocities for low frequency flows if the larger storm event (100-year) controls culvert design.

    Figure 4-5 provides a convenient form to organize culvert design calculations.

    For an example of a design which incorporates roadway overtopping, see Appendix 4A - example application of the HY8 Culvert Analysis Microcomputer Program.



STANDARD FORM SF-4
CULVERT RATING

PROJECT: ______ LOCATION: ______ STATION: ______

CULVERT DATA

TYPE: ______

INLET: ______

K_e: ______

n: ______

Q_{FULL}: ______

V_{FULL}: ______

OUTLET CONTROL EQUATIONS

(1) $H_w = H + h_o - LS_o$

(2) For $T_w < D$; $h_o = \frac{d_c + D}{2}$ or T_w (whichever is greater)

$T_w > D$; $h_o = T_w$

(3) For Box Culvert: $d_c = 0.315(Q/B)^{2/3} \leq D$

Q	INLET CONTROL		OUTLET CONTROL						CONT.	CONTROL	ELEV.
					$T_w < D$		$T_w > D$				
	$\frac{H_w}{D}$	H_w	H	T_w	d_c	$\frac{d_c+D}{2}=h_o$	h_o	H_w	H_w		
1	2	3	4	5	6	7	8	9	10	11	12

STORM DRAINAGE DESIGN AND TECHNICAL CRITERIA

WRC ENGINEERING

Figure 4-5 Culvert Calculation Form

4.9 Design Of Improved Inlets

4.9.1 Introduction

A culvert operates in either inlet or outlet control. As previously discussed under outlet control, headwater depth, tailwater depth, entrance configuration, and barrel characteristics all influence a culvert's capacity. The entrance configuration is defined by the barrel cross sectional area, shape, and edge condition, while the barrel characteristics are area, shape, slope, length, and roughness.

4.9.2 Outlet Control

The flow condition for outlet control may be full or partly full for all or part of the culvert length. The design discharge usually results in full flow. Inlet improvements in these culverts reduce the entrance losses, which are only a small portion of the total headwater requirements. Therefore, only minor modifications of the inlet geometry which result in little additional cost are justified.

4.9.3 Inlet Control

In inlet control, only entrance configuration and headwater depth determine the culvert's hydraulic capacity. Barrel characteristics and tailwater depth are of no consequence. These culverts usually lie on relatively steep slopes and flow only partly full. Entrance improvements can result in full, or nearly full flow, thereby increasing culvert capacity significantly.

4.9.4 Common Entrances

The figure below illustrates the performance of a 30-in. circular culvert in inlet control with three commonly used entrances: thin-edged projecting, square-edged, and groove-edged.

4.9.5 Capacity Determinations

It is clear that inlet type and headwater depth determine the capacities of many culverts. For a given headwater, a groove-edged inlet has a greater capacity than a square-edged inlet, which in turn-out performs a thin-edged projecting inlet.

The performance of each inlet type is related to the degree of flow contraction. A high degree of contraction requires more energy, or headwater, to convey a given discharge than a low degree of contraction.

4.9.6 Improved Inlets

Improved inlets include inlet geometry refinements beyond those normally used in conventional culvert design practice. Several degrees of improvements are possible, including bevel-edged, side-tapered, and slope-tapered inlets.

4.9.6.1 Bevel-Edged Inlet

The first degree of inlet improvement is a beveled edge. The bevel is proportioned based on the culvert barrel or face dimension and operates by decreasing the flow contraction at the inlet. A bevel is similar to a chamfer except that a chamfer is smaller and is generally used to prevent damage to sharp concrete edges during construction.

Adding bevels to a conventional culvert design with a square-edged inlet increases culvert capacity by 5 to 20 percent. The higher increase results from comparing a bevel-edged inlet with a square-edged inlet at high headwaters. The lower increase is the result of comparing inlets with bevels, with structures having wingwalls of 30 to 45 degrees. Although the bevels referred to in this publication are plane surfaces, rounded edges which approximate the bevels are also acceptable.

As a minimum, bevels should be used on all culverts which operate in inlet control, both conventional and improved inlet types. The exception to this is circular concrete culverts where the socket end performs much the same as a beveled edge.

Culverts flowing in outlet control cannot be improved as much as those in inlet control, but the entrance loss coefficient, ke, is reduced from 0.5 for a square edge to 0.2 for beveled edges.

It is recommended that bevels be used on all culvert entrances if little additional cost is involved.

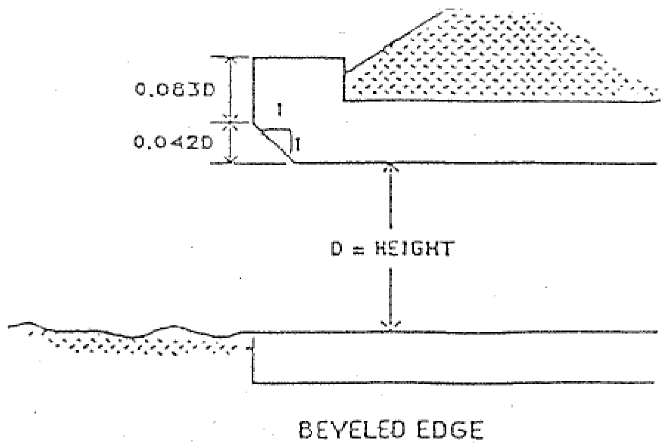


Figure 4-6 Beveled Inlet Detail

4.9.6.2 Side-Tapered Inlet

The second degree of improvement is a side-tapered inlet. This inlet has an enlarged face area with the transition to the culvert barrel accomplished by tapering the sidewalls. The inlet face has the same height as the barrel, and its top and bottom are extensions of the top and bottom of the barrel. The intersection of the sidewall tapers and barrel is defined as the throat section. If a headwall and wingwalls are going to be used at the culvert entrance, side-tapered inlets should add little if any to the overall cost while significantly increasing hydraulic efficiency.

The side-tapered inlet provides an increase in flow capacity of 25 to 40 percent over that of a conventional culvert with a square-edged inlet.

Whenever increased inlet efficiency is needed or when a headwall and wing walls are planned to be used for a culvert installation, a side-tapered inlet should be considered.

4.9.6.3 Slope-Tapered Inlet

A slope-tapered inlet is the third degree of improvement. Its advantage over the side-tapered inlet without a depression is that more head is available at the inlet. This is accomplished by incorporating a fall in the enclosed entrance section.

The slope-tapered inlet can have over a 100 percent greater capacity than a conventional culvert with square edges. The degree of increased capacity depends largely upon the amount of fall available. Since this fall may vary, a range of increased capacities is possible.

Side- and slope-tapered inlets should be used in culvert design when they can economically be used to increase the inlet efficiency over a conventional design.

For a complete discussion of tapered inlets, including figures and illustrations, see pages 65-93, Federal Highway Administration, HDS-5, 1985.

4.9.6.4 Improved Inlet Performance

The two tables below compare the inlet control performance of the different inlet types. The first half of Table 4-4 shows the increase in discharge that is possible for a headwater depth of 8 feet. The bevel-edged inlet, side-tapered inlet and slope-tapered inlet show increases in discharge over the square-edged inlet of 16.7, 30.4 and 55.6 percent, respectively. It should be noted that the slope-tapered inlet incorporates only a minimum fall. Greater increases in capacity are often possible if a larger fall is used.

The second half of Table 4-4 depicts the reduction in headwater that is possible for a discharge of 500 cfs. The headwater varies from 12.5 ft for the square-edged inlet to 7.6 ft for the slope-tapered inlet. This is a 39.2 percent reduction in required headwater.

Table 4-4 Comparison of Inlet Performance

Comparison of Inlet Performance at Constant Headwater for 6 ft x 6 ft Concrete Box Culvert**

Inlet Type	Headwater	Discharge	% Improvement
Square-edge	8.0 feet	336 cfs	0
Bevel-edge	8.0 feet	392 cfs	16.7
Side-tapered	8.0 feet	438 cfs	30.4
Slope-tapered*	8.0 feet	523 cfs	55.6

* Minimum fall in inlet = D/4 = 6/4 = 1.5 ft

Comparison of Inlet Performance at Constant Discharge for 6 ft x 6 ft Concrete Box Culvert**

Inlet Type	Discharge	Headwater	% Improvement
Square-edge	500 cfs	12.5 feet	0
Bevel-edge	500 cfs	10.1 feet	19.2
Side-tapered	500 cfs	8.8 feet	29.6
Slope-tapered*	500 cfs	7.6 feet	39.2

* Minimum fall in inlet = D/4 = 6/4 = 1.5 ft
** Substantially less improvement in capacity can be accomplished if the culvert functions under outlet control.

4.10 Design Procedures For Beveled-Edged Inlets

4.10.1 Introduction

This section will outline the procedures and charts to use when incorporating bevel-edged inlets in the design of culverts. Those designers interested in using side- and slope-tapered inlets should consult the detailed design criteria and example designs outlined in the U. S. Department of Transportation publication Hydraulic Engineering Circular No. 5 entitled, "Hydraulic Design of Highway Culverts."

4.10.2 Design Figures

Four inlet control figures for culverts with beveled edges are included in Appendix C at the end of this chapter.

Figure	Use for
1	circular pipe culverts with beveled rings
2	90° headwalls (same for 90° wingwalls)
3	skewed headwalls
4	wingwalls with flare angles of 18 to 45 degrees

4.10.3 Design Procedure

The figures for bevel-edged inlets are used for design in the same manner as the conventional inlet design nomographs discussed earlier.

Note: Figures 2, 3, and 4 apply only to bevels having either a 33^o angle (1.5:1) or a 45^o angle (1:1).

For box culverts, the dimensions of the bevels to be used are based on the culvert's dimensions. The top-bevel dimension is determined by multiplying the height of the culvert by a factor. The side-bevel dimensions are determined by multiplying the width of the culvert by a factor. For a 1:1 bevel, the factor is 1/2 in./ft. For a 1.5:1 bevel the factor is 1 in./ft.

For example, the minimum bevel dimensions for a 8 ft x 6 ft box culvert with 1:1 bevels would be:

Top Bevel = d = 6 ft x 1/2 in./ft = 3 inches

Side Bevel = b = 8 ft x 1/2 in./ft = 4 inches

For a 1.5:1 bevel, computations would result in d = 6 and b = 8 inches.

4.10.4 Design Figure Limits

The improved inlet design figures are based on research results from culvert models with barrel width, B, to depth, D, ratios of from 0.5:1 to 2:1.

For box culverts with more than one barrel, the figures are used in the same manner as for a single barrel, except that the bevels must be sized on the basis of the total clear opening rather than on individual barrel size.

For example, in a double 8 ft by 8 ft box culvert:

Top Bevel - is proportioned based on the height of 8 ft which results in a bevel of 4 in. for the 1:1 bevel and 8 in. for the 1.5:1 bevel.

Side Bevel - is proportioned based on the clear width of 16 ft which results in a bevel of 8 in. for the 1:1 bevel and 16 in. for the 1.5:1 bevel.

4.10.5 Area Ratios

The ratio of the inlet face area to the barrel area remains the same as for a single barrel culvert. Multibarrel pipe culverts should be designed as a series of single barrel installations since each pipe requires a separate bevel.

4.10.6 Multibarrel Installations

For multibarrel installations exceeding a 3:1 width to depth ratio, the side bevel becomes excessively large when proportioned on the basis of the total clear width. For these structures, it is recommended that the side bevel be sized in proportion to the total clear width, B, or three times the height, whichever is smaller.

The top bevel dimension should always be based on the culvert height.

The shape of the upstream edge of the intermediate walls of multibarrel installations is not as important to the hydraulic performance of a culvert as the edge condition of the top and sides. Therefore, the edges of these walls may be square, rounded with a radius of one-half their thickness, chamfered, or beveled. The intermediate walls may also project from the face and slope downward to the channel bottom to help direct debris through the culvert.

4.10.7 Skewed Inlets

Skewed inlets should be avoided whenever possible, and should not be used with side- or slope-tapered inlets.

4.11 HYDRAIN Culvert Computer Program

4.11.1 Introduction

The HYDRAIN culvert analysis microcomputer program is available at nominal cost from McTrans Software or Dodson & Associates. It will perform the calculation for the following:

1. culvert analysis (including independent multiple barrel sizing)
2. hydrograph generation
3. hydrograph routing
4. roadway overtopping
5. outlet scour estimates

The example problems in Appendix A provide the user with analysis approaches to be used with the culvert analysis portion of the program. The examples provide instruction in data entry, file modification and culvert performance analysis. The three examples presented use the same site characteristics and discharge range, which are described in Example 1. The user should work through the problems on the computer while following the text so as to become familiar with the program. New users should consult the README file that accompanies the program for further help and directions.

4.11.2 User-Friendly Features

HYDRAIN's culvert analysis has several user-friendly features which permit easy data entry, editing and comparison of several design alternatives. Data are entered by selecting options on a menu or by entering numeric data at prompts. These data are periodically summarized in tables. Any incorrect entry can be changed, and design variations can be quickly analyzed. Another feature of HYDRAIN's culvert analysis is that plots of irregular cross sections, channel rating curves and culvert performance curves can be obtained if the terminal has graphics capabilities.

4.11.3 Examples

The following three examples are given in Appendix A:

- Example 1 - Reinforced Concrete Box Culvert Design,
- Example 2 - Irregular Culvert Cross Section,
- Example 3 - Multiple Independent Barrels.

The culvert alternatives for these examples were chosen to illustrate the features of the software and do not necessarily represent cost effective designs. Since the program is still being developed, some of the screens shown in the examples may differ slightly from the version you obtain.

4.12 Construction And Maintenance Considerations

An important step in the design process involves identifying whether special provisions are warranted to properly construct or maintain proposed facilities. Culverts located on and aligned with the natural channel generally do not have a sedimentation problem. A stable channel is expected to balance erosion and sedimentation. A culvert resting on such a channel behaves in a similar manner. In a degrading channel, erosion, not sedimentation, is a potential problem. A culvert located in an agrading channel may encounter some sedimentation. Multi-barrel culverts and culverts with depressed inlets may encounter sedimentation problems. It is common for one or more barrels to accumulate sediment. Culverts built with an upstream depression have a barrel slope less than the stream slope and sediment accumulation is likely. Both usually are self-cleaning during periods of high discharge. Maintenance concerns of storm sewer system design center on adequate physical access for cleaning and repair.

Culverts must be kept free of obstructions. Sand or sediment deposits should be removed as soon as possible During major storms, critical areas should be patrolled and the inlets kept free of debris. Inlet and outlet channels should be kept in alignment and vegetation should be controlled in order to prevent any significant restriction of flow.

Provision for a smooth, well designed inlet and avoidance at multiple barrels and skewed inlets will help align and pass most floating debris. Preventative maintenance should be used to inspect for structural problems, replacement needs, and scheduling of needed repairs.

References

American Association of State Highway and Transportation Officials. Highway Drainage Guidelines. 1982.

Federal Highway Administration. Hydraulics of Bridge Waterways. Hydraulic Design Series No. 1. 1978.

Federal Highway Administration. Hydraulic Design of Highway Culverts. Hydraulic Design Series No. 5. 1985.

Federal Highway Administration. Debris-Control Structures. Hydraulic Engineering Circular No. 9. 1971.

Federal Highway Administration. HY8 Culvert Analysis Microcomputer Program Applications Guide. Hydraulic Microcomputer Program HY8. 1987.

HYDRAIN Culvert Computer Program (HY8). Available from McTrans Software, University of Florida, 512 Weil Hall, Gainesville, Florida 32611.

U. S. Department of Interior. 1983. Design of Small Canal Structures.

APPENDIX 4-A

HY-8 CULVERT ANALYSIS - COMPUTER PROGRAM

Data Input For Culvert

As an initial size estimate, try a 24 inch circular culvert. For the culvert assume that a conventional inlet with head-wall and square edges will be used. As each group of data are entered the user is allowed to edit any incorrect entries. The following will show the computer screens that the user will see.

After inputting the word CULVERT to start the program the following will appear on the computer screen.

```
        CULVERT FILE MENU

TYPE LETTER OF DESIRED OPTION
<E>   EDIT OR USE A FILE
<C>   CREATE A FILE

<ESC> FOR MAIN PROGRAM MENU
```

Input a name for the file that will store all input data and press return to have the computer input the current date.

```
TYPE NEW CULVERT FILE NAME
FILE NAME ---> TEST

TYPE DATE OR <ENTER> FOR CURRENT DATE
DATE --->

<ESC> TO RETURN TO CULVERT FILE MENU
```

Input the design discharge (10-year peak discharge) and the maximum discharge (100-year peak discharge).

```
ENTER DESIGN AND MAXIMUM FLOW

<1> MINIMUN DISCHARGE (CFS)          0.0

<2> DESIGN  DISCHARGE (CFS)         60.0

<3> MAXIMUM DISCHARGE (CFS)        110.0


<NUMBER> TO EDIT DISCHARGE
<ENTER>  TO CONTINUE
```

Select <2> Culvert Invert Data

Input the culvert invert data which will be used to determine the length, slope, and elevations associated with this culvert installation.

```
          CULVERT INVERT DATA

NO.     ITEM                              VALUE

<1> INLET STATION (FT)                   100.00
<2> INLET ELEVATION (FT)                 950.00
<3> OUTLET STATION (FT)                  175.00
<4> OUTLET ELEVATION (FT)                947.00
<5> ENTER NUMBER OF BARRELS                 1
```

Select a culvert shape (for this example a circular culvert was selected)

```
SELECT A CULVERT SHAPE:

   <1> CIRCULAR
   <2> BOX
   <3> ELLIPTICAL
   <4> PIPE ARCH
   <5> USER DEFINED (COORDINATES)
   <6> ARCH
   <7> LOW-PROFILE ARCH
   <8> HIGH-PROFILE ARCH
   <9> METAL BOX
```

Specify the culvert diameter to be used for the first analysis.

```
        CIRCULAR CULVERT

CULVERT DIAMETER (FT) ---> 2*


<ESC> TO RETURN TO SHAPE MENU
```

Select a culvert material (for this example a concrete culvert was selected).

```
SELECT A CULVERT MATERIAL:

<1> CONCRETE

<2> CORRUGATED STEEL PLATE

<3> CORRUGATED ALUMINUM PLATE
```

Select an inlet type (for this example a conventional inlet was selected).

```
SELECT AN INLET TYPE:

<1> CONVENTIONAL

<2> SIDE-TAPERED, CIRCULAR

<3> SIDE-TAPERED, RECTANGULAR

<4> SLOPE TAPERED
```

Select inlet conditions (for this example square edge with headwall was selected). Then specify if there is inlet depression (this example did not include any inlet depression).

```
SELECT AN INLET CONDITION:

   CONVENTIONAL INLETS

<3>   SQUARE EDGE WITH HEADWALL
<4>   GROOVED END PROJECTION
<5>   GROOVED END IN HEADWALL
<6>   BEVELED EDGE (1:1)
<7>   BEVELED EDGE (1.5:1)
```

Following is a summary table of the culvert input data.

```
    ITEM                        SELECTED CULVERT
<1> BARREL SHAPE:               CIRCULAR
<2> BARREL SIZE:                 2.00 FT DIAMETER
<3> BARREL MATERIAL:            CONCRETE
<4> MANNING'S n:                .012
<5> INLET TYPE:                 CONVENTIONAL
<6> INLET EDGE AND WALL:        SQUARE EDGE WITH HEADWALL
<7> INLET DEPRESSION:           NONE

<NUMBER> TO EDIT ITEM
<ENTER>  TO CONTINUE DATA LISTING
```

At this point the user can edit any of the input data or press <ENTER> to continue.

Data Input Downstream Channel

Following are the data related to the channel downstream from the culvert.

For the calculation of tailwater conditions, select the shape of the channel directly downstream from the outlet of the culvert (for this example a rectangular channel was selected).

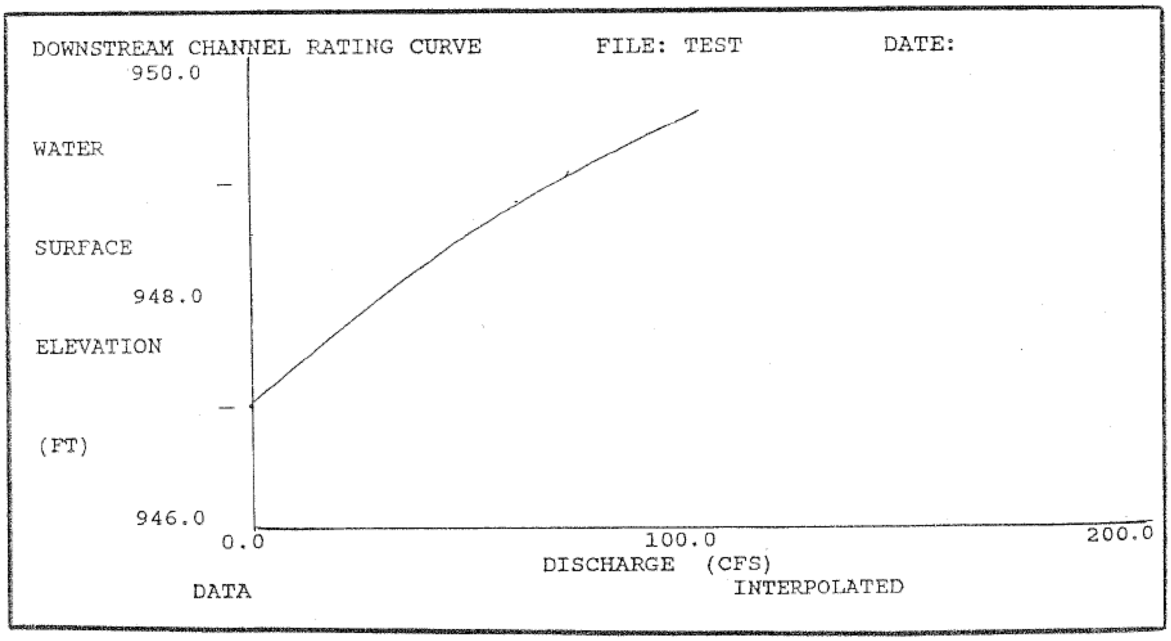
```
TAILWATER RATING CURVE

PRESS NUMBER OF OPTION

<1> RECTANGULAR CHANNEL

<2> TRAPEZOIDAL CHANNEL

<3> TRIANGULAR CHANNEL

<4> IRREGULAR CHANNEL (MAX. 15 COORDINATES)

<5> ENTER RATING CURVE (11 POINTS)

<6> ENTER CONSTANT TAILWATER ELEVATION
```

For the downstream channel, input the bottom width, channel slope, Manning's n, and channel invert elevation.

```
   ENTER TAILWATER CHANNEL DATA

NO.       ITEM                                 VALUE

<1> BOTTOM WIDTH (FT)                            5

<2> SIDE SLOPE  H:V  __:1

<3> CHANNEL SLOPE (FT/FT)                       .04

<4> MANNING'S N (.01-0.1)                       .045

<5> CHANNEL INVERT ELEVATION (FT)               947

    CULVERT INVERT ELEVATION (FT)              947.00
    (CULVERT NO. 1 OUTLET)

<NUMBER>  TO EDIT ITEM
 <ENTER>  TO CONTINUE DATA INPUT
   <ESC>  FOR CHANNEL SHAPE MENU
```

The table showing the Tailwater Rating Curve Data will now be calculated by the model. By pressing P you can obtain a plot of the Tailwater Data (Downstream Channel Rating Curve).

```
                TAILWATER RATING CURVE

NO.    FLOW(CFS)  T.W.E.(FT)  VEL.(FPS)  SHEAR(PSF)
 1        0.00      947.00      0.00        0.00
 2       11.00      947.56      3.92        1.40
 3       22.00      947.88      4.97        2.21
 4       33.00      948.16      5.67        2.91
 5       44.00      948.42      6.19        3.55
 6       55.00      948.67      6.60        4.16
 7       60.00      948.77      6.77        4.43
 8       77.00      949.13      7.24        5.31
 9       88.00      949.35      7.50        5.86
10       99.00      949.56      7.73        6.40
11      110.00      949.77      7.93        6.93

       PRESS:
           <D> FOR DATA
           <P> TO PLOT RATING CURVE
         <ESC> FOR CHANNEL SHAPE MENU
       <ENTER> TO CONTINUE
```



Data Input Roadway

For overtopping analysis additional data will be needed about the roadway over the culvert installation.

Select a profile shape for the roadway above the culvert installation (for this example a constant roadway elevation was selected).

```
ROADWAY PROFILE SHAPE FOR OVERTOPPING ANALYSIS

        SELECT PROFILE SHAPE:

        <1> CONSTANT ROADWAY ELEVATION

        <2> IRREGULAR (3 TO 15 COORDINATES)
```

Input the crest length and overtopping crest elevation.

```
        ENTER PROFILE DATA

NO.     ITEM                     VALUE

<1> CREST LENGTH (FT)             50

<2> OVERTOPPING CREST
             ELEVATION (FT)       955*
```

Select a weir coefficient for the roadway section (for this example a paved roadway surface was selected).

```
              WEIR COEFFICIENTS

SELECT ROADWAY SURFACE OR A WEIR COEFFICIENT:

<1> PAVED ROADWAY SURFACE

<2> GRAVEL ROADWAY SURFACE

<3> INPUT COEFFICIENT OF DISCHARGE (2.5 - 3.095)


<ESC> FOR LAST MENU
```

Select an overtopping crest elevation.

```
     SELECTED OVERTOPPING CREST

  <1> SHAPE:  CONSTANT ROADWAY ELEVATION 955 FT

  <2> CROSS-SECTION DATA

  <3> ROADWAY SURFACE:  P

  <4> EMBANKMENT TOP WIDTH (FT): 60*
```

Following is a summary table of the data input.

```
CULVERT ANALYSIS 3.1                                          DATE:
CULVERT FILE: TEST                SUMMARY TABLE               CULVERT NO. 1
---------------------------------------------------------------------------------
 C |    <S>  SITE DATA            |    <C>  CULVERT SHAPE, MATERIAL, INLET
 U |------------------------------|----------------------------------------------
 L |  INLET   OUTLET   CULVERT    | BARRELS
 V |  ELEV.    ELEV.   LENGTH     | SHAPE        SPAN   RISE   MANNING   INLET
NO.|  (FT)     (FT)     (FT)      | MATERIAL     (FT)   (FT)      n      TYPE
 1 | 950.00   947.00     75       | 1 - RCP      2.00   2.00    .012  CONVENTIONAL
 2 |                              |
 3 |                              |
 4 |                              |
 5 |                              |
 6 |                              |
---------------------------------------------------------------------------------
     PRESS   TO REVIEW                        PRESS   TO
      <C>    CULVERT DATA                      <E>    EDIT CULVERT SIZE
      <D>    DISCHARGE DATA                    <M>    MINIMIZE CULVERT SPAN
      <O>    OVERTOPPING DATA                  <A>    ADD OR DELETE CULVERTS
      <S>    SITE DATA
      <T>    TAILWATER RATING CURVE

                          <ENTER>  TO CONTINUE
```

The user can now edit any data input either to correct errors or to try different analysis for different conditions or culvert designs.

Analysis

Overtopping Analysis - If the user wants to take into account roadway overtopping, then this alternative should be selected. Otherwise select the no overtopping analysis option. The model will then perform the flow analysis and calculate how much flow will pass through the culvert and how much flow will overtop the roadway. The user can then plot the culvert rating curve and/or print a summary of the culvert data and analysis.

```
                              CULVERT PROGRAM OPTIONS

                           PRESS LETTER OF DESIRED OPTION

                           <O> OVERTOPPING ANALYSIS
                               Inlet Control   - HDS5 Nomographs
                               Outlet Control  - Full Barrel Flow
                               Overtopping     - Wier Equation

                           <N> NO OVERTOPPING ANALYSIS (CULVERT NUMBER 1)
                               Inlet Control   - HDS5 Nomographs
                               Outlet Control  - Full Barrel Flow


                           <B> OUTLET CONTROL METHOD
                           <S> SAVE MENU
                           <D> DATA SUMMARY
                           <F> FILE MENU      SAVE FILE BEFORE <F> OR <M>
                           <M> MAIN MENU      OR FILE WILL BE LOST


1HELP   2       3       4       5END    6       7       8       9SHELL  10

--------------------------------------------------------------------------------
SUMMARY OF CULVERT FLOWS (CFS)          FILE: TEST                  DATE:
--------------------------------------------------------------------------------
ELEV (FT)    TOTAL       1       2       3       4       5       6  OVERTOP ITER
  950.00         0       0       0       0       0       0       0        0   0
  951.77        11      11       0       0       0       0       0        0   1
  953.23        22      22       0       0       0       0       0        0   1
  955.07        33      31       0       0       0       0       0        3   2
  955.20        44      31       0       0       0       0       0       13   2
  955.30        55      32       0       0       0       0       0       24   2
  955.33        60      32       0       0       0       0       0       29   2
  955.45        77      32       0       0       0       0       0       46   2
  955.52        88      32       0       0       0       0       0       56   2
  955.58        99      33       0       0       0       0       0       67   2
  955.64       110      33       0       0       0       0       0       78   2
--------------------------------------------------------------------------------
  955.00        30      30       0       0       0       0       0 (OVERTOPPING)
--------------------------------------------------------------------------------
       PRESS: <1> TO PLOT TOTAL RATING CURVE
              <2> TO DETERMINE SPECIFIC INFORMATION ABOUT EACH CULVERT
              <3> TO SEE MULTIPLE CULVERT COMPUTATIONAL ERROR TABLE
              <4> TO PRINT CULVERT SUMMARY
              <ENTER> TO RETURN FOR NEW RUN OR EXIT
```

Changing Data

After reviewing the data for the first analysis, it is obvious that too much flow is overtopping the road with very little flow passing through the culvert. Thus another analysis will be performed with the overtopping elevation increased to 955.

Before leaving the first analysis be sure to save the file (for this example the file was saved under the name TEST).

```
            SAVE MENU

TYPE LETTER OF DESIRED OPTION

<R>  TO RENAME FILE

<F>  FOR FILE LISTING

<S>  TO SAVE FILE

<ENTER>  TO RETURN
```

Return to the Culvert File Menu and select < E > to edit an existing file.

```
        CULVERT FILE MENU


TYPE LETTER OF DESIRED OPTION

<E>     EDIT OR USE A FILE

<C>     CREATE A FILE


<ESC> FOR MAIN PROGRAM MENU
```

Select C for culvert data and select #2 to change the culvert size from 2 to 3 feet.

```
          CIRCULAR CULVERT

CULVERT DIAMETER (FT)  --->  3*
```

Proceed with the overtopping analysis and obtain the following results.

SUMMARY OF CULVERT FLOWS (CFS) FILE: TEST DATE:

ELEV (FT)	TOTAL	1	2	3	4	5	6	OVERTOP	ITER
950.00	0	0	0	0	0	0	0	0	0
951.39	11	11	0	0	0	0	0	0	1
952.17	22	22	0	0	0	0	0	0	1
952.82	33	33	0	0	0	0	0	0	1
953.51	44	44	0	0	0	0	0	0	1
954.35	55	55	0	0	0	0	0	0	1
954.79	60	60	0	0	0	0	0	0	1
955.21	77	64	0	0	0	0	0	14	3
955.30	88	65	0	0	0	0	0	24	3
955.38	99	66	0	0	0	0	0	35	3
955.45	110	67	0	0	0	0	0	45	3
955.00	62	62	0	0	0	0	0 (OVERTOPPING)		

PRESS: <1> TO PLOT TOTAL RATING CURVE
<2> TO DETERMINE SPECIFIC INFORMATION ABOUT EACH CULVERT
<3> TO SEE MULTIPLE CULVERT COMPUTATIONAL ERROR TABLE
<4> TO PRINT CULVERT SUMMARY
<ENTER> TO RETURN FOR NEW RUN OR EXIT

This table shows that the design flow of 60 cfs will pass through the culvert while the additional flow from the 100-year storm will overtop the roadway.

Additional Analysis

Numerous other design selections could be analyzed included different culvert materials, improved inlets, different size culverts, etc. In addition, the model has another feature that is very useful. When the Summary Table appears on the screen there is an option <M> which will calculate the minimum size culvert need to accommodate the design flow given the other data provided in the analysis. Using this option, the following table shows that a 2.5 ft (30 inch) culvert could be used if the allowable headwater elevation could be increased to 957.21.

CULVERT ANALYSIS 3.1 DATE:
CULVERT FILE: TEST SUMMARY TABLE CULVERT NO. 1

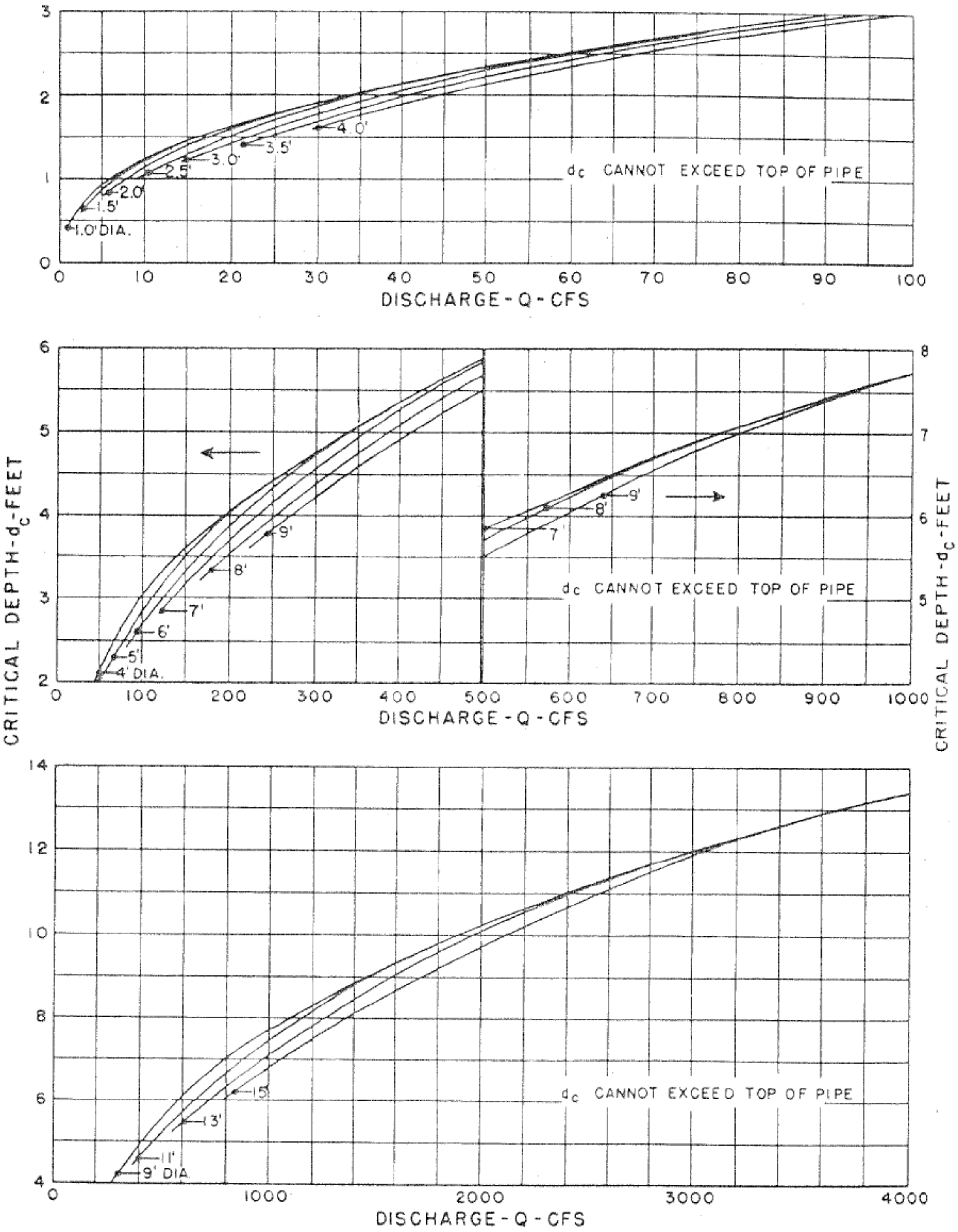
CULV NO.	<S> SITE DATA: INLET ELEV. (FT)	OUTLET ELEV. (FT)	CULVERT LENGTH (FT)	<C> CULVERT SHAPE, MATERIAL, INLET: BARRELS SHAPE MATERIAL	SPAN (FT)	RISE (FT)	MANNING n	INLET TYPE
1	950.00	947.00	75	1 - RCP	2.50	2.50	.012	CONVENTIONAL
2								
3								
4								
5								
6								

HEADWATER ELEVATION		FLOW VELOCITY		FLOW DEPTHS	
ENTER ALLOWABLE =	958.00	CULVERT =	19.46	CULVERT =	1.50
CONTROLLING =	957.56	CHANNEL =	6.77	CHANNEL =	1.77
INLET CONTROL =	957.56	DISCHARGE =	60.00	NORMAL =	1.50
OUTLET CONTROL =	954.34	SLOPE =	0.0400	CRITICAL =	2.50

MAXIMUM HEADWATER <ENTER> TO CHANGE HEADWATER <ANY KEY> TO RETURN

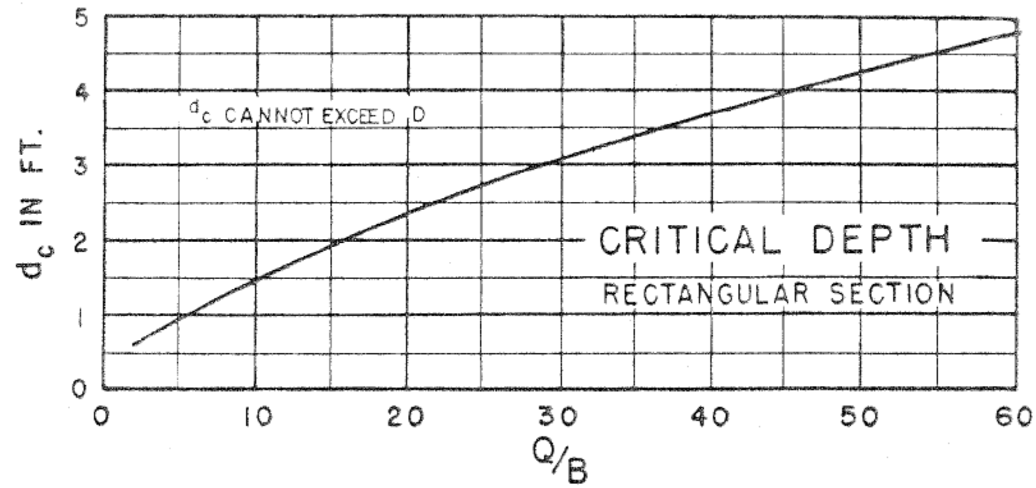
APPENDIX 4-B

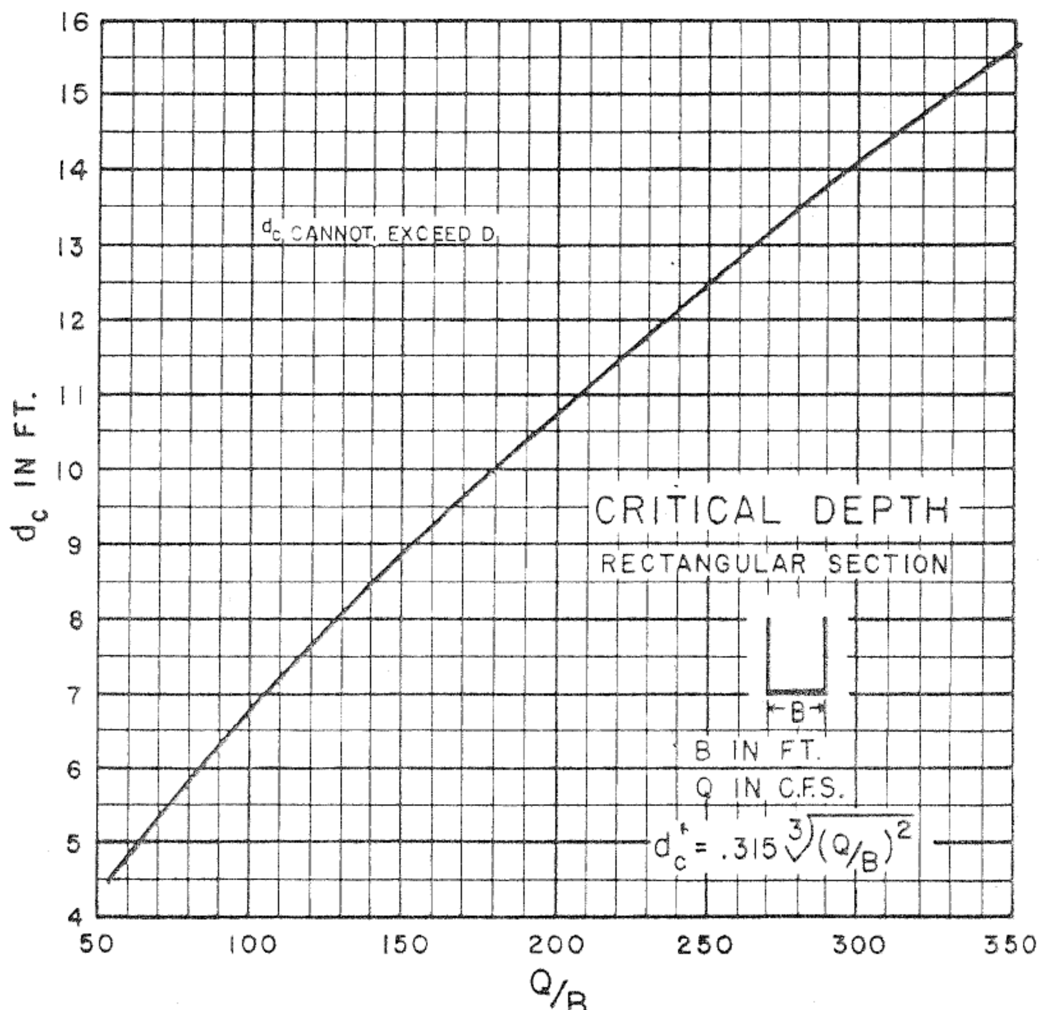
CRITICAL DEPTH CHARTS



BUREAU OF PUBLIC ROADS
JAN. 1964

CRITICAL DEPTH
CIRCULAR PIPE



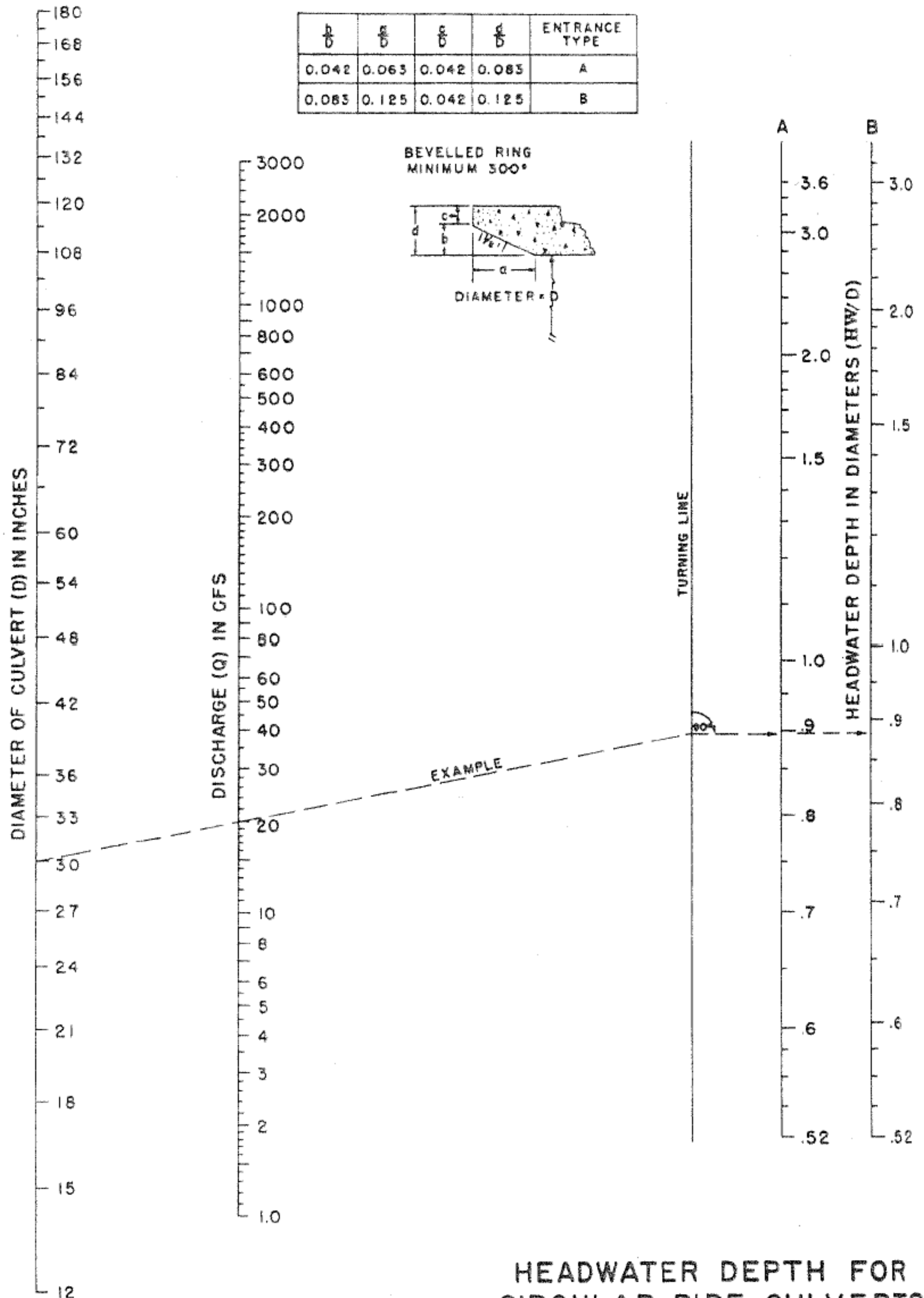


BUREAU OF PUBLIC ROADS JAN 1963

CRITICAL DEPTH
RECTANGULAR SECTION

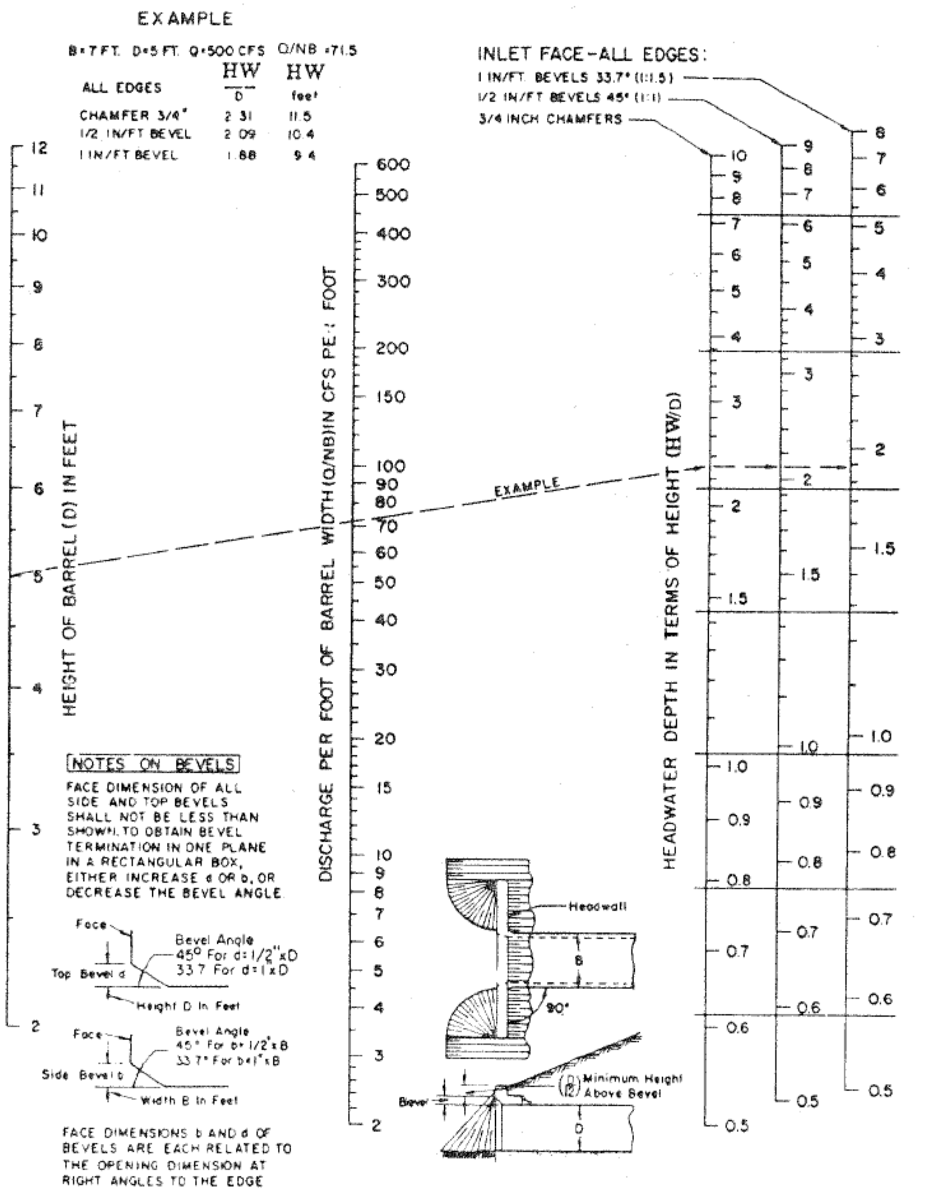
APPENDIX 4-C

CULVERT DESIGN - INTLET CONTROL



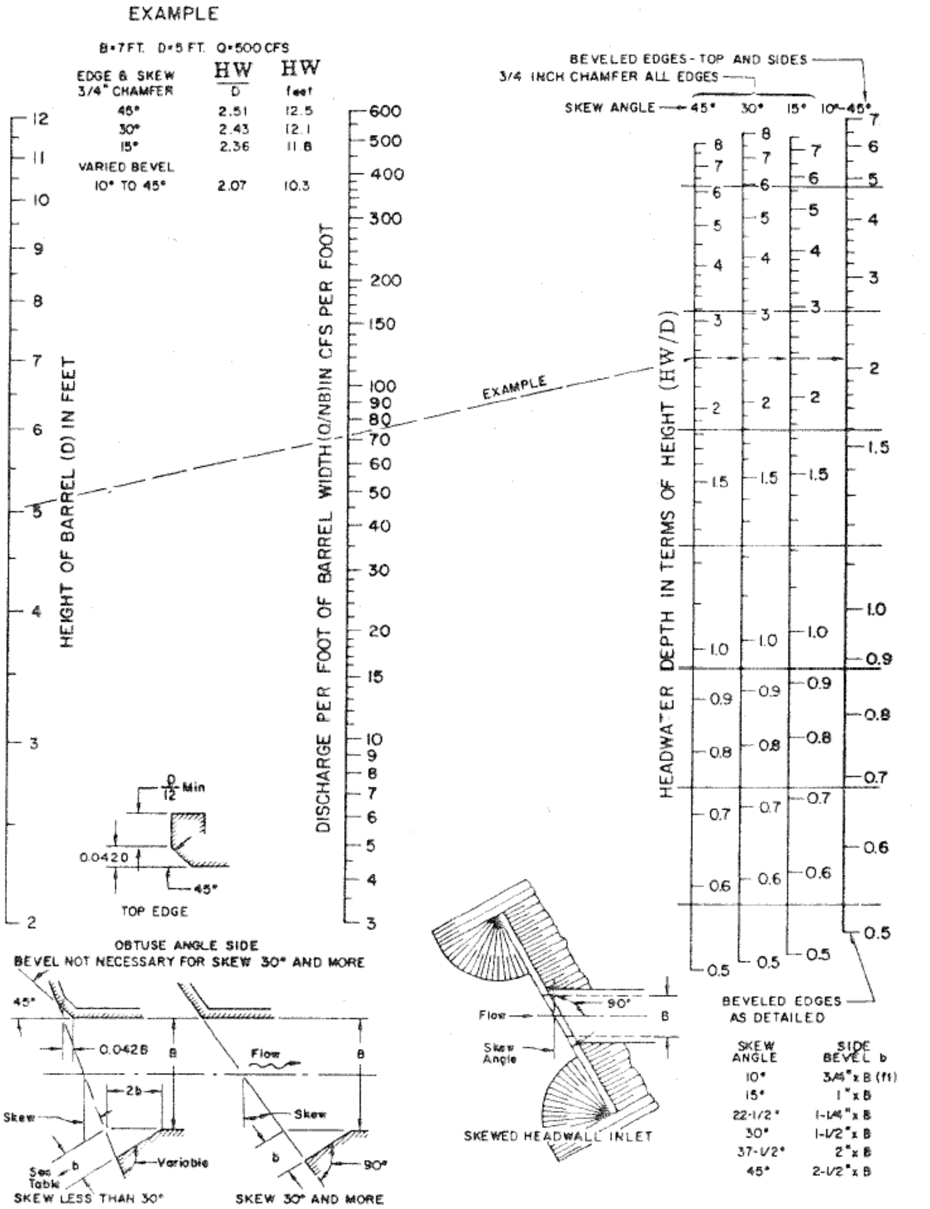
)ERAL HIGHWAY ADMINISTRATION
MAY 1973

HEADWATER DEPTH FOR
CIRCULAR PIPE CULVERTS
WITH BEVELED RING
INLET CONTROL



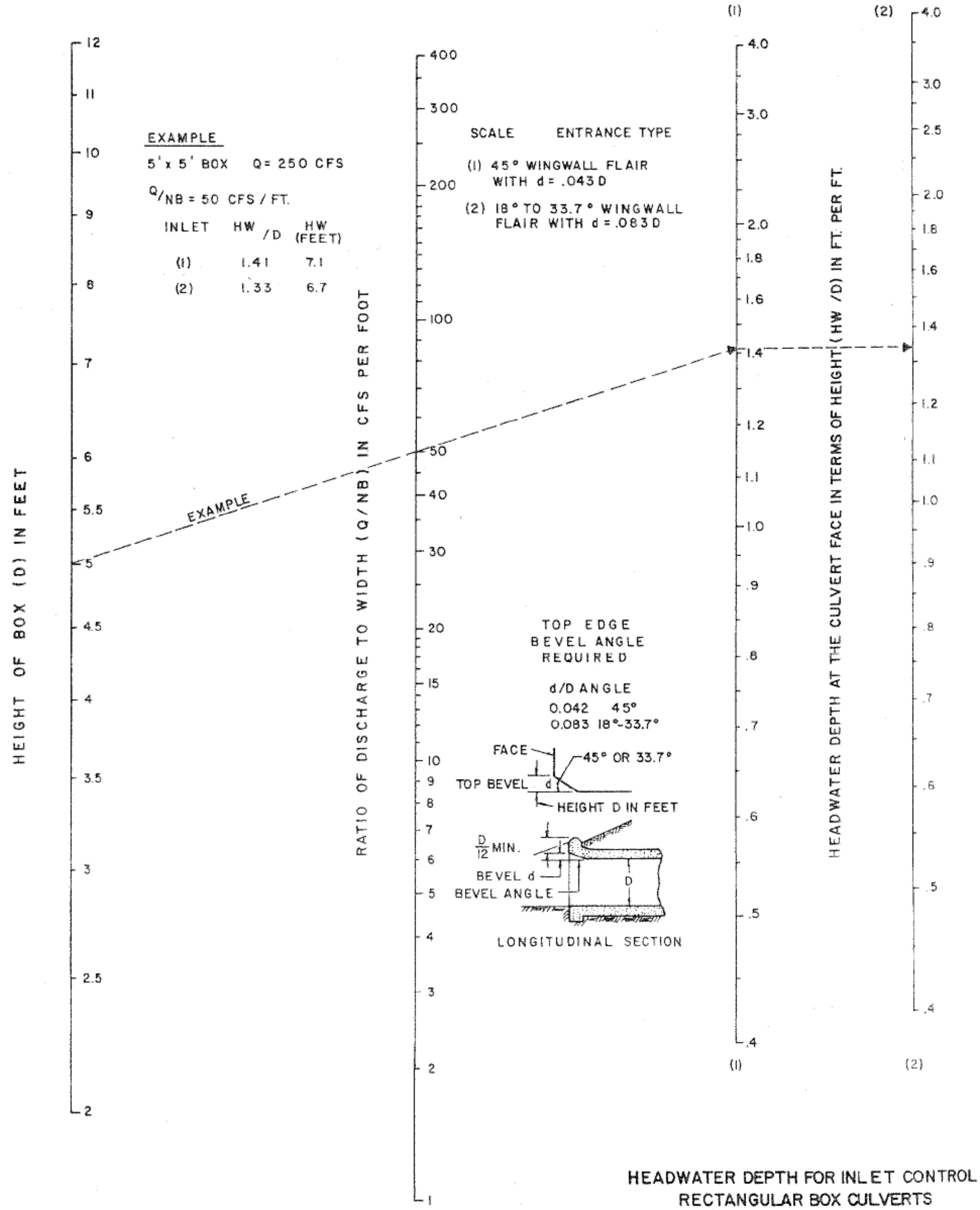
HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
90° HEADWALL
CHAMFERED OR BEVELED INLET EDGES

FEDERAL HIGHWAY ADMINISTRATION
MAY 1973



HEADWATER DEPTH FOR INLET CONTROL
SINGLE BARREL BOX CULVERTS
SKEWED HEADWALLS
CHAMFERED OR BEVELED INLET EDGES

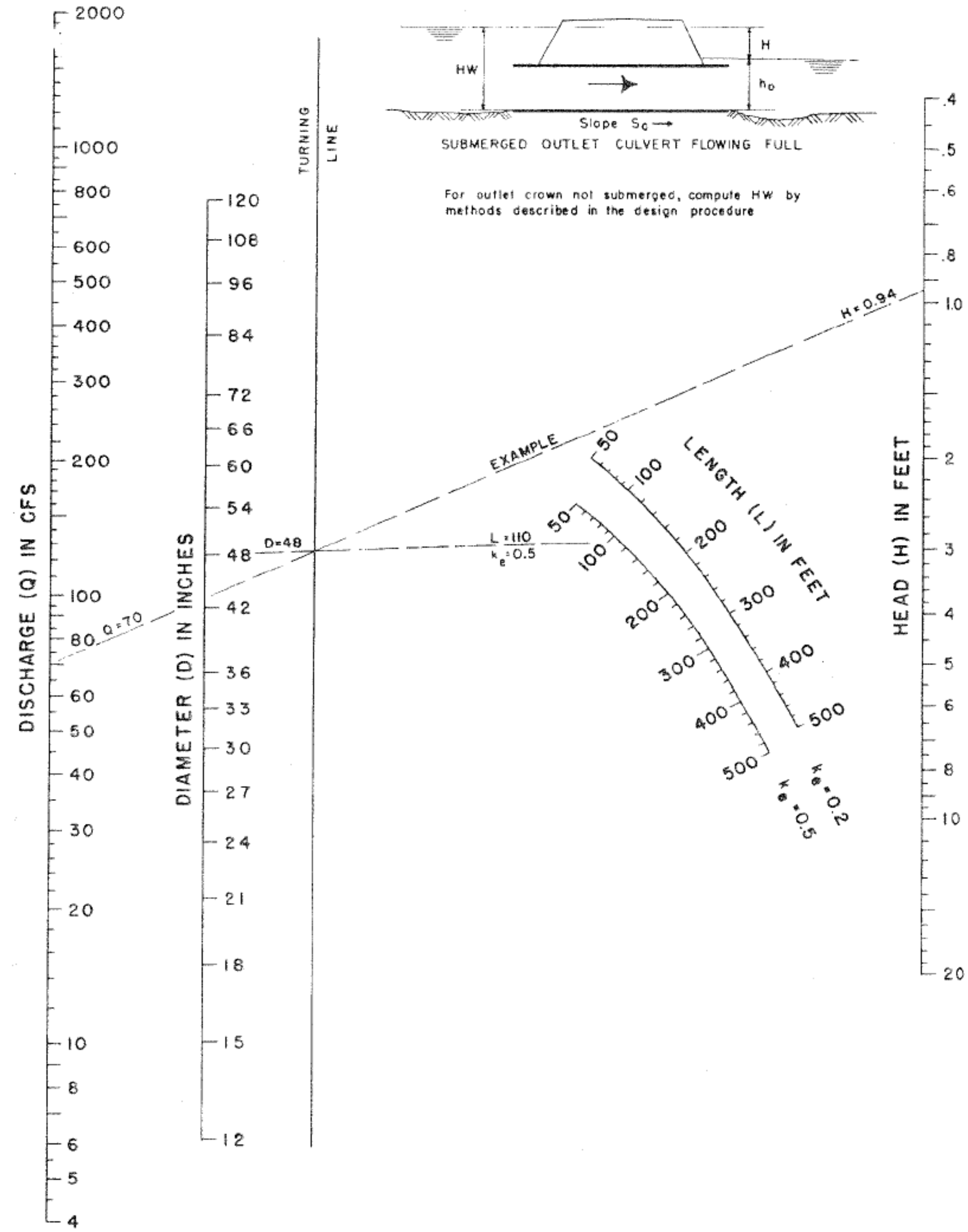
FEDERAL HIGHWAY ADMINISTRATION
MAY 1973



HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS 18° TO 33.7° & 45°
WITH BEVELED EDGE AT TOP OF INLET

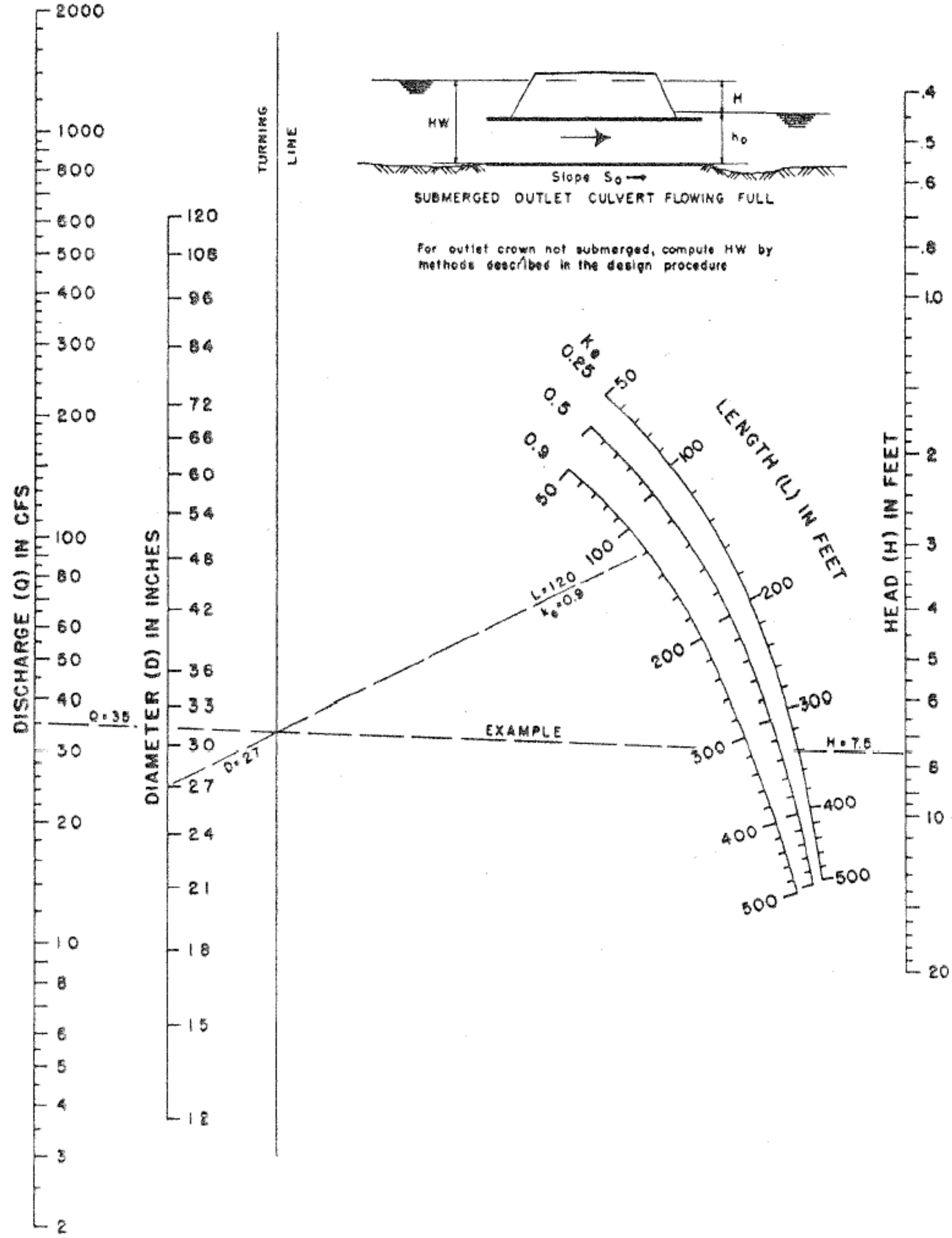
APPENDIX 4-D

CULVERT DESIGN - OUTLET CONTROL



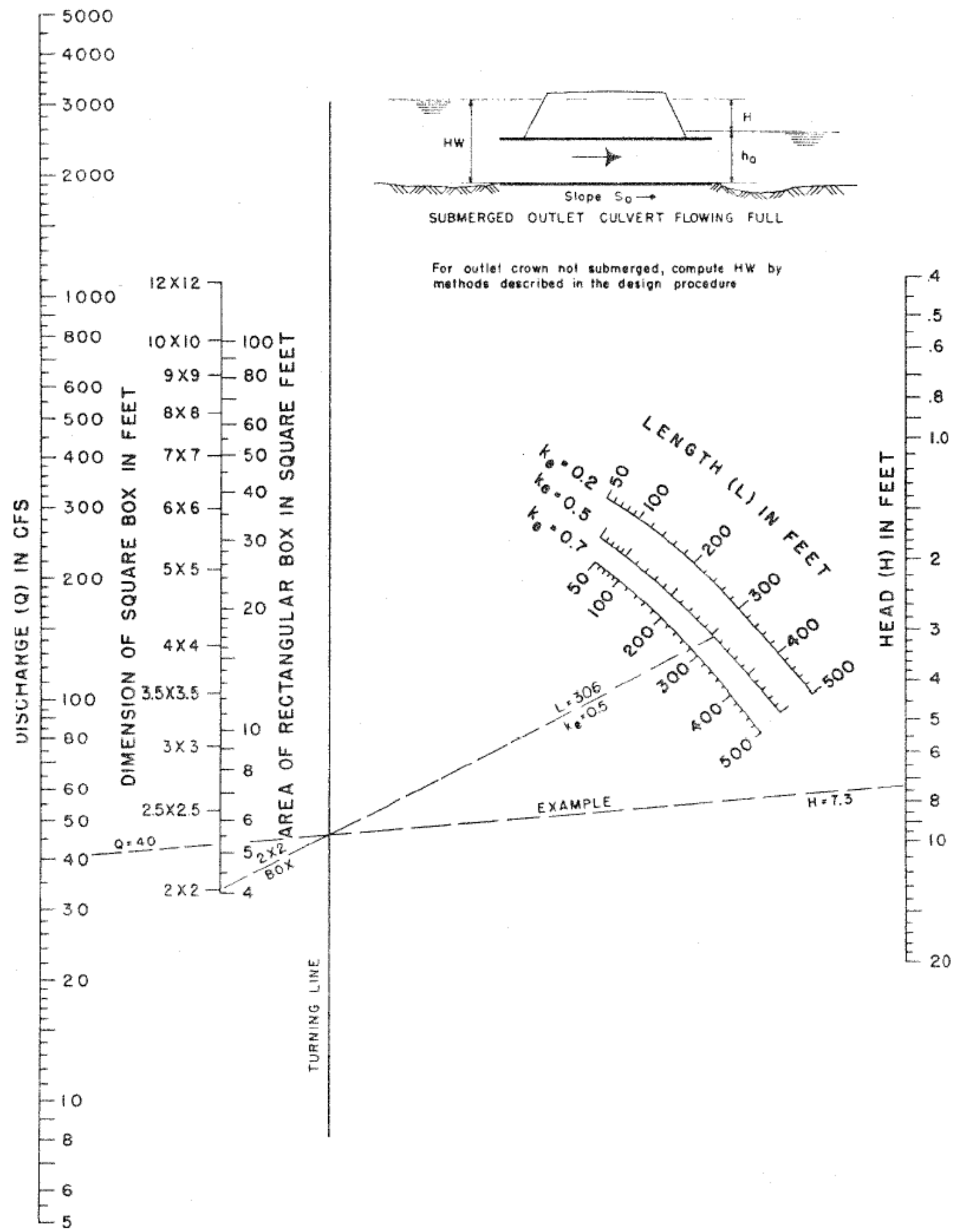
HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963



HEAD FOR
STANDARD
C. M. PIPE CULVERTS
FLOWING FULL
n = 0.024

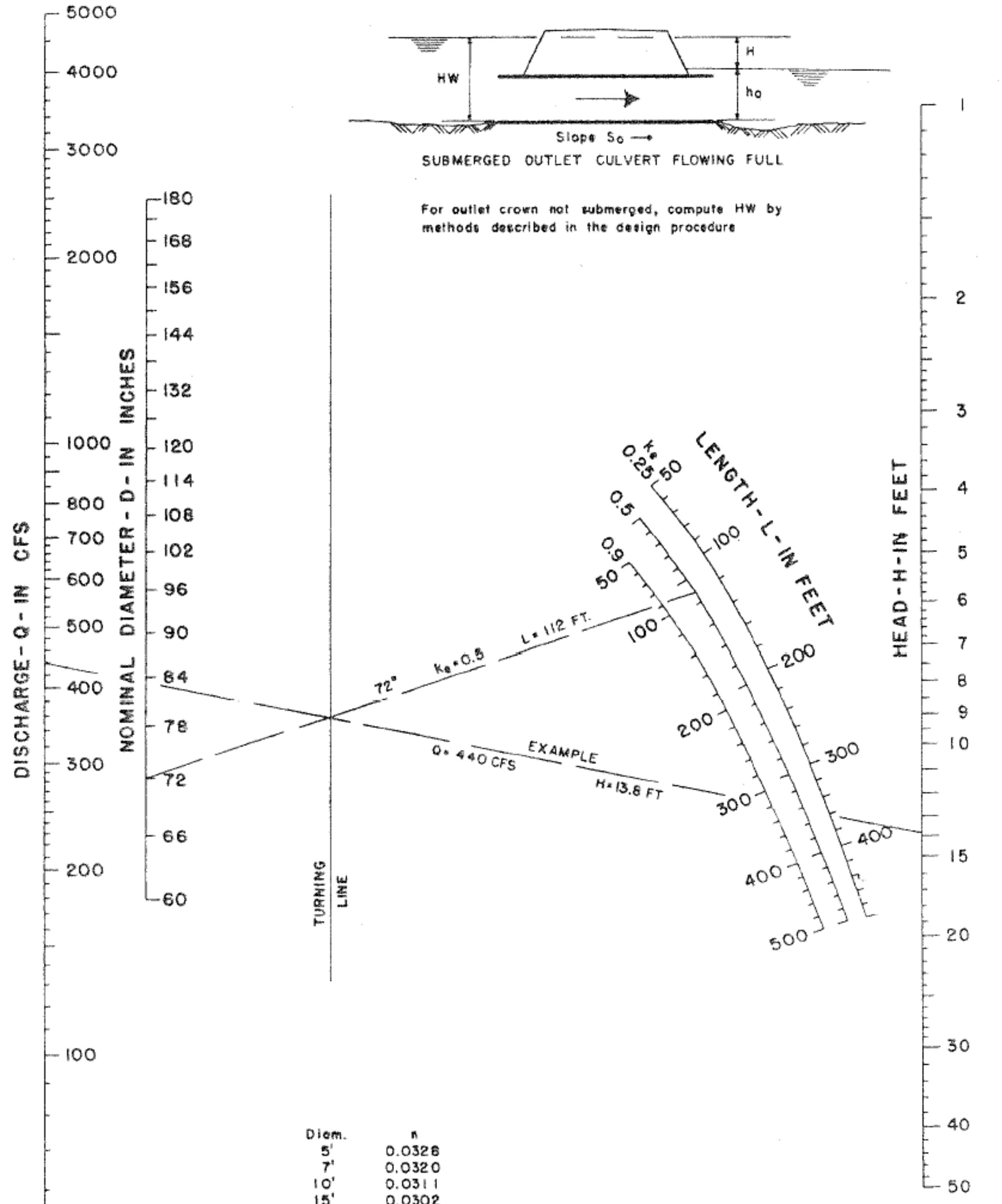
BUREAU OF PUBLIC ROADS JAN. 1963



HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL

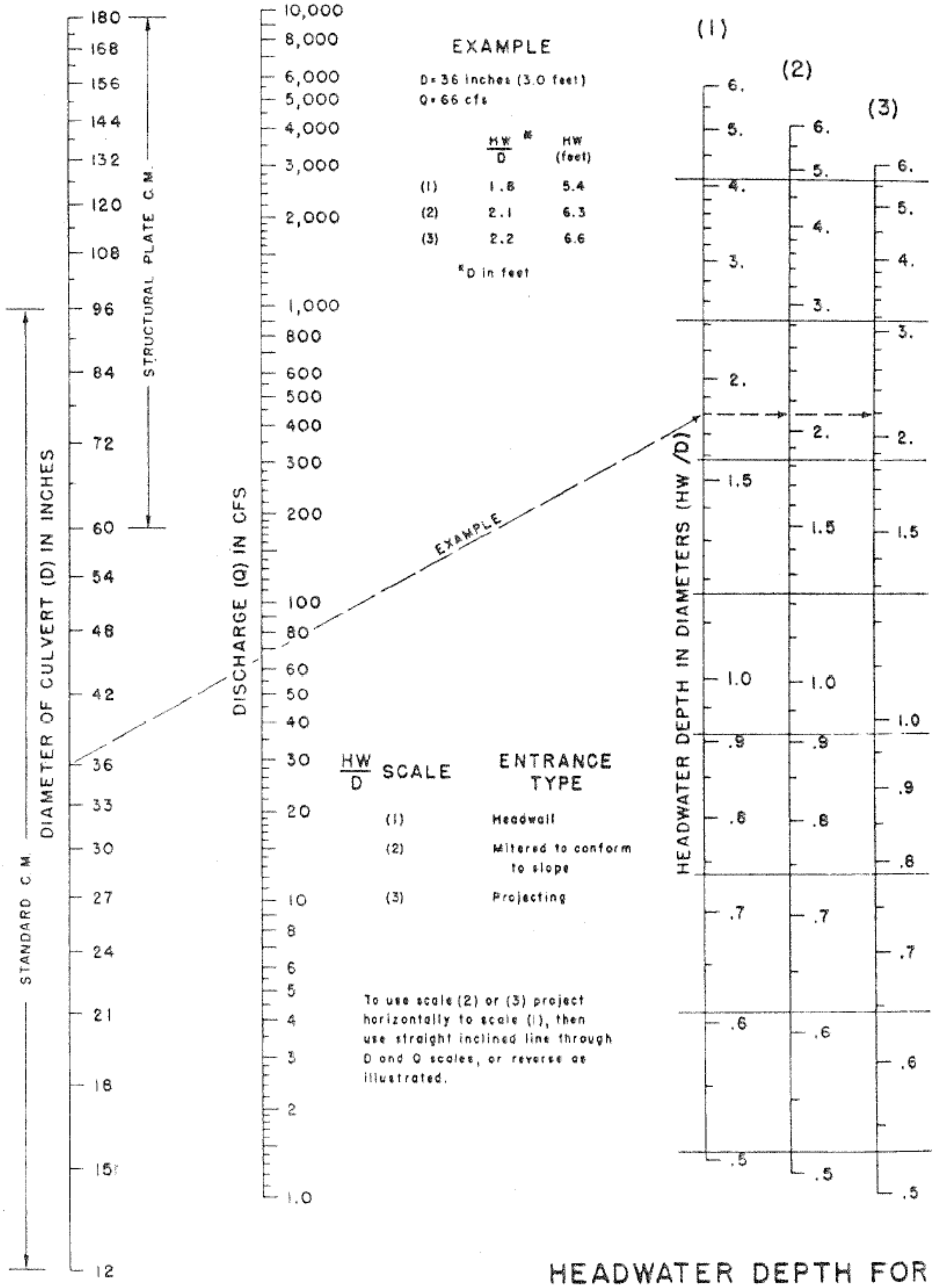
$n = 0.012$

AU OF PUBLIC ROADS JAN. 1963



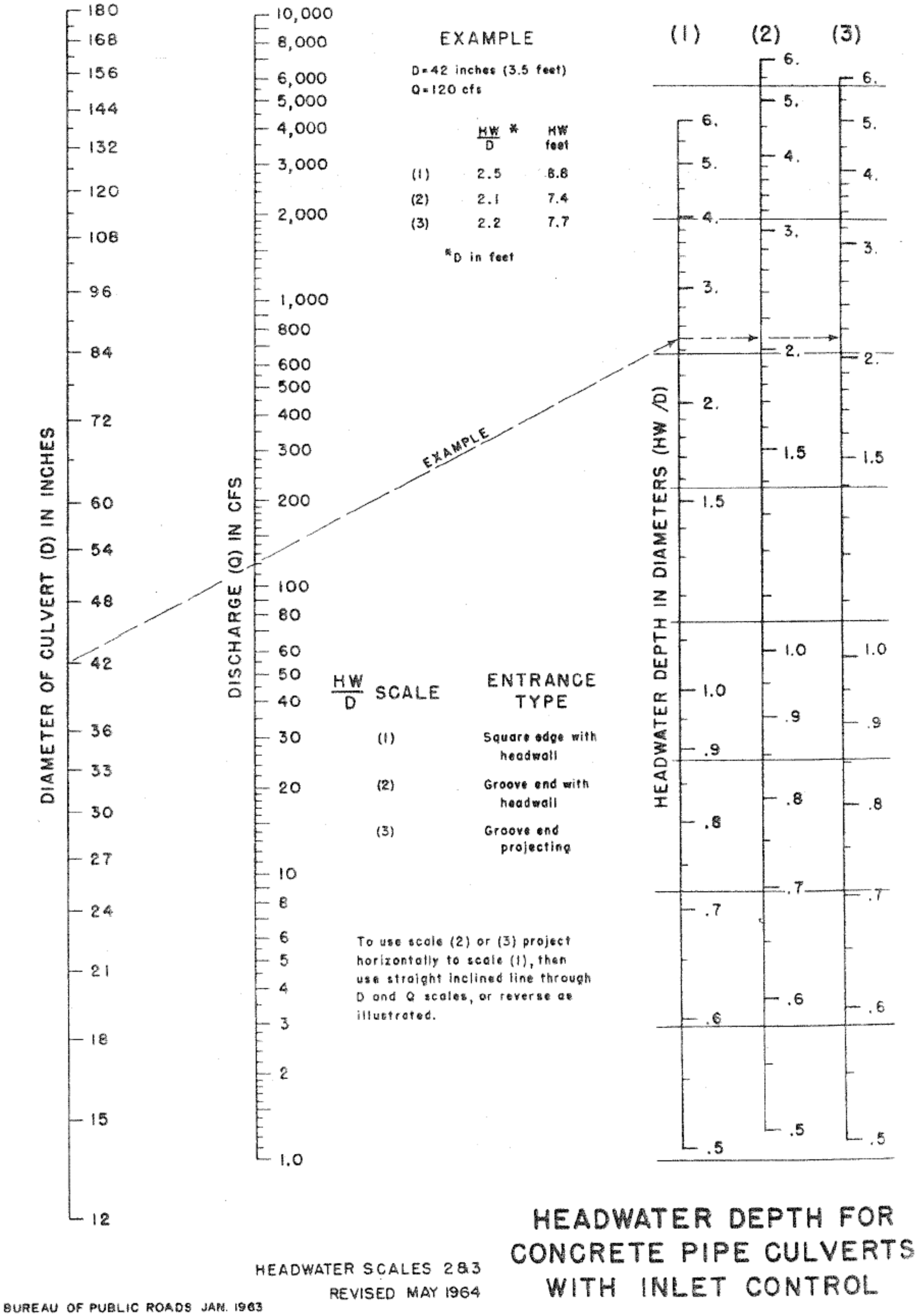
HEAD FOR
STRUCTURAL PLATE
CORR. METAL PIPE CULVERTS
FLOWING FULL
n = 0.0328 TO 0.0302

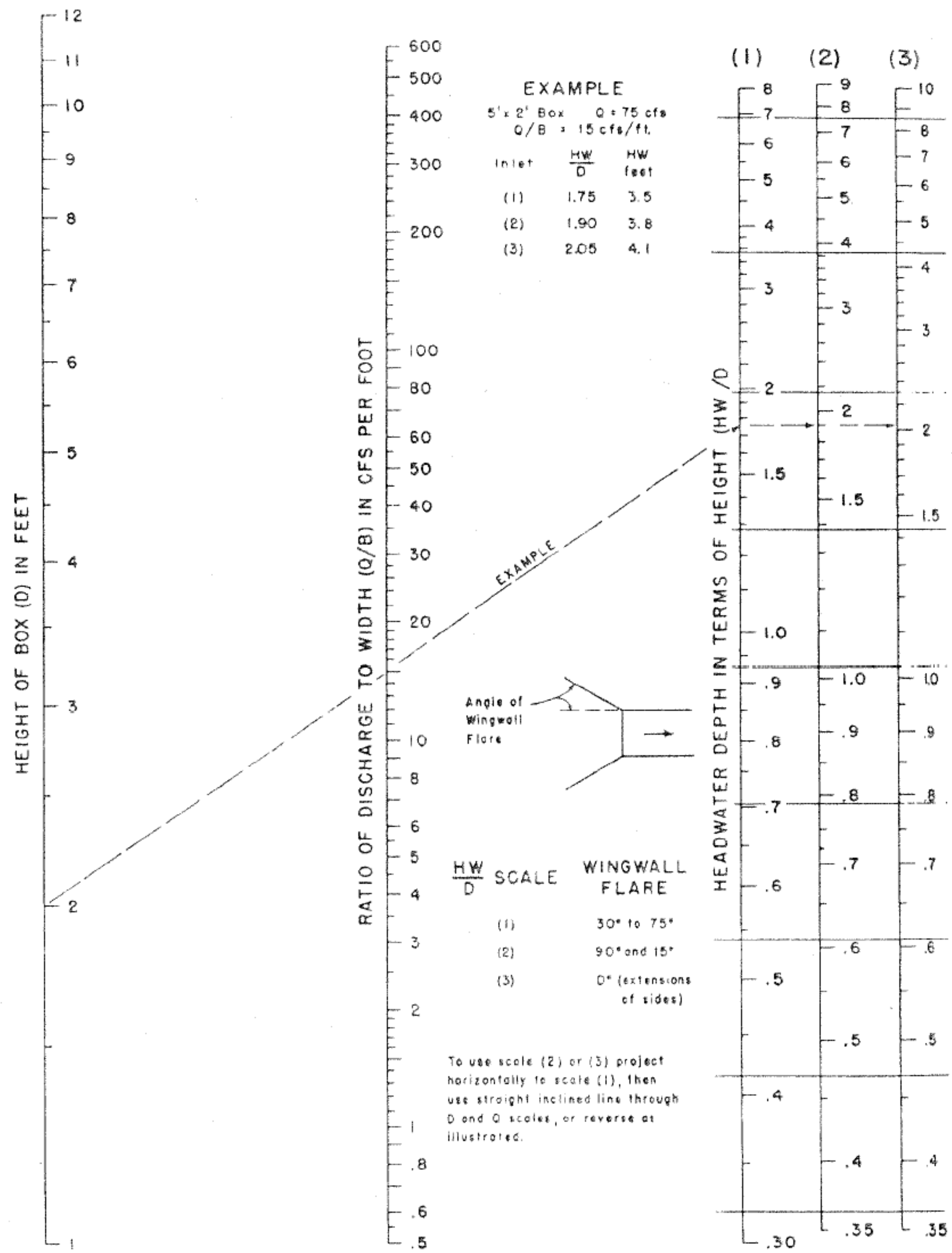
BUREAU OF PUBLIC ROADS JAN. 1963



HEADWATER DEPTH FOR
C. M. PIPE CULVERTS
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963


EXAMPLE
D=42 inches (3.5 feet)
Q=120 cfs
HW/D * HW feet
(1) 2.5 8.8
(2) 2.1 7.4
(3) 2.2 7.7
*D in feet
DIAMETER OF CULVERT (D) IN INCHES
DISCHARGE (Q) IN CFS
HEADWATER DEPTH IN DIAMETERS (HW/D)
EXAMPLE
HW/D SCALE
ENTRANCE TYPE
(1) Square edge with headwall
(2) Groove end with headwall
(3) Groove end projecting
To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated.
HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL
HEADWATER SCALES 2&3 REVISED MAY 1964
BUREAU OF PUBLIC ROADS JAN. 1963



HEADWATER DEPTH
FOR BOX CULVERTS
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

CHAPTER 5

OPEN CHANNELS

March 7, 2011

Chapter Five - Open Channels

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Chapter Five - Open Channels

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5.1 Overview

5.1.1 Introduction

Consideration of open channel hydraulics is an integral part of projects in which artificial channels and improvements to natural channels are a primary concern. Open channels are encouraged for use, especially in the major drainage system, and can have advantages in terms of cost, capacity, multiple use (i.e., recreation, wildlife habitat, etc.), and flow routing storage. Disadvantages include right-of-way needs and maintenance requirements.

Where natural channels are not well defined, runoff flow paths can usually be determined and used as the basis for location and construction of channels. In some cases the well-planned use of natural channels and flow paths in the development of a major drainage system may obviate the need for an underground storm sewer system.

For any open channel conveyance, channel stability must be evaluated to determine what measures are needed so as to avoid bottom scour and bank cutting. This chapter emphasizes procedures for performing uniform flow calculations that aid in the selection or evaluation of appropriate channel linings, depths, and grades for natural or man-made channels. Allowable velocities are provided, along with procedures for evaluating channel capacity using Manning's equation.

Even where streams retains a relatively natural state, streambanks may need to be stabilized while vegetation recovers. To preserve riparian characteristics of channels, channel improvement or stabilization projects should minimize the use of visible concrete, riprap or other hard stabilization materials.

Hydraulic analysis software such as the Corps of Engineers HEC-RAS program may be useful when preparing preliminary and final channel designs.

For any open channel conveyance, channel stability must be evaluated to determine what measures are needed to avoid bottom scour and bank cutting. Channels shall be designed for long term stability, but be left in as near a natural condition as possible. The use of open, natural channels is especially encouraged in the major drainage system and can have advantages in terms of cost, capacity, multiple use (i.e., recreation, wildlife habitat, etc.) and flow routing storage. It shall be demonstrated that the natural condition or an alternative channel design will provide stable stream bed and bank conditions. Where this cannot be demonstrated, a concrete low flow liner with a nonerosive crossection may be required by the City. Even where streams retain a relatively natural state, streambanks may need to be stabilized while vegetation recovers. To preserve riparian characteristics of channels, channel improvement or stabilization projects should minimize the use of visible concrete, riprap or other hard stabilization materials. The main classifications of open channel types are natural, bio-technical vegetated grass-lined, rock-lined, and concrete. Grass-lined channels include grass with mulch and/or sod, reinforced turf, and wetland bottom. Rock-lined channels include riprap, grouted riprap, and wire-enclosed rock.

Open channels shall be sized to handle the 100-year storm. Open channels shall be maintained by the developer or a property-owners’ association unless an alternative ownership/maintenance arrangement has been approved by the City, Planning Commission and the City Council.

5.1.2 Channel Types

The main classifications of open channel types are natural, bio-technical vegetated grass-lined, rock-lined, and concrete. Grass-lined channels include grass with mulch and/or sod, reinforced turf, and wetland bottom channel. Rock-lined channels include riprap, grouted riprap, and wire-enclosed rock. Concrete low flow liners are required, unless the engineer can clearly demonstrate an alternative channel design will provide stable stream bed and bank conditions.

5.1.2.1 Natural Channels

Natural channels are carved or shaped by nature prior to urbanization. Often, natural channels have mild slopes and are relatively stable. With increased flows due to urbanization, natural channels may experience erosion and may need grade control checks and localized bank protection to provide stabilization (UDFCD, 1990).

5.1.2.2 Grass-lined Channels

Grass-lined channels are the most desirable type of artificial channel. Vegetative linings stabilize the channel body, consolidate the soil mass of the bed, check erosion on the channel surface, and control the movement of soil particles along the channel bottom. Conditions under which vegetative linings may not be acceptable, however, include but are not limited to:

1. Flow conditions in excess of the maximum shear stress for bare soils,
2. Lack of the regular maintenance necessary to prevent domination by taller vegetation,
3. Lack of nutrients and inadequate topsoil,
4. Excessive shade,
5. High velocities, and
6. Right-of-way limitations

For grass-lined channels, proper seeding, mulching, and soil preparation are required during construction to assure establishment of a healthy stand of grass. Soil testing should be performed and the results evaluated by an agronomist to determine soil treatment requirements for pH, nitrogen, phosphorus, potassium, and other factors. In many cases, temporary erosion control measures are required to provide time for the seeding to establish a viable vegetative lining. Commercially available turf reinforcement products can be used to control erosion while vegetation is being established and to increase the erosion resistance of established vegetation.

Sodding, when implemented, should be staggered, to avoid seams in the direction of flow. Lapped or shingle sod should be staggered and overlapped by approximately 25 percent. Staked sod is usually only necessary for use on steeper slopes to prevent sliding. Low flow areas may need to be concrete or rock-lined to minimize erosion and maintenance problems.

Wetland bottom channels are a subset of grass-lined channels that are designed to encourage the development of wetlands and other riparian species in the channel bottom. In low flow areas, the banks may need protection against undermining (UDFCD, 1990).

5.1.2.3 Trickle Channel Linings

Under continuous baseflow conditions when a vegetative lining alone would not be appropriate, a small concrete pilot or trickle channel could be used to handle the continuous low flows. Vegetation could then be maintained for handling larger flows. The trickle channel allows for easier maintenance and reduces erosion caused by a meandering low flow channel. Sometimes rock-lined channels are used for trickle channels, but may require more maintenance and can encourage sediment deposition. Rock imbedded in concrete can obtain the best of both designs, but at greater cost. Trickle channel capacity should be roughly 1 to 5 percent of the design flow. Trickle flows may be conveyed in storm sewers (see Chapter 3).

5.1.2.4 Rock-lined Channels

Rock riprap, including clean rubble, is a common type of rock-lined channel. It presents a rough surface that can dissipate energy and mitigate increases in erosive velocity. These linings are usually less expensive than rigid concrete linings and have self-healing qualities that reduce maintenance. They typically require use of filter fabric and allow the infiltration and exfiltration of water. The growth of grass and weeds through the lining may present maintenance problems. The use of rock-lined channels may be restricted where right-of-way is limited, since the higher roughness values create larger cross sections. Wire-enclosed rock and grouted riprap are other examples of commonly used rock-lined channels

5.1.2.5 Concrete Channels

Concrete channels are used where smoothness offers a higher capacity for a given cross-sectional area. Higher velocities, however, create the potential for scour at channel lining transitions. A concrete lining can be destroyed by flow undercutting the lining, channel headcutting, or the buildup of hydrostatic pressure behind the rigid surfaces. Filter fabric may be required to prevent soil loss through pavement cracks. When properly designed, concrete linings may be appropriate where the channel width is restricted.

5.1.2.6 Maintenance

Open channels shall be maintained by the developer or a property-owners' association unless an alternative ownership/maintenance arrangement has been approved by the City, Planning Commission and the City Council.

5.2 Symbols And Definitions

To provide consistency within this chapter, as well as throughout this manual, the following symbols will be used. These symbols were selected because of their wide use in open channel publications.

Table 5-1 Symbols And Definitions

Symbol	Definition	Units
A	Cross-sectional area	ft^2
b	Bottom width	ft
C_x	Correction factor	-
D	Depth of flow	ft
d_{avg}	Average flow depth in the main flow channel	ft
d_x	Diameter of stone for which x percent, by weight, of the gradation is finer	ft
Fr	Froude number	-
g	Acceleration of gravity	32.2 ft/s^2
h	Superelevation	ft
K_1	Correction term reflecting bank angle	-
L	Length of channel	ft
L_p	Length of downstream protection	ft
n	Manning's roughness coefficient	-
P	Wetted perimeter	ft
Q	Discharge rate	cfs
R	Hydraulic radius	ft
r_c	Mean radius of the bend	ft
S	Slope	ft/ft
S_f	Friction slope or energy grade line slope	ft/ft
SF	Stability factor	-
S_s	Specific gravity of the riprap material	lb/ft^2
Tw	Top width	ft
V or v	Velocity of flow	ft/s
W_{50}	Weight of the median particle	lb
y_c	Critical depth	ft
y_n	Normal depth	ft
Z	Critical flow section factor	-
θ	Bank angle with the horizontal	degrees
Φ	Riprap materials angle of repose	degrees

5.3 Hydraulic Terms

5.3.1 Introduction

An open channel is a channel or conduit in which water flows with a free surface. The hydraulics of an open channel can be very complex, encompassing many different flow conditions from steady-state uniform flow to unsteady, rapidly varied flow. Most of the problems in stormwater drainage involve uniform, gradually varied or rapidly varied flow states. The calculations for uniform and gradually varied flow are relatively straight forward and are based upon similar assumptions (e.g., parallel streamlines). Rapidly varied flow computations, such as hydraulic jumps and flow over spillways, however, can be very complex and the solutions are generally empirical in nature (Tulsa, 1993).

This section will present the basic equations and computational procedures for uniform, gradually varied, and rapidly varied flow. For more detailed discussion, the user is referred to references such as Chow’s Open-Channel Hydraulics (1959) and French’s Open-Channel Hydraulics (1985). Many proprietary and non-proprietary computer software packages are available that may be used to evaluate the hydraulics of open channels.

5.3.2 Steady And Unsteady Flow

Flow in open channels is classified as steady flow or unsteady flow. Steady flow occurs when discharge or rate of flow at any cross section is constant with time. In unsteady flow the discharge or rate of flow varies from one cross section to another, with time.

5.3.3 Uniform Flow And Normal Depth

Open channel flow is said to be uniform if the depth of flow is the same at every section. For a given channel geometry, roughness, slope, and discharge, there is only one possible depth for maintaining uniform flow. This depth is referred to as normal depth (Tulsa, 1993).

True uniform is difficult to observe in the field because not all of the parameters remain the same. However, channels are often designed assuming uniform flow. This approximation is generally adequate for drainage purposes. The engineer must be aware that uniform flow computation provides only an approximation of what will occur.

Manning's Equation, presented below, is recommended for evaluating uniform flow conditions in open channels.

$$\mathbf{Q = (1.49/n)\ A\ R^{2/3}\ S^{1/2}} \tag{5.1}$$

Where:
Q = discharge rate for design conditions (cfs)
n = Manning's roughness coefficient
A = cross-sectional area (ft^2)
R = hydraulic radius A/P (ft)
P = wetted perimeter (ft)
S = slope of the energy grade line (EGL) (ft/ft)

The Manning's n value is an important variable in open channel flow computations. Variation in this variable can significantly affect discharge, depth, and velocity estimates. Since Manning's n values depend on many different physical characteristics of natural and man-made channels, care and good engineering judgment must be exercised in the selection process.

For prismatic (e.g., trapezoid, rectangular) channels, in the absence of backwater conditions, the slope of the energy grade line, water surface and channel bottom are equal.

Since normal depth is computed so frequently, special tables and figures (see Table 5-2 and Figure 5-1) have been developed using the Manning's formula for various uniform cross sections to eliminate the need for trial and error solutions, which are time consuming. Table 5-2 is applicable only for trapezoidal channels.

5.3.3.1 Uniform Flow And Normal Depth Example

A trapezoidal channel has a bottom width of 8 feet and 4 to 1 side slopes. The grade is 0.005 feet per foot. Manning's n is 0.035. What is the normal depth for discharge of 100 cfs?

<u>Solve using Table 5-2:</u>

1. Calculate:

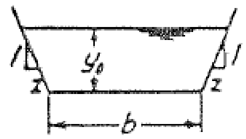
$$\frac{Q\,n}{b^{8/3}s_0^{1/2}} = \frac{100\ \times 0.035}{8^{8/3}0.005^{1/2}} = 0.19$$

2. From Table 5-2 with the above value of side slope horizontal dimension, z, equal to 4, it is found that:

$$\frac{y_0}{b} = 0.235; rearranging\ yields\ y_0 = 0.235\ \times b = 0.235\ \times 8 = 1.88\ ft$$

The designer should be aware that as the roughness coefficient increases, the same discharge will flow at a greater depth. Conversely, flow at the computed depth will result in less discharge if the roughness coefficient increases.

UNIFORM FLOW IN TRAPEZOIDAL CHANNELS BY MANNING FORMULA



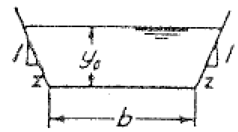
$\frac{y_0}{b}$	Values of $\frac{Qn}{b^{8/3}S_0^{1/2}}$										
	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1 1/4	z = 1 1/2	z = 2	z = 2 1/2	z = 3	z = 4
0.02	0.00213	0.00215	0.00216	0.00217	0.00218	0.00219	0.00220	0.00221	0.00222	0.00223	0.00225
0.03	0.00414	0.00419	0.00423	0.00426	0.00429	0.00431	0.00433	0.00437	0.00440	0.00443	0.00449
0.04	0.00661	0.00670	0.00679	0.00685	0.00690	0.00696	0.00700	0.00707	0.00715	0.00722	0.00735
0.05	0.00947	0.00964	0.00980	0.00991	0.0100	0.0101	0.0102	0.0103	0.0104	0.0106	0.0109
0.06	0.0127	0.0130	0.0132	0.0134	0.0136	0.0137	0.0138	0.0141	0.0143	0.0145	0.0149
0.07	0.0162	0.0166	0.0170	0.0173	0.0176	0.0177	0.0180	0.0183	0.0186	0.0190	0.0196
0.08	0.0200	0.0206	0.0211	0.0215	0.0219	0.0222	0.0225	0.0231	0.0235	0.0240	0.0250
0.09	0.0240	0.0249	0.0256	0.0262	0.0267	0.0271	0.0275	0.0282	0.0289	0.0296	0.0310
0.10	0.0283	0.0294	0.0305	0.0311	0.0318	0.0324	0.0329	0.0339	0.0348	0.0358	0.0375
0.11	0.0329	0.0342	0.0354	0.0364	0.0373	0.0380	0.0387	0.0400	0.0413	0.0424	0.0448
0.12	0.0376	0.0393	0.0408	0.0420	0.0431	0.0441	0.0450	0.0466	0.0482	0.0497	0.0527
0.13	0.0425	0.0446	0.0464	0.0480	0.0493	0.0505	0.0516	0.0537	0.0556	0.0575	0.0613
0.14	0.0476	0.0501	0.0524	0.0542	0.0559	0.0573	0.0587	0.0612	0.0636	0.0659	0.0705
0.15	0.0528	0.0559	0.0585	0.0608	0.0628	0.0645	0.0662	0.0692	0.0721	0.0749	0.0805
0.16	0.0582	0.0619	0.0650	0.0676	0.0699	0.0720	0.0740	0.0776	0.0811	0.0845	0.0912
0.17	0.0638	0.0680	0.0717	0.0748	0.0775	0.0800	0.0823	0.0867	0.0907	0.0947	0.103
0.18	0.0695	0.0744	0.0786	0.0822	0.0854	0.0883	0.0910	0.0961	0.101	0.105	0.115
0.19	0.0753	0.0809	0.0857	0.0900	0.0936	0.0970	0.100	0.106	0.112	0.117	0.128
0.20	0.0813	0.0875	0.0932	0.0979	0.102	0.106	0.110	0.116	0.123	0.129	0.141
0.21	0.0873	0.0944	0.101	0.106	0.111	0.115	0.120	0.127	0.134	0.142	0.156
0.22	0.0935	0.101	0.109	0.115	0.120	0.125	0.130	0.139	0.147	0.155	0.171
0.23	0.0997	0.109	0.117	0.124	0.130	0.135	0.141	0.151	0.160	0.169	0.187
0.24	0.106	0.116	0.125	0.133	0.139	0.146	0.152	0.163	0.173	0.184	0.204
0.25	0.113	0.124	0.133	0.142	0.150	0.157	0.163	0.176	0.187	0.199	0.222
0.26	0.119	0.131	0.142	0.152	0.160	0.168	0.175	0.189	0.202	0.215	0.241
0.27	0.126	0.139	0.151	0.162	0.171	0.180	0.188	0.203	0.218	0.232	0.260
0.28	0.133	0.147	0.160	0.172	0.182	0.192	0.201	0.217	0.234	0.249	0.281
0.29	0.139	0.155	0.170	0.182	0.193	0.204	0.214	0.232	0.250	0.267	0.302
0.30	0.146	0.163	0.179	0.193	0.205	0.217	0.227	0.248	0.267	0.286	0.324
0.31	0.153	0.172	0.189	0.204	0.217	0.230	0.242	0.264	0.285	0.306	0.347
0.32	0.160	0.180	0.199	0.215	0.230	0.243	0.256	0.281	0.304	0.327	0.371
0.33	0.167	0.189	0.209	0.227	0.243	0.257	0.271	0.298	0.323	0.348	0.396
0.34	0.174	0.198	0.219	0.238	0.256	0.272	0.287	0.315	0.343	0.369	0.422
0.35	0.181	0.207	0.230	0.251	0.270	0.287	0.303	0.334	0.363	0.392	0.450
0.36	0.190	0.216	0.241	0.263	0.283	0.302	0.319	0.353	0.384	0.416	0.477
0.37	0.196	0.225	0.251	0.275	0.297	0.317	0.336	0.372	0.406	0.440	0.507
0.38	0.203	0.234	0.263	0.289	0.311	0.333	0.354	0.392	0.429	0.465	0.536
0.39	0.210	0.244	0.274	0.301	0.326	0.349	0.371	0.412	0.452	0.491	0.568
0.40	0.218	0.254	0.286	0.314	0.341	0.366	0.389	0.433	0.476	0.518	0.600
0.41	0.225	0.263	0.297	0.328	0.357	0.383	0.408	0.455	0.501	0.545	0.634
0.42	0.233	0.279	0.310	0.342	0.373	0.401	0.427	0.478	0.526	0.574	0.668
0.43	0.241	0.282	0.321	0.356	0.389	0.418	0.447	0.501	0.553	0.604	0.703
0.44	0.249	0.292	0.334	0.371	0.405	0.437	0.467	0.524	0.579	0.634	0.739
0.45	0.256	0.303	0.346	0.385	0.422	0.455	0.487	0.548	0.607	0.665	0.778
0.46	0.263	0.313	0.359	0.401	0.439	0.475	0.509	0.574	0.635	0.696	0.816
0.47	0.271	0.323	0.371	0.417	0.457	0.494	0.530	0.600	0.665	0.729	0.856
0.48	0.279	0.333	0.384	0.432	0.475	0.514	0.552	0.626	0.695	0.763	0.897
0.49	0.287	0.345	0.398	0.448	0.492	0.534	0.575	0.652	0.725	0.797	0.939
0.50	0.295	0.356	0.411	0.463	0.512	0.556	0.599	0.679	0.758	0.833	0.983
0.52	0.310	0.377	0.438	0.496	0.548	0.599	0.646	0.735	0.820	0.906	1.07
0.54	0.327	0.398	0.468	0.530	0.590	0.644	0.696	0.795	0.891	0.984	1.17
0.56	0.343	0.421	0.496	0.567	0.631	0.690	0.748	0.856	0.963	1.07	1.27
0.58	0.359	0.444	0.526	0.601	0.671	0.739	0.802	0.922	1.04	1.15	1.37
0.60	0.375	0.468	0.556	0.640	0.717	0.789	0.858	0.988	1.12	1.24	1.49
0.62	0.391	0.492	0.590	0.679	0.763	0.841	0.917	1.06	1.20	1.33	1.60
0.64	0.408	0.516	0.620	0.718	0.809	0.894	0.976	1.13	1.28	1.43	1.72

Table 5-2 Uniform Flow for Trapezoidal Channels by Manning Formula

Source: UDFCD, 1990

NORMAL DEPTH FOR UNIFORM FLOW $\frac{y_0}{b}$ = 0.66 to 5.00 (4)

UNIFORM FLOW IN TRAPEZOIDAL CHANNELS BY MANNING FORMULA



$\frac{y_0}{b}$	Values of $\frac{Qn}{b^{8/3}S_0^{1/2}}$										
	$z = 0$	$z = \frac{1}{4}$	$z = \frac{1}{2}$	$z = \frac{3}{4}$	$z = 1$	$z = 1\frac{1}{4}$	$z = 1\frac{1}{2}$	$z = 2$	$z = 2\frac{1}{2}$	$z = 3$	$z = 4$
0.66	0.424	0.541	0.653	0.759	0.858	0.951	1.04	1.21	1.37	1.53	1.85
0.68	0.441	0.566	0.687	0.801	0.908	1.01	1.10	1.29	1.47	1.64	1.98
0.70	0.457	0.591	0.722	0.842	0.958	1.07	1.17	1.37	1.56	1.75	2.12
0.72	0.474	0.617	0.757	0.887	1.01	1.13	1.24	1.45	1.66	1.87	2.27
0.74	0.491	0.644	0.793	0.932	1.07	1.19	1.31	1.55	1.77	1.98	2.41
0.76	0.508	0.670	0.830	0.981	1.12	1.26	1.39	1.64	1.88	2.11	2.57
0.78	0.525	0.698	0.868	1.03	1.18	1.32	1.46	1.73	1.98	2.24	2.73
0.80	0.542	0.725	0.906	1.08	1.24	1.40	1.54	1.83	2.10	2.37	2.90
0.82	0.559	0.753	0.945	1.13	1.30	1.47	1.63	1.93	2.22	2.51	3.07
0.84	0.576	0.782	0.985	1.18	1.36	1.54	1.71	2.03	2.34	2.65	3.25
0.86	0.593	0.810	1.03	1.23	1.43	1.61	1.79	2.14	2.47	2.80	3.44
0.88	0.610	0.839	1.07	1.29	1.49	1.69	1.88	2.25	2.60	2.95	3.63
0.90	0.627	0.871	1.11	1.34	1.56	1.77	1.98	2.36	2.74	3.11	3.83
0.92	0.645	0.898	1.15	1.40	1.63	1.86	2.07	2.48	2.88	3.27	4.04
0.94	0.662	0.928	1.20	1.46	1.70	1.94	2.16	2.60	3.03	3.43	4.25
0.96	0.680	0.960	1.25	1.52	1.78	2.03	2.27	2.73	3.17	3.61	4.48
0.98	0.697	0.991	1.29	1.58	1.85	2.11	2.37	2.85	3.33	3.79	4.70
1.00	0.714	1.02	1.33	1.64	1.93	2.21	2.47	2.99	3.48	3.97	4.93
1.05	0.759	1.10	1.46	1.80	2.13	2.44	2.75	3.33	3.90	4.45	5.55
1.10	0.802	1.19	1.58	1.97	2.34	2.69	3.04	3.70	4.34	4.96	6.21
1.15	0.846	1.27	1.71	2.14	2.56	2.96	3.34	4.09	4.82	5.52	6.91
1.20	0.891	1.36	1.85	2.33	2.79	3.24	3.68	4.50	5.32	6.11	7.68
1.25	0.936	1.45	1.99	2.52	3.04	3.54	4.03	4.95	5.86	6.73	8.48
1.30	0.980	1.54	2.14	2.73	3.30	3.85	4.39	5.42	6.42	7.39	9.34
1.35	1.02	1.64	2.29	2.94	3.57	4.18	4.76	5.90	7.01	8.10	10.2
1.40	1.07	1.74	2.45	3.16	3.85	4.52	5.18	6.43	7.65	8.83	11.2
1.45	1.11	1.84	2.61	3.39	4.15	4.88	5.60	6.98	8.30	9.62	12.2
1.50	1.16	1.94	2.78	3.63	4.46	5.26	6.04	7.55	9.02	10.4	13.3
1.55	1.20	2.05	2.96	3.88	4.78	5.65	6.50	8.14	9.74	11.3	14.4
1.60	1.25	2.15	3.14	4.14	5.12	6.06	6.99	8.79	10.5	12.2	15.6
1.65	1.30	2.27	3.33	4.41	5.47	6.49	7.50	9.42	11.3	13.2	16.8
1.70	1.34	2.38	3.52	4.69	5.83	6.94	8.02	10.1	12.2	14.2	18.1
1.75	1.39	2.50	3.73	4.98	6.21	7.41	8.57	10.9	13.0	15.2	19.5
1.80	1.43	2.62	3.93	5.28	6.60	7.89	9.13	11.6	14.0	16.3	20.9
1.85	1.48	2.74	4.15	5.59	7.01	8.40	9.75	12.4	15.0	17.4	22.4
1.90	1.52	2.86	4.36	5.91	7.43	8.91	10.4	13.2	15.9	18.7	24.0
1.95	1.57	2.99	4.59	6.24	7.87	9.46	11.0	14.0	17.0	19.9	25.6
2.00	1.61	3.12	4.83	6.58	8.32	10.0	11.7	14.9	18.0	21.1	27.2
2.10	1.71	3.39	5.31	7.30	9.27	11.2	13.1	16.8	20.3	23.9	30.8
2.20	1.79	3.67	5.82	8.06	10.3	12.5	14.6	18.7	22.8	26.8	34.7
2.30	1.89	3.96	6.36	8.86	11.3	13.8	16.2	20.9	25.4	30.0	38.8
2.40	1.98	4.26	6.93	9.72	12.5	15.3	17.9	23.1	28.3	33.4	43.3
2.50	2.07	4.58	7.52	10.6	13.7	16.8	19.8	25.6	31.3	37.0	48.0
2.60	2.16	4.90	8.14	11.6	15.0	18.4	21.7	28.2	34.5	40.8	53.0
2.70	2.26	5.24	8.80	12.6	16.3	20.1	23.8	31.0	37.9	44.8	58.4
2.80	2.35	5.59	9.49	13.6	17.8	21.9	25.9	33.8	41.6	49.1	64.0
2.90	2.44	5.95	10.2	14.7	19.3	23.8	28.2	36.9	45.3	53.7	70.1
3.00	2.53	6.33	11.0	15.9	20.9	25.8	30.6	40.1	49.4	58.4	76.4
3.20	2.72	7.12	12.5	18.3	24.2	30.1	35.8	47.1	58.0	68.9	90.3
3.40	2.90	7.97	14.2	21.0	27.9	34.8	41.5	54.6	67.7	80.2	105
3.60	3.09	8.86	16.1	24.0	32.0	39.9	47.8	63.0	78.2	92.8	122
3.80	3.28	9.81	18.1	27.1	36.3	45.5	54.6	72.4	89.6	107	141
4.00	3.46	10.8	20.2	30.5	41.1	51.6	61.9	82.2	102	122	160
4.50	3.92	13.5	26.2	40.1	54.5	68.8	82.9	111	136	164	217
5.00	4.39	16.7	33.1	51.5	70.3	89.2	108	145	181	216	287

Table 5-2 (continued) Uniform Flow for Trapezoidal Channels by Manning Formula

Source: UDFCD, 1990

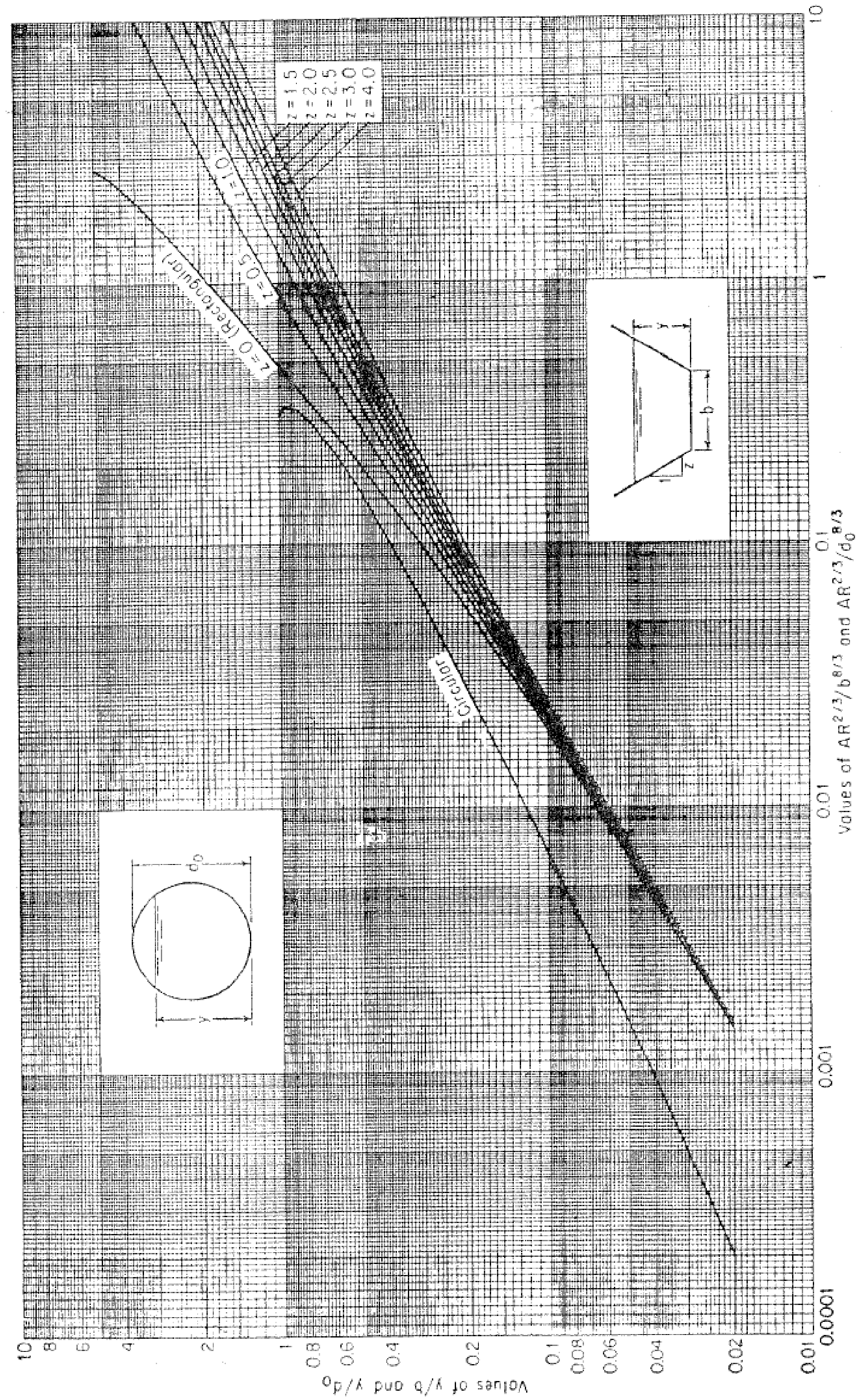


Figure 5-1 Normal Depth for Uniform Flow in Open Channels

Source: Chow, 1959

5.3.4 Critical Flow

Critical flow in an open channel or covered conduit with a free water surface is characterized by the following conditions:

- The specific energy is a minimum for a given discharge.
- The discharge is a maximum for a given specific energy.
- The specific force is a minimum for a given discharge.
- The velocity head is equal to half the hydraulic depth in a channel of small slope.
- The Froude number is equal to 1.0.
- The velocity of flow in a channel of small slope is equal to the celerity of small gravity waves in shallow waters.

If the critical state of flow exists throughout an entire reach, the channel flow is critical and the channel slope is at critical slope S_c. A slope less than S_c will cause subcritical flow, while a slope greater than S_c will cause supercritical flow. Under subcritical flow, surface waves propagate upstream as well as downstream, and control of subcritical flow depth is always downstream. Under supercritical flow, surface disturbance can propagate only in the downstream direction, and control of supercritical flow depth is always at the upstream end of the critical flow region. A flow at or near the critical state is not 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.

The criteria of minimum specific energy for critical flow results in the definition of the Froude number, which is expressed by the following equation:

$$\mathbf{Fr = v / (gD)^{0.5}} \qquad \textbf{(5.2)}$$

Where: Fr = Froude number
v = mean velocity of flow (ft/s)
g = acceleration of gravity (32.2 ft/s^2)
D = hydraulic depth (ft) - defined as the cross sectional area of water normal to the direction of channel flow divided by free surface width.

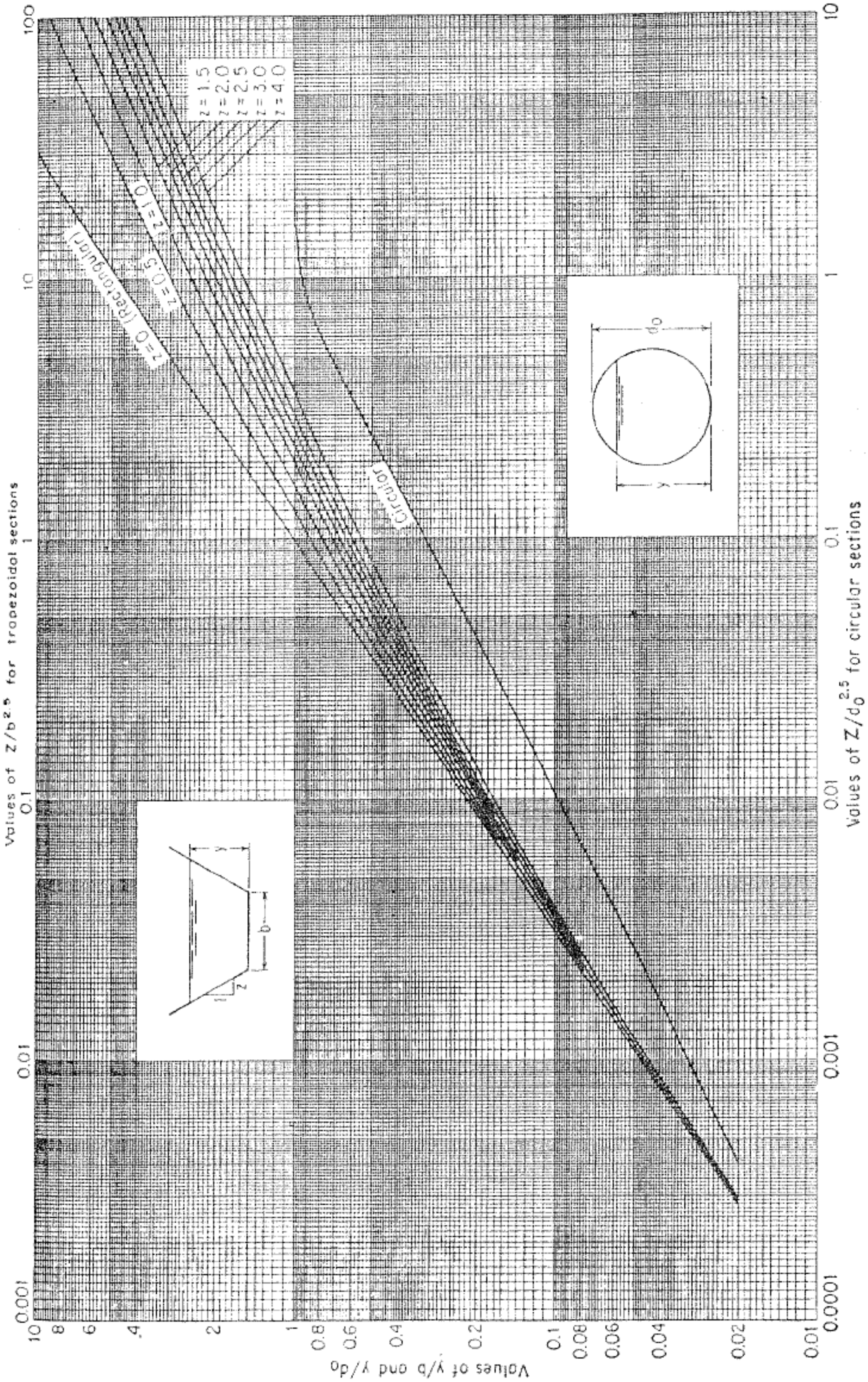
Since the Froude number is a function of depth, the equation indicates there is only one possible critical depth for maintaining a given discharge in a given channel. When the Froude number equals 1.0, the flow is critical. The Froude number should be calculated for the design of open channels to check the flow state. The computation of critical flow for trapezoidal and circular sections can be performed with the use of Figure 5-2 (Chow, 1959).

5.3.5 Gradually Varied Flow

The most common occurrence of gradually varied flow in storm drainage is the backwater created by culverts, storm sewer inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel and the water surface profile should be computed using backwater techniques.

Many computer programs are available for computation of backwater curves. The most general and widely used program is, HEC-RAS, River Analysis System, developed by the U.S. Army Corps of Engineers (USACE, 1995) and is the program recommended for floodwater profile computations. HEC-RAS will compute water surface profiles for natural and man-made channels. Bridge Waterways Analysis Model (WSPRO) and HY-8 are programs developed for the Federal Highway Administration that can also be used to perform backwater calculations for both natural and artificial channels.

For prismatic channels, the backwater calculation can be computed manually using the direct step method, as presented by Chow (1959). For an irregular nonuniform channel, the standard step method is recommended, although it is a more tedious and iterative process. The use of HEC-RAS is recommended for non-uniform channel analysis. The reader is directed to the HEC-RAS documentation for proper use of the model.



NOTE: $Z = Q / \sqrt{g}$

Figure 5-2 Critical Depth in Open Channels

Source: Chow, 1959

5.3.6 Rapidly Varied Flow

Rapidly varied flow is characterized by pronounced curvature of streamlines. The change in curvature may become so abrupt that the flow profile is virtually broken, resulting in high turbulence. Empirical solutions are usually relied on to solve specific, rapidly varying flow problems. Hydraulic jump is an example of rapidly varied flow that commonly occurs in urban storm drainage.

5.3.6.1 Hydraulic Jump

Hydraulic jumps occur when a supercritical flow rapidly changes to subcritical flow. The result is usually an abrupt rise of the water surface with an accompanying loss of kinetic energy. The hydraulic jump is an effective energy dissipation device which is often used to control erosion at drainage structures.

In urban hydraulics, the jump may occur at grade control structures, inside of or at the outlet of storm sewers or concrete box culverts, or at the outlet of an emergency spillway for detention ponds. The evaluation of a hydraulic jump should consider the high energy loss and erosive forces that are associated with the jump. For rigid-lined facilities such as pipes or concrete channels, the forces and the change in energy can affect the structural stability or the hydraulic capacity. For grass-lined channels, unless the erosive forces are controlled, serious damage can result. Control of jump location is usually obtained by check dams or grade control structures that confine the erosive forces to a protected area. Flexible material such as riprap or rubble usually affords the most effective protection.

5.3.6.1.1 Storm Sewers

The analysis of the hydraulic jump inside storm sewers is approximate, because of the lack of data for circular, elliptical, or arch sections. The jump can be approximately located by intersecting the energy grade line of the super-critical and subcritical flow reaches. The primary concerns are whether the pipe can withstand the forces which may separate the joint or damage the pipe wall, and whether the jump will affect the hydraulic characteristics. The effect on pipe capacity can be determined by evaluating the energy grade line, taking into account the energy lost by the jump. In general, for Froude numbers less than 2.0, the loss of energy is less than 10 percent. French (1985) provides semi-empirical procedures to evaluate the hydraulic jump in circular and other non-rectangular channel sections. "Hydraulic Analysis of Broken Back Culverts", Nebraska Department of Roads, January 1998 provides guidance for analysis of hydraulic jump in pipes.

5.3.6.1.2 Box Culverts

For long box culverts with a concrete bottom, the concerns about jump are the same as for storm sewers. However, the jump can be adequately defined for box culverts/drains and for spillways using the jump characteristics of rectangular sections. The relationship between variables for a hydraulic jump in rectangular sections can be expressed as:

$$\mathbf{D_2 = -(D_1/2) + [(D_1^2/4) + (2v_1^2 D_1/g)]^{1/2}} \qquad \textbf{(5.3)}$$

Where:
D_2 = depth below jump (ft)
D_1 = depth above jump (ft)
v_1 = velocity above jump (ft/s)
g = acceleration due to gravity (32.2 ft/s^2)

Additional details on hydraulic jumps can be found in HEC-14 (1983), Chow (1959), Peterska (1978), and French (1985).

5.3.6.1.3 Vertical Drop Structures

Chow (1959) used experimental data to determine hydraulic jump conditions at vertical drop structures. The aerated free-falling nappe in a vertical check drop structure will reverse the curvature and turn smoothly into supercritical flow on the apron, which may form a hydraulic jump downstream. Based on the relationships developed by Chow, the length of the hydraulic jump can be determined. A good approximation of the hydraulic jump length is six times the sequent depth (UDFCD, 1990). The reader is referred to Chow for a more detailed presentation.

5.4 General Open Channel Design Criteria

5.4.1 Introduction

In general, the following criteria should be used for open channel design:

1. Trapezoidal cross sections are preferred and triangular shapes should be avoided.

2. Channel side slopes shall be stable throughout the entire length and side slope shall depend on the channel material. A maximum of 4H:1V is recommended for vegetation and 2H:1V for riprap, unless otherwise justified by calculations.

3. If relocation of a stream channel is unavoidable, the cross-sectional shape, meander, pattern, roughness, sediment transport, and slope should generally conform to the existing conditions insofar as practicable, after giving consideration to increased flows from urbanization. Energy dissipation may be necessary.

4. Streambank stabilization should be provided, when appropriate, as a result of any stream disturbance such as encroachment and should include both upstream and downstream banks as well as the local site.

5. A low flow or trickle channel is recommended for all grass-lined channels.

6. Low flow sections shall be used in the design of channels with large cross sections.

7. New channels with bottom widths greater than 10 feet shall be designed with a minimum bottom cross slope of 12 to 1 to discourage meandering.

8. Superelevation of the water surface at horizontal curves shall be accounted for by increased freeboard.

9. Computation of water surface profiles shall be presented for all open channels utilizing standard backwater methods, taking into consideration losses due to changes in velocity, drops, and obstructions. The hydraulic and energy grade lines shall also be shown on preliminary and construction drawings. When potential erosion and flood capacity problems are identified, modifications to the channel may be necessary (Tulsa 1993).

5.4.2 Channel Transitions

The following criteria should be considered at channel transitions:

1. Transition to channel sections should be smooth and gradual.

2. A straight line connecting flow lines at the two ends of the transition should not make an angle greater than 12.5 degrees with the axis of the main channel.

3. Transition sections should be designed to provide a gradual transition to avoid turbulence and eddies.

4. Energy losses in transitions should be accounted for as part of the water surface profile calculations.

5. Scour downstream from rigid-to-natural and steep-to-mild slope transition sections should be accounted for through velocity-slowing and energy-dissipating devices.

5.4.3 Return Period Design Criteria

Open channels shall be sized to handle the 100-year storm. When comprising the minor drainage system, open channels shall be sized to handle the 5-year storm in residential areas and the 10-year storm in downtown areas and industrial/commercial developments. For major drainage systems, open channels shall be sized to handle the 100-year storm.

5.4.3.1 Approximate Flood Limits Determination

Refer to Section 1.5.6 Flood Corridor Management for guidance on policy requirements for flood limit determination. For cases when the design engineer can demonstrate that a complete backwater analysis is unwarranted, approximate methods may be used.

A generally accepted method for approximating the 100-year flood elevation is outlined as follows:

1. Divide the stream or tributary into reaches that may be approximated using average slopes, cross sections, and roughness coefficients for each reach.

2. Estimate the 100-year peak discharge for each reach using the appropriate hydrologic method.

3. Compute normal depth for uniform flow in each reach using Manning's equation for the reach characteristics from Step 1 and peak discharge from Step 2.

4. Use the normal depths computed in Step 3 to approximate the 100-year flood elevation in each reach. The 100-year flood elevation is then used to delineate the floodplain.

This approximate method is based on several assumptions, including, but not limited to, the following:

1. A channel reach is accurately approximated by average characteristics throughout its length.

2. The cross-sectional geometry, including area, wetted perimeter, and hydraulic radius, of a reach may be approximated using typical geometric properties that can be used in Manning's equation to solve for normal depth.

3. Uniform flow can be established and backwater effects are negligible between reaches.

4. Expansion and contraction effects are negligible.

As indicated, the approximate method is based on a number of restrictive assumptions that may limit the accuracy of the approximation and applicability of the method. The engineer is responsible for appropriate application of this method to get reliable results.

Where a complete backwater analysis is warranted, the engineer is encouraged to use the USACE HEC-RAS model.

5.4.4 Velocity Limitations

Sediment transport requirements must be considered for conditions of flow below the design frequency, minimum channel flow velocity for the 2-year storm shall be 2.0 feet per second. A low flow channel component within a larger channel can reduce maintenance by improving sediment transport in the channel. Channel flow velocities shall be non erosive for the 2-, 10- and 100-year storms. Trickle channel design flow rate shall be 1% of the major storm flow rate and shall be non erosive. Grade control structures, streambank protection, and construction and maintenance considerations shall be determined during design.

The final design of artificial open channels should be consistent with the velocity limitations for the selected channel lining. Maximum velocity values for selected lining categories are presented in Table 5-3. Velocity limitations for established vegetative linings are reported in Table 5-4.

Table 5-3 Maximum Design Velocities for Comparing Lining Materials
(all values in feet per second)

Material	Clear Water	Water with Colloidal Silt	Water with Non-colloidal Silt, Sand or Gravel
Fine Sand (colloidal)	1.5	2.5	1.5
Sand Loam (noncolloidal)	1.45	2.5	2.0
Silt Loam (noncolloidal)	2.0	3.0	2.0
Alluvial Silt (noncolloidal)	2.0	3.5	2.0
Alluvial Silt (colloidal)	3.75	5.0	3.0
Firm Loam	2.5	3.5	2.25
Fine Gravel	2.5	5.0	3.75
Stiff Clay (very colloidal)	3.75	5.0	3.0
Graded Loam to Cobbles(noncol)	3.75	5.0	5.0
Graded Silt to Cobbles (colloidal)	3.75	5.0	3.0
Coarse Gravel	4.0	6.0	6.5
Cobbles and Shingles	5.0	5.5	6.5
Shales and Hard Pans	6.0	6.0	5.0

Source: Fortier and Scoby, 1926.

Table 5-4 Maximum Velocities For Vegetative Channel Linings

Vegetation Type	Slope Range (%)[1]	Maximum Velocity[2] (ft/s)	
		Erosion Resistant Soils	Easily Eroded Soils
Bermuda grass	0-5	8	6
	5-10	7	5
	>10	6	4
Kentucky bluegrass	0-5	7	5
Buffalo grass	5-10	6	4
	>10	5	3
Grass mixture	0-5[1]	5	4
	5-10	4	3
Kudzu, alfalfa	0-5[3]	3.5	2.5
Annuals	0-5	3.5	2.5
Sod		4.0	4.0
Lapped sod		5.5	5.5

Source: USDA, TP-61, 1954.
[1] Do not use on slopes steeper than 10 percent except for side-slope in combination channel.
[2] Use velocities exceeding 5 ft/s only where good stands can be established and maintained.
[3] Do not use on slopes steeper than 5 percent except for side-slope in combination channel.

5.4.5 Grade Control Structures

Grade control structures are used to prevent streambed degradation. This is accomplished in two ways. First, the structures provide local base levels that prevent bed erosion and subsequent slope increases. Second, some structures provide controlled dissipation of energy between upstream and downstream sides of the structure. Structure choice depends on existing or anticipated erosion, cost, and environmental objectives. Design guidance for grade control structures is provided in Section 5.10. Additional guidance can be found in the National Engineering Handbook, Section 11, Drop Spillways and Section 14, Chute Spillways.

Examples of grade control structures include:

Sills or Check Structures - A sill is a structure that extends across a channel and has a surface that is flush with the channel invert or that extends a foot or two above the invert. Because sills are intended to prevent scouring of the bed, they should be placed close enough together to control the energy grade line and prevent scour between structures. Sills may be notched at the lowest flow point location to concentrate low flows to improve aquatic habitat and water quality or for aesthetic reasons. In highly visible locations, sills extending above the channel invert may be constructed of, or faced with, materials such as natural stone that create an attractive appearance. Sills may also be modified to allow for passage of boats or fish, if desired.

Drop Structures, Chutes, and Flumes - Drop structures provide for a vertical drop in the channel invert between the upstream and downstream sides, whereas chutes and flumes provide for a more gradual change in invert elevation. Because of the high energies that must be dissipated, pre-formed scour holes or plunge pools are required below these structures.

The design of hydraulic structures, such as drop structures, must consider safety of the general public, especially when multiple uses are allowed (i.e., boating and fishing). There are certain hazards that can be associated with drop structures, such as the “reverse roller” phenomenon which can trap an individual and result in drowning. As a result, it may be necessary to sign locations accessible by the public to warn of the danger associated with the hydraulic structure.

5.4.6 Streambank Protection

Streambanks subject to erosion are protected by stabilizing eroding soils, planting vegetation, covering the banks with various materials, or building structures to deflect stream currents away from the bank. Placement and type of bank protection vary, depending on the cause of erosion, environmental objectives, and cost. Section 5-11 identifies different streambank protection measures that are recommended for bank stability.

5.4.7 Construction And Maintenance Considerations

Open channels shall be maintained by the developer or a property-owners’ association unless an alternative ownership/maintenance arrangement has been approved by the City, Planning Commission and the City Council.

An important step in the design process involves identifying whether special provisions are warranted to properly construct or maintain proposed facilities.

Open channels can lose hydraulic capacity without adequate maintenance. Maintenance may include repairing erosion damage, mowing grass, cutting brush, and removing sediment or debris. Brush, sediment, or debris can reduce design capacity and can harm or kill vegetative linings, thus creating the potential for erosion damage during large storm events. Maintenance of vegetation should include mowing, the appropriate application of fertilizer, irrigation during dry periods, and reseeding or resodding to restore the viability of damaged areas. Extra sizing may be used to account for future vegetation growth.

Implementation of a successful maintenance program is directly related to the accessibility of the channel system and the easements necessary for maintenance activities. The easement cross-section must accommodate the depth and width of flow from the 100-year storm. The width must also be designed to allow for access of maintenance equipment.

5.5 Natural Channel Design Criteria

Natural channels in the Waverly area are sometimes found to have erodible banks and bottoms which tend to result in steep vertical banks. Other channels may have mild slopes and are reasonably stable. If natural channels are to be used in urbanized and to-be-urbanized areas to convey stormwater runoff, it can be assumed that there will be increased flow peaks and volumes that will result in increased channel erosion. As such, an hydraulic analysis during the planning and design phase is necessary to address the potential for erosion, and will usually result in the need for some stabilization measures.

The following criteria and analysis techniques are recommended for natural channel evaluation and stabilization:

- The channel and over-bank areas must have adequate capacity for the 100-year post-development storm runoff.
- The water surface profiles must be defined and delineated so that the 100-year floodplain can be identified and managed. Plan and profile drawings should be prepared of the FEMA floodplain, and allowances should be made for future bridges or culverts.
- Filling of the floodplain is subject to the restriction of floodplain regulations.
- Manning's n roughness factors representative of maintained channel conditions should be used. Table 5-5 provides representative values of the roughness factor in natural streams.
- Erosion control structures such as drop structures and grade control checks should be provided as necessary to control flow velocities and channel erosion.

Table 5-5 Uniform Flow Values Of Roughness Coefficient - n

Type Of Channel And Description	Minimum	Normal	Maximum
Minor streams (top width at flood stage < 100 ft)			
a. Streams on Plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and some stones	0.035	0.045	0.050
5. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
6. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
Floodplains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated area			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees	0.040	0.060	0.080
3. Medium to dense brush	0.070	0.100	0.160
d. Trees			
1. Dense willows, straight	0.110	0.150	0.200
2. Cleared land, tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
Major Streams (top width at flood stage > 100 ft).			
a. Regular section with no boulders or brush	0.025	.....	0.060
b. Irregular and rough section	0.035	.....	0.100

Natural channels should be left in as near a natural condition as feasible. However, with most natural channels, grade control structures will need to be constructed at regular intervals to limit channel degradation and to maintain what is expected to be the final stable longitudinal slope after full urbanization of the watershed. In addition, the engineer is reminded that modification of the channel may require a US Army Corps of Engineers Section 404 permit.

Use of natural channels in the drainage system requires thoughtful planning, as they offer multiple-use opportunities. Certain criteria pertaining to artificial channels, such as freeboard depth and curvature, may not apply to natural channels in order to meet some of the multi-purpose objectives. Special consideration shall be given to transitions from “hard”to “soft” stabilization materials.

5.6 Grassed-Lined Channel Design Criteria

Grass-lined channels are encouraged when designing artificial channels. Advantages include: channel storage, lower velocities, provision of wildlife habitat, and aesthetic and recreational values. Design considerations include velocity, longitudinal slopes, roughness coefficients, depth, freeboard, curvature, cross-section shape, and channel lining material (vegetation and trickle channel considerations).

5.6.1 Design Velocity and Froude Number

It is recommended that the maximum normal depth velocity for grass-lined channels during the major design storm (i.e., 100-year) not exceed 7.0 feet per second for erosion-resistant soils and 5.0 per second for easily eroded soils. These velocity limitations assume a well-maintained, good stand of grass. The Froude number should not exceed 0.8 for erosion-resistant soils and 0.6 for easily eroded soils (UDFCD, 1990).

5.6.2 Longitudinal Slopes

Grass-lined channels should have longitudinal slopes of less than 1 percent, but will ultimately be dictated by velocity and Froude number considerations. In locations where the natural topography is steeper than desirable, drop structures should be implemented.

5.6.3 Roughness Coefficients

Table 5-6 provides guidance for roughness coefficients for grass-lined channels. The roughness coefficient for grass-lined channels depends on length and type of vegetation and flow depth. Roughness coefficients are smaller for higher flow depths due to the fact that at higher depths the grass will lay down to form a smoother bottom surface.

Table 5-6 Manning's Roughness Coefficients for Grass-Lined Channels - n
n - Value With Flow Depth Ranges

Grass Type	Length	0.0-1.5 ft	>3.0 ft
Bermuda grass, Buffalo grass,	Mowed to 2 inches	0.035	0.030
Kentucky bluegrass	Length 4 to 6 inches	0.040	0.030
Good stand any grass	Length of 12 inches	0.070	0.035
	Length of 24 inches	0.100	0.035
Fair stand any grass	Length of 12 inches	0.060	0.035
	Length of 24 inches	0.070	0.035

Source: UDFCD, 1990.

5.6.4 Freeboard

A minimum freeboard of 1 foot should be provided between the water surface and top of bank or the elevation of the lowest opening of adjacent structures. In some areas, localized overflow may be desirable for additional ponding/storage benefits.

Superelevation of the water surface should be determined at horizontal curves. An approximation of the superelevation can be made from the following equation:

$$\mathbf{h = V^2 Tw/gr_c} \qquad \mathbf{(5.4)}$$

Where:
h = superelevation (ft)
V = velocity (ft/s)
T_w = top width of channel (ft)
g = acceleration due to gravity (32.2 ft/sec^2)
r_c = centerline radius of curvature (ft)

5.6.5 Curvature

It is recommended that the centerline curves of channels have a radius of two to three times the design flow top width or at least 100 feet.

5.6.6 Cross-sections

Channel shape may be almost any type suitable to the site-specific conditions, and can be designed to meet multi-purpose uses, such as recreational needs and wildlife habitat. However, limitations to the design include the following:

- Side slopes should be 4 (horizontal) to 1 (vertical) or flatter. Slopes as steep as 3H:1V may be considered in areas where development already exists and there are right-of-way limitations.
- The bottom width should be designed to accommodate the hydraulic capacity of the cross-section, recognizing the limitations on velocity and depth. Width must be adequate to allow necessary maintenance (ASCE, 1992).
- Maintenance/access roads should be provided for along all major drainageways.
- Trickle channels or underdrain pipes should be provided on grass-lined channels to minimize erosion. As an alternative, low flow channels can be provided (low flow channels are particularly applicable for larger conveyances). Figure 5-3 shows typical cross-sections suitable for grass-lined channels. Trickle channels should be designed to carry base flow originating from lawn watering, low intensity rainfall events, and snow melt.

5.6.7 Grass Species

Seed mixes for the channel lining should be selected to be sturdy, easy to establish, and able to spread and develop a strong turf layer after establishment. A thick root structure is necessary to control weed growth and erosion. Seed mixes should meet all state and local seed regulations. For additional guidance on seed mixes and seed rates the reader is referred to the local Natural Resources Conservation Service (NRCS) branch office and the LPSNRD. Table 5-7 provides suggested seed mixtures.

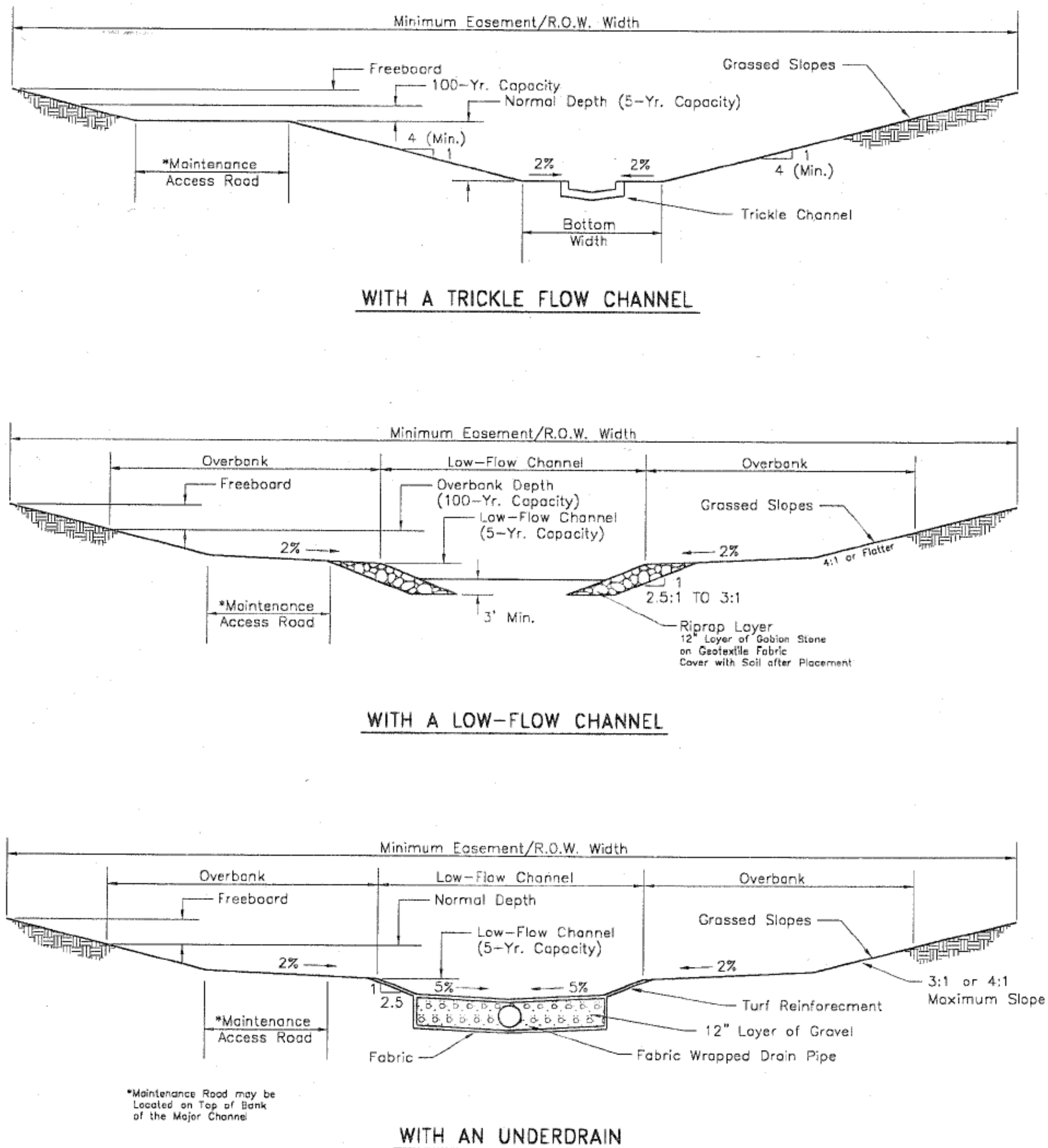


Figure 5-3 Typical Grass-Lined Channel Details

Source: UDFCD, 1990

Seeding Dates	Suggested Mixtures	Lbs. to Furnish 60 PLS/sq. ft. Per Acre	Loams, Clayloams, and Clays						Sands & Loamy Sand				Remarks
			A	B	C	D	E	F	A	D	E	F	
	Warm Season Dominant:												
October 1 - June 15	Big bluestem	4.0	X	0	-	-	X	-	X	-	X	-	For sands and loamy sands substitute sand bluestem for big bluestem, 1.8 lbs. of prairie sandreed for sideoats grama and 0.2 lb. PLS sand lovegrass for tall fescue.
	Sideoats grama	2.9	X	0	-	-	X	-	X	-	X	-	
	Switchgrass	0.7	X	0	-	-	X	-	X	-	X	-	
	Indiangrass	3.6	X	0	-	-	X	-	X	-	X	-	
	Tall fescue	2.1	X	0	-	-	X	-	X	-	X	-	
	Buffalograss (burs)	38.0	-	-	-	X	X	X	-	X	-	X	
	Blue grama	0.6	-	-	-	X	X	X	-	X	-	X	
	Buffalograss (burs)	32.3	0	-	-	X	X	X	-	X	-	X	
	Blue grama	0.3	0	-	-	X	X	X	-	X	-	X	
	Sideoats grama	2.7	0	-	-	X	X	X	-	X	-	X	
	Switchgrass	3.8	-	X	X	-	-	-	X	-	X	-	Wet areas
	Reed canarygrass	2.0	-	X	X	-	-	-	X	-	X	-	
	Cool-Season Dominant:												
August 15 - April 30	Smooth brome	14.4	X	0	-	-	X	-	X	-	0	-	Add 10 lbs. of western wheatgrass to replace 7.2 lbs. of brome or 3.6 lbs. of tall fescue.
	Switchgrass	1.7	X	0	-	-	X	-	X	-	0	-	
	Tall fescue	7.3	X	X	0	-	X	-	0	-	0	-	
	Switchgrass	2.4	X	X	0	-	X	-	0	-	0	-	
	Perennial ryegrass	5.2	-	-	-	X	-	X	-	X	-	X	Less than 5 years
	Alfalfa	10.8	-	-	-	X	-	X	-	X	-	X	
	Red clover	3.9	-	-	-	X	-	X	-	X	-	X	Substitute 0.6 lbs. PLS sand lovegrass to replace tall fescue for sands and loamy sands.
	Birdsfoot trefoil	1.9	-	-	-	X	-	X	-	X	-	X	
	Tall fescue	6.3	-	-	-	X	-	X	-	X	-	X	

KEY:
X Best
0 Fair
- Poor

A Dam, Diversion, Dike
B Channels
C Shoreline & Low Areas
D Heavy Traffic & Recreation
E Roadside
F Residential & Development Sites

Table 5-7 Suggested Seed Mixtures

Source: LPSNRD, 1994

5.7 Wetland Bottom Channel Design Criteria

Wetland bottom channels should be considered as the design approach in circumstances where existing wetland areas are affected or natural channels are modified. In fact, the USACE may mandate the use of wetland bottom vegetation in the channel design as mitigation for wetland damages elsewhere. Wetland bottom channels are in essence grass-lined channels, with the exception that wetland-type vegetation is encouraged in the channel bottom (this is usually accomplished by removing the trickle channel and slowing velocities). Increased water quality and habitat benefits are realized with the implementation of wetland bottom channels; however, they can become difficult to maintain (i.e., mow) and may be potential mosquito breeding areas.

Due to the abundant vegetation associated with wetland channels, flow conveyance will decrease and channel bottom agradation will increase. Consequently, channel cross-sections and right-of-way requirements will be larger than those associated with grass-lined channels.

The recommended procedures for wetland bottom channel design are quite similar to the design of grass-lined channels. For wetland channel design, the engineer must accommodate two flow roughness conditions to account for channel stability during a “new channel” condition and channel capacity during a “mature channel” condition.

5.7.1 Design Velocity

It is recommended that the maximum normal depth velocity for wetland bottom “new channel” conditions during the major design storm (i.e., 100-year) not exceed 7.0 feet per second for erosion resistant soils and 5.0 per second for easily eroded soils. The Froude number should not exceed 0.8 for erosion resistant soils and 0.6 for easily eroded soils under “new channel” conditions.

5.7.2 Longitudinal Slopes

The longitudinal slopes of wetland bottom channels should be dictated by velocity and Froude number considerations under “new channel” conditions.

5.7.3 Roughness Coefficients

As previously mentioned, wetland bottom channel design requires consideration of two roughness coefficient scenarios. To determine longitudinal slope and initial cross-section area, a “new channel” coefficient should be used. To determine design water surface, and final cross-section area, a “mature channel” coefficient should be used. The “mature channel” coefficient will likely be a composite coefficient. The following provides guidance for roughness coefficients for wetland bottom channels:

- New channel condition, use n = 0.030
- Mature channel condition, calculate a composite based on the following relation and Figure 5-4 (UDFCD 1990):

$$\mathbf{n_c = (n_0 p_0 + n_w p_w)/(p_0 + p_w)} \tag{5.5}$$

Where: n_c = composite Manning’s n
n_0 = Manning’s n for areas above wetland (refer to Table 5.5)
n_w = Manning’s n for the wetland area (see Figure 5-4)
p_0 = wetted perimeter of channel above wetland area
p_w = wetted perimeter of wetland area (approximated as bottom width plus 10 feet)

5.7.4 Design Depth

As a preliminary design criteria, the maximum design depth of flow for the major storm runoff should not exceed 5.0 feet in areas of the channel cross-section outside the low flow channel area. Scour potential should also be analyzed when determining the design depth.

5.7.5 Freeboard

A minimum freeboard of 1 foot should be provided between the water surface and top of bank or the elevation of the lowest opening of adjacent structures. Freeboard should be determined based on the major storm water surface elevation under “mature channel” conditions.

5.7.6 Curvature

It is recommended that the centerline curves of channels have a radius of two to three times the design flow top width or at least 100 feet.

5.7.7 Cross-sections

Channel shape may be almost any type suitable to the site-specific conditions, and can be designed to meet multi-purpose uses, such as recreational needs and wildlife habitat. However, limitations to the design include the following:

- Side slopes should be 4 (horizontal) to 1 (vertical) or flatter.
- It is recommended that the low flow channel be designed to convey the minor storm (i.e., 5- or 10-year storm) runoff.
- The bottom width should be designed to accommodate the hydraulic capacity of the cross-section, recognizing the limitations on velocity and depth. It is recommended that bottom widths not be less than 8.0 feet.
- Side slope banks of low flow channels should be lined with riprap or turf reinforcement material (at 2.5H:1V or 3H:1V) to minimize erosion. Figure 5-5 shows a typical cross-section suitable for wetland bottom channels.

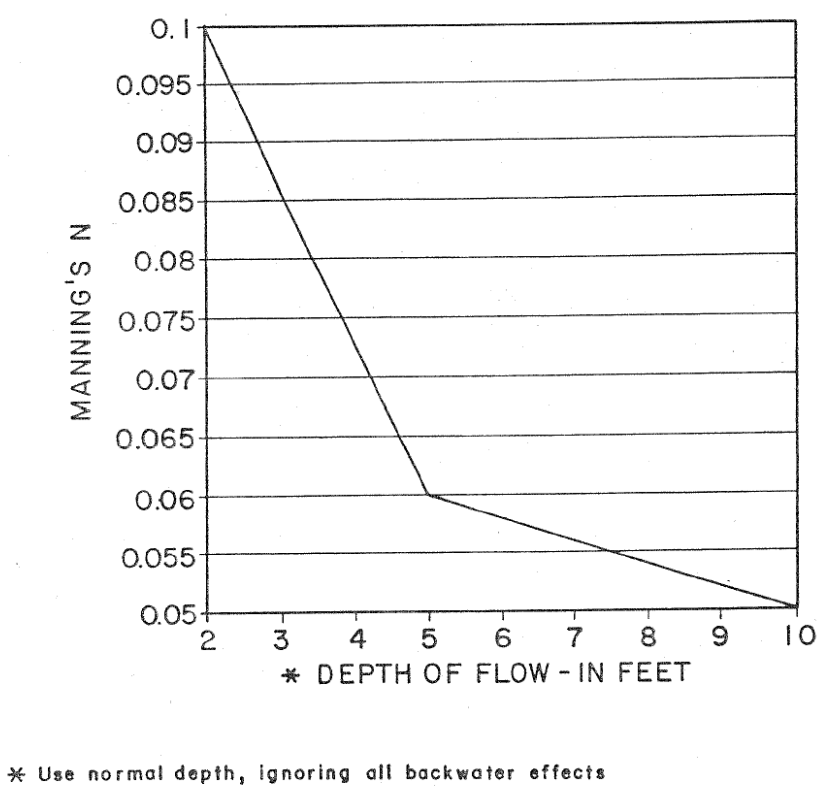


Figure 5-4 Depth of Flow vs. Manning’s n for Wetland Bottom

Source: UDFCD, 1990

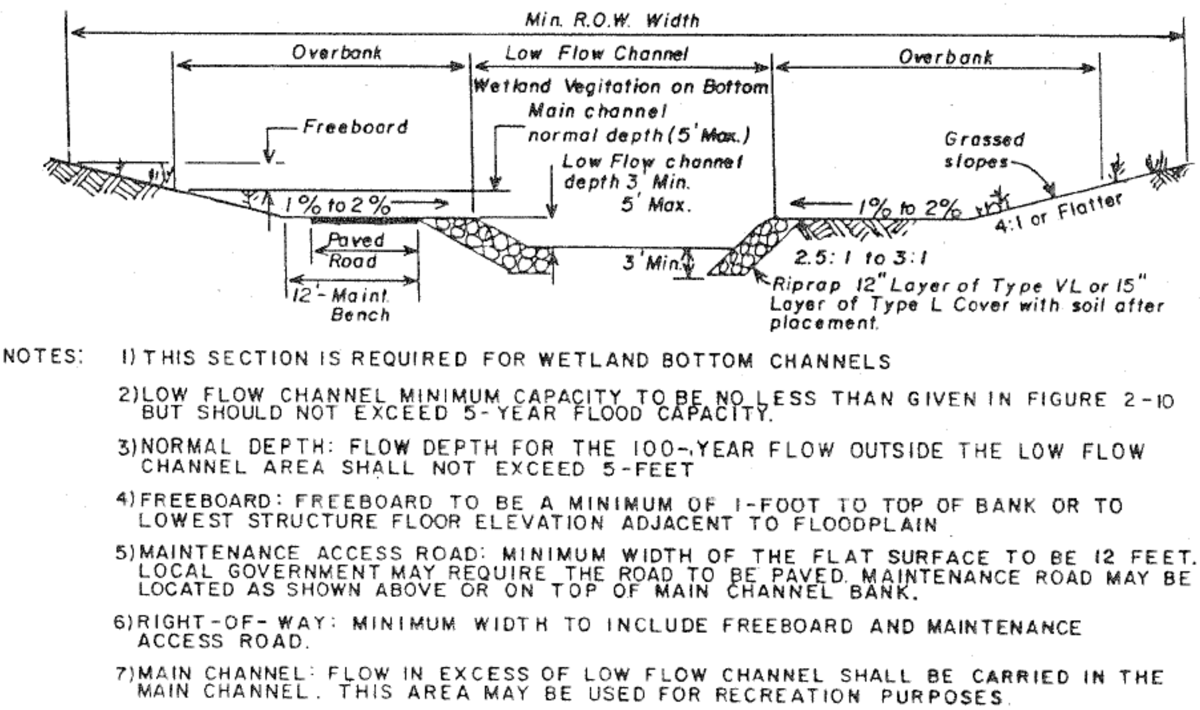


Figure 5-5 Typical Cross-Section of Wetland Bottom Channel

Source: UDFCD, 1990

5.8 Rock-Lined Channel Design

Rock-lined channels constructed from riprap, grouted riprap, or wire-enclosed rock can be cost effective at controlling erosion along short channel reaches. These rock-lined channels might be appropriate in the following scenarios:

- Where major flows generate velocities in excess of allowable non-eroding values.
- Where right-of-way restrictions necessitate channel side slopes to be steeper than 3H:1V.
- Where rapid changes in channel geometry occur such as at channel bends and transitions.
- For low flow channels.

For hydraulic calculations, the following equation can be used for Manning's n values for riprap (this equation does not apply to situations involving very shallow flow where the roughness coefficient will be greater):

$$\mathbf{n = 0.0395\ (d_{50})^{1/6}} \qquad \textbf{(5.6)}$$

Where: n = Manning's roughness coefficient for stone riprap
d_{50} = diameter of stone for which 50 percent, by weight, of the gradation is finer (ft)

A Manning's n value of 0.035 can be used for wire-enclosed rock and a value of 0.023 to 0.030 can be used for grouted riprap.

Riprap requirements for a stable channel lining can be based on the following relationship (UDFCD 1984):

$$\frac{V\,S^{0.17}}{d_{50}^{0.5}\,(S_s-1)^{0.66}} = 4.5 \tag{5.6}$$

Where: V = mean channel velocity (ft/s)
S = longitudinal channel slope (ft/ft)
S_s = specific gravity of rock (minimum S_s = 2.5)
d_{50} = diameter of stone for which 50 percent, by weight, of the gradation is finer (ft)

Rock sizing requirements are based on rock having a specific gravity of at least 2.5. Gradation and classification for riprap are shown in Tables 5-8 and 5-9.

Table 5-8 Rock Riprap Gradation Limits

Stone Size Range (ft.)	Stone Weight Range (lb)	Percent of Gradation Smaller Than
1.5 d_{50} to 1.7 d_{50}	3.0 W_{50} to 5.0 W_{50}	100
1.2 d_{50} to 1.4 d_{50}	2.0 W_{50} to 2.75 W_{50}	85
1.0 d_{50} to 1.15 d_{50}	1.0 W_{50} to 1.5 W_{50}	50
0.4 d_{50} to 0.6 d_{50}	0.1 W_{50} to 0.2 W_{50}	15

Table 5-9 Riprap Gradation Classes

Riprap Class	Rock Size[1] (ft.)	Rock Size[2] (lbs.)	Percent of Riprap Smaller Than
Facing	1.30	200	100
	0.95	75	50
	0.40	5	10
Light	1.80	500	100
	1.30	200	50
	0.40	5	10
1/4 ton	2.25	1000	100
	1.80	500	50
	0.95	75	10
½ ton	2.85	2000	100
	2.25	1000	50
	1.80	500	5
1 ton	3.60	4000	100
	2.85	2000	50
	2.25	1000	5
2 ton	4.50	8000	100
	3.60	4000	50
	2.85	2000	5

[1] Assuming a specific gravity of 2.65.
[2] Based on AASHTO gradations.

Rock-lined side slopes steeper than 2H:1V are considered unacceptable because of stability, safety, and maintenance considerations. Proper bedding is required along both the side slopes and channel bottom. The riprap blanket thickness should be at least 1.75 times d50 and should extend up the side slopes at least one foot above the design water surface. The upstream and downstream flanks require special treatment to prevent undermining. Details on these considerations are presented in section 5.11.2.

5.9 Concrete Channels

Concrete linings are used where smoothness offers a higher capacity for a given cross-sectional area. When properly designed, rigid linings may be appropriate where the channel width is restricted. Use of concrete linings is not encouraged due to the lack of water quality benefits as well as the propensity for higher velocities, which create the potential for scour at channel lining transitions.

5.10 Grade Control Structures

The most common use of channel drop structures or grade control structures is to control the longitudinal slope of grass-lined channels to keep design velocities within acceptable limits. Baffle chute drops, grouted sloping boulder drops, and vertical riprap drops are all examples of possible structures to use. The focus of this section will be on vertical riprap drops. The guidance presented in this section for design of vertical riprap drops was obtained from the City of Tulsa Stormwater Management Manual (1993). Other design approaches exist which are also appropriate for vertical drops and other types of grade control structures. For example, the reader is referred to the SCS National Engineering Handbook for more detail on chute and sloping boulder drops. Also, Chapter 7 of this Manual provides guidance for more substantial energy dissipator structures used for larger flows and channel transitions.

The design of hydraulic structures, such as drop structures, must consider safety of the general public, especially when multiple uses are allowed (i.e., boating and fishing). There are certain hazards that can be associated with drop structures, such as the “reverse roller” phenomenon which can trap an individual and result in drowning. As a result, it may be necessary to sign locations accessible by the public to warn of the danger associated with the hydraulic structure and should be evaluated on a project by project basis.

5.10.1 Vertical Riprap Drops

An example of a vertical riprap drop is presented in Figure 5-6. The design of the drop is based upon the height of the drop and the normal depth and velocity of the approach and exit channels. The channel should be prismatic from the upstream channel through the drop to the downstream channel. The maximum recommended side slope for the stilling basin area is 4:1. Flatter side slopes are encouraged when available right-of-way exists. When riprap is grouted, the stilling basin side slopes can be steepened to 3:1. The riprap should extend up the side slopes to a depth 1 foot above the normal depth projected upstream from the downstream channel. For safety considerations, the maximum fall recommended at any one drop structure is 4 feet from the upper channel bottom to the lower channel bottom, excluding the trickle channel. Table 5-10 is a design chart to be used in conjunction with Figure 5-6 for sizing of the riprap basin and retaining wall structure. Rock-filled wire baskets may be a likely alternative to be considered by the designer for some structures.

5.10.1.1 Approach Depth

The upstream and downstream channels will normally be grass-lined trapezoidal channels with trickle channels to convey normal low flow water. The maximum normal depth, y_n, is 5 feet and the maximum normal velocity, v_n, is 7 ft/s for erosion-resistant soils and 5 ft/s for easily eroded soils.

5.10.1.2 Trickle Channel

The trickle channel (shown as a concrete channel in Figure 5-6) ends at the upstream end of the upstream riprap apron. A combination cutoff wall and foundation wall is provided to give the end of the trickle channel additional support. The water is allowed to flow across the upstream apron and over the vertical wall. The trickle channel is ended at the upstream end of the apron to minimize the concentration of flows.

5.10.1.3 Approach Apron

A 10-foot long riprap apron (d_{50} = 12 inches is recommended) is provided upstream of the cutoff wall to protect against the increasing velocities and turbulence which result as the water approaches the vertical drop. Grouted riprap can also be used for the approach apron.

5.10.1.4 Crest Wall

The vertical wall should have the same trapezoidal shape as the approach channel. The wall distributes the flow evenly over the entire width of the drop structure, which minimizes flow concentrations that could adversely affect the riprap basin. The trickle flows pass through the wall via a series of notches in order to prevent ponding (see Figure 5-6).

The wall must be designed as a structural retaining wall, with the top of the wall above the upstream channel bottom. This is done to create a higher water surface elevation upstream, thereby reducing the draw-down effects normally caused by a sudden drop. The distance, P, that the top of the wall should be above the upstream channel, can be determined from Table 5-10 or from a backwater analysis.

5.10.1.5 Stilling Basin

The riprap stilling basin is designed to force the hydraulic jump to occur within the basin, and is designed for minimal scour. The floor of the basin is depressed an amount, B, below the downstream channel bottom, excluding the trickle channel. This is done to create a deeper downstream sequent depth which helps keep the hydraulic jump in the basin. This arrangement will cause ponding in the basin; however, a trickle channel can relieve all or some of the ponding.

The riprap basin can be sized using Table 5-10. To use the table, determine the required height of the drop, C, the normal velocity of the approach, v_n and the upstream and downstream normal depths, y_n and y_2, respectively. Both upstream and downstream channels must have the same geometry and y_n and y_2 must be equal to use Table 5-10. Select the appropriate riprap classification based on the row with the correct C, v_n, y_n,, and y_2. The riprap should be placed on bedding and filter fabric and should extend up the channel side slopes a distance y_2 + 1 foot as projected from the downstream channel. The basin side slopes should be the same as those in the downstream channel (i.e., 4:1 or flatter).

When riprap is grouted to within approximately 4 inches of the riprap surface, then the rock size requirement can be reduced by one size from that specified in Table 5-10. However, if the grout has been placed such that much of the rock surface is smooth, a larger basin than specified in Table 5-10 would be required.

5.10.1.6 Exit Depth

The downstream channel design should be the same as the upstream channel, including a trickle channel. For concrete trickle channels, a cutoff wall similar to the one used for the upstream trickle channel should be used. This may also serve to control seepage and piping.

5.10.1.7 Design Example

The following example demonstrates the use of Table 5-10 and Figure 5-6 for the sizing of riprap basin dimensions and selection of riprap.

Given a Q_{100} = 400 cfs and the following upstream and downstream channel dimensions:

- bottom width = 8 ft
- longitudinal slope = 0.004 ft/ft
- side slopes = 4:1
- y_c = 2.8 ft
- y_n = 4 ft

Open Channels

- v_n = 4.2 ft/s
- channel drop, C = 3 ft

From Table 5-10, for C = 3.0 ft, v_n = 4.2 ft/s (assume v_n = 5 ft/s on table), and y_n and y_2 = 4.0 ft, the following dimensions can be determined:

- P = 0.1 ft
- B = 1.0 ft
- A = 2.5 ft
- LB = 20 ft
- D = 5 ft
- E = 4 ft
- Riprap = d_{50} of 18 inches

Table 5-10 Vertical Riprap Channel Drop Design Chart

C (ft)	v_n (ft/s)	y_n & y_2 (ft)	P (ft)	B (ft)	A (ft)	L_B (ft)	D (ft)	E (ft)	Riprap d_{50} (in)
2	5	4	0.1	0.6	2.0	20	4	3	12
2	5	5	*	0.8	2.5	25	5	4	18
2	5; 7	4	0.1	0.8	2.5	20	5	4	18
2	5; 7	5	*	0.8	2.5	25	5	4	18
3	5	4	0.1	1.0	2.5	20	5	4	18
3	5	5	*	1.0	2.5	25	5	4	18
3	5; 7	4	0.1	1.0	2.5	20	5	4	18
3	5; 7	5	*	1.0	2.5	25	5	4	18
4	5	4	0.1	1.2	3.5	20	7	5	18
4	5	5	*	1.2	3.5	25	7	5	18
4	5; 7	4	0.1	1.4	3.5	20	7	6	18
4	5; 7	5	*	1.4	3.5	25	7	6	18

* See crest wall elevation chart below

Crest Wall Elevation Chart

approach bottom width (ft)	P (ft) at V_n = 5 ft/s	P (ft) at V_n = 7 ft/s
5	0.2	0.2
4	0.4	0.2
100	0.5	0.3

Notes:
1. See Figure 5-6 for definition of symbols.
2. Maximum allowable C = 4.0 ft.
3. This chart is for ordinary riprap structures only. Other types of drop structures require their own hydraulic analysis.

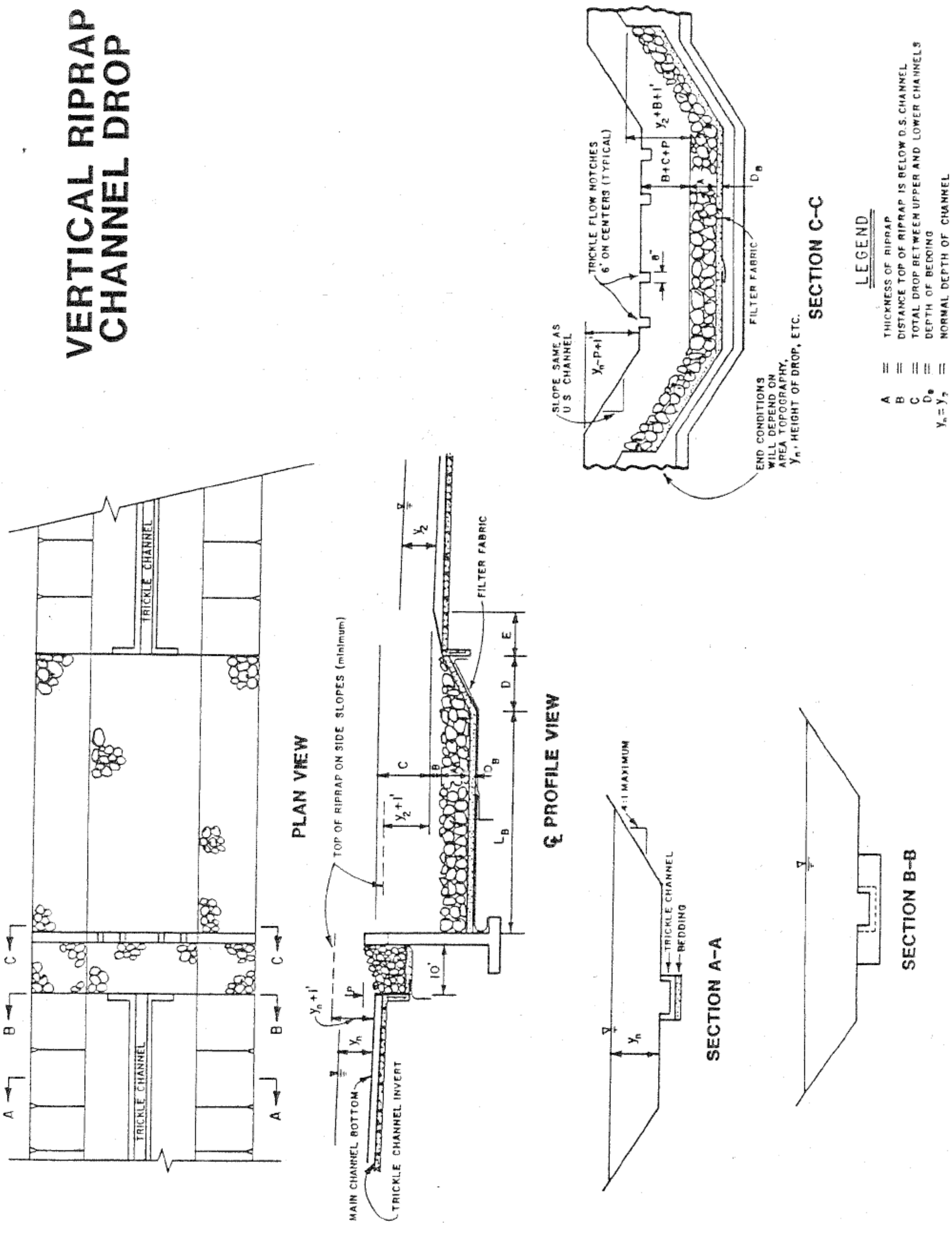


Figure 5-6 Vertical Riprap Channel Drop

Source: City of Tulsa, 1993

5.11 Stability And Bank Protection

5.11.1 Channel Stability Guidelines

The best way to avoid instability problems in urban stream channels and to maximize environmental benefits is to maintain streams in as natural a condition as possible, and when channel modification is necessary, to avoid altering channel dimensions, channel alignment, and channel slope as much as possible, except to account for impacts caused by urbanization. When channel modification is necessary, the following set of guidelines should be followed to minimize erosion problems and maximize environmental benefits.

- When channels must be enlarged, avoid streambed excavation that would significantly increase streambed slope or streambank height.
- When channel bottom widths are increased more than 25 percent, provide for a low flow channel to concentrate flows during critical low flow periods.
- Avoid channel realignment whenever feasible.

When unstable banks exist, several stabilization measures can be employed to provide the needed erosion protection and bank stability. The types of slope protection or revetment commonly used for bank stabilization include:

- turf reinforcement,
- rock and rubble riprap,
- wire-enclosed rock (gabions),
- pre-formed concrete blocks,
- grouted rock, and
- bioengineering methods
- poured-in-place concrete
- grout-filled fabric mattress

5.11.2 Rock Riprap

Placement of riprap is often used as bank or bed stabilization. Design of riprap size and thickness has been presented in numerous documents including those by Reese (1984 and 1988). Filter material is installed beneath riprap in all cases.

<u>Filter Fabric Placement</u>

To provide good performance, a properly selected cloth should be installed in accordance with manufacturer recommendations with due regard for the following precautions:

- Heavy riprap may stretch the cloth as it settles, eventually causing bursting of the fabric in tension. A 4-inch to 6-inch gravel bedding layer should be placed beneath the riprap layer for riprap gradations having d50 greater than 3.00 ft.
- The filter cloth should not extend into the channel beyond the riprap layer; rather, it should be wrapped around the toe material as illustrated in Figure 5-7.
- Adequate overlaps must be provided between individual fabric sheets.
- A sufficient number of folds should be included during placement to eliminate tension and stretching under settlement.
- Securing pins with washers are recommended at 2- to 5-ft intervals along the midpoint of the overlaps.
- Proper stone placement on the filter requires beginning at the toe and proceeding up the slope. Dropping stone from heights greater than 2 ft can rupture fabrics (greater drop heights are allowable under water).

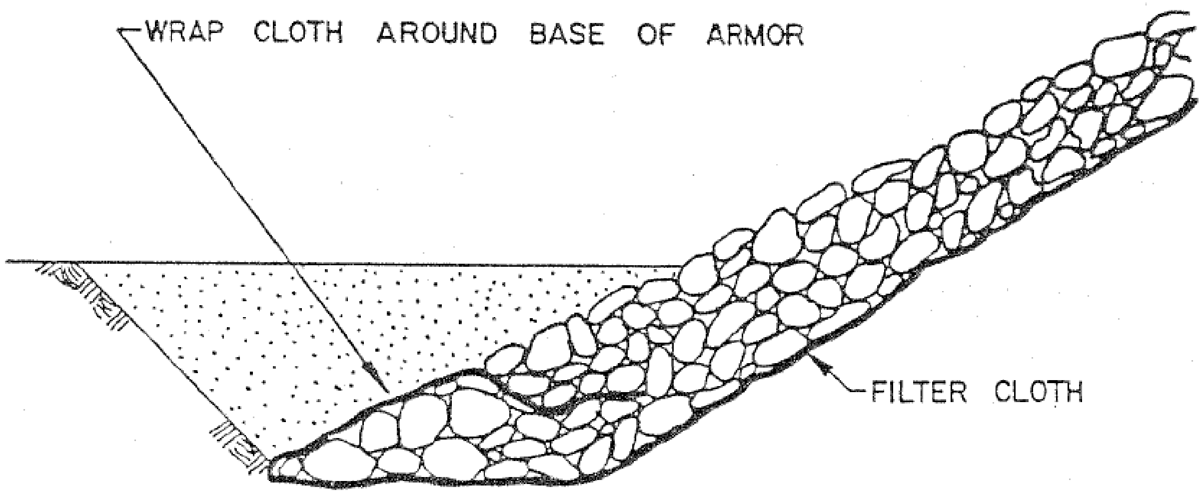


Figure 5-7 Filter Fabric Placement

5.11.2.1 Edge Treatment

The edges of riprap revetments (flanks, toe, and head) require special treatment to prevent undermining. The flanks of the revetment should be designed as illustrated in Figure 5-8. The upstream flank is illustrated in section (a) and the downstream flank is illustrated in section (b) of this figure. A more constructable flank section uses riprap rather than compacted fill.

Undermining of the revetment toe is one of the primary mechanisms of riprap failure. The toe of the riprap should be designed as illustrated in Figure 5-9. The toe material should be placed in a toe trench along the entire length of the riprap blanket.

Where a toe trench cannot be dug, the riprap blanket should terminate in a thick, stone toe at the level of the streambed (see alternate design in Figure 5-9). Care must be taken during the placement of the stone to ensure that the toe material does not mound and form a low dike; a low dike along the toe could result in flow concentration along the revetment face which could stress the revetment to failure. In addition, care must be exercised to ensure that the channel's design capability is not impaired by placement of too much riprap in a toe mound.

The size of the toe trench or the alternate stone toe is controlled by the anticipated depth of scour along the revetment. As scour occurs (and in most cases it will) the stone in the toe will launch into the eroded area. Observation of the performance of these types of rock toe designs indicates that the riprap will launch to a final slope of approximately 2:1.

The volume of rock required for the toe must be equal to or exceed one and one-half times the volume of rock required to extend the riprap blanket (at its design thickness and on a slope of 2:1) to the anticipated depth of scour. Dimensions should be based on the required volume using the thickness and depth determined by the scour evaluation. The alternate location can be used when the amount of rock required would not constrain the channel.

5.11.2.2 Construction Considerations

Construction considerations related to the construction of riprap revetments include bank slope or angle, bank preparation, and riprap placement.

The area should be prepared by first clearing all trees and debris, and grading the surface to the desired slope. In general, the graded surface should not deviate from the specified slope line by more than 6 inches. However, local depressions larger than this can be accommodated since initial placement of filter material and/or rock for the revetment will fill these depressions. In addition, any debris found buried near the edges of the revetment should be removed.

The common methods of riprap placement are hand placing; machine placing, such as from a skip, dragline, or some form of bucket; and dumping from trucks and spreading by bulldozer. Hand placement produces the best riprap revetment, but it is the most expensive method except when labor is unusually cheap. Steeper side slopes can be used with hand placed riprap than with other placing methods. Where steep slopes are unavoidable (when channel widths are constricted by existing bridge openings or other structures, and when rights-of-way are costly), hand placement should be considered.

In the machine placement method, sufficiently small increments of stone should be released as close to their final positions as practical. Rehandling or dragging operations to smooth the revetment surface tend to result in segregation and breakage of stone, and can result in an overly rough revetment surface. Stone should not be dropped from an excessive height as this may result in the same undesirable conditions. Riprap placement by dumping with spreading may be satisfactory provided the required layer thickness is achieved. Riprap placement by dumping and spreading is the least desirable method as a large amount of segregation and breakage can occur and is not recommended. In some cases, it may be economical to increase the layer thickness and stone size somewhat to offset the shortcomings of this placement method.

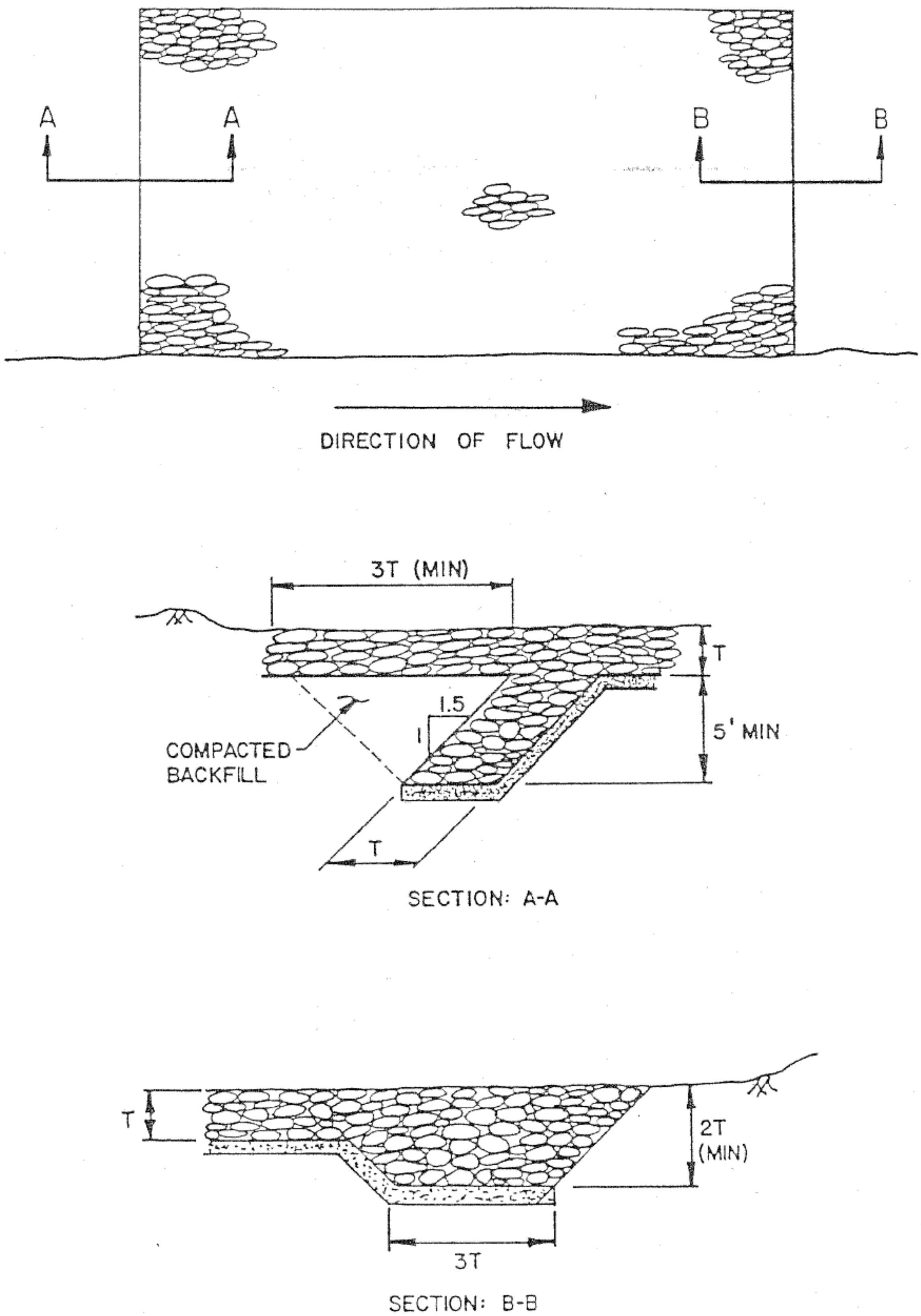


Figure 5-8 Typical Riprap Installation: Plan And Flank Details

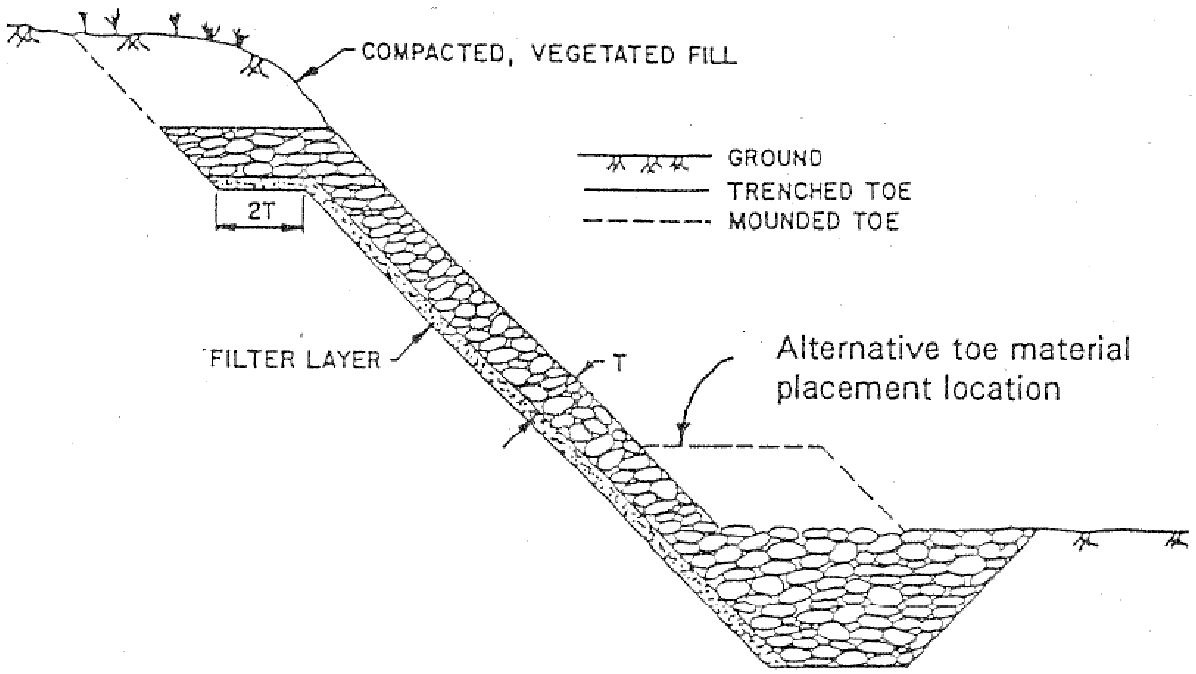


Figure 5-9 Typical Riprap Installation: End View (Bank Protection Only)

5.11.2.3 Design Procedure

The rock riprap design procedure outlined in the following sections is comprised of three primary sections: preliminary data analysis, rock sizing, and revetment detail design. The individual steps in the procedure are numbered consecutively throughout each of the sections.

Preliminary Data

Step 1 Compile all necessary field data including (channel cross section surveys, soils data, aerial photographs, history of problems at site, etc.).

Step 2 Determine design discharge.

Step 3 Develop design cross section(s). Note: The rock sizing procedures described in the following steps are designed to prevent riprap failure from particle erosion.

Step 4 Compute design water surface.

(a) When evaluating the design water surface, Manning's "n" shall be estimated. If a riprap lining is being designed for the entire channel perimeter, an estimate of the rock size may be required to determine the roughness coefficient "n".

(b) If the design section is a regular trapezoidal shape, and flow can be assumed to be uniform, use design procedures delineated in this chapter.

(c) If the design section is irregular or flow is not uniform, backwater procedures must be used to determine the design water surface.

(d) Any backwater analysis conducted must be based on conveyance weighing of flows in the main channel, right bank and left bank.

Step 5 Determine design average velocity and depth.

(a) Average velocity and depth should be determined for the design section in conjunction with the computations of step 4. In general, the average depth and velocity in the main flow channel should be used.
(b) If riprap is being designed to protect channel banks, abutments, or piers located in the floodplain, average floodplain depths and velocities should be used.

Step 6 Compute the bank angle correction factor

$$\mathbf{K_1 = [1 - (\sin^2 \theta / \sin^2 \Phi)]^{0.5}} \qquad \textbf{(5.8)}$$

Where:
θ = the bank angle with the horizontal
Φ = the riprap material's angle of repose

Step 7 Determine riprap size required to resist particle erosion

$$\mathbf{d_{50} = 0.001\ V^3 / d_{avg}{}^{0.5} K_1{}^{1.5})} \qquad \textbf{(5.9)}$$

Where:
d_{50} = the median riprap particle size, ft
V = the average velocity in the main channel, ft/s
d_{avg} = the average flow depth in the main flow channel ft,
K_1 = bank angle correction factor

(a) Initially assume no corrections.
(b) Evaluate correction factor for rock riprap specific gravity and stability factor $C = C_{sg}C_{sf}$).

$$\mathbf{C_{sg} = 2.12 / S_s - 1)^{1.5}}$$

Where: S_s = the specific gravity of the rock riprap

$$\mathbf{C_{sf} = (SF / 1.2)^{1.5}}$$

Where: SF = the stability factor to be applied

Step 8 If the entire channel perimeter is being stabilized, and an assumed d_{50} was used in determination of Manning's 'n' for backwater computations, return to step 4 and repeat steps 4 through 7.

Step 9 Select final d_{50} riprap size, set material gradation, and determine riprap layer thickness.

Step 10 Determine longitudinal extent of protection required.

Step 11 Determine appropriate vertical extent of revetment.

Step 12 Design filter layer.

(a) Determine appropriate filter material size and gradation.
(b) Determine layer thickness.

Step 13 Design edge details (flanks and toe).

5.11.3 Wire-enclosed Rock

Wire-enclosed rock (gabion) revetments consist of rectangular wire mesh baskets filled with rock. The most common types of wire-enclosed revetments are mattresses and stacked blocks. The wire cages which make up the mattresses and gabions are available from commercial manufacturers.

Rock and wire mattress revetments consist of flat wire baskets or units filled with rock that are laid end to end and side to side on a prepared channel bed and/or bank. The individual mattress units are wired together to form a continuous revetment mattress.

Stacked block gabion revetments consist of rectangular wire baskets which are filled with stone and stacked in a stepped-back fashion to form the revetment surface. They are also commonly used at the toe of embankment slopes as toe walls which help to support other upper bank revetments and prevent undermining.

The rectangular basket or gabion units used for stacked configurations are more equidimensional than those typically used for mattress designs. That is, they typically have a square cross section. Commercially available gabions used in stacked configurations are available in various sizes but the most common have a 3-ft width and thickness.

Follow manufacturers recommended practice for design of gabions.

5.11.4 Pre-cast Concrete Blocks

Pre-cast concrete block revetments consist of pre-formed sections which interlock with each other, are attached to each other, or butt together to form a continuous blanket or mat. The concrete blocks which make up the mats differ in shape and method of articulation, but share certain common features. These features include flexibility, rapid installation, and provisions for the establishment of vegetation within the revetment.

Pre-cast revetments are bound using a variety of techniques. In some cases the individual blocks are bound to rectangular sheets of filter fabric (referred to as fabric carrier). Other manufacturers use a design which interlocks individual blocks. Other units are simply butted together at the site. The most common method is to join individual blocks with wire cable or synthetic fiber rope. Follow manufacturers recommended design procedure.

5.11.5 Grouted Rock

Grouted rock revetment consists of rock slope-protection having voids filled with concrete grout to form a monolithic armor.

Components of grouted rock riprap design include layout of a general scheme or concept, bank preparation, bank slope, rock size and blanket thickness, rock grading, rock quality, grout quality, edge treatment, filter design, and pressure relief.

Grouted riprap designs are rigid monolithic bank protection schemes. When complete they form a continuous surface. A typical grouted riprap section is shown in Figure 5-10. Grouted riprap should extend from below the anticipated channel bed scour depth to the design high water level, plus additional height for freeboard.

During the design phase for a grouted riprap revetment, special attention needs to be paid to edge treatment, foundation design, and mechanisms for hydrostatic pressure relief.

<u>Bank And Foundation Preparation</u>

The area to be stabilized should be prepared by first clearing all trees and debris, and grading the surface to the desired slope. In general, the graded surface should not deviate from the specified slope line by more than 6 inches. However, local depressions larger than this can be accommodated since initial placement of filter material and/or rock for the revetment will fill these depressions.

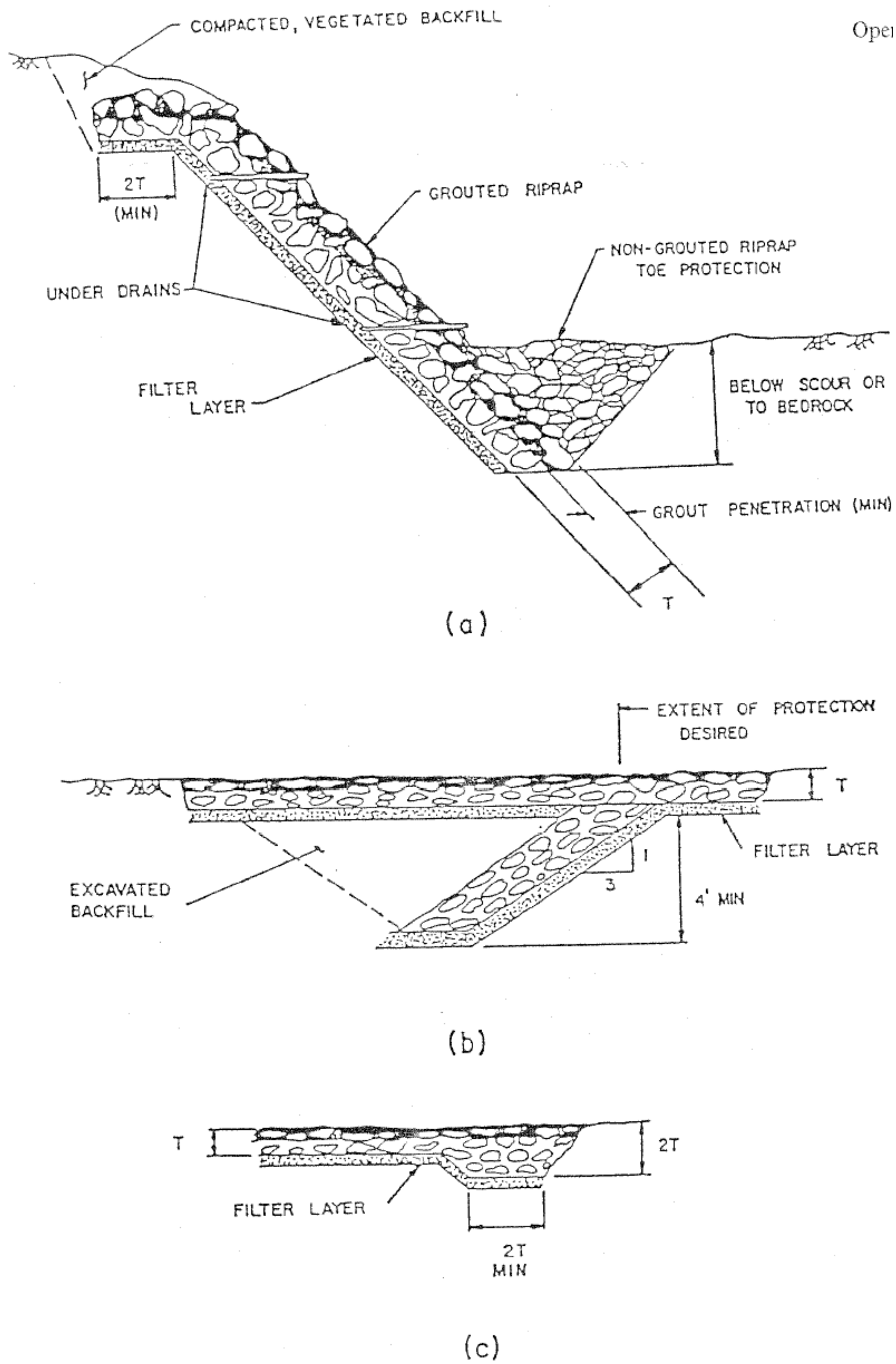


Figure 5-10 Grouted Riprap Sections: (a) Section; (b) Upstream Flank; and (c) Downstream Flank

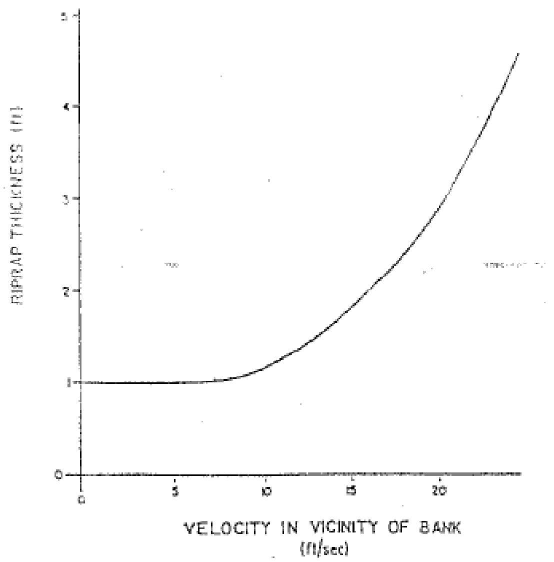


Figure 5-11 Required Blanket Thickness As A Function Of Flow Velocity

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Since grouted riprap is rigid but not extremely strong, support by the embankment must be maintained. To form a firm foundation, it is recommended that the bank surface be tamped or lightly compacted. Care must be taken during bank compaction to maintain a soil permeability similar to that of the natural, undisturbed bank material. The foundation for the grouted riprap revetment should have a bearing capacity sufficient to support either the dry weight of the revetment alone, or the submerged weight of the revetment plus the weight of the water in the wedge above the revetment for design conditions, whichever is greater.

Bank Slope

Bank slopes for grouted riprap revetments should not exceed 1.5:1. The soil stability slope will likely determine the maximum bank slope.

Rock Size And Blanket Thickness

Blanket thickness and rock size requirements for grouted riprap installation are interrelated. Figure 5-11 illustrates a relationship between the design velocity and the required riprap blanket thickness for grouted riprap designs. The median rock size in the revetment should not exceed 0.67 times the blanket thickness. The largest rock used in the revetment should not exceed the blanket thickness.

Rock Grading

Grouted riprap should meet all of the requirements for ordinary riprap except that the smallest rock fraction (i.e., smaller than the 10 percent size) should be eliminated from the gradation. A reduction of riprap size by one size designation is acceptable for grouted rock.

Rock Quality

Rock used in grouted rock slope-protection is usually the same as that used in ordinary rock slope-protection. However, the specifications for specific gravity and hardness may be lowered if necessary as the rocks are protected by the surrounding grout. In addition, the rock used in grouted riprap installations should be free of fines in order that penetration of grout may be achieved.

Grout Quality And Characteristics

Grout should consist of good strength concrete using a maximum aggregate size of 3/4 inch and a slump of 3 to 4 inches. Sand mixes may be used where roughness of the grout surface is unnecessary, provided sufficient cement is added to give good strength and workability.

The volume of grout required will be that necessary to provide penetration to the full depth of the riprap layer or at least 2 feet where the riprap layer is thicker than 2 feet. The finished grout should leave face stones exposed for one-fourth to one-third their depth and the surface of the grout should expose a matrix of coarse aggregate.

Edge Treatment

The edges of grouted rock revetments (the head, toe, and flanks) require special treatment to prevent undermining. The revetment toe should extend to a depth below anticipated scour depths or to bedrock. The toe should be designed as illustrated in Figure 5-10(a). After excavating to the desired depth, the riprap slope protection should be extended to the bottom of the trench and grouted. The remainder of the excavated area in the toe trench should be filled with ungrouted riprap. The ungrouted riprap provides extra protection against undermining at the bank toe.

To prevent outflanking of the revetment, various edge treatments are required. Recommended designs for these edge treatments are illustrated in Figure 5-10, parts (a), (b), and (c).

Filter Design

Filters are required under all grouted riprap revetments to provide a zone of high permeability to carry off seepage water and prevent damage to the overlying structure from uplift pressures. A 6-inch granular filter is required beneath the pavement to provide an adequate drainage zone. The filter can consist of well-graded granular material or uniformly-graded granular material with an underlying filter fabric. The filter should be designed to provide a high degree of permeability while preventing base material particles from penetrating the filter, thus causing clogging and failure of the protective filter layer.

Pressure Relief

Weep holes should be provided in the revetment to relieve hydrostatic pressure build-up behind the grout surface (see Figure 5-10(a)). Seeps should extend through the grout surface to the interface with the gravel underdrain layer. Weeps should consist of 2-inch minimum diameter pipes having a maximum horizontal spacing of 6 ft and a maximum vertical spacing of 10 ft. The buried end of the weep should be covered with wire screening or a fabric filter of a gage that will prevent passage of the gravel underlayer.

5.11.5.1 Construction

Construction details for grouted riprap revetments are illustrated in Figure 5-10. The following construction procedures should be followed:

Step 1 Normal construction procedures include (a) bank clearing and grading; (b) development of foundations; (c) placement of the rock slope protection; (d) grouting of the interstices; (e) backfilling toe and flank trenches; and (f) vegetation of disturbed areas.

Step 2 The rock should be set immediately prior to commencing the grouting operation.

Step 3 The grout may be transported to the place of final deposit by chutes, tubes, buckets, pneumatic equipment, or any other mechanical method which will control segregation and uniformity of the grout.

Step 4 Spading and rodding are necessary where penetration is achieved by gravity flow into the interstices.

Step 5 No loads should be allowed upon the revetment until good strength has been developed.

5.11.6 Bioengineering Methods

Bioengineering combines mechanical, biological, and ecological concepts to construct "living" structures for bank and slope protection. Bioengineering methods use structural support to hold live plantings in place while the root structure grows and the plants are established. This is done through the use of sprigging, live crib walls, cut brush layers, live fascines, live stakes, and other methods.

Advantages of bioengineering include: natural appearance, the self-healing properties, habitat enrichment, and resistance to slope failure. Disadvantages include: labor-intensive installation, need for stability control until the roots are established, and dependence on materials to root and grow. Bioengineering is gaining in popularity throughout the country, locally, the LPSNRD initiated a pilot project along Beal Slough near the 40th Street Bridge in 1997 that employed bioengineering techniques for bank stabilization.

Soil-bioengineered bank stability systems have not been standardized, the decision of whether and how to use the requires careful consideration. Two excellent references for detailed bioengineering design guidelines entitled "Stream Restoration: Principles, Processes, and Practices, Final Manuscript Draft, 1998" and "Part 650, Engineering Field Handbook, Chapter 16, Streambank and Shoreline Protection, 1996", are published by the Natural Resources Conservation Service. The first document is available at www.usda.gov on the NRCS webpage for downloading. These documents provide background on fundamental concepts necessary for planning, designing and applying bio-engineering techniques on many streams. Expertise in soils, biology, plant sciences, landscape architecture, geology, engineering and hydrology may be required for projects where the stream is large or the erosion is severe (NRCS Stream Corridor Restoration Final Manuscript Draft 1998). Several examples of bio-engineering techniques are presented in Figures 5-12- through 5-18.

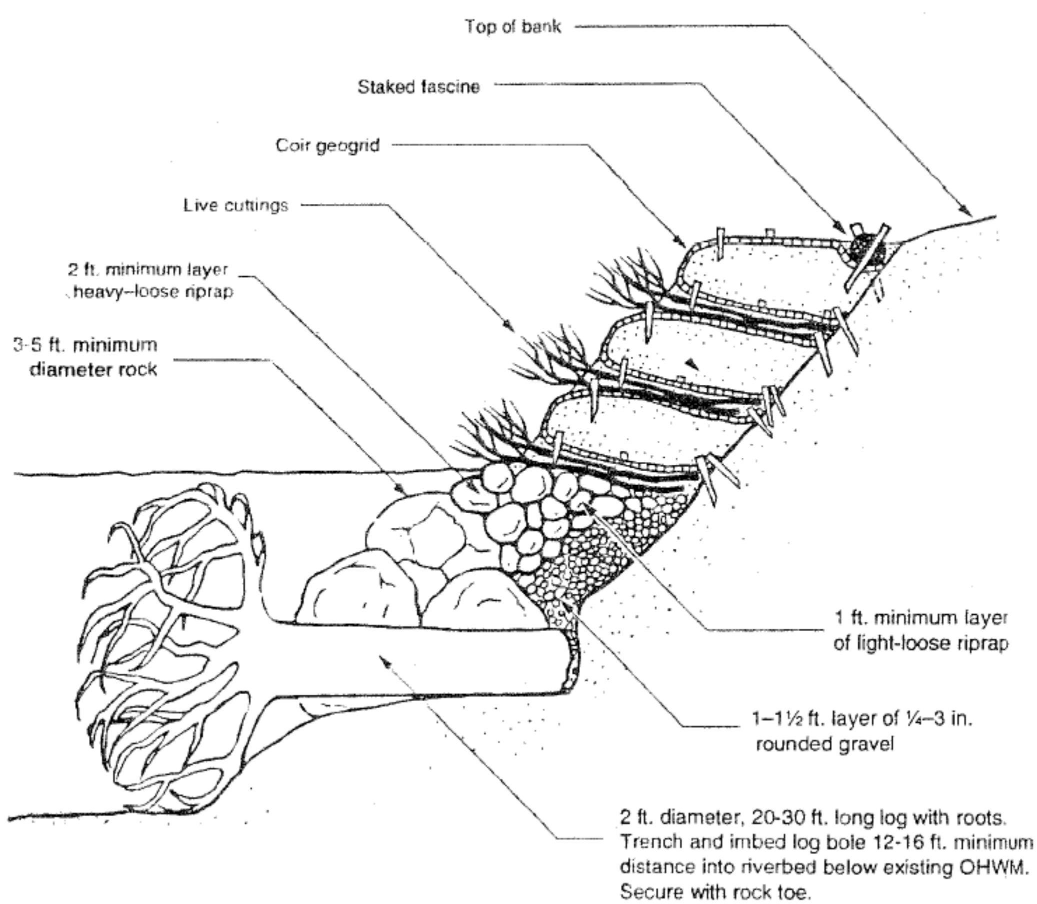


Figure 5-12 Integrated System with Large Woody Debris

Source: NRCS, 1996

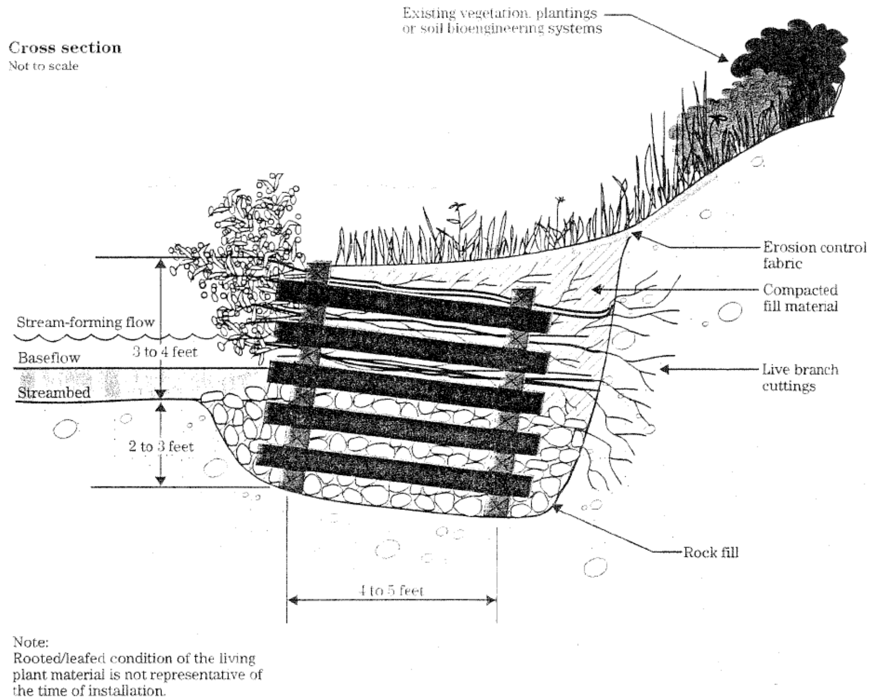


Figure 5-13 Live Cribwall Details

Source: NRCS, 1996

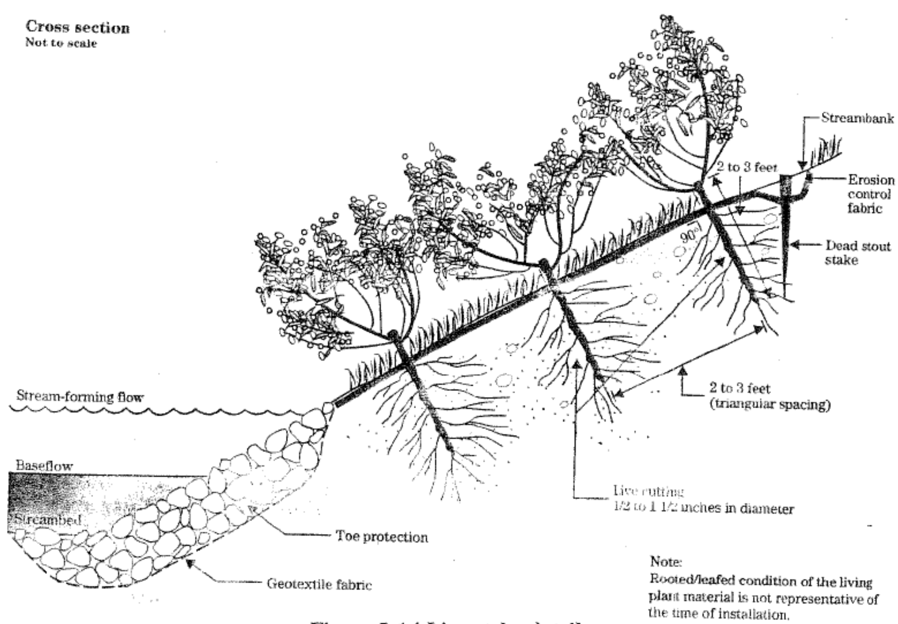


Figure 5-14 Live stake details

Source: NRCS, 1996

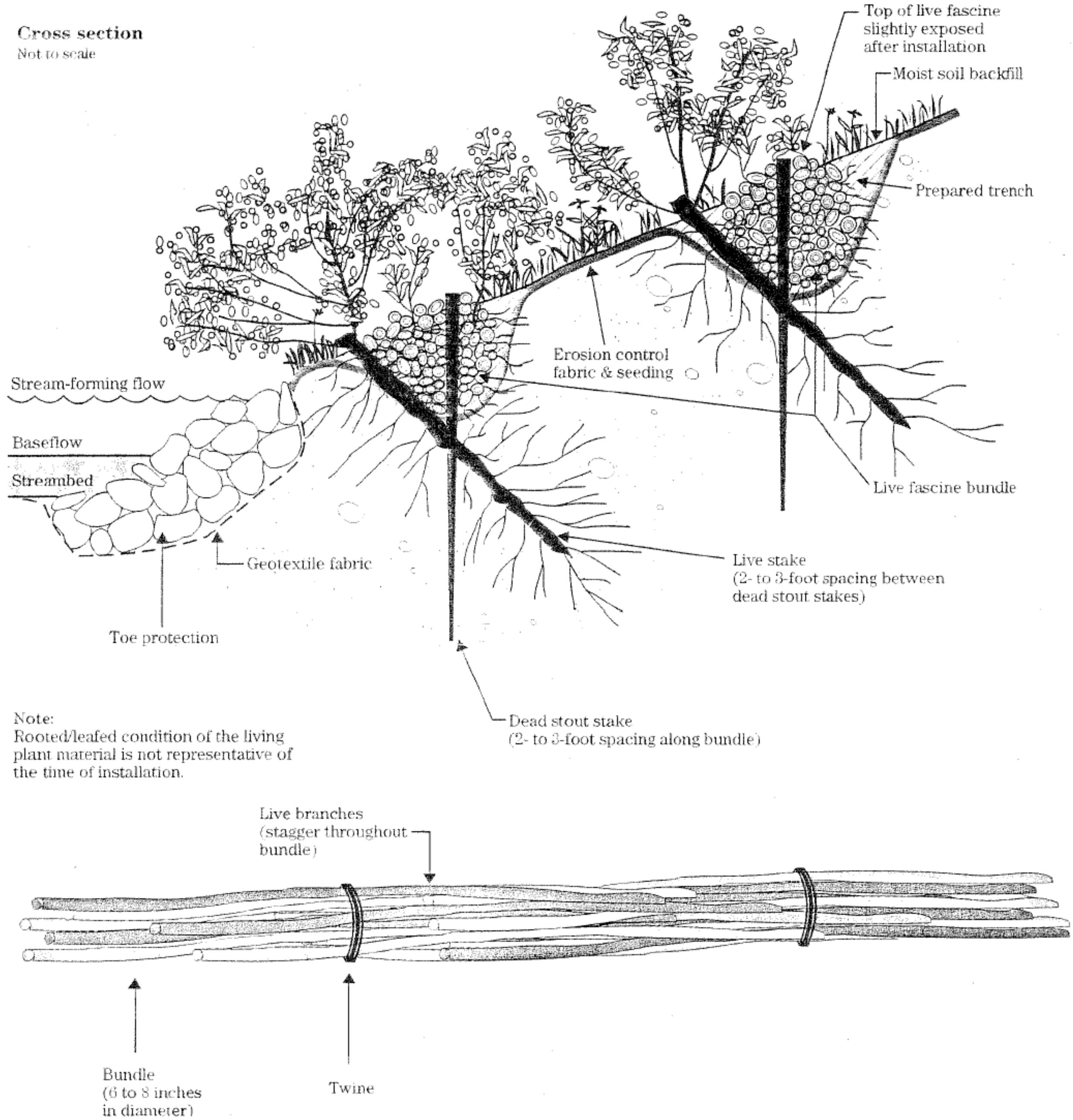
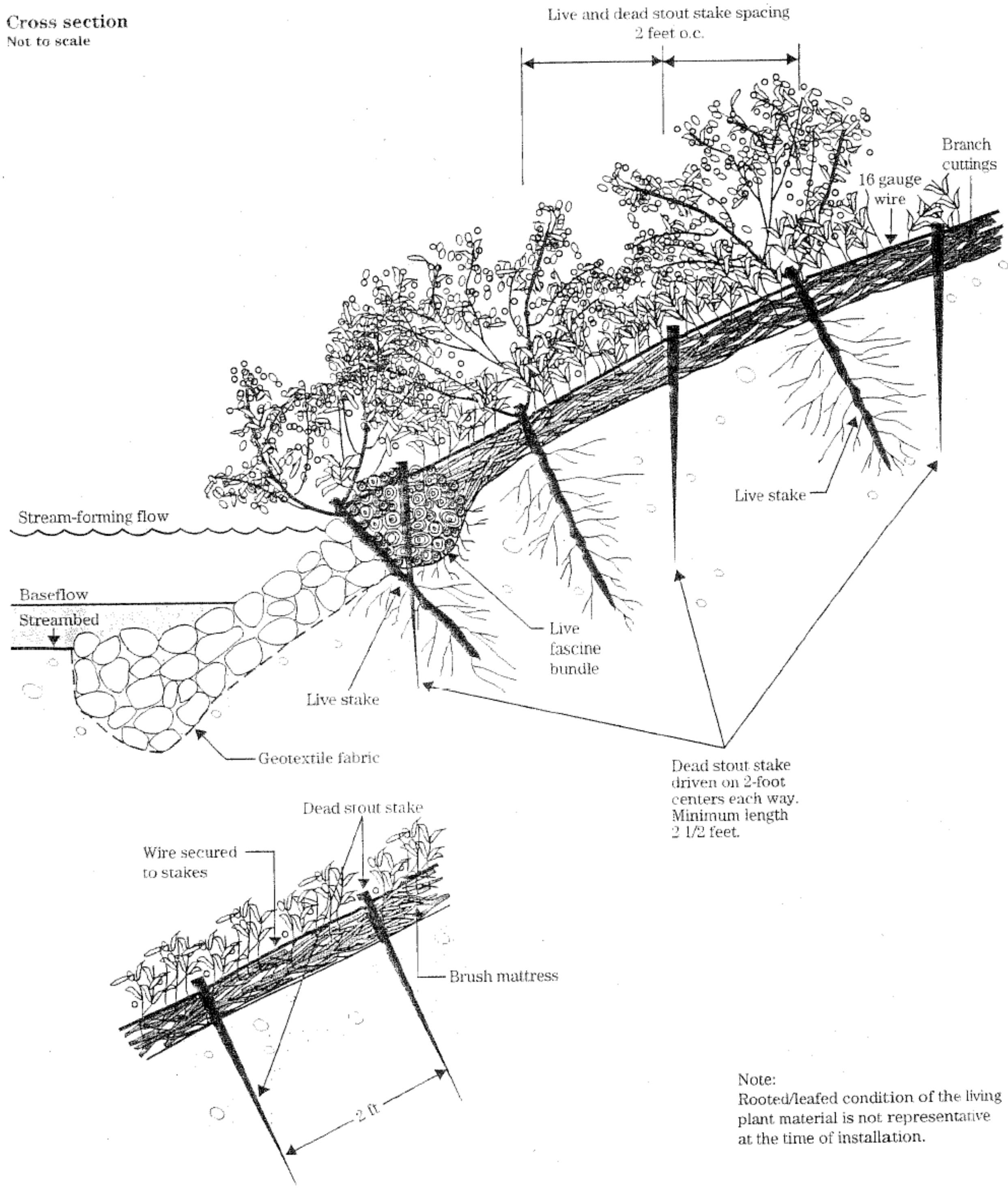
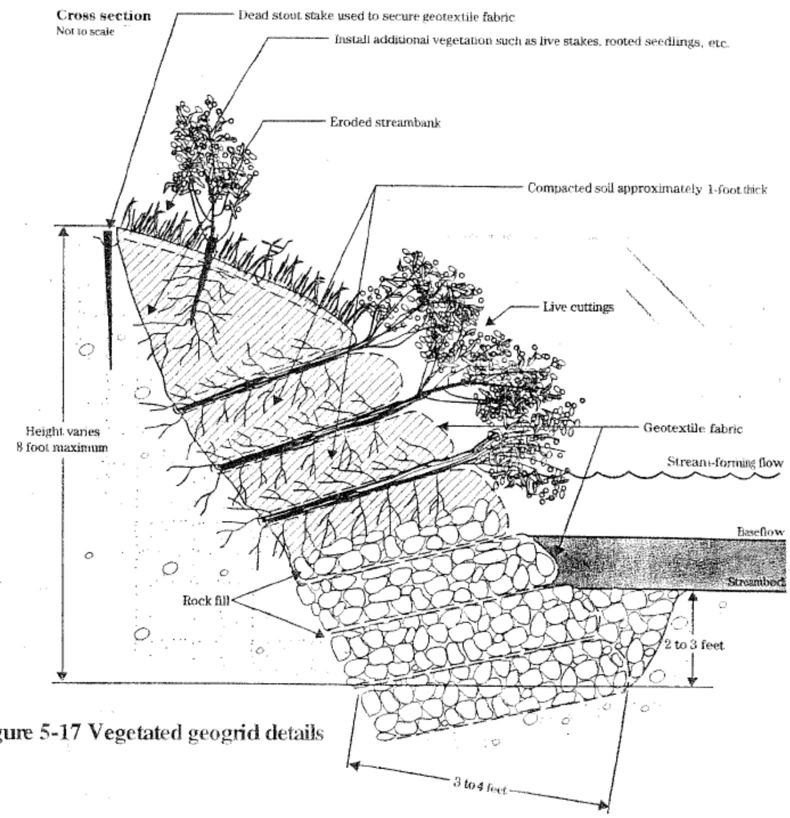


Figure 5-15 Live fascine details

Source: NRCS, 1996


Cross section
Not to scale
Live and dead stout stake spacing
2 feet o.c.
Branch
cuttings
16 gauge
wire
Stream-forming flow
Baseflow
Streambed
Live stake
Live
fascine
bundle
Live stake
Geotextile fabric
Dead stout stake
driven on 2-foot
centers each way.
Minimum length
2 1/2 feet.
Dead stout stake
Wire secured
to stakes
Brush mattress
2 ft
Note:
Rooted/leafed condition of the living
plant material is not representative
at the time of installation.

Open Channels



Note: Rooted/leafed condition of the living plant material is not representative of the time of installation.

Figure 5-17 Vegetated geogrid details

Source: NRCS, 1996

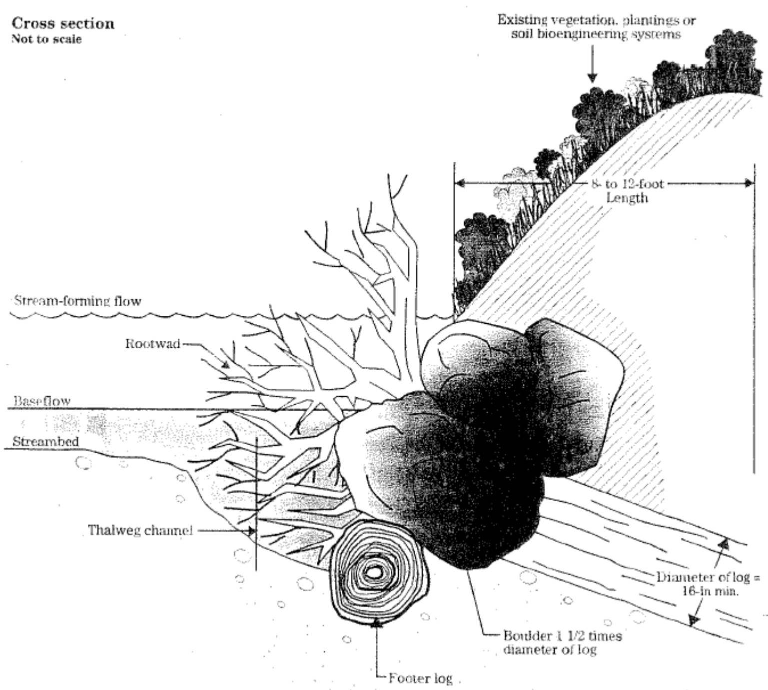


Figure 5-18 Log, rootwad, and boulder revetment details

Source: NRCS, 1996

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CHAPTER 6

STORAGE FACILITIES

March 7, 2011

Chapter 6 - Storage Facilities

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Chapter 6 - Storage Facilities

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6.1 Introduction

6.1.1 Overview

The traditional design of storm drainage systems has been to collect and convey storm runoff as rapidly as possible to a suitable location where it can be discharged. As areas urbanize, this type of design may result in major drainage and flooding problems downstream. The engineering community is now more conscious of the quality of the environment and the impact that uncontrolled increases in runoff can have on land owners. The temporary storage of some of the storm runoff can decrease downstream flows and often the cost of the downstream conveyance system. Storage facilities can range from small facilities contained in parking lots or other on-site facilities to large lakes and reservoirs. This appendix provides general design criteria for detention/retention storage basins as well as procedures for performing preliminary and final sizing and reservoir routing calculations. On-site storage facilities are required unless the master planning process or regional analysis as shown that the detention requirements can be transferred to a regional facility, which is determined to be of regional benefit to the drainage system by the City and NRD. On-site facilities may still be necessary to provide maintenance of receiving stream channel stability, maintenance and water quality.

6.1.2 Location Considerations

It should be noted that the location of storage facilities is very important as it relates to the effectiveness of these facilities to control downstream flooding. Small facilities will only have minimal flood control benefits and these benefits will quickly diminish as the flood wave travels downstream. Multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system, which could decrease or increase flood peaks in different downstream locations. Thus, it is important for the engineer to design storage facilities as drainage structures that both control runoff from a defined area and interact with other drainage structures within the drainage basin. Effective stormwater management must be coordinated on a regional, or basin-wide, planning basis.

6.1.3 Detention And Retention

Urban stormwater storage facilities are often referred to as either detention or retention facilities. For the purposes of this appendix, detention facilities are those that are designed to reduce the peak discharge and only detain runoff for some short period of time. These facilities are designed to completely drain after the design storm has passed. Recharge basins are a special type of detention basin designed to drain into the groundwater table; these are not addressed in this manual. Retention facilities are designed to contain a permanent pool of water. Since most of the design procedures are the same for detention and retention facilities, the term, “storage facilities,” will be used in this chapter to include detention and retention facilities. If special procedures are needed for detention or retention facilities, these will be specified.

6.1.4 Computer Programs

Routing calculations needed to design storage facilities, although not extremely complex, are time consuming and repetitive. To assist with these calculations there are many available reservoir routing computer programs. Also, the storage indication method can be used, which makes calculations simple. All storage facilities shall be designed and analyzed using reservoir routing calculations. Watershed routing for storage facilities shall be performed manually using the procedures outlined in this appendix or using HEC-HMS.

6.1.5 Plan Review

- Detention or retention storage construction plans shall be submitted by the owner to the Nebraska Department of Natural Resources for approval, or shall be certified by the owner that Nebraska Department of Natural Resources approval is not required.
- Supporting calculations for hydrologic and hydraulic analysis and design shall be submitted by the owner to the City for review and approval. As a minimum, supporting calculations shall include; design storm inflow and outflow hydrographs, stage-storage-discharge curves, and cumulative inflow / outflow elevation curves for the design storms.
- Appropriate soil investigation (i.e., suitability for water storage, settlement potential, slope stability, and

influence of groundwater) for the structure hazard classification.

- Construction plans for detention or retention storage, including the outlet structure, shall be submitted by the owner to the City for review and approval.
- The owner shall provide, at the end of construction, a separate written statement prepared by a licensed surveyor or engineer to the City that the grading and construction of storage facilities has been completed in conformance with the approved construction plans.

6.1.6 Ownership and Maintenance of Storage Facilities

Storage facilities proposed in a development, along with all inlet and outlet structures and/or channels, are to be owned and maintained by the developer or a property-owners' association unless a different ownership/maintenance arrangement has been approved by the City. Because the downstream storm sewer system will be designed assuming detention storage upstream, a storage facility in the storm sewer system shall remain functional as a storage facility site permanently. Provisions shall be made in the approval of development by the Planning Commission and City Council for the permanence of the storage facilities and ongoing maintenance of the storage facilities.

6.2 Uses

6.2.1 Introduction

The use of storage facilities for stormwater management has increased dramatically in recent years. The benefits of storage facilities can be divided into two major control categories of quality and quantity.

6.2.2 Quality

Control of stormwater quality using storage facilities offers the following potential benefits:

- decreased downstream channel erosion (with proper design) through velocity control and flow reduction,
- reduced pollution loading through deposition, chemical reaction and biological uptake mechanisms,
- aesthetic and ecological habitat benefits at multi-objective sites,
- control of sediment deposition, and
- improved water quality through stormwater filtration.

6.2.3 Quantity

Controlling the quantity of stormwater using storage facilities can provide the following potential benefits:

- prevention or reduction of peak runoff rate increases caused by urban development,
- mitigation of downstream drainage capacity problems,
- recharge of groundwater resources,
- reduction or elimination of the need for downstream outfall improvements, and
- maintenance of historic low flow rates by controlled discharge from storage.

6.3 Symbols And Definitions

To provide consistency, the following symbols will be used. These symbols were selected because of their wide use in technical publications. In some cases the same symbol is used in existing publications for more than one definition. Where this occurs in this appendix, the symbol will be defined where it occurs in the text or equations.

Table 6-1 Symbols And Definitions

Symbol	Definition	Units
A	Cross-sectional or surface area	ft^2
C	Weir coefficient	-
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
g	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_c	Height of weir crest above channel bottom	ft
I	Inflow rate	cfs
L	Length	ft
Q, O	Flow or outflow rate	cfs
S, Vs	Storage volume	ft^3, ac-ft
t	Routing time period	sec
t_b	Time base on hydrograph	hr
T_c	Time of concentration	hr
T_i	Duration of basin inflow	hr
t_p	Time to peak	hr
V_S, S	Storage volume	ft^3
W	Width of basin	ft
z	Side slope factor	-

6.4 Design Criteria

6.4.1 General Criteria

Storage may be concentrated in large basin-wide (or regional) facilities or distributed throughout an urban drainage system. Storage may be developed in depressed areas in parking lots, behind road embankments, freeway interchanges, parks and other recreation areas, and small lakes, ponds and depressions within urban developments. The utility of any storage facility depends on the amount of storage, its location within the system and its operational characteristics. An analysis of such storage facilities shall consist of comparing the design flow at a point or points downstream of the proposed storage site with and without storage. In addition to the design flow, other flows in excess of the design flow that might be expected to pass through the storage facility shall be included in the analysis. The design criteria for storage facilities shall include the following list. Compute inflow hydrograph for runoff from the 2-, 10- and 100-year design storms using the procedures outlined in Urban Hydrology for Small watersheds TR-55. Both predevelopment and post development hydrographs are required.

- release rate,
- storage volume,
- grading and depth requirements,
- safety considerations and landscaping,
- outlet works and location, and
- efficiency of maintenance.

6.4.2 Release Rate

Control structure release rates shall be such that peak discharge rates for post development conditions do not exceed predevelopment peak runoff rates for the 2-year, 10-year and 100-year discharges at the project property line and in accordance with paragraph 6.4.6, unless waived by the City. Parameters for predevelopment conditions shall be determined for actual site conditions existing on the site as of 1 August 1999. In addition, structures must provide detention of the initial ½-inch per impervious acre of storm runoff for 24-hours if the facility will be used for water quality purposes. Design calculations are required to demonstrate that runoff from the 2-, 10- and 100-year design storms is controlled. Runoff from intermediate storm return periods can be assumed to be adequately controlled. Multi stage control structures may be required to control runoff from all three storm events.

6.4.3 Storage

Storage volume shall be adequate to attenuate the post development peak discharge rates to predevelopment discharge rates for the 2-year, 10-year and 100-year storms, depending on the downstream system design capacity. Routing calculations must be used to demonstrate that the storage volume is adequate. Storage volume shall allow for the sediment load anticipated from the contributing watershed. Proper implementation of site erosion and sediment measures will greatly reduce the sediment load. If sedimentation during construction causes loss of detention volume, design dimensions shall be restored before completion of the project. For storage facilities, all temporarily stored runoff shall be drained within 72-hours.

6.4.4 Grading And Depth

Following is a discussion of the general grading and depth criteria for storage facilities, followed by criteria related to detention and retention facilities.

6.4.4.1 General

The construction of storage facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Dams shall be designed as per the applicable Department of Water Resources requirements. Specific City of Waverly requirements are that vegetated embankments shall have side slopes no steeper than 4:1 (horizontal to vertical), that the top width of any embankment shall be no narrower than 14 feet, and traversable vehicular access for maintenance purposes shall be provided from public right-of-way.

Other considerations when setting depths include flood elevation requirements, public safety, land availability, land value, present and future land use, water table fluctuations, soil characteristics, maintenance requirements and required freeboard. New development shall be designed so the lowest opening of adjacent new buildings is a minimum of one foot above the calculated 100-year flood elevation. Aesthetically pleasing features are also important in urbanizing areas. Fencing of basins is addressed in section 6.14.

6.4.4.2 Detention

Areas above the normal high-water elevations of storage facilities shall slope at a minimum of 2% toward the facilities to allow drainage and to prevent standing water. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of storage facilities shall be graded toward the outlet to prevent standing water conditions. A minimum 2% bottom slope is required on unpaved areas. A low flow or pilot channel constructed across the facility bottom from the inlet to the outlet is required to convey low flows, and prevent standing water conditions.

6.4.4.3 Retention

Retention facilities are conducive to establishment of wetland and open water habitats. Site-specific criteria relating to such things as depth, habitat, and bottom and shoreline geometry shall be selected to encourage establishment of desired habitat. Where wetland habitat is desired, vegetative and geometric conditions shall be provided to minimized the propagation of undesired vegetation. Plant and wildlife experts should be contacted for site specific guidance. If the facility provides open water conditions, a depth sufficient to discourage growth of vegetation, except along the shoreline, (without creating undue potential for anaerobic bottom conditions) shall be provided. A depth of 5 to 10 feet is generally reasonable unless fishery requirements dictate otherwise. Aeration may be required in permanent pools to prevent anaerobic conditions. The maximum depth of permanent storage facilities will be determined by site conditions, design constraints, and environmental needs.

6.4.5 Outlet Works

Outlet works selected for storage facilities shall include a principal spillway and an emergency overflow, and must be able to accomplish the design functions of the facility unless adequate supporting documentation is provided to the satisfaction of the City. Principal spillway discharge must be released in a nonerosive manner. Outlet works can be combinations of drop inlets, pipes, weirs, orifices, chutes, and channels. Slotted riser pipes are discouraged because of clogging problems, but curb openings may be used for parking lot storage facilities. Storage facilities shall pass the 2-year, 10-year and 100-year design storms for post development conditions without allowing flow to enter an emergency outlet through a combination of available storage and outlet works capacity. Outlet works must operate without requiring attendance or operation. The emergency spillway crest elevation shall be set at the maximum water surface elevation for the 100-year design storm. Minimum freeboard of three feet above the emergency spillway crest elevation will be necessary for embankment structures which are large enough to require review and permitting by NDWR. For large storage facilities, selecting a flood magnitude for sizing the emergency outlet shall be consistent with the potential threat to downstream life and property if the basin embankment were to fail. The sizing of a particular outlet works shall be based on results of hydrologic routing calculations.

Outlets such as V-notch weirs are preferred to pipes since they provide for multiple flood events, including events less than the 2-year storm, while pipes sized for the 2-year flood will not provide any control of smaller storm events.

6.4.6 Location and Downstream Analysis

Although storage facilities are designed to control the discharge at the outlet device, the discharge may need to be routed downstream to be sure that the downstream drainage system provides an adequate outlet for the discharge without causing drainage or flooding problems. This is particularly important where discharge from the storage facility may exceed the downstream drainage system capacity and overtop roadways, causing a hazard or property damage. Storage facilities will change the timing of the entire hydrograph. If several storage facilities are located within a particular basin, it is important to determine what effects a particular facility may have on combined hydrographs in downstream locations. If the storage facility being designed is located in a drainage basin that has a master plan, the discharge hydrographs from the outlet works shall be routed down stream to the bottom of the master plan subbasin. The resulting 2-, 10-, and 100-year peak flows with the proposed facility in place shall be compared to the master plan peak flows to verify the development drainage and storage facility plan is acceptable. If the resulting peak flows exceed the master plan flows, the designs shall be improved to be consistent with the master plan.

Detention can be located within floodplains and still effectively control flooding through the use of timing calculations. In this situation, the flood peak coming down the stream rarely coincides with local on-site flooding. Allowing the on-site water to pass is often advantageous, implementing only simple erosion control and a properly sized conveyance system. Then design and locate detention to "skim" the peak from the oncoming flood hydrograph through the use of a side-channel weir or a simple flow-through depression along the banks.

6.5 Safe Dams Act

6.5.1 Background

National responsibility for the promotion and coordination of dam safety lies with the Federal Emergency Management Agency (FEMA). State responsibility for administration of the provisions of the Federal Dam Safety Act is governed by State of Nebraska Department of Natural Resources, Surface Water Chapter 46, Article 2. Rules and regulations relating to applicable dams are promulgated by State of Nebraska Department of Natural Resources.

Under the state regulations, a dam is an artificial barrier that does or may impound water that is 25 ft or greater in height or has a maximum storage volume of 50 ac-ft or more (including surcharge storage). A number of exemptions are allowed from the Safe Dams Act and the appropriate state office should be contacted to resolve questions. Detention or retention storage embankments which fall under the jurisdiction of the Department of Natural Resources must be designed and constructed in accordance with Safe Dam criteria with review and permitting by NDNR. An owner proposing a detention or retention embankment shall submit to the City documentation of compliance with

NDNR review and permitting requirements, or documentation why the embankment does not fall under NDNR jurisdiction.

6.5.2 Classification

Dams are classified as either new or existing, by hazard potential, and by size. The State of Nebraska Department of Water Resources classifies dams under the Rules for Surface Water (August 1995), Title 457, Chapter 19 (entitled Dam Hazard Class). These classifications are presented below.

High Hazard Dam - A dam located where failure may cause loss of life, or serious damage to homes, normally occupied industrial and commercial buildings, important public utilities, main highways, or major railroads.

Significant Hazard Dam - A dam located in areas where failure may damage isolated homes, occasionally occupied buildings, main highways, minor railroads or interrupt public utility use or service.

Low Hazard Dam - A dam located in areas where failure may damage normally unoccupied buildings, undeveloped land, or township and county roads.

6.5.3 New Dams

Detailed engineering requirements are given in the regulations for new dams. Regulations that shall be consulted for further details and engineering requirements are State of Nebraska Department of Natural Resources, Surface Water Chapter 46, Article 2 and State of Nebraska Department of Natural Resources Rules for Surface Water (August 1995), Title 457.

6.6 General Hydraulic Design Procedure

6.6.1 Data Needs

The following data will be needed to complete storage design and routing calculations for submittal to the Cityt.

- Inflow hydrograph for all selected design storms.
- Stage-storage curve for proposed storage facility (see Figure 6-1 for an example). For large storage volumes (such as for reservoirs), use acre-feet, otherwise use cubic feet.
- Stage-discharge curve for all outlet control structures (see Figure 6-2 for an example).

Using these data, a design procedure is used to route the inflow hydrograph through the storage facility with different basin and outlet geometry until the desired outflow hydrograph is achieved (see example in section 6.9).

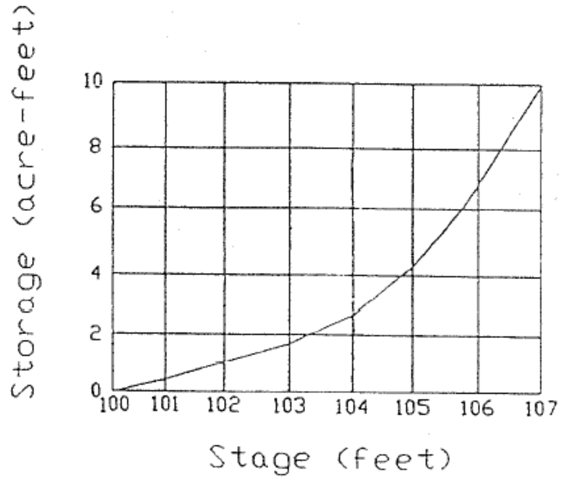


Figure 6-1 Example Stage-Storage Curve

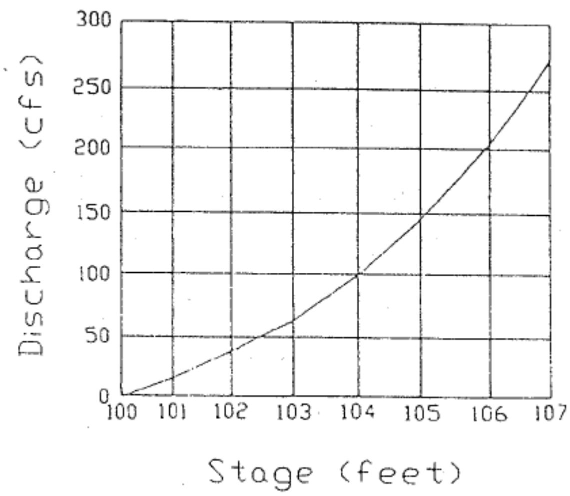


Figure 6-2 Example Stage-Discharge Curve

6.6.2 Stage-storage Curve

A stage-storage curve defines the relationship between the depth of water and storage volume in a reservoir. The data for this type of curve are usually developed using a topographic map and one of the following formulas; the average-end area, frustum (i.e., cross-sectional slice) of a pyramid, or prism. Storage basins are often irregular in shape to blend well with the surrounding terrain and to improve aesthetics. Therefore, the average-end area formula applied to elevational contour slices is usually preferred as the method to be used on non-geometric areas. The average-end area formula is expressed as:

$$\mathbf{V_{1,2} = [(A_1 + A_2)/2]d} \qquad \textbf{(6.1)}$$

Where: $V_{1,2}$ = storage volume between elevations 1 and 2, ft^3
$A_{1,2}$ = surface area at elevations 1 and 2, respectively, ft^2
d = change in elevation between points 1 and 2, ft

The frustum of a pyramid is expressed as:

$$\mathbf{V = d/3\ [A_1 + (A_1 \times A_2)^{0.5} + A_2]} \qquad \textbf{(6.2)}$$

Where: V = volume of frustum of a pyramid, ft^3
d = change in elevation between points 1 and 2, ft
$A_{1,2}$ = surface area at elevations 1 and 2 respectively, ft^2

The prism formula for trapezoidal basins is expressed as:

$$\mathbf{V = LWD + (L + W)\ ZD^2 + 4/3\ Z^2\ D^3} \qquad \textbf{(6.3)}$$

Where: V = volume of trapezoidal basin, ft^3
L = length of basin at base, ft
W = width of basin at base, ft
D = depth of basin, ft
Z = side slope factor, ratio of vertical to horizontal

6.6.3 Stage-discharge Curve

Stage-discharge curves define the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility has two spillways: principal and emergency. A pipe culvert, rectangular weir, v-notch weir or other appropriate outlet can be used for the principal spillway or outlet. Tailwater influences and structure losses must be considered when developing discharge curves.

The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal spillway. This spillway must be designed taking into account the potential threat to downstream life and property if the storage facility were to fail. The stage-discharge curve shall take into account the discharge characteristics of both the principal and emergency spillways.

6.6.4 Procedure

A general procedure for the design of storage facilities follows.

Step 1: Compute inflow hydrograph for runoff from the 2-, 10- and 100-year design storms using the procedures outlined in Urban Hydrology for Small watersheds TR-55. Both pre- and post-development hydrographs are required.

Step 2: Perform preliminary calculations to estimate detention storage requirements for the hydrographs from Step 1 (see section 6.8). If storage requirements are satisfied for runoff from the 2-, 10-, and 100-year design storms, runoff from intermediate storms is assumed to be controlled.

Step 3: Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 shall be used.

Step 4: Size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure shall be sized to convey the allowable discharge at this stage.

Step 5: Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using the storage routing equations. If any of the routed postdevelopment peak discharges from the 2-, 10- and 100-year design storms exceed the corresponding predevelopment peak discharges, or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise the estimated volume and return to Step 3.

Step 6: Design for emergency overflow and established freeboard requirements.

Step 7: Evaluate the downstream effects of detention outflow to ensure that the routed hydrograph does not cause downstream flooding problems. The exit hydrograph from the storage facility shall be routed downstream for a distance defined by the criteria provided in section 6.4.6.

Step 8: Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity or duration will cause erosion problems downstream.

This procedure can involve a significant number of reservoir routing calculations to obtain the desired results.

6.7 Outlet Hydraulics

6.7.1 Outlets

Outlet control structures combine the hydraulic function of one or more individual control elements such as, sharp-crested weirs, broad-crested weirs, v-notch weirs, orifices, and pipes to produce integrated stage-discharge behavior required to meet the discharge limits for developments. Weir and orifice design equations are presented below. If culverts are used as outlets works, procedures presented in the NDOR Roadway Design Manual should be used to develop stage-discharge data. When analyzing release rates, the tailwater influence of the principal spillway culvert on the control structure (orifice and/or weirs) must be considered to determine the effective head on each opening. Slotted riser pipe outlet facilities shall be avoided.

6.7.2 Sharp-crested Weirs

A sharp-crested weir with no end contractions is illustrated in Figure 6-3. The discharge equation for this configuration is (Chow, 1959):

$$\mathbf{Q = [3.27 + 0.4(H/H_c)]\ LH^{1.5}} \qquad \textbf{(6.4)}$$

Where: Q = discharge, cfs
H = head above weir crest excluding velocity head, ft
H_c = height of weir crest above channel bottom, ft
L = horizontal weir length, ft

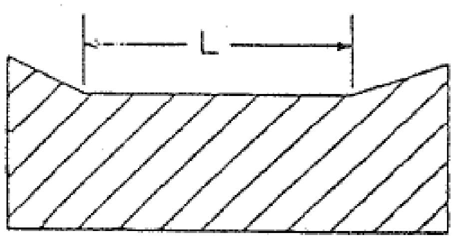


Figure 6-3
Sharp-crested Weir (No End Contractions)

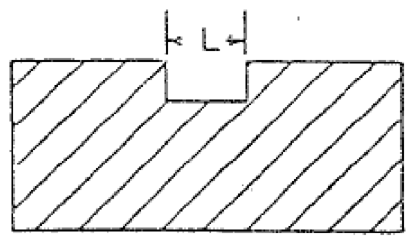


Figure 6-4
Sharp-crested Weir (Two End Contractions)

A sharp-crested weir with two end contractions is illustrated in Figure 6-4. The discharge equation for this configuration is (Chow, 1959):

$$\mathbf{Q = [3.27 + 0.4(H/H_c)]\ (L - 0.2H)\ H^{1.5}} \qquad \textbf{(6.5)}$$

Where: Variables are the same as equation 6.4.

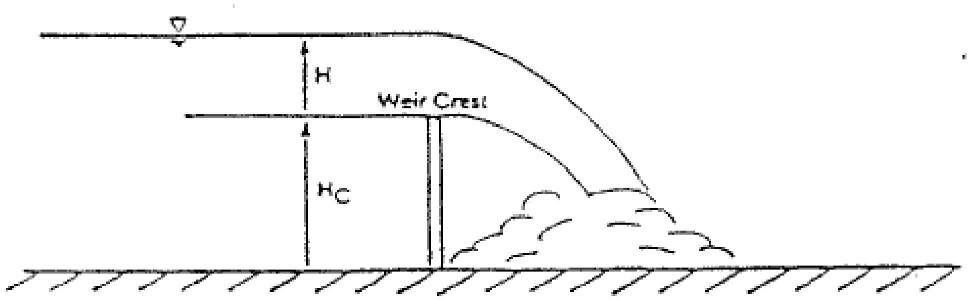


Figure 6-5 Sharp-crested Weir And Head

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation (see Figure 6-5). The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$$\mathbf{Q_S = Q_f\ (1 - (H_2/H_1)^{1.5})^{0.385}} \qquad \textbf{(6.6)}$$

Where: Q_s = submergence flow, cfs
Q_f = free flow, cfs
H_1 = upstream head above crest, ft
H_2 = downstream head above crest, ft

6.7.3 Broad-crested Weirs

The equation generally used for the broad-crested weir is (Brater and King, 1976):

$$\mathbf{Q = CLH^{1.5}} \tag{6.7}$$

Where: Q = discharge, cfs
C = broad-crested weir coefficient
L = broad-crested weir length, ft
H = head above weir crest, ft

Information on C values as a function of weir crest breadth and head is given in Table 6-2.

6.7.4 V-Notch Weirs

The discharge through a v-notch weir can be calculated from the following equation (Brater and King, 1976).

$$\mathbf{Q = 2.5\ tan(\theta/2)H^{2.5}} \tag{6.8}$$

Where: Q = discharge, cfs
θ = angle of v-notch, degrees
H = head on apex of notch, ft

6.7.5 Proportional Weirs

Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head (example shown in Figure 6-6). Refer to "Proportional Weirs for Stormwater Pond Outlets" (Sandvik, 1985) for a full discussion.

Design equations for proportional weirs are (Sandvik, 1985):

$$\mathbf{Q = 4.97\ a^{0.5}\ b\ (H - a/3)} \tag{6.9}$$
$$\mathbf{x/b = 1 - (1/3.17)\ (arctan\ (y/a)^{0.5})} \tag{6.10}$$

Where: Q = discharge, cfs
Dimensions a, b, h, x and y are shown below

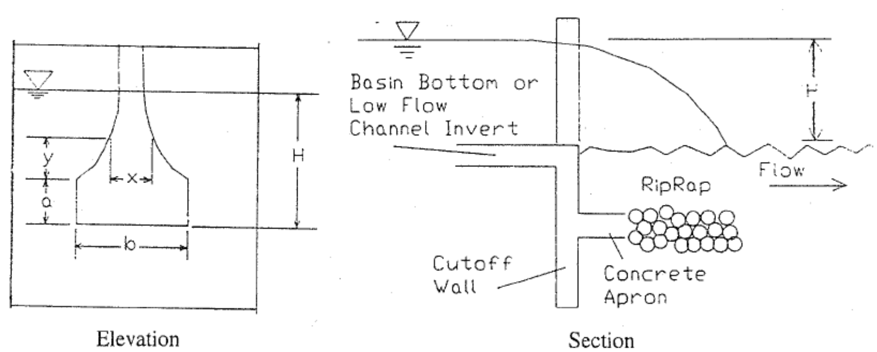


Figure 6-6 Proportional Weir Dimensions

6.7.6 Orifices

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

$$Q = 0.6A(2gH)^{0.5} = 3.78D^2H^{0.5} \tag{6.11}$$

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

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

Table 6-2 Broad-Crested Weir Coefficient C Values As A Function Of Weir Crest Breadth And Head (ft)

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

[1]Measured at least 2.5H upstream of the weir.
Reference: Brater and King (1976).

6.8 Preliminary Detention Calculations

6.8.1 Estimating Storage Volume

When a detention basin is installed, hydrologic routing procedures can be used to estimate the effect on hydrographs. Both the HEC-HMS and DAMS2 (SCS 1982) computer programs provide accurate methods of analysis. Programmable calculator and computer programs are available for routing hydrographs through dams.

This chapter contains a manual method for quick estimates of the effects of temporary detention on peak discharges. The method is based on average storage and routing effects for many structures. Figure 6-7a relates two ratios: peak outflow to peak inflow discharge (qo /qi) and storage volume runoff volume (Vs /Vr) for all rainfall distributions.

The relationships in figure 6-7a were determined on the basis of single stage outflow devices. Some were controlled by pipe flow, others by weir flow. Verification runs were made using multiple stage outflow devices, and the variance was similar to that in the base data. The method can therefore be used for both single- and multiple-stage outflow devices. The only constraints are that (1) each stage requires a design storm and a computation of the storage required for it and (2) the discharge if the upper stage(s) includes the discharge of the lower stage(s).

Use figure 6-7a to estimate storage volume (Vs) required or peak outflow discharge (qo). The most frequent application is to estimate Vs , for which the required inputs are runoff volume (Vr), qo , and peak inflow discharge (qi). To estimate qo , the required inputs are Vr , Vs , and qi

6.8.1.1 Estimating Vs

Use the worksheet shown on figure 6-7b to estimate Vs , storage volume required, by the following procedure:

1. Determine qo . Many factors may dictate the selection of peak outflow discharge. The most common is to limit downstream discharges to a desired level, such as predevelopment discharge.

2. Estimate qi by procedures in Chapter 2.6. Do not use peak discharges developed by other procedures.

3. Compute qo/qi and determine Vs/Vr from figure 6-7a.

4. Q (in inches) was determined when computing qi in step 2, but now it must be converted to the units in which Vs is to be expressed—most likely, acre-feet.

5. Use the results of steps 3 to 4 to compute Vs: Vs = Vr(Vs/Vr) where Vs = storage volume required (acre-ft).

6. The stage in the detention basin corresponding to Vs must be equal to the stage used to generate qo. In most situations a minor modification of the outflow device can be made. If the device has been preselected. repeat the calculations with a modified qo value.

6.8.1.2 Limitations

This routing method is less accurate as the qo/qi ration approaches the limits shown in figure 6-7a. The curve in figure 6-7a depends on the relationship between available storage, outflow device, inflow volume, and shape of the inflow hydrograph. When storage volume (Vs) required is small, the shape of the outflow hydrograph is sensitive to the rate of the inflow hydrograph. Conversely, when Vs is large, the inflow hydrograph shape has little effect on the outflow hydrograph. In such instances, the outflow hydrograph is controlled by the hydraulics of the outflow device and the procedure therefore yields consistent results. When the peak outflow discharge (qo) approaches the peak flow discharge (qi) parameters that affect the rate of rise of a hydrograph, such as rainfall volume, curve number, and time of concentration, become especially significant The procedure should not be used to perform final design if an error in storage of 25 percent cannot be tolerated. It is adequate, however, for final design of small detention basins. Figure 6-7a is biased to prevent under sizing of outflow devices, but it may significantly overestimate the required storage capacity. More detailed hydrograph development and routing will often pay for itself through reduced construction costs.

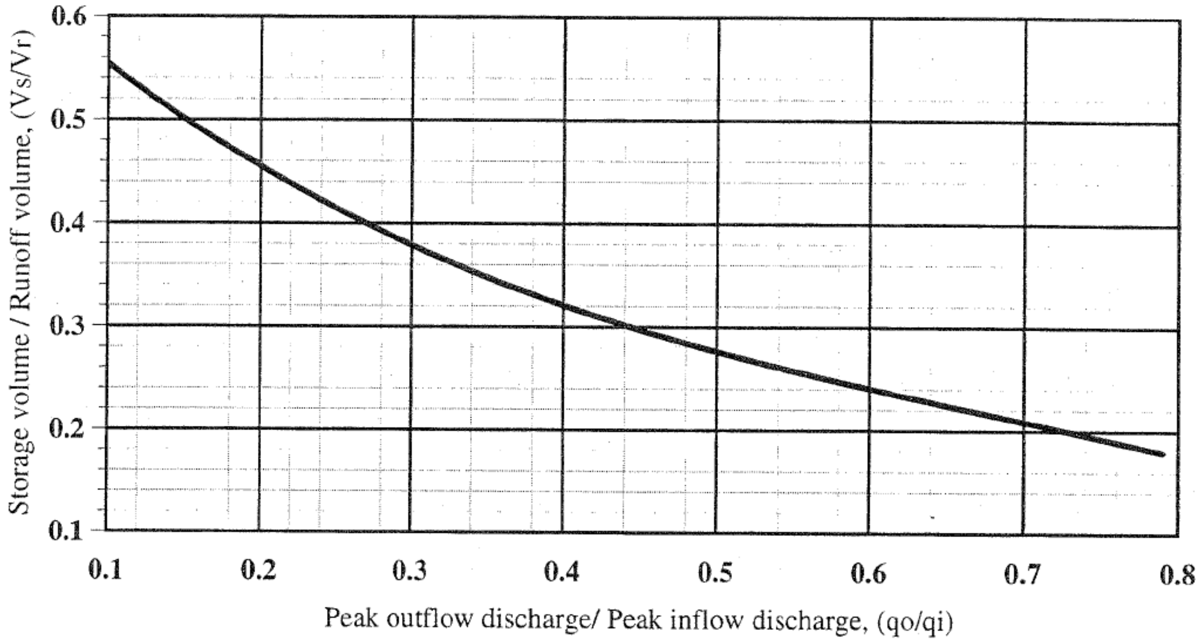


Figure 6-7a Approximate stormwater storage basin routing for type II rainfall
(Source 210-VI-TR-55, Second Ed., June 1986)

6.8.1.3 Example Estimating Vs for Two-Stage Structure

Given: 2 -year		100-year	
predevelopment	= 50 cfs	predevelopment	= 180 cfs
post development	= 91 cfs	post development	= 360 cfs
post development runoff volume	= 1.5 inches	post development runoff volume	= 3.4 inches

A rectangular concrete weir outlaw device was selected; the device could have been another type, but it is important to remember that the flows through the first stage are part of the total discharge of the higher stage.

Figure 6-7b shows how the worksheet is used to compute the Vs of 2.4 acre-ft and Emax of 103.6 for the stage. Emax of 103.6 is the weir crest elevation for the second stage. Equation 6.7 is used to compute Lw for the first stage. The weir crest elevation for the first stage is 100.00 ft and qo = 50 cfs. The first-stage computations for Hw and Lw are

Hw = Emax - weir crest elevation = 103.6 - 100.0 = 3.6 ft;

and, from equation 6.7 , $Lw = \dfrac{50}{3.2(3\ 6)^{1.5}} = 2.3$ ft

The second stage is then proportioned to discharge correct amount at 105.7 feet.

Compute the discharge through the first stage for elevation 105.7 feet using

L = 2.3 ft (first stage) and Hw = 105.7 - 100.0 = 5.7 ft

By substituting these values in equation 6.7 , discharge (qo) through the first stage at 105.7 feet is calculated:
$qo = 3.2(2.3(5.7)^{1.5} = 100$ cfs

Now compute the required weir crest length (Lw) for the second stage, using equation 6.7 . Since the second stage crest elevation is 103.6 feet,

Hw = Emax - weir crest elevation = 105.7 - 103.6 = 2.1 ft;

and, since qo for the second stage equals the total discharge minus discharge through the first stage,

qo = 180 -100 = 80 cfs

Finally, substituting these Hw and qo values in equation 6.7 results in

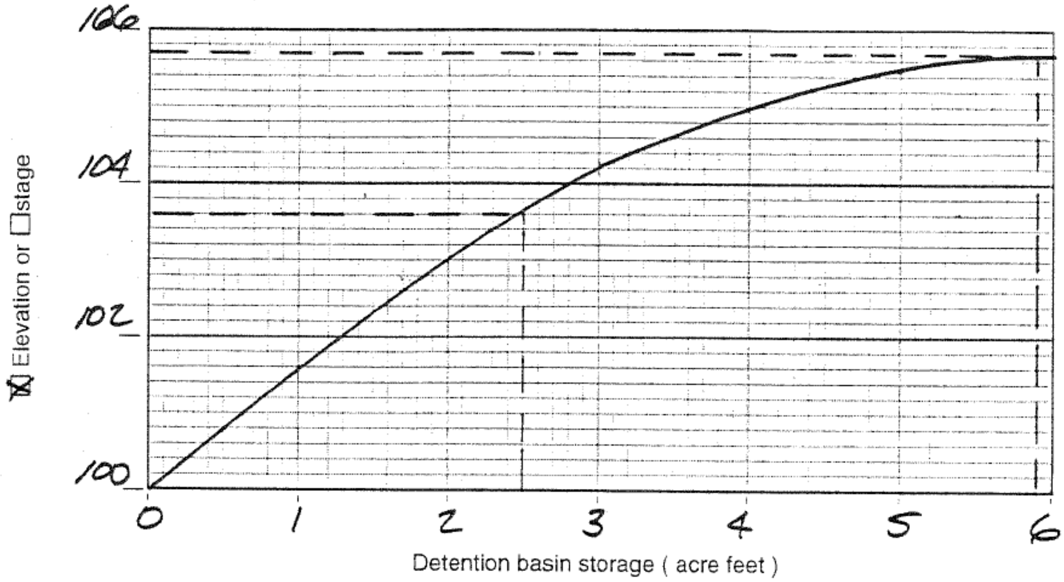
$$Lw = \frac{80}{3.2(2.1)^{1.5}}$$

In summary, the outlet structure is a two-stage rectangular weir with first stage crest length of 2.3 feet at elevation 100.0, and second stage crest length of 8.2 feet at elevation 103.6 feet.

The weir equation used is probably less accurate for the two-stage example than for the single-stage ex- ample. The actual second-stage discharge will be slightly more that the one computed. but a discussion of hydraulics of outflow devices is outside the scope of this technical release. This example is presented only to illustrate the interrelationship of outflow discharges and storage volume and to show how to develop preliminary estimates of storage requirements for two-the stage outlet structures.

Project: Two-stage structure	By: ENGINEER	Date: 02/22/2000
Location:	Checked: ENGINEER2	Date: 02/22/2000

Check one: ☐ Present ☒ Developed



1. Data:
   Drainage area .... A_m = 0.117 mi²
   Rainfall distribution type (I, IA, II, III) = II

	1st Stage	2nd Stage
2. Frequency .... yr	2	100
3. Peak inflow discharge q_i .... ft³/s (from worksheet 4 or 5b)	91	360
4. Peak outflow discharge q_o .... ft³/s	50	180 [1/]
5. Compute $\frac{q_o}{q_i}$	0.55	0.50
6. $\frac{V_s}{V_r}$ .... (Use $\frac{q_o}{q_i}$ with figure 6-1)	0.26	0.28
7. Runoff, Q .... in (From worksheet 2)	1.5	3.4
8. Runoff volume .... ac ft ($V_r = QA_m$ 53.33)	9.4	21.2
9. Storage volume, V_s .... ac-ft ($V_s = V_s(\frac{V_s}{r})$)	2.4	5.9
10. Maximum storage E_{max} (from plot)	103.6	105.7

1/ 2nd stage q_o includes 1st stage q_o.

Figure 6-7b Stormwater storage facility basin storage volume, peak outflow discharge (qo) known
(Source 210-VI-TR-55, Second Ed., June 1986)

6.8.2 Preliminary Basin Dimensions

- Plot the control structure location on a contour map.
- Select desired depth(s) of ponding for the design storm(s).
- Divide the estimated storage volume needed by the desired depth to estimate the required surface area of the reservoir.
- Based on site conditions and contours, estimate the geometric shape(s) required to provide the estimated reservoir surface area.

6.9 Routing Calculations

The following procedure is used to perform routing through a reservoir or storage facility (Puls Method of storage routing).

Step 1: Develop an inflow hydrograph, stage-discharge curve and stage-storage curve for the proposed storage facility. Example stage-storage and stage-discharge curves are shown below as Figure 6-8 and Figure 6-9.

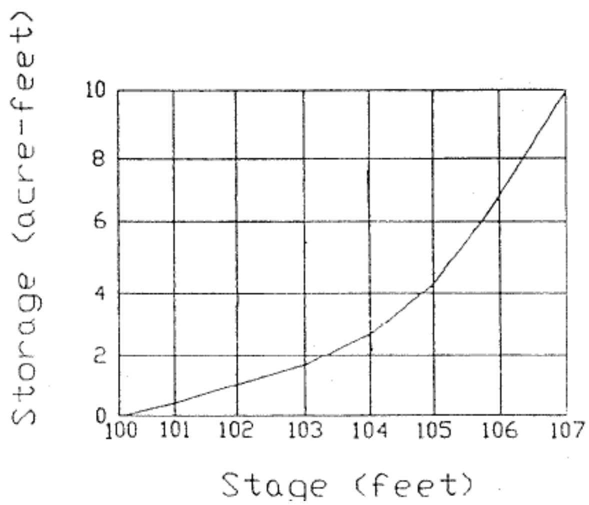


Figure 6-8 Example Stage-Storage Curve

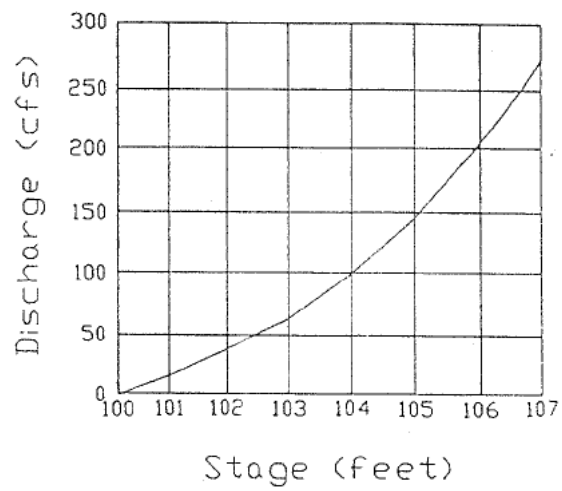


Figure 6-9 Example Stage-Discharge Curve

Step 2: Select a routing time period, Δt, to provide at least five points on the rising limb of the inflow hydrograph ($\Delta t < T_c/5$).

Step 3: Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of $S \pm (O/2)\Delta t$ versus stage. An example tabulation of storage characteristics curve data is shown in Table 6-3.

Table 6-3 Storage Characteristics

(1) Stage (H_1) (ft)	(2) Storage[1] (ft^3)	(3) Discharge[2] (cfs)	(ac-ft/hr)	(4) S-(O/2)Δt (ft^3)	(5) S+(O/2)Δt (ft^3)
100	0.05	0	0	0.05	0.05
101	0.3	15	1.24	0.20	0.40
102	0.8	35	2.89	0.56	1.04
103	1.6	63	5.21	1.17	2.03
104	2.8	95	7.85	2.15	3.45
105	4.4	143	11.82	3.41	5.39
106	6.6	200	16.53	5.22	7.98
107	10.0	275	22.73	8.11	11.89

[1] Obtained from the Stage-Storage Curve.
[2] Obtained from the Stage-Discharge Curve.

Note: For this example, Δt = 10 min = 0.167 hours and 1 cfs = 0.0826 ac-ft/hr.

Step 4: For a given time interval, I_1 and I_2 are known. Given the depth of storage or stage, H_1, at the beginning of that time interval, $S_1-(O_1/2)\Delta t$ can be determined from the appropriate storage characteristics curve (Figure 6-10).

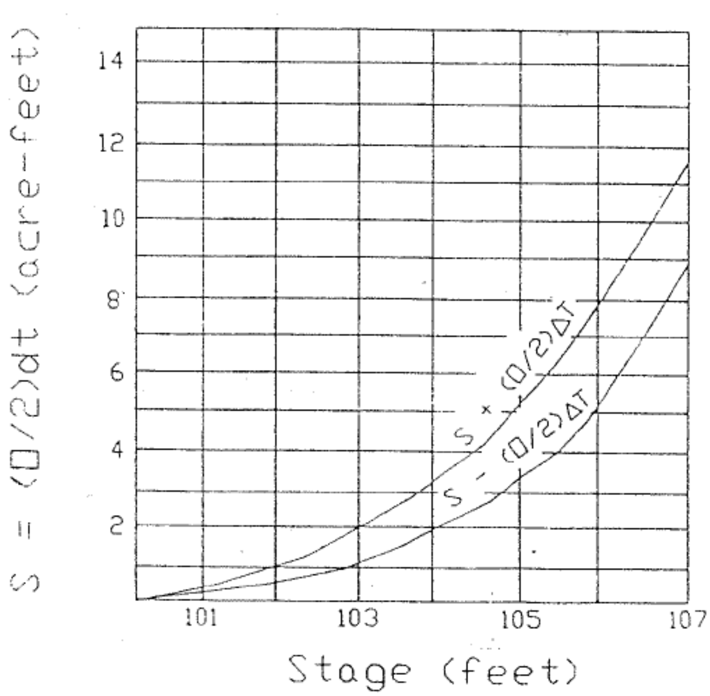


Figure 6-10 Storage Characteristic Curve

Step 5: Determine the value of $S_2 + (O_2/2)\ \Delta t$ from the following equation:

$$\mathbf{S_2 + (O_2/2)\ \Delta t = [S_1 - (O_1/2)\ \Delta t] + [(I_1 + I_2)/2]\Delta t} \quad \textbf{(6.15)}$$

Where: S_2 = storage volume at time 2, ft^3
O_2 = outflow rate at time 2, cfs
Δt = routing time period, sec
S_1 = storage volume at time 1, ft^3
O_1 = outflow rate at time 1, cfs
I_1 = inflow rate at time 1, cfs
I_2 = inflow rate at time 2, cfs
(Other consistent units are equally appropriate.)

Step 6: Enter the storage characteristics curve at the calculated value of $S_2+(O_2/2)\Delta t$ determined in Step 5 and read off a new depth of water, H_2.

Step 7: Determine the value of O_2, which corresponds to a stage of H_2 determined in Step 6, using the stage-discharge curve.

Step 8: Repeat Steps 1 through 7 by setting new values of I_1, O_1, S_1 and H_1 equal to the previous I_2, O_2, S_2 and H_2, and using a new I_2 value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

There are numerous proprietary and non-proprietary (HEC-HMS, DAMS2) computer models that perform reservoir routing. However, the designer must still have a thorough understanding of hydrology and hydraulics design procedures to properly interpret the results.

6.10 Example Problem

6.10.1 Example

This example demonstrates the application of the methodology presented in this chapter for the design of a typical detention storage facility. Example inflow hydrographs and associated peak discharges for both pre- and post-development conditions are assumed to have been developed using hydrologic methods provided in SCS TR-55.

6.10.2 Design Discharge And Hydrographs

Storage facilities are to be designed for runoff from the 2-, 10- and 100-year design storms. The following example only shows the calculations for the 2- and 10-year storms. Actual designs would also include a similar analysis for the 100-year storm. Example peak discharges from the 2- and 10-year design storm events are as follows:

- Pre-development 2-year peak discharge = 150 cfs
- Pre-development 10-year peak discharge = 200 cfs
- Post-development 2-year peak discharge = 190 cfs
- Post-development 10-year peak discharge = 250 cfs

Since the post-development peak discharge must not exceed the pre-development peak discharge, the allowable design discharges are 150 and 200 cfs for the 2- and 10-year storms, respectively.
Example runoff hydrographs are shown in Table 6-4 below. Inflow durations from the post-development hydrographs are about 1.2 and 1.25 hours, respectively, for runoff from the 2- and 10-year storms.

Table 6-4 Example Runoff Hydrographs

Pre-DevelopmentRunoff			Post-DevelopmentRunoff	
(1) Time (Hours)	(2) 2-Year (cfs)	(3) 10-Year (cfs)	(4) 2-Year (cfs)	(5) 10-Year (cfs)
0	0	0	0	0
0.1	18	24	38	50
0.2	61	81	125	178
0.3	127	170	190>150	250>200
0.4	150	200	125	165
0.5	112	150	70	90
0.6	71	95	39	50
0.7	45	61	22	29
0.8	30	40	12	16
0.9	21	28	7	9
1.0	13	18	4	5
1.1	10	15	2	3
1.2	8	13	0	1

6.10.3 Preliminary Volume Calculations

Preliminary estimates of required storage volumes are obtained using the simplified method outlined in section 6.8. For runoff from the 2- and 10-year storms, the required storage volumes, V_S, are computed using equation 6.12:

$$V_S = 0.5T_i(Q_i - Q_o)$$

2-year storm: $V_S = [0.5(1.2 \times 3600)(190 - 150)]/43560$ cu-ft/ac-ft = 1.98 acre-feet
10-year storm: $V_S = [0.5(1.25 \times 3600)(250 - 200)]/43560$ cu-ft/ac-ft = 2.58 acre-feet

6.10.4 Design And Routing Calculations

Stage-discharge and stage-storage characteristics of a storage facility that should provide adequate peak flow attenuation for runoff from both the 2- and 10-year design storms are presented in Table 6-5, below. The storage-discharge relationship was developed by requiring the preliminary storage volume estimates of runoff for both the 2- and 10-year design storms to be provided when the corresponding allowable peak discharges occurred. Discharge values were computed by solving the broad-crested weir equation for head, H, assuming a constant discharge coefficient of 3.1, a weir length of 4 ft and no tailwater submergence.

Table 6-5 Stage-Discharge-Storage Data

(1) Stage (ft)	(2) Q (cfs)	(3) S (ac-ft)	(4) $S_1+(O/2)\Delta t$ (ac-ft)	(5) $S_1-(O/2)\Delta t$ (ac-ft)
0.0	0	0.00	0.00	0.00
0.9	10	0.26	0.30	0.22
1.4	20	0.42	0.50	0.33
1.8	30	0.56	0.68	0.43
2.2	40	0.69	0.85	0.52
2.5	50	0.81	1.02	0.60
2.9	60	0.93	1.18	0.68
3.2	70	1.05	1.34	0.76
3.5	80	1.17	1.50	0.84
3.7	90	1.28	1.66	0.92
4.0	100	1.40	1.81	0.99
4.5	120	1.63	2.13	1.14
4.8	130	1.75	2.29	1.21
5.0	140	1.87	2.44	1.29
5.3	150	1.98	2.60	1.36
5.5	160	2.10	2.76	1.44
5.7	170	2.22	2.92	1.52
6.0	180	2.34	3.08	1.60
6.4	200	2.58	3.41	1.76
6.8	220	2.83	3.74	1.92
7.0	230	2.95	3.90	2.00
7.4	250	3.21	4.24	2.17

Storage routing was conducted for runoff from both the 2- and 10-year design storms to confirm the preliminary storage volume estimates and to establish design water surface elevations. Routing results using the Stage-Discharge-Storage data above and the Storage Characteristics Curves given on Figures 6-8, 6-9, and 6-10, and 0.1-hr time steps are shown in Tables 6-6 and 6-7 below for runoff from the 2- and 10-year design storms, respectively. The preliminary design provides adequate peak discharge attenuation for both the 2- and 10-year design storms.

Table 6-6 Storage Routing For The 2-Year Storm

(1) Time (hrs)	(2) Inflow (cfs)	(3) $[(I_1+I_2)/2]\Delta t$ (acre-ft)	(4) H_1 (ft)	(5) $S_1-(O_1/2)\Delta t$ (acre-ft)	(6) $S_2+(O_2/2)\Delta t$ (acre-ft) (3)+(5)	(7) H_2 (ft)	(8) Outflow (cfs)
0.0	0	0.00	0.00	0.00	0.00	0.00	0
0.1	38	0.16	0.00	0.00	0.16	0.43	3
0.2	125	0.67	0.43	0.10	0.77	2.03	36
0.3	190	1.30	2.03	0.50	1.80	4.00	99
0.4	125	1.30	4.00	0.99	2.29	4.80	130<150 (OK)
0.5	70	0.81	4.80	1.21	2.02	4.40	114
0.6	39	0.45	4.40	1.12	1.57	3.60	85
0.7	22	0.25	3.60	0.87	1.12	2.70	55
0.8	12	0.14	2.70	0.65	0.79	2.02	37
0.9	7	0.08	2.08	0.50	0.58	1.70	27
1.0	4	0.05	1.70	0.42	0.47	1.03	18
1.1	2	0.02	1.30	0.32	0.34	1.00	12
1.2	0	0.01	1.00	0.25	0.26	0.70	7
1.3	0	0.00	0.70	0.15	0.15	0.40	3

Table 6-7 Storage Routing For The 10-Year Storm

(1) Time (hrs)	(2) Inflow (cfs)	(3) $[(I_1+I_2)/2]\Delta t$ (acre-ft)	(4) H_1 (ft)	(5) $S_1-(O_1/2)\Delta t$ (acre-ft)	(6) $S_2+(O_2/2)\Delta t$ (acre-ft) (3)+(5)	(7) H_2 (ft)	(8) Outflow (cfs)
0.0	0	0.00	0.00	0.00	0.00	0.00	0
0.1	50	0.21	0.21	0.00	0.21	0.40	3
0.2	178	0.94	0.40	0.08	1.02	2.50	49
0.3	250	1.77	2.50	0.60	2.37	4.90	134
0.4	165	1.71	4.90	1.26	2.97	2.97	173<200 (OK)
0.5	90	1.05	5.80	1.30	2.35	4.00	137
0.6	50	0.58	4.95	1.25	1.83	4.10	103
0.7	29	0.33	4.10	1.00	1.33	3.10	68
0.8	16	0.19	3.10	0.75	0.94	2.40	46
0.9	9	0.10	2.40	0.59	0.69	1.90	32
1.0	5	0.06	1.90	0.44	0.50	1.40	21
1.1	3	0.03	1.40	0.33	0.36	1.20	16
1.2	1	0.02	1.20	0.28	0.30	0.90	11
1.3	0	0.00	0.90	0.22	0.22	0.60	6

For the routing calculations, the following equation was used: $S_2 + (O_2/2)\Delta t = [S_1 - (O_1/2)\Delta t] + [(I_1+I_2)/2]\Delta t$
Also, column (6) = column (3) + column (5)

Since the routed peak discharge is lower than the maximum allowable peak discharges for both design storm events, the weir length could be increased or the storage decreased. If revisions are desired, routing calculations must be repeated.

Although not shown for this example, runoff from the 100-year storm must be routed through the storage facility to determine if the routed peak discharge is lower than the maximum allowable peak discharge. In addition, the preliminary design provides hydraulic details only. Final design shall consider site constraints such as depth to groundwater, side slope stability and maintenance, grading to prevent standing water and provisions for public safety.

6.11 Retention Storage Facilities

6.11.1 Introduction

Design of retention storage facilities must allow for performance of maintenance activities. The owner’s capability for performing required maintenance shall be considered Provisions for weed control and aeration for prevention of anaerobic conditions shall be considered.

6.11.2 Water Budget

Water budget calculations are required for all permanent pool facilities and shall consider performance for average annual conditions to demonstrate that adequate runoff is available for maintenance of a permanent pool. The water budget shall consider all significant inflows and outflows including, but not limited to, rainfall, runoff, infiltration, exfiltration, evaporation and outflow.

Average annual runoff may be computed using a weighted runoff coefficient for the tributary drainage area, multiplied by the average annual rainfall volume. Infiltration and exfiltration shall be based on site-specific soils testing data. Evaporation may be approximated using the mean monthly pan evaporation or free water surface evaporation data.

6.12 Example Problem

A shallow basin with an average surface area of 3 acres and a bottom area of 2 acres is planned for construction at the outlet of a 10-acre watershed. The watershed is estimated to have a post-development runoff coefficient of 0.3. Site-specific soils testing indicates that the average infiltration rate is about 0.1 in./hr. Determine for average annual conditions if the facility will function as a retention facility with a permanent pool.

<u>Solution</u>

1. From rainfall records, the average annual rainfall is about 30 in.
2. The mean annual evaporation is 19 in.
3. The average annual runoff is estimated as:
   Runoff = (0.3) (30 in.) (100 acres) = 900 ac-in.
4. The average annual evaporation is estimated as:
   Evaporation = (19 in.) (3 ac) = 57 ac-in.
5. The average annual infiltration is estimated as:
   Infiltration = (0.1 in./hr) (24 hrs/day) (365 days/yr) (2 acres)
   Infiltration = 1,752 ac-in.
6. Neglecting basin outflow and assuming no change in storage, the runoff (or inflow) less evaporation and infiltration losses is:
   Net Budget = 900 - 57 - 1,752 = -909 ac-in.
   Thus, the proposed facility will not function as a retention facility with a permanent pool.
7. Revise pool design as follows:
   Average surface area = 1.7 acres and bottom area = 0.8 acre
8. Recompute the evaporation and infiltration
   Evaporation = (19) (1.7) = 32 ac-in.
   Infiltration = (0.1) (24) (365) (0.8) = 700 ac-in.
9. The revised runoff less evaporation and infiltration losses is:
   Net Budget = 900 - 30 - 700 = + 162 ac-in.

The revised facility is assumed to function as a retention facility with a permanent pool.

6.13 Construction And Maintenance Considerations

An important step in the design process is identifying whether special provisions are warranted to properly construct or maintain proposed storage facilities. To provide for acceptable performance and function, storage facilities that require extensive maintenance are discouraged. However, the following maintenance considerations should be viewed generally and should not limit efforts in the creation or enhancement of wetlands, open water habitats, plantings, or other natural/conservation design techniques that can contribute positively to the aesthetic or environmental elements of storage areas, particularly retention basins. In general, facilities shall be designed to minimize maintenance problems typical of urban detention facilities such as:

- weed growth,
- grass and vegetation maintenance,
- sedimentation control,
- bank deterioration,
- standing water or soggy surfaces,
- mosquito control,
- blockage of outlet structures,
- litter accumulation and
- maintenance of fences and perimeter plantings.

Proper design focuses on elimination or reduction of maintenance requirements by addressing the potential for problems to develop.

- Both weed growth and grass maintenance may be addressed by constructing side slopes that can be maintained using available power-driven equipment, such as tractor mowers.
- Sedimentation shall be controlled by constructing traps to contain sediment for easy removal or low-flow channels to reduce erosion and sediment transport.
- Bank deterioration can be controlled with protective lining or by limiting bank slopes.
- Standing water or soggy surfaces may be eliminated by sloping basin bottoms toward the outlet, constructing low-flow pilot channels across basin bottoms from the inlet to the outlet, or by constructing underdrain facilities to lower water tables.
- In general, when the above problems are addressed, mosquito control will not be a major problem.
- Outlet structures shall be selected to minimize the possibility of blockage (i.e., very small pipes tend to block quite easily and shall be avoided).
- One way to deal with the maintenance associated with litter and damage to fences and perimeter plantings is to locate the facility for easy access so this maintenance can be conducted on a regular basis.
- Access easements shall be provided for heavy equipment when facilities do not abut public right-of-way Access for vehicular maintenance shall be provided to the control structure, along side(s) of the storage pond as necessary (15-foot minimum width), and to the basin bottom for facilities with bottom widths greater than 15 feet. When a facility abuts a City right-of-way such as a local or arterial street, maintenance access from the abutting City right-of-way is an option which may be acceptable if it will not result in an unsafe or otherwise unworkable conditions.
- Retention storage, which proposes a permanent pool in addition to detention, shall be constructed to facilitate silt removal and disposal.
- An outlet shall be provided that will allow the retention facilities to be completely drained when required for silt removal, maintenance, or inspection.
- Provisions shall be made for the deposit of silt removed from the stilling basin and/or the main pool.

6.14 Protective Treatment

Protective treatment may be required to prevent entry to facilities that present a hazard to children and, to a lesser extent, all persons. Fences may be required for detention areas where one or more of the following conditions exist:

- Rapid stage increases would make escape practically impossible where small children frequent the area.
- Water depths either exceed 2.5 ft for more than 24 hrs or are permanently wet.
- A low-flow watercourse or ditch passing through the detention area has a depth greater than 5 ft or a flow velocity greater than 5 ft/s.

Guards or grates may be appropriate for other conditions, but in all circumstances heavy debris must be transported through the detention area. In some cases, it may be advisable to fence the watercourse or ditch rather than the detention area.

Fencing should be considered for normally dry storage facilities with design depths in excess of 2.5 ft for 24 hrs, unless the area is within a fenced, limited access facility.

6.15 Trash Racks And Safety Grates

Trash racks and safety grates serve several functions:

- They trap larger debris well away from the entrance to the outlet works where they will not clog the critical portions of the works;
- They trap debris in such a way that relatively easy removal is possible;
- They keep people and large animals out of confined conveyance and outlet areas; and
- They provide a safety system whereby persons caught in them will be stopped prior to the very high velocity flows immediately at the entrance to outlet works and persons will be carried up and onto the outlet works allowing for a possibility to climb to safety.

Well-designed trash racks serve these purposes without interfering significantly with the hydraulic capacity of the outlet (or inlet in the case of conveyance structures) (ASCE, 1985, Allred-Coonrod, 1991). The location and size of the trash rack depends on a number of factors including: head losses through the rack, structural convenience, safety, and size of outlet.

Trash racks at entrances to pipes and conduits should be sloped at about 3:1 to 5:1 to allow trash to slide up the rack with flow pressure and rising water level, the slower the approach flow, the flatter the angle. Rack opening rules-of-thumb abound in the literature. Figure 6-11 gives opening estimates based on outlet diameter (UDFCD, 1992). Judgment should be used in areas with larger debris (e.g. a wooded area) that may require more opening space.

The bar opening space for small pipes shall be less than the pipe diameter. Collapsible racks have been used in some places if clogging becomes excessive or a person becomes pinned to the rack. Alternately, debris for culvert openings can be caught upstream from the opening by using pipes placed in the ground or a chain safety net (USBR, 1978, UDFCD, 1991). Racks can be hinged on top to allow for easy opening and cleaning.

The control for the outlet shall not shift to the grate. Nor shall the grate cause the headwater to rise above planned levels. Therefore, headlosses through the grate shall be calculated. A number of empirical loss equations exist, though many have difficult-to-estimate variables. For a discussion of headloss related to grates with example empirical loss equations, see Debo & Reese, 1994.

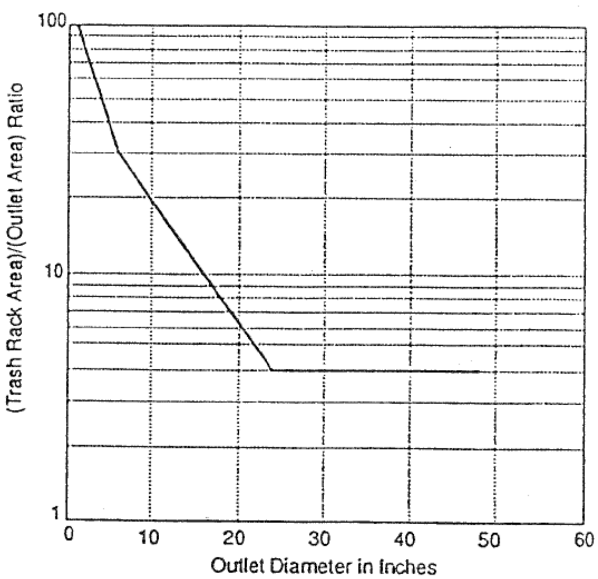


Figure 6-11 Minimum Rack Size vs. Outlet Diameter

Source: UDCFD, 1992

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CHAPTER 7

ENERGY DISSIPATORS

March 7, 2011

Chapter 7 - Energy Dissipators

Table Of Contents

7.1 Introduction

7.1.1 Overview

The failure or damage of many culverts and detention basin outlet structures can be traced to unchecked erosion. Erosive forces which are at work in the natural drainage network are often exacerbated by the construction of a highway or by other urban development. Interception and concentration of overland flow and constriction of natural waterways inevitably results in an increased erosion potential. To protect the culvert or other outlet device and adjacent areas, it is sometimes necessary to employ an energy dissipator.

A common failure made at storm sewer outlets to streams is erosion of the area immediately beneath the outlet of the storm sewer. Design to resist this failure mode is presented in section 7.10.

7.1.2 Definition

Energy dissipators are any device designed to protect downstream areas from erosion by reducing the velocity of flow to acceptable limits.

7.1.3 Purpose

This chapter provides:

- Design procedures which are based on FHWA Hydraulic Engineering Circular Number 14 (HEC 14) "Hydraulic Design of Energy Dissipators for Culverts and Channels," September 1983, revised in 1995.
- Results of analysis using the HYDRAIN system and the HY8 software.

7.1.4 Symbols

Table 7-1 Symbols, Definitions And Units

Symbol	Definition	Units
A	Cross sectional area	ft^2
A_o	Area of flow at culvert outlet	ft^2
d_E	Equivalent depth at brink	ft
d_o	Normal flow depth at brink	ft
D	Height of culvert	ft
d_{50}	Mean diameter of riprap	ft
DI	Discharge Intensity Modified	-
Fr	Froude Number	-
g	Acceleration due to gravity	ft/s^2
h_S	Depth of dissipator pool	ft
L	Length of culvert	ft
L_B	Overall length of basin	ft
L_S	Length of dissipator pool	ft
Q	Rate of discharge	cfs
Q_{OT}	Overtopping flow	cfs
S_o	Slope of streambed	ft/ft
TW	Tailwater depth	ft
V_d	Velocity downstream	ft/s
V_L	Velocity — (L) feet from brink	ft/s
V_o	Normal velocity at brink	ft/s
W_B	Width of basin	ft
W_o	Diameter or width of culvert	ft
W_S	Width of scour hole	ft

7.2 Design Criteria

7.2.1 Overview

Energy dissipators should be employed whenever the velocity of flow leaving a stormwater management facility exceeds the erosion velocity of the downstream channel system. Several standard energy dissipator designs have been documented by the U.S. Department of Transportation including hydraulic jump, forced hydraulic jump, impact basins, drop structures, stilling wells, and riprap. This chapter will concentrate on those energy dissipators most applicable to urban stormwater management problems. In addition, throughout this chapter culvert outlets will be referred for energy dissipation design. However, dissipators can be used downstream from any outlet device.

7.2.2 Dissipator Type Selection

The dissipator type selected for a site must be appropriate to the location. In this chapter, the terms “internal” and “external” are used to indicate the location of the dissipator in relationship to the culvert. An external dissipator is located outside of the culvert and an internal dissipator is located within the culvert barrel.

1. Internal dissipators

Containing the hydraulic jump within the culvert is a form of internal energy dissipation. The Nebraska Department of Roads has issued a report titled "Hydraulic Analysis of Broken-Back Culverts" (January 1998) that contains a design procedure for this type of culvert.

Internal dissipators are used where:

- the scour hole at the culvert outlet is unacceptable,
- the right-of way is limited,
- debris is not a problem, and
- moderate velocity reduction is needed.

2. Natural scour holes

Natural scour holes are used where:

- undermining of the culvert outlet will not occur or it is practicable to be checked by a cutoff wall,
- the expected scour hole will not cause costly property damage, and
- there is no nuisance effect.

3. External dissipators

External dissipators are used where:

- the outlet scour hole is not acceptable,
- moderate amount of debris is present, and
- the culvert outlet velocity (V_o) is moderate, $Fr < 3$.

4. Stilling Basins

Stilling Basins are used where:

- the outlet scour hole is not acceptable,
- debris is present, and
- the culvert outlet velocity (V_o) is high, $Fr > 3$.

7.2.3 Design Limitations

Ice Buildup

If ice buildup is a factor, it shall be mitigated by:

- sizing the structure to not obstruct the winter low flow, and
- using external dissipators.

Debris Control

Debris control shall be designed using Hydraulic Engineering Circular No. 9, "Debris-Control Structures" and shall be considered:

- where clean-out access is limited, and
- if the dissipator type selected cannot pass debris.

Flood Frequency

The flood frequency used in the design of the energy dissipator device shall be the same flood frequency used for the culvert design. The use of a greater frequency is permitted, if justified by:

- low risk of failure of the crossing,
- substantial cost savings,
- limited or no adverse effect on the downstream channel, and
- limited or no adverse effect on downstream development.

Maximum Culvert Exit Velocity

The culvert exit velocity shall be consistent with the maximum velocity in the natural channel or shall be mitigated by using:

- channel stabilization (See Chapter 5, Open Channels), and
- energy dissipation.

Tailwater Relationship

The hydraulic conditions downstream shall be evaluated to determine a tailwater depth and the maximum velocity for a range of discharges. Refer to:

- Open channels (See Chapter 5, Open Channels).
- Lake, pond, or large water body shall be evaluated using the high water elevation that has the same frequency as the design flood for the culvert. (See the Lancaster County Flood Insurance Study for the appurtenant stream information).

7.2.4 Design Options

Material Selection

The material selected for the dissipator shall be based on a comparison of the total cost over the design life of alternate materials and shall not be made using first cost as the only criteria. This comparison shall consider replacement cost and the difficulty of construction as well as traffic delay.

Culvert Outlet Type

In choosing a dissipator, the selected culvert end treatment has the following implications.

- Culvert ends which are projecting or mitered to the fill slope offer no outlet protection.

Energy Dissipators

- Headwalls provide embankment stability and erosion protection. They provide protection from buoyancy and reduce damage to the culvert.
- Commercial end sections add little cost to the culvert and may require less maintenance, retard embankment erosion and incur less damage from maintenance.
- Aprons do not reduce outlet velocity, but if used shall extend at least one culvert height downstream. They shall not protrude above the normal streambed elevation.
- Wingwalls are used where the side slopes of the channel are unstable, where the culvert is skewed to the normal channel flow, to redirect outlet velocity, or to retain fill.

Safety Considerations

Traffic shall be protected from external energy dissipators by locating them outside the appropriate "clear zone" distance per the AASHTO Roadside Design Guide or shielding them with a traffic barrier.

Weep Holes

If weep holes are used to relieve uplift pressure, they shall be designed in a manner similar to underdrain systems.

7.2.5 Related Designs

Culvert

The culvert shall be designed independently of the dissipator design (see Design of Culverts, Chapter 4) with the exception of internal dissipators which may require an iterative solution. The culvert design shall be completed before the outlet protection is designed and shall include computation of outlet velocity.

Downstream Channel

The downstream channel protection shall be designed concurrently with dissipator design (See Chapter 5, Open Channel Hydraulics).

7.2.6 Computational Methods

Charts

- Charts are required for a manual solution.
- Charts required for the design of scour holes, riprap basins, USBR type VI impact basins and SAF basins are included in this Chapter. Charts required for the design of other types of energy dissipators are found in HEC 14.

Computer Software

- HY-8 (FHWA Culvert Analysis Software) Version 4.1 or greater, contains an energy dissipator module which can be used to analyze most types of energy dissipators in HEC 14.

7.3 Design Equations

7.3.1 Culvert Outlet Conditions

The culvert design establishes the outlet flow conditions. However, these parameters may require closer analysis for energy dissipator design.

Depth (ft), d_o.

- The normal depth assumption should be reviewed and a water surface profile calculated if L < 50 do.
- The brink depth (see HEC 14 for curves) should be used for mild slopes and low tailwater, not critical depth.

Area (ft^2), A_o.

The cross sectional area of flow at the culvert outlet should be calculated using (d_o).

Velocity (ft/s), V_o

The culvert outlet velocity should be calculated as follows:

$$\mathbf{V_o = Q/A_o} \tag{7.1}$$

Where: Q = discharge, cfs

Froude Number, Fr

The Froude number is a flow parameter traditionally used to design energy dissipators and is calculated using:

$$\mathbf{Fr = V_o/[(g\ d_o)^{0.5}]} \tag{7.2}$$

Where: g = acceleration of gravity, 32.2 ft/s^2

Equivalent Depth (ft), $d_E = (A_o/2)^{0.5}$

Equivalent depth is an artificial depth calculated for culverts which are not rectangular so a reasonable Fr can be determined.

Discharge Intensity, $DI_{c.}$

Discharge intensity is a flow parameter similar to Fr, used for circular culverts of diameter (D) which are flowing full.

$$\mathbf{DI_c = Q/(g^{0.5}D^{2.5})} \tag{7.3}$$

Discharge Intensity Modified, DI.

Referring to the new Chapter V, HEC 14, 1995, the Modified Discharge Intensity, DI, for all culvert shapes are:

$$\mathbf{DI = Q/(g^{0.5}\ R_c^{\ 2.5})} \tag{7.4}$$

Where: Q = discharge, cfs
A_c = culvert area, ft^2
P_c = culvert perimeter, ft
R_c = (A_c/P_c)

7.3.2 Scour Hole Estimation

Chapter V of HEC 14 (revised version, 1995) contains an estimating procedure for scour hole geometry based on soil, flow data and culvert geometry. This scour prediction procedure is intended to serve together with the maintenance history and site reconnaissance information for determining energy dissipator needs.

Only scour holes on cohesionless material will be discussed in this Chapter. For scour holes on cohesive soil, the designer can refer to the above-mentioned Chapter V, HEC 14 for detail.

The results of the tests made by the US Army Waterways Experiment Station, Vicksburg, Mississippi indicate that the scour hole geometry varies with the tailwater conditions. The maximum scour geometry occurs at tailwater depths less than half the culvert height. The maximum depth of scour, d_S, occurs at a location approximately $0.4L_S$ downstream of the culvert, where L_S is the length of the scour.

The following empirical equations defining the relationship between the culvert discharge intensity, time and the length, width, depth and volume of the scour hole are presented for the maximum or extreme scour case.

$$\left[\frac{d_s}{R_c}, \frac{W_s}{R_c}, \frac{L_s}{R_c}\right] = C_s\, C_h \left(\frac{\alpha}{\sigma^{1/3}}\right)\left(\frac{Q}{\sqrt{g}R_c^{2.5}}\right)^{\beta}\left(\frac{t}{316}\right)^{\theta} \tag{7.5}$$

Where: d_S = maximum depth of scour hole, ft
L_S = length of scour hole, ft
W_S = width of scour hole, ft

$$\mathbf{d_S, W_S, \text{ or } L_S = (F_1)(F_2)(F_3)R_c} \tag{7.6}$$

Where:

$$F_1 = C_s C_h \left(\frac{\alpha}{\sigma^{1/3}}\right)$$

$$F_2 = \left(\frac{Q}{\sqrt{g}R_c^{2.5}}\right)^{\beta} = (DI)^{\beta}$$

$$F_3 = \left(\frac{t}{316}\right)^{\theta}$$

Where: t = 30 min or the time of concentration, if longer
R_c = hydraulic radius of drainage structure flowing full
σ = material standard deviation, generally, $\sigma = 2.10$ for gravel and 1.87 for sand
α, ß, θ, C_S and C_h are coefficients, as shown in Table 7-2
F_1 , F_2 and F_3 are factors to aid the computation, as shown in Step 7B, Figure 7-1

7.4 Design Procedure

The following design procedures are intended to provide a convenient and organized method for designing energy dissipators by hand. The designer should be familiar with all the equations in section 7.3 before using these procedures. In addition, application of the following design method without an understanding of hydraulics can result in an inadequate, unsafe, or costly structure.

Step 1: Assemble Site Data And Project File

Step 2: Determine Hydrology

Step 3: Select Design Q

Step 4: Design Downstream Channel
a. Determine channel slope, cross section, normal depth and velocity.
b. Check bed and bank materials stability.

Step 5: Design Culvert/Outlet

Step 6: Summarize Data On Design Form, Figure 7-1

Step 7: Estimate Scour Hole Size
a. Enter input for scour equation on Figure 7-1.
b. Calculate d_S, W_S, L_S , using equations 7.5 or 7.6

Step 8: Determine Need For Dissipator

An energy dissipator is needed if:

a. the estimated scour hole dimensions, which exceed the allowable right-of-way, undermine the culvert cutoff wall, or present a safety or aesthetic problem;
b. downstream property is threatened; or
c. V_o is much greater than V_d.

Step 9: Select Design Alternative

a. Calculate Froude number, Fr.
b. Choose energy dissipator types.
   If Fr > 3, design a SAF stilling basin.
   If Fr < 3, design a riprap basin or design a USBR Type VI, if Q < 400 cfs for each barrel and little debris is expected. If these are not acceptable or economical, try other dissipators in HEC 14.

Step 10: Design Dissipators

a. Use the following design procedures and charts:
   - Section 7.6 for the SAF.
   - Section 7.7 for the RIPRAP.
   - Section 7.8 for the USBR Type VI, (Baffled Outlet)
   - Section 7.9 for the Riprap Aprons.

Step 11: Design Riprap Transition

a. Most dissipators require some protection adjacent to the basin exit.
b. The length of protection can be judged based on the difference between V_o and V_d. The riprap should be designed using HEC 11.

Step 12: Review Results

a. If downstream channel conditions (velocity, depth and stability) are exceeded, either:
   - design riprap for channel, Step 4, or
   - select another dissipator, Step 9.
b. If preferred energy dissipator affects culvert hydraulics, return to Step 5 and calculate culvert performance.
c. If debris-control structures are required upstream, consult HEC 9.
d. If a check Q was used for the culvert design, assess the dissipator performance with this discharge.

Step 13: Documentation

Table 7-2

A. Coefficient for Culvert Outlet Scour - Cohesionless Materials

	α	β	θ
Depth, d_S	2.27	0.39	0.06
Width, W_S	6.94	0.53	0.08
Length, L_S	17.10	0.47	0.10
Volume, V_S	127.08	1.24	0.18

B. Coefficient C_S for Outlets Above the Bed

H_S	Depth	Width	Length	Volume
0	1.00	1.00	1.00	1.00
1	1.22	1.51	0.73	1.28
2	1.26	1.54	0.73	1.47
4	1.34	1.66	0.73	1.55
H_S is the height above bed in pipe diameters, ft				

C. Coefficient C_h for Culvert Slope

Slope %	Depth	Width	Length	Volume
0	1.00	1.00	1.00	1.00
2	1.03	1.28	1.17	1.30
5	1.08	1.28	1.17	1.30
>7	1.12	1.28	1.17	1.30

Figure 7-1 : ENERGY DISSIPATOR CHECKLIST

Project ______________________________

Designer ______________________________ Date __________

Reviewer ______________________________ Date __________

SCOUR EQUATIONS

$$\left[\frac{d_s}{R_c},\frac{W_s}{R_c},\frac{L_s}{R_c}\right] = C_s\,C_h\left(\frac{\alpha}{\sigma^{1/3}}\right)\left(\frac{Q}{\sqrt{g}R_c^{2.5}}\right)^{\beta}\left(\frac{t}{316}\right)^{\theta}$$

$$d_s, W_s, L_s = C_s\,C_h\left(\frac{\alpha}{\sigma^{1/3}}\right)(DI)^{\beta}\left(\frac{t}{316}\right)^{\theta} R_c$$

d_S, W_S, or $L_S = (F_1)(F_2)(F_3)R_c$

STEP 6 - DATA SUMMARY

Parameters	Culvert	Channel
Station		
Control		
Type		
Height, D		
Width, B		
Length, L		
Material		
Manning's n		
Side Slope		
Discharge, Q		
Depth, d		
Velocity, V		
$Fr=V/(gd)^{0.5}$		
Flow Area, A		
Slope		

STEP 7A - EQUATION INPUT DATA

FACTOR	VALUE
Q = Discharge, cfs	
A_c= Culvert area, ft^2	
P_c= Perimeter, ft	
$R_c = A_c / P_c$	
DI= Discharge Intensity	
t = time of concentration	

STEP 7B - SCOUR COMPUTATION

Factor	Depth	Width	Length
α			
β			
θ			
F_1			
F_2			
F_3			
$[F_1][F_2][F_3]R_c$			
Allowable			

If calculate scour > Allowable and:

1. Fr > 3, design a SAF basin
2. Fr < 3, design a riprap basin
3. Fr < 3, design a USBR type VI

7.5 Design Example

7.5.1 Design Example Steps

Step 1: Assemble Site Data And Project File

a. Site survey - The culvert project file contains USGS site and location maps, roadway profile and embankment cross sections. Site visit notes indicate no sediment or debris problems and no nearby structures.
b. Studies by other agencies - none.
c. Environmental, risk assessment shows no problems.
d. Design criteria:
   - 25-year frequency for design, and
   - 100-year frequency for check.

Step 2: Determine Hydrology

For the purpose of this example, use

- Q_{25} = 400 cfs
- Q_{100} = 500 cfs

Step 3: Select Design Q

Use Q_{25} = 400 cfs, as requested by the design criteria.

Step 4: Design Downstream Channel

a. Cross section of channel with slope = 0.05 ft/ft

Point	Station, ft	Elevation, ft
1	12	180
2	22	175
3	32	174.5
4	34	172.5
5	39	172.5
6	41	174.5
7	51	175
8	61	180

b. Rating Curve for Channel
Calculated using methods contained in Chapter 5.

Q (cfs)	TW (ft)	V (ft/s)
100	1.4	11
200	2.1	14
300	2.5	16
400	2.8	18
500	3.1	19

c. At a V_{25} = 18 ft/s, the 3-inch gravel material which makes up the channel boundary is not stable and riprap is needed (See Chapter 5, Open Channels) for a transition.

Step 5: Design Culvert

A 7 ft × 6 ft RCB with a beveled entrance on a slope of 0.05 ft/ft was the selected design. The FHWA HY8 program showed that this culvert is operating at inlet control and has:

	Q (cfs)	HW_i (ft)	V_o (ft/s)
Q_{25} =	400	7.6	32
Q_{ot} =	430	8.5	
Q_{100} =	500	8.6	34

Step 6: Summarize Data On Design Form

See Figure 7-2.

Step 7: Size Scour Hole

The size of the scour hole is determined using equations 7.5 and 7.6. For channel with gravel bed, the standard deviation of the material, σ is 2.10. Table 7-2 shows that the value of C_S =1.00 and C_h = 1.08 for depth, 1.28 for width, 1.17 for length and 1.30 for volume calculations. See Figure 7-2 for a summary of the computation.

Step 8: Determine Need For Dissipator

The scour hole dimensions are excessive, and since V_o = 32 ft/s is much greater than V_d = 18 ft/s, an energy dissipator is needed.

Step 9: Select Design Alternative

Since the Fr > 3, an SAF stilling basin should be used.

Step 10: Design Dissipators

The design of an SAF stilling basin is as shown in Section 7.6, Figure 7-3.

Step 11: Design Riprap Transition

Protection is required (See HEC 11).

Step 12: Review Results

The downstream channel conditions are matched by the dissipator.

Step 13: Documentation

Figure 7-2 : ENERGY DISSIPATOR CHECKLIST

Project ____________________

Designer ____________________ Date ________

Reviewer ____________________ Date ________

SCOUR EQUATIONS

$$\left[\frac{d_s}{R_c},\frac{W_s}{R_c},\frac{L_s}{R_c}\right] = C_s\,C_h\left(\frac{\alpha}{\sigma^{1/3}}\right)\left(\frac{Q}{\sqrt{g}R_c^{2.5}}\right)^{\beta}\left(\frac{t}{316}\right)^{\theta}$$

$$d_s, W_s, L_s = C_s\,C_h\left(\frac{\alpha}{\sigma^{1/3}}\right)(DI)^{\beta}\left(\frac{t}{316}\right)^{\theta}R_c$$

d_S, W_S, or $L_S = (F_1)(F_2)(F_3)R_c$

STEP 6 - DATA SUMMARY

Parameters	Culvert	Channel
Station	125+50	4+00
Control	Inlet	Super.
Type	RCB	Natural
Height, D	6 ft	7.5 ft
Width, B	7 ft	29 ft
Length, L	300 ft	---
Material	Concrete	Gravel
Manning's n	0.012	0.03 & 0.08
Side Slope	---	1:1
Discharge, Q	400 cfs	400 cfs
Depth, d	1.8 ft	2.8 ft
Velocity, V	32 ft/s	18 ft/s
$Fr=V/(gd)^{0.5}$	4.2	1.9
Flow Area, A	12.5 ft^2	22.2 ft^2
Slope	0.05 ft/ft	0.05 ft/ft

STEP 7A - EQUATION INPUT DATA

FACTOR	VALUE
Q = Discharge, cfs	400 cfs
A_c= Culvert area, ft^2	42 ft^2
P_c= Perimeter, ft	26 ft
$R_c = A_c / P_c$	1.62
DI= Discharge Intensity	1.32
t = time of concentration	30 min

STEP 7B - SCOUR COMPUTATION

Factor	Depth	Width	Length
α	7.96	26.42	64.54
β	0.26	0.62	0.56
θ	0.09	0.06	0.17
F_1	0.63	0.54	0.62
F_2	8.6	31.4	75.4
F_3	0.8	0.9	0.7
$[F_1][F_2][F_3]R_c$	7	28	53
Allowable	7 (OK)	29 (OK)	60 (OK)

If calculated scour > Allowable and:

1. Fr > 3, design a SAF basin
2. Fr < 3, design a riprap basin
3. Fr < 3, design a USBR type VI

* These values are not standards. They may vary, depending on design criteria. In this case, calculated scour > Allowable and Fr>3: **Recommend a SAF Basin**.

7.5.2 Computer Output

The scour hole geometry can also be computed by using the "Energy Dissipators" module of the FHWA microcomputer program HY-8, Culvert Analysis, Version 4.0 or later. A hardcopy of the output of the module is shown below. The dimensions of the scour hole computed by the HY-8 program are in agreement with the values calculated in the previous section.

FHWA CULVERT ANALYSIS, HY-8, VERSION 6.0

CURRENT DATE	CURRENT TIME	FILE NAME	FILE DATE

CULVERT AND CHANNEL DATA

CULVERT NO. 1	DOWNSTREAM CHANNEL
CULVERT TYPE: 7.0 ft × 6.0 ft BOX	CHANNEL TYPE : IRREGULAR
CULVERT LENGTH = 300 ft	BOTTOM WIDTH = 7.0 ft
NO. OF BARRELS = 1.0	TAILWATER DEPTH = 2.8 ft
FLOW PER BARREL = 400 cfs	TOTAL DESIGN FLOW = 400.0 cfs
INVERT ELEVATION = 172.5 ft	BOTTOM ELEVATION = 172.5 ft
OUTLET VELOCITY = 31.3 ft/s	NORMAL VELOCITY = 17.5 ft/s
OUTLET DEPTH = 2.02 ft	

SCOUR HOLE GEOMETRY AND SOIL DATA

LENGTH = 91.4 ft	WIDTH = 49.3 ft
DEPTH = 9.2 ft	VOLUME = 4609.7 ft^3

MAXIMUM SCOUR OCCURS 36.6 ft DOWNSTREAM OF CULVERT

SOIL TYPE : NONCOHESIVE

SAND SIZES:
D16 = 8 mm
D50 = 14 mm
D84 = 18 mm

7.6 Riprap Aprons

7.6.1 Uses

A flat riprap apron can be used to prevent erosion at the transition from a pipe or box culvert outlet to a natural channel. Protection is provided primarily by having sufficient length and flare to dissipate energy by expanding the flow. Riprap aprons are appropriate when the culvert outlet Fr is less than or equal to 2.5. The HY-8 computer program does not include design of riprap aprons.

7.6.2 Design Procedure

The procedure presented in this section is taken from USDA, SCS (1975). Two sets of curves, one for minimum and one for maximum tailwater conditions, are used to determine the apron size and the median riprap diameter, d_{50}. If tailwater conditions are unknown, or if both minimum and maximum conditions may occur, the apron should be designed to meet criteria for both. Although the design curves are based on round pipes flowing full, they can be used for partially full pipes and box culverts. The design procedure consists of the following steps:

Step 1: If possible, determine tailwater conditions for the channel. If tailwater is less than one-half the discharge flow depth (pipe diameter if flowing full), minimum tailwater conditions exist and the curves in Figure 7-3 apply. Otherwise, maximum tailwater conditions exist and the curves in Figure 7-4 should be used.

Step 2: Determine the correct apron length and median riprap diameter, d_{50}, using the appropriate curves from Figures 7-3 and 7-4. If tailwater conditions are uncertain, find the values for both minimum and maximum conditions and size the apron as shown in Figure 7-5.

a. For pipes flowing full:

Use the depth of flow, d, which equals the pipe diameter, in feet, and design discharge, in cfs, to obtain the apron length, L_a, and median riprap diameter, d_{50}, from the appropriate curves.

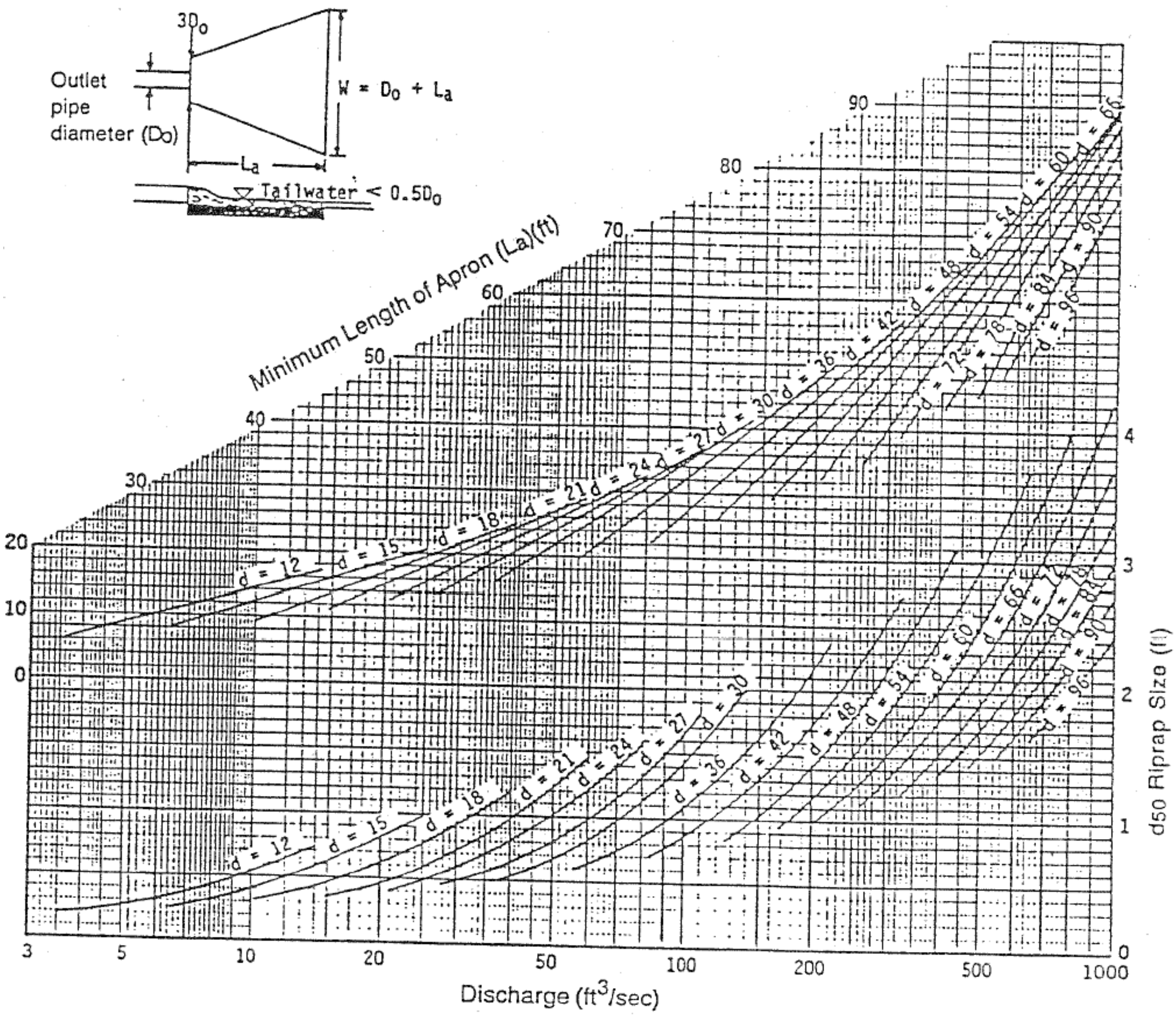
b. For pipes flowing partially full:

Use the depth of flow, d, in feet, and velocity, v, in feet/second. On the lower portion of the appropriate figure, find the intersection of the d and v curves, then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curves until intersecting the curve for the correct flow depth, d. Find the minimum apron length, $L_{a,}$ from the scale on the left.

c. For box culverts:

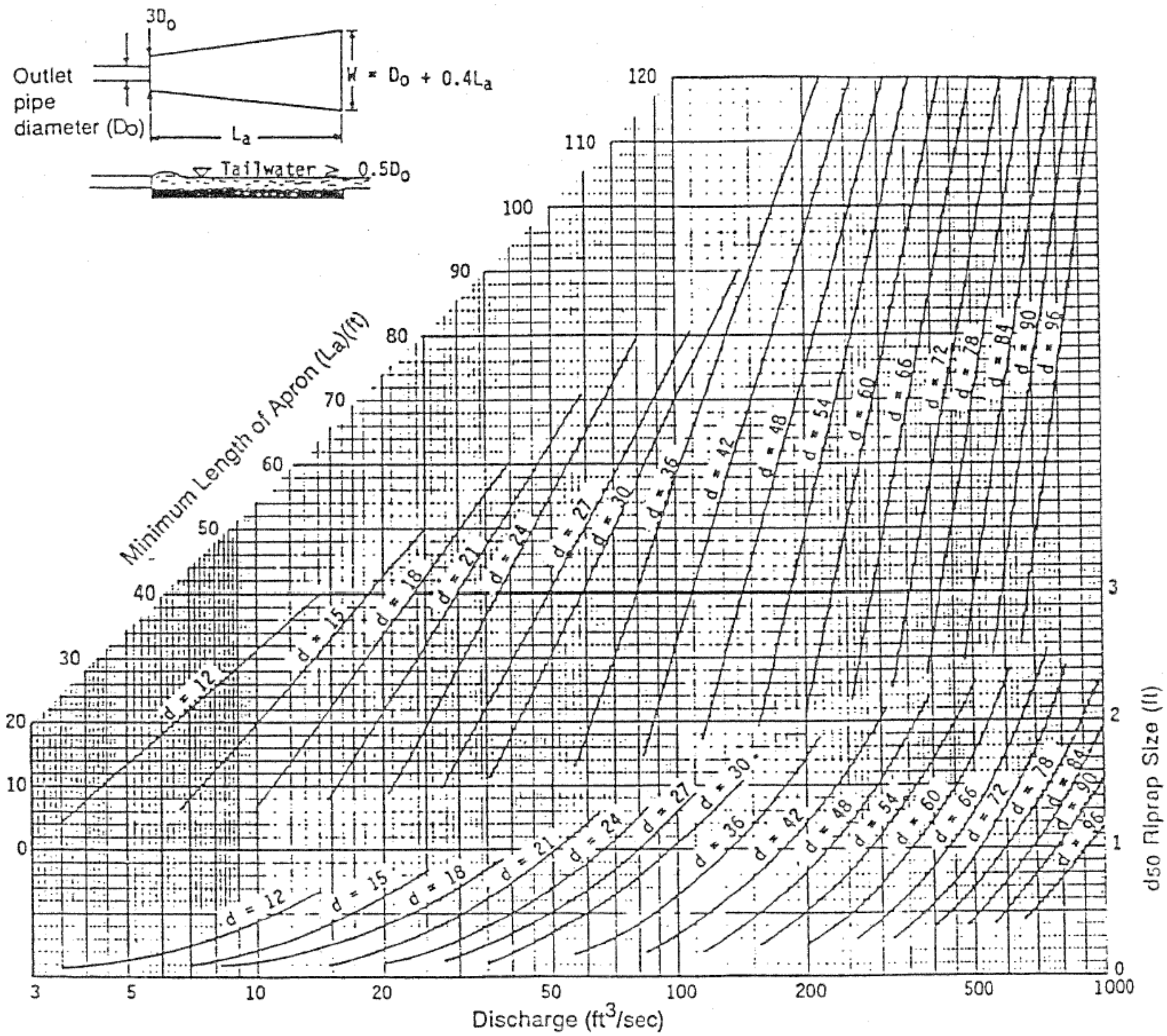
Use the depth of flow, d, in feet, and velocity, v, in feet/second. On the lower portion of the appropriate figure, find the intersection of the d and v curves, then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curve until intersecting the curve equal to the flow depth, d. Find the minimum apron length, L_a, using the scale on the left.

Step 3: If tailwater conditions are uncertain, the median riprap diameter should be the larger of the values for minimum and maximum conditions. The dimensions of the apron will be as shown in Figure 7-5. This will provide protection under either of the tailwater conditions.



Curves may not be extrapolated.

Figure 7-3 Design of Riprap Apron Under Minimum Tailwater Conditions



Curves may not be extrapolated.

Figure 7-4 Design Of Riprap Apron Under Maximum Tailwater Conditions

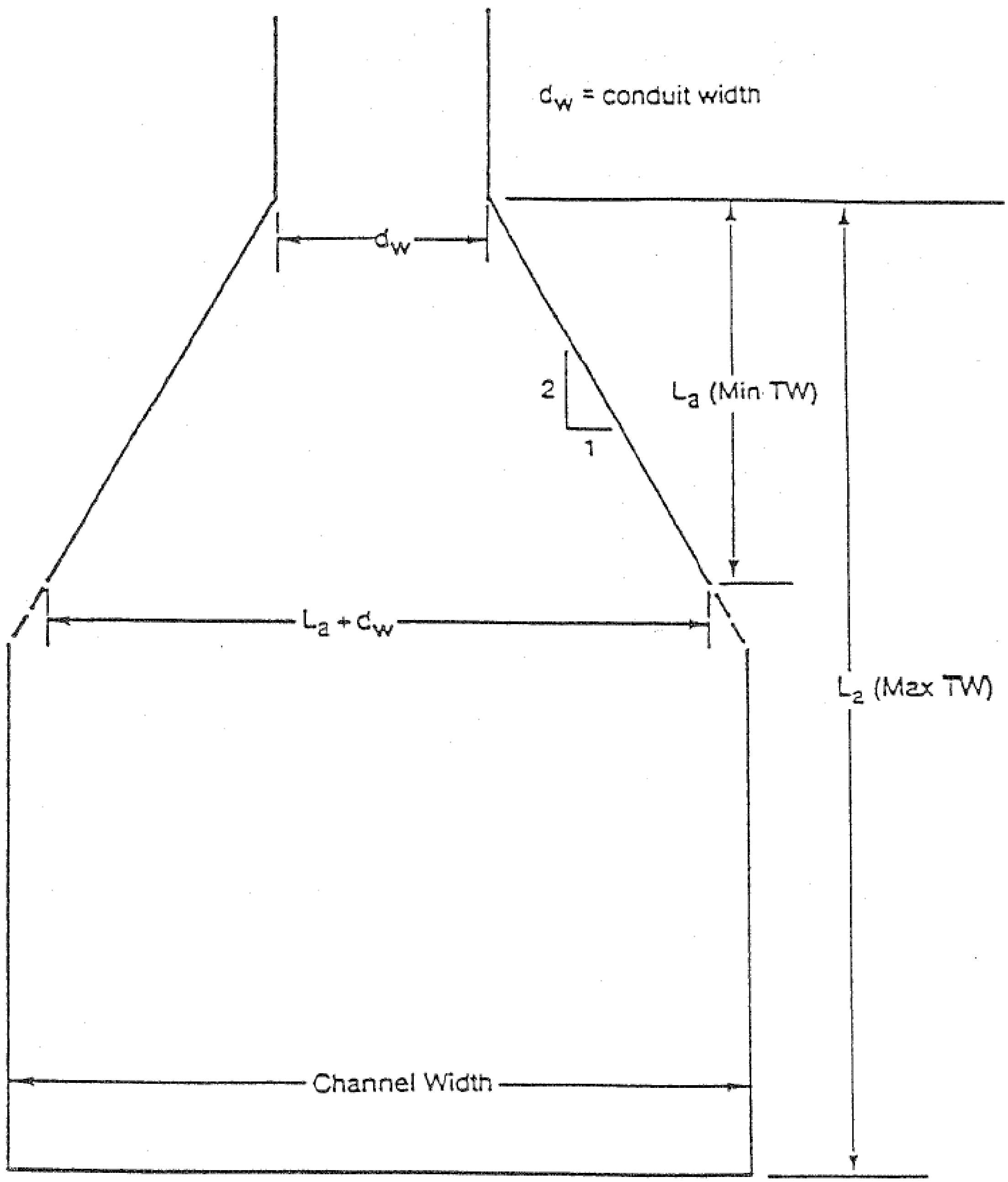


Figure 7-5 Riprap Apron Schematic For Uncertain Tailwater Conditions

7.6.3 Design Considerations

The following items should be considered during riprap apron design:

1. The maximum stone diameter should be 1.5 times the median riprap diameter.

   $d_{max} = 1.5 \times d_{50}$

   d_{50} = the median stone size in a well-graded riprap apron.

2. The riprap thickness should be 1.5 times the maximum stone diameter or 6 inches, whichever is greater.

   Apron thickness = $1.5 \times d_{max}$

   (Apron thickness may be reduced to $1.5 \times d_{50}$ when an appropriate filter fabric is used under the apron.)

3. The apron width at the discharge outlet should be at least equal to the pipe diameter or culvert width, d_w. Riprap should extend up both sides of the apron and around the end of the pipe or culvert at the discharge outlet at a maximum slope of 2:1 and a height not less than the pipe diameter or culvert height, and should taper to the flat surface at the end of the apron.

4. If there is a well-defined channel, the apron length should be extended as necessary so that the downstream apron width is equal to the channel width. The sidewalls of the channel should not be steeper than 2:1.

5. If the ground slope downstream of the apron is steep, channel erosion may occur. The apron should be extended as necessary until the slope is gentle enough to prevent further erosion.

6. The potential for vandalism should be considered if the rock is easy to carry. If vandalism is a possibility, the rock size must be increased or the rocks held in place using concrete or grout.

7.6.4 Example Problems

Example - Riprap Apron Design for Minimum Tailwater Problem Conditions

A flow of 280 cfs discharges from a 66-inch pipe with a tailwater of 2 ft above the pipe invert. Find the required design dimensions for a riprap apron.

1. Minimum tailwater conditions = $0.5\ d_o$ = 66 in = 5.5 ft, therefore, $0.5\ d_o$ = 2.75 ft.

2. Since TW = 2 ft, use Figure 7-3 for minimum tailwater conditions.

3. By Figure 7-3, the apron length, L_a, and median stone size, d_{50}, are 38 ft and 1.2 ft, respectively.

4. The downstream apron width equals the apron length plus the pipe diameter:

   $W = d + L_a = 5.5 + 38 = 43.5$ ft

5. Maximum riprap diameter is 1.5 times the median stone size:

   $1.5\ (d_{50}) = 1.5\ (1.2) = 1.8$ ft

6. Riprap depth = $1.5\ (d_{max}) = 1.5\ (1.8) = 2.7$ ft.

Example - Riprap Apron Design for Maximum Tailwater Conditions

A concrete box culvert 5.5 ft high and 10 ft wide conveys a flow of 600 cfs at a depth of 5.0 ft. Tailwater depth is 5.0 ft above the culvert outlet invert. Find the design dimensions for a riprap apron.

1. Compute 0.5 d_o = 0.5 (5.0) = 2.5 ft.

2. Since TW = 5.0 ft is greater than 2.5 ft, use Figure 7-4 for maximum tailwater conditions.

   v = Q/A = [600/(5) (10)] = 12 ft/s

3. On Figure 7-4, at the intersection of the curve, d_o = 60 in and v = 12 ft/s, d_{50} = 0.4 foot.

7.7 Riprap Basin

7.7.1 Overview

Following are the principal features of the riprap basin:

- Preshaping and lining with riprap of median size, d_{50}.
- Constructing the floor at a depth of h_S below the invert, where h_S is the depth of scour that would occur in a pad of riprap of size d_{50}.
- Sizing d_{50} so that $2 < h_S/d_{50} < 4$.
- Sizing the length of the dissipating pool to be $10(h_S)$ or $3(W_o)$, whichever is larger, for a single barrel. The overall length of the basin is $15(h_S)$ or $4(W_o)$, which ever is larger.
- Angular rock results were approximately the same as the results of rounded material.
- Layout details are shown on Figure 7-6.

<u>High Tailwater</u> ($TW/d_o > 0.75$)

- The high velocity water emerging from the culvert retains its jetlike character as it passes through the basin.
- The scour hole is not as deep as with low tailwater and is generally longer.
- Riprap may be required for the channel downstream of the rock-lined basin.

7.7.2 Design Procedure

Step 1: <u>Determine Input Flow</u>
- a. d_o or d_E, V_o, Fr at the culvert outlet (d_E = the equivalent depth at the brink = $(A/2)^{0.5}$).

Step 2: <u>Check TW</u>
- a. Determine if $TW/d_o \leq 0.75$. (See Chapter 5, Open Channels)

Step 3: <u>Determine d_{50}</u>
- a. Use Figure 7-7.
- b. Select d_{50}/d_E. Satisfactory results will be obtained if $0.25 < d_{50}/d_E < 0.45$.
- c. Obtain h_S/d_E using Froude number Fr and Figure 7-7.
- d. Check if $2 < h_S/d_{50} < 4$ and repeat until a d_{50} is found within the range.

Step 4: <u>Size Basin</u>
- a. As shown in Figure 7-6.
- b. Determine length of the dissipating pool, L_S.
  $L_S = 10h_S$ or $3W_o$ minimum.
- c. Determine length of basin, L_B.
  $L_B = 15h_S$ or $4W_o$ minimum.
- d. Thickness of riprap: Approach = $3d_{50}$ or 1.5 d_{max} Remainder = $2d_{50}$ or 1.5 d_{max}

Step 5: Determine V_B

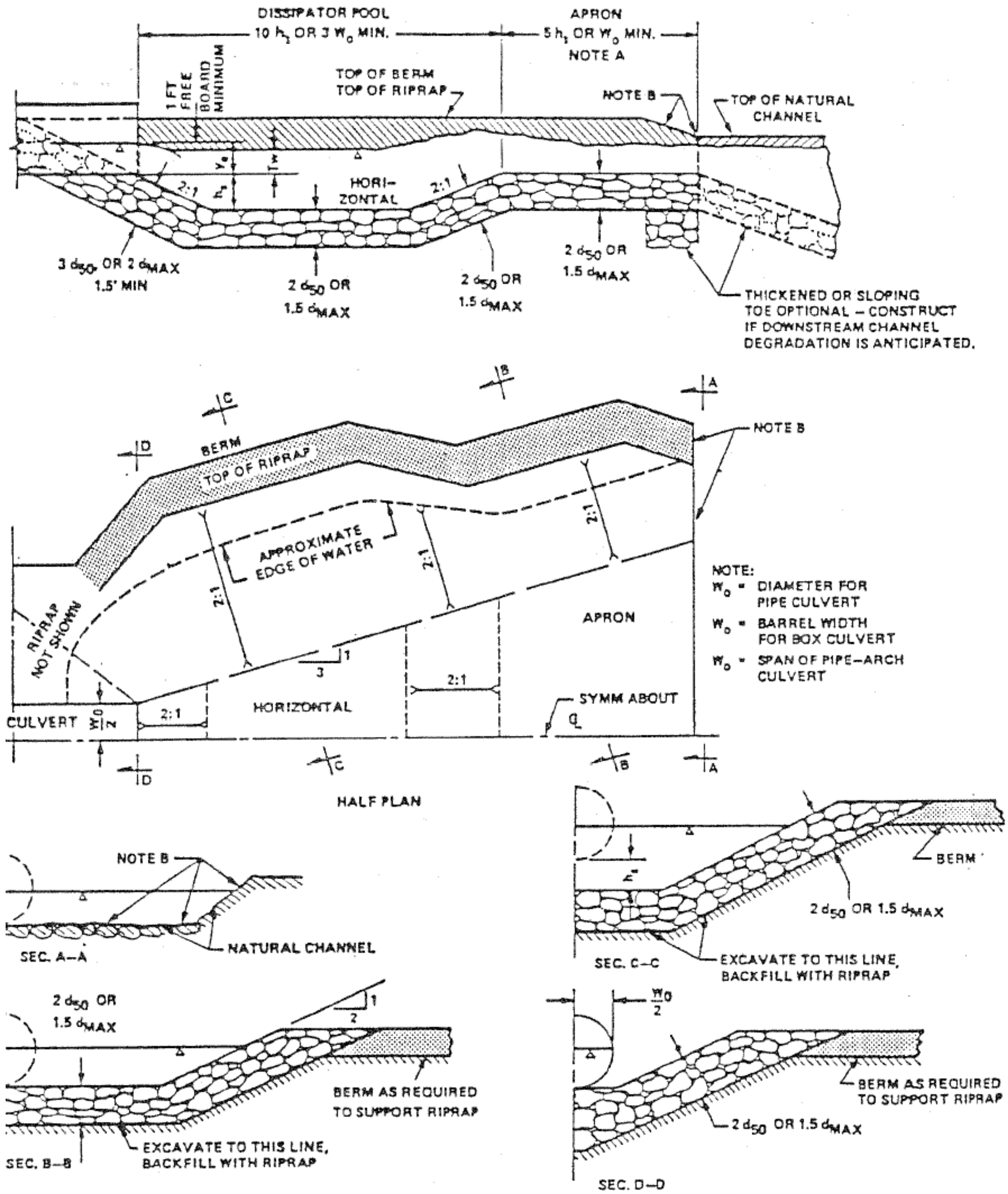
a. Basin exit depth, d_B = critical depth at basin exit.
b. Basin exit velocity, $V_B = Q/(W_B)(d_B)$.
c. Compare V_B with the average normal flow velocity in the natural channel, V_d.

Step 6: High Tailwater Design

a. Design a basin for low tailwater conditions, Steps 1-5.
b. Compute equivalent circular diameter D_E for brink area from:
   $A = \pi D_E^2/4 = d_o(W_o)$.
c. Estimate centerline velocity at a series of downstream cross sections using Figure 7-9.
d. Size riprap using HEC 11 "Use of Riprap For Bank Protection" or Chapter 5.

Step 7: Design Filter

a. Unless the streambed material is sufficiently well graded.
b. Follow instructions in section 4.4, HEC 11.



NOTE A – IF EXIT VELOCITY OF BASIN IS SPECIFIED, EXTEND BASIN AS REQUIRED TO OBTAIN SUFFICIENT CROSS–SECTIONAL AREA AT SECTION A–A SUCH THAT Q_{des}/(CROSS SECTION AREA AT SEC. A–A) = SPECIFIED EXIT VELOCITY.

NOTE B – WARP BASIN TO CONFORM TO NATURAL STREAM CHANNEL. TOP OF RIPRAP IN FLOOR OF BASIN SHOULD BE AT THE SAME ELEVATION OR LOWER THAN NATURAL CHANNEL BOTTOM AT SEC. A–A.

Figure 7-6 Details Of Riprap Basin Energy Dissipator

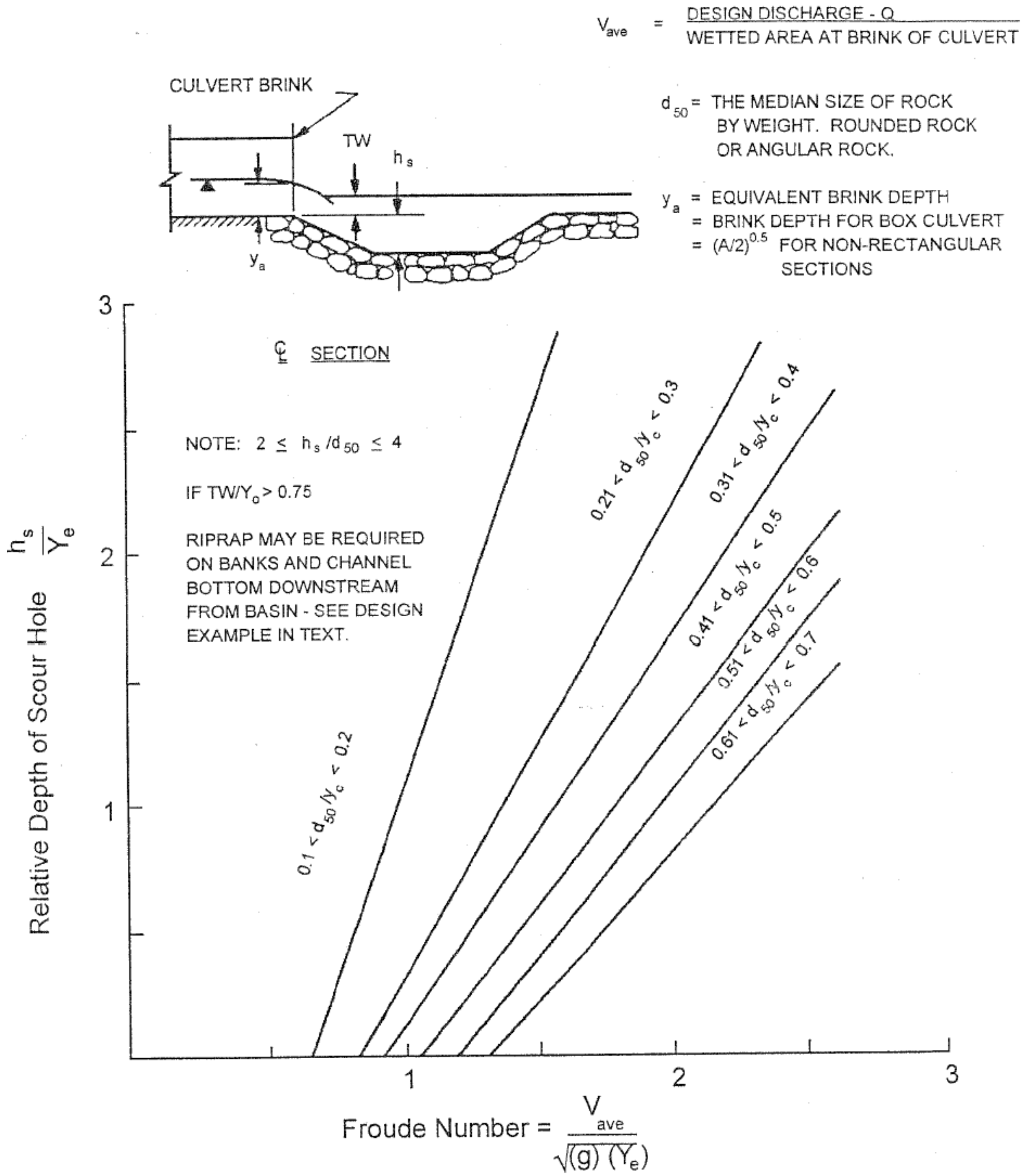
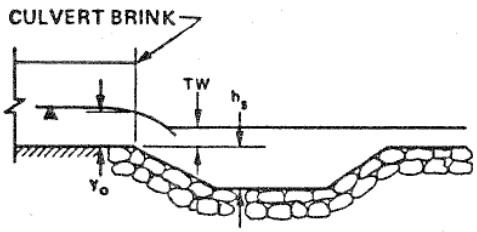
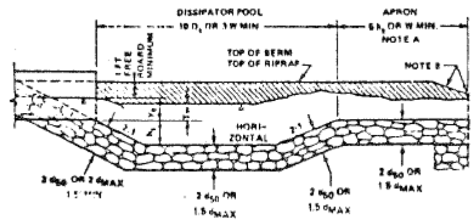


Figure 7-7 Riprap Basin Depth of Scour

RIPRAP BASIN		
PROJECT____________________________	PLAN SHEET NO.______	DATE______________
____________________________________	DESIGNER____________	REVIEWER____________





DESIGN VALUES	TRIALS		
	1	2	3
D_{50}/d_e			
D_{50}			
F_r			
h_s/d_e			
h_s			
h_s/D_{50}			
$2 < h_s/D_{50} < 4$			

BASIN DIMENSIONS		FEET	
POOL LENGTH IS THE LARGER OF:	$10h_s$		
	$3W_o$		
BASIN LENGTH IS THE LARGER OF:	$10h_s$		
	$3W_o$		
THICKNESS APPROACH	$3D_{50}$		
THICKNESS APPROACH	$3D_{50}$		

TAILWATER CHECK	
TW	
d_e	
TW/d_e	
IF $TW/d_e > 0.75$ CALCULATE RIPRAP DOWNSTREAM USING	
$D_e = (4A_c/\pi)^{.5}$	

DOWNSTREAM RIPRAP				
L/D_e	L	V_L/V_o	V_L	D_{50}

Figure 7-8 Riprap Basin Design Checklist

7.7.3 Design Example - Low Tailwater

Low Tailwater

Box culvert — 8 ft × 6 ft
Design Discharge Q = 800 cfs
Supercritical flow in culvert
Normal flow depth d_o = brink depth d_E = 4 ft
Tailwater depth, TW = 2.8 ft

Step 1: Determine Input Flow

a. $d_o = d_E$ for rectangular section.
   $d_o = d_E = 4$ ft.
   $V_o = Q/A = 800/(4)8 = 25$ ft/s.
   $Fr = V/(g\ d_E)^{0.5} = 25/[(32.2)4]^{0.5} = 2.20 < 3.0$, O.K.

Step 2: Check TW

a. Determine if $TW/d_o \leq 0.75$.
   $TW/d_E = 2.8/4.0 = 0.7$.
   Therefore $TW/d_E < 0.75$ O.K.

Step 3: Determine d_{50}

a. Use Figure 7-7.
b. Select $d_{50}/d_E = 0.45$.
   $d_{50} = 0.45(4) = 1.8$ ft.
c. Obtain h_S/d_E using Fr = 2.2.
   $h_S/d_E = 1.6$.
d. Check if $2 < h_S/d_{50} < 4$.
   $h_S = 4(1.6) = 6.4$ ft.
   $h_S/d_{50} = 6.4/1.8 = 3.6$ ft.
   $2 < 3.6 < 4$, O.K.

Step 4: Size Basin

a. As shown in Figure 7-6.
b. Determine length of dissipating pool, L_S.
   $L_S = 10h_S = 10(6.4) = 64$ ft,
   min $= 3W_o = 3(8) = 24$ ft,
   Therefore, use $L_S = 64$ ft.
c. Determine length of basin, L_B.
   $L_B = 15h_S = 15(6.4) = 96$ ft,
   min $= 4W_o = 4(8) = 32$ ft,
   Therefore use $L_B = 96$ ft.
d. Thickness of riprap:
   Approach $= 3d_{50} = 3(1.8) = 5.4$ ft,
   Remainder $= 2d_{50} = 2(1.8) = 3.6$ ft.

Step 5: Determine V_B

a. d_B = critical depth at basin exit = 3.3 ft (Assuming a rectangular cross section with width W_B = 24 ft).
b. $V_B = Q/(W_B d_B) = 800/(24 \times 3.3) = 10$ ft/s.
c. $V_B = 10$ ft/s $< V_d = 18$ ft/s.

7.7.4 Design Example - High Tailwater

High Tailwater

Data on the channel and the culvert are the same as above except the new tailwater depth, TW = 4.2 ft.
$TW/d_o = 4.2/1.05 = 1.05 > 0.75$
Downstream channel can tolerate only 7 ft/s.

Steps 1 through 5 are the same as above.

Step 6: High Tailwater Design

a. Design a basin for low tailwater conditions, steps 1-5 as above.
$d_{50} = 1.8$ ft, $h_S = 6.4$ ft.
$L_S = 64$ ft, $L_B = 96$ ft.

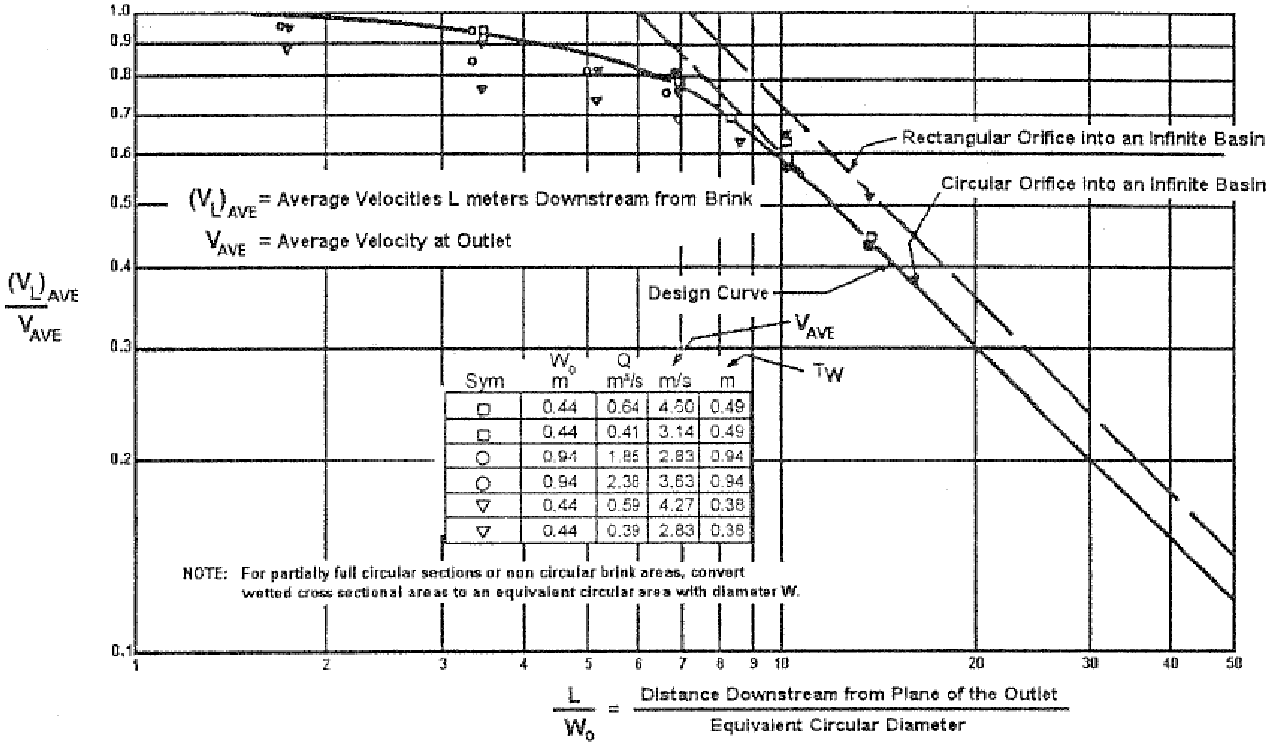
b. Compute equivalent circular diameter, D_E, for brink area from:
$A = \pi D_E^2/4 = d_o(W_o) = 4(8) = 32$ ft^2.
$D_E = [32(4)/\pi]^{0.5} = 6.4$ ft.
$V_o = 25$ ft/s.

c. Estimate centerline velocity at a series of downstream cross sections using Figure 7-9.

L/D_E [1]	L	V_L/V_o	V_L	d_{50} [2]
10	64	0.59	14.7	1.4
15 [3]	96	0.37	9.0	0.6
20	128	0.30	7.5	0.4
21	135	0.28	7.0	0.4

[1] Use $W_o = D_E$ in Figure 7-9.
[2] from Figure 7-10
[3] is on a logarithmic scale so interpolations must be made logarithmically.

d. Size riprap using HEC 11. The channel can be lined with the same size rock used for the basin. Protection must extend at least 135 ft downstream.



Note: To be used for predicting channel velocities downstream from culvert outlets where high tailwater prevails. Velocities obtained from the use of this figure can be used with HEC No. 11 for sizing riprap.

Figure 7-9 Distribution Of Centerline Velocity For Flow From Submerged Outlets

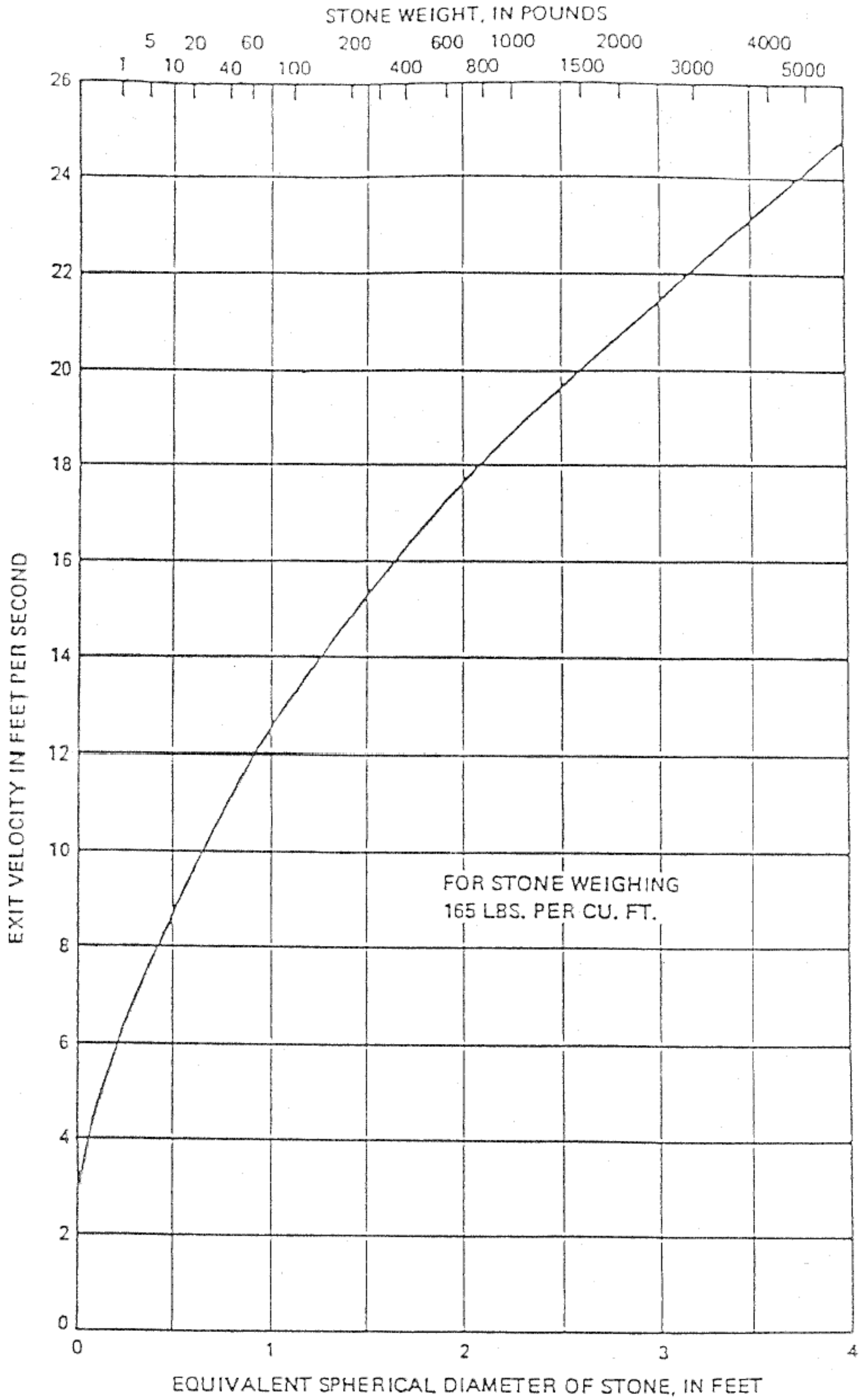
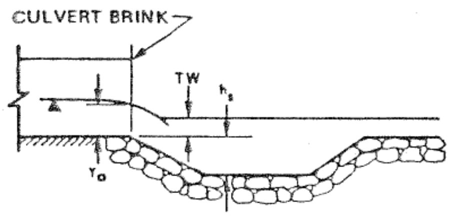
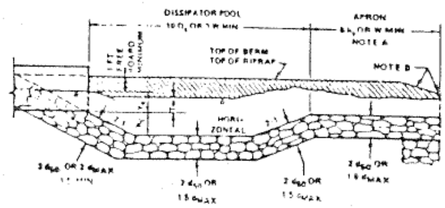


Figure 7-10 Riprap Size Versus Exit Velocity (after HEC 14)

RIPRAP BASIN		
PROJECT	PLAN SHEET NO.	DATE
	DESIGNER	REVIEWER





DESIGN VALUES	TRIALS		
	1	2	3
D_{50}/d_e	.45		
D_{50}	1.8'		
F_r	2.2		
h_s/d_e	1.6		
h_s	6.4'		
h_s/D_{50}	3.6'		
$2 < h_s/D_{50} < 4$	OK		

BASIN DIMENSIONS		FEET	
POOL LENGTH IS THE LARGER OF:	$10h_s$	64'	
	$3W_o$	24'	64'
BASIN LENGTH IS THE LARGER OF:	$10h_s$	96'	
	$3W_o$	32'	96'
THICKNESS APPROACH	$3D_{50}$	5.4'	
THICKNESS BASIN	$2D_{50}$	3.6'	

TAILWATER CHECK	
TW	2.8'
d_e	4.2'
TW/d_e	1.05'
IF $TW/d_e > 0.75$ CALCULATE RIPRAP DOWNSTREAM USING	
$D_e=(4A_c/\pi)^{.5}$	

DOWNSTREAM RIPRAP				
L/D_e	L	V_L/V_o	V_L	D_{50}
10	64	0.59	14.7	1.4
15	96	0.37	9.0	0.6
20	128	0.30	7.5	0.4
21	135	0.28	7.0	0.4

Figure 7-11 Riprap Basin Design Example

7.7.5 Computer Output

The dissipator geometry can be computed using the "Energy Dissipator" module which is available in microcomputer program HY-8, Culvert Analysis. The output of the culvert and channel input data, and computed geometry using this module, are shown below.

FHWA CULVERT ANALYSIS, HY-8, VERSION 4.5

CURRENT DATE	CURRENT TIME	FILE NAME	FILE DATE

CULVERT AND CHANNEL DATA

CULVERT NO. 1	DOWNSTREAM CHANNEL
CULVERT TYPE: 8 ft × 6 ft BOX	CHANNEL TYPE : IRREGULAR
CULVERT LENGTH = 300 ft	BOTTOM WIDTH = 8.0 ft
NO. OF BARRELS = 1.0	TAILWATER DEPTH = 3.7 ft
FLOW PER BARREL = 800 cfs	TOTAL DESIGN FLOW = 800 cfs
INVERT ELEVATION = 172.5 ft	BOTTOM ELEVATION = 172.5 ft
OUTLET VELOCITY = 25 ft/s	NORMAL VELOCITY = 21.8 ft/s

RIPRAP STILLING BASIN — FINAL DESIGN

THE LENGTH OF THE BASIN	= 93.4 ft
THE LENGTH OF THE POOL	= 62.2 ft
THE LENGTH OF THE APRON	= 31.1 ft
THE WIDTH OF THE BASIN AT THE OUTLET	= 8.0 ft
THE DEPTH OF POOL BELOW CULVERT INVERT	= 6.2 ft
THE THICKNESS OF THE RIPRAP ON THE APRON	= 5.4 ft
THE THICKNESS OF THE RIPRAP ON THE REST OF THE BASIN	= 3.6 ft
THE BASIN OUTLET VELOCITY	= 10 ft/s
THE DEPTH OF FLOW AT BASIN OUTLET	= 4.4 ft

7.8 Impact Basin USBR Type VI

7.8.1 Overview

The USBR Type VI basin, Figure 7-12, developed by the U.S. Bureau of Reclamation (USBR):

- is referred to as the USBR Type VI basin or hanging baffle,
- is contained in a relatively small box-like structure,
- requires no tailwater for successful performance,
- may be used in open channels, as well, and
- is not recommended where debris or ice buildup may cause substantial clogging.

Hanging Baffle

Energy dissipation is initiated by flow striking the vertical hanging baffle and being deflected upstream by the horizontal portion of the baffle and by the floor, creating horizontal eddies.

Notches in Baffle

Notches are provided to aid in cleaning the basin. The notches provide concentrated jets of water for cleaning. The basin is designed to carry the full discharge over the top of the baffle if the space beneath the baffle becomes completely clogged.

Equivalent Depth

This depth must be calculated for a pipe or irregular shaped conduit. The cross section flow area in the pipe is converted into an equivalent rectangular cross section in which the width is twice the depth of flow.

Limitations

Discharges up to 400 cfs per barrel and velocities as high as 50 ft/s can be used without subjecting the structure to cavitation damage.

Tailwater

A moderate depth of tailwater will improve performance. For best performance, set the basin so that maximum tailwater does not exceed h3 + (h2/2).

Slope

If culvert slope is greater than 15°, a horizontal section of at least four culvert widths should be provided upstream.

End Treatment

An end sill with a low-flow drainage slot, 45° wingwalls and a cutoff wall should be provided at the end of the basin.

Riprap

Riprap should be placed downstream of the basin for a length of at least four conduit widths.

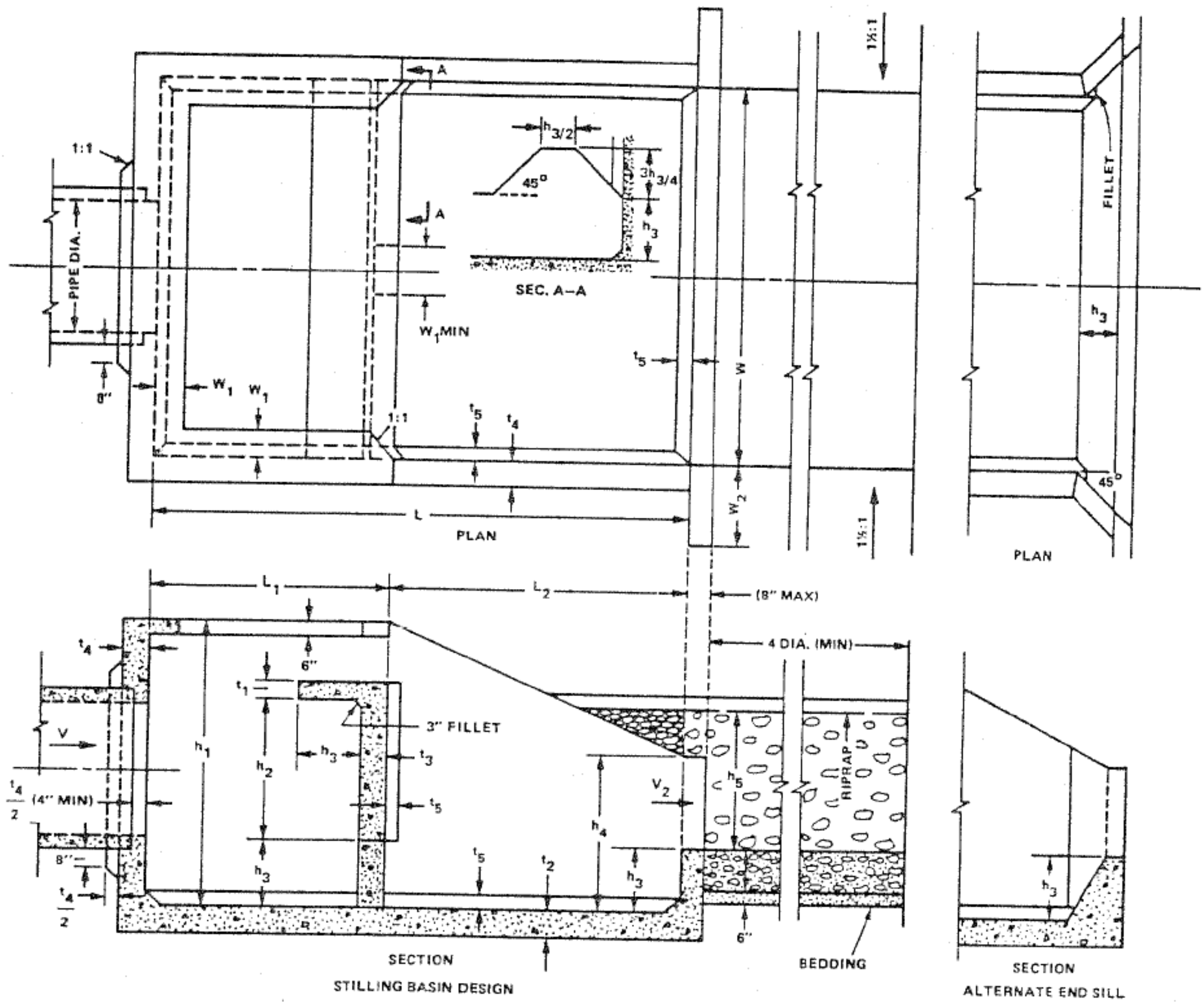


Figure 7-12 USBR Type VI (Impact) Dissipator

7.8.2 Design Procedure

Step 1: Calculate equivalent depth, d_E
a. Rectangular section, $d_E = d_o = y_o$.
b. Other sections, $d_E = (A/2)^{0.5}$.

Step 2: Determine Input Flow
a. Froude number, $Fr = V_o/(gd_E)^{0.5}$.
b. Specific energy, $H_o = d_E + V_o^2/2g$.

Step 3: Determine Basin Width, W
a. Use Figure 7-15.
b. Enter with Fr and read H_o/W.
c. $W = H_o/(H_o/W)$.

Step 4: Size Basin
a. Use Table 7-3 and W.
b. Obtain the remaining dimensions.

Step 5: Energy Loss
a. Use Figure 7-14.
b. Enter with Fr and read H_L/H_o.
c. $H_L = (H_L/H_o)H_o$.

Step 6: Exit Velocity (V_B)
a. Exit energy (H_E) = H_o - H_L.
b. $H_E = d_B + V_B^2/2g$.
$V_B = (Q/W)/d_B$.

7.8.3 Design Example

Inputs
D = 48 inch pipe, S_o = 0.15 ft/ft, n = 0.015.
Q = 300 cfs, d_o = 2.3 ft, V_o = 40 ft/s.

Step 1: Calculate equivalent depth, d_E.
b. Other sections, $d_E = (A/2)^{0.5}$.
$A = Q/V_o = 300/40 = 7.5$ ft^2.
$d_E = (7.5/2)^{0.5} = 1.94$ ft.

Step 2: Determine Input Flow
a. Froude number, $Fr_o = V_o/(gd_E)^{0.5}$.
$Fr = 40/[32.2(1.94)]^{0.5} = 5.05$.
b. Specific energy, $H_o = d_E + V_o^2/2g$.
$H_o = 1.94 + (40)^2/(2)(32.2) = 26.8$ ft.

Step 3: Determine basin width, W.
a. Use Figure 7-15.
b. Enter with Fr = 5.05 and read H_o/W = 1.68.
c. $W = H_o/(H_o/W) = 26.8/1.68 = 16$ ft.

Table 7-3 Dimensions Of USBR Type VI Basin

(Dimensions, ft)
(See Figure 7-12)

W	h_1	h_2	h_3	h_4	L	L_1	L_2
4	3-1	1-6	0-8	1-8	5-5	2-4	3-1
5	3-10	1-11	0-10	2-1	6-8	2-11	3-10
6	4-7	2-3	1-0	2-6	8-0	3-5	4-7
7	5-5	2-7	1-2	2-11	9-5	4-0	5-5
8	6-2	3-0	1-4	3-4	10-8	4-7	6-2
9	6-11	3-5	1-6	3-9	12-0	5-2	6-11
10	7-8	3-9	1-8	4-2	13-5	5-9	7-8
11	8-5	4-2	1-10	4-7	14-7	6-4	8-5
12	9-2	4-6	2-0	5-0	16-0	6-10	9-2
13	10-2	4-11	2-2	5-5	17-4	7-5	10-0
14	10-9	5-3	2-4	5-10	18-8	8-0	10-9
15	11-6	5-7	2-6	6-3	20-0	8-6	11-6
16	12-3	6-0	2-8	6-8	21-4	9-1	12-3
17	13-0	6-4	2-10	7-1	21-6	9-8	13-0
18	13-9	6-8	3-0	7-6	23-11	10-3	13-9
19	14-7	7-1	3-2	7-11	25-4	10-10	14-7
20	15-4	7-6	3-4	8-4	26-7	11-5	15-4

W	W_1	W_2	t_1	t_2	t_3	t_4	t_5
4	0-4	1-1	0-6	0-6	0-6	0-6	0-3
5	0-5	1-5	0-6	0-6	0-6	0-6	0-3
6	0-6	1-8	0-6	0-6	0-6	0-6	0-3
7	0-6	1-11	0-6	0-6	0-6	0-6	0-3
8	0-7	2-2	0-6	0-7	0-7	0-6	0-3
9	0-8	2-6	0-7	0-7	0-8	0-7	0-3
10	0-9	2-9	0-8	0-8	0-9	0-8	0-3
11	0-10	3-0	0-8	0-9	0-9	0-8	0-4
12	0-11	3-0	0-8	0-10	0-10	0-9	0-4
13	1-0	3-0	0-8	0-11	0-10	0-10	0-4
14	1-1	3-0	0-8	1-0	0-11	0-11	0-5
15	1-2	3-0	0-8	1-0	1-0	1-0	0-5
16	1-3	3-0	0-9	1-0	1-0	1-0	0-6
17	1-4	3-0	0-9	1-1	1-0	1-0	0-6
18	1-4	3-0	0-9	1-1	1-1	1-1	0-7
19	1-5	3-0	0-10	1-2	1-1	1-1	0-7
20	1-6	3-0	0-10	1-2	1-2	1-2	0-8

Step 4: <u>Size Basin</u>
a. Use Table 7-3 and W.
b. Obtain the remaining dimensions.

Step 5: <u>Energy Loss</u>
a. Use Figure 7-14.
b. Enter with Fr = 5.05 and read $H_L/H_o = 0.67$.
c. $H_L = (H_L/H_o)H_o = 0.67(26.8) = 18$ ft.

Step 6: <u>Exit Velocity</u> (V_B)

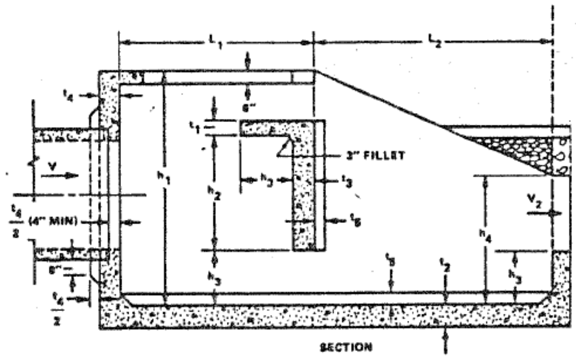
a. Exit energy (H_E) = H_o - H_L = 26.8 - 18 = 8.8 ft.

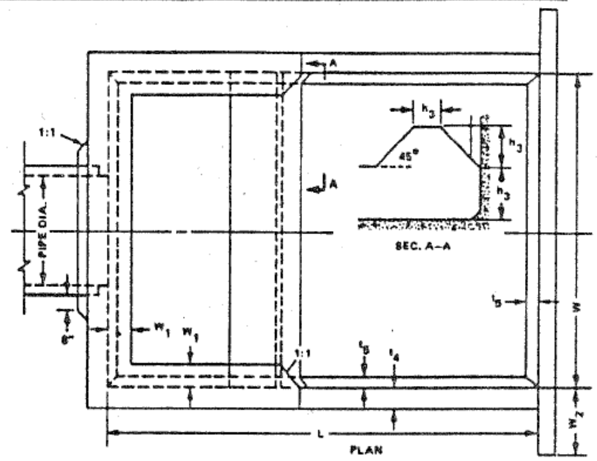
b. $H_E = d_B + V_B^2/2g = 8.8$ ft.

$V_B = (Q/W)/d_B = (300/16)/d_B = 18.75/d_B$.

d_B	V_B	$d_B+V_B^2/2g = 2.69$
$2.3 = d_o$	8.1	3.3
1.0	18.8	6.5
0.8	23.4	9.3 (use)
0.9	20.8	7.6

USBR TYPE VI DISSIPATOR COMPUTATION FORM		
PROJECT	PLAN SHEET NO.	DATE
	DESIGNER	REVIEWER





CHOOSE BASIN WIDTH (W)	TRIALS		
	1	2	3
$d_e = y_e$			
V_o			
$H_o = d_e + V_o 2/2g$			
F_r			
H_o/W			
$W = H_o/(H_o/W)$			

CHECK OUTLET VELOCITY (V_B)			
H_L/H_o			
$H_L = (H_L/H_o) H_o$			
$H_e = H_o - H_L$			
d_B			
$V_B = (Q/W)/d_B$			
$(H_e)_T = d_B + V_B 2/2g$			
IF $(H_e)_T \neq H_e$ CHOOSE ANOTHER D_B			

BASIN DIMENSIONS (FEET-INCHES)							
W	h_1	h_2	h_3	h_4	L	L_1	L_2
W	W_1	W_2	t_1	t_2	t_3	t_4	t_5

Figure 7-13 Impact Basin Type VI Checklist

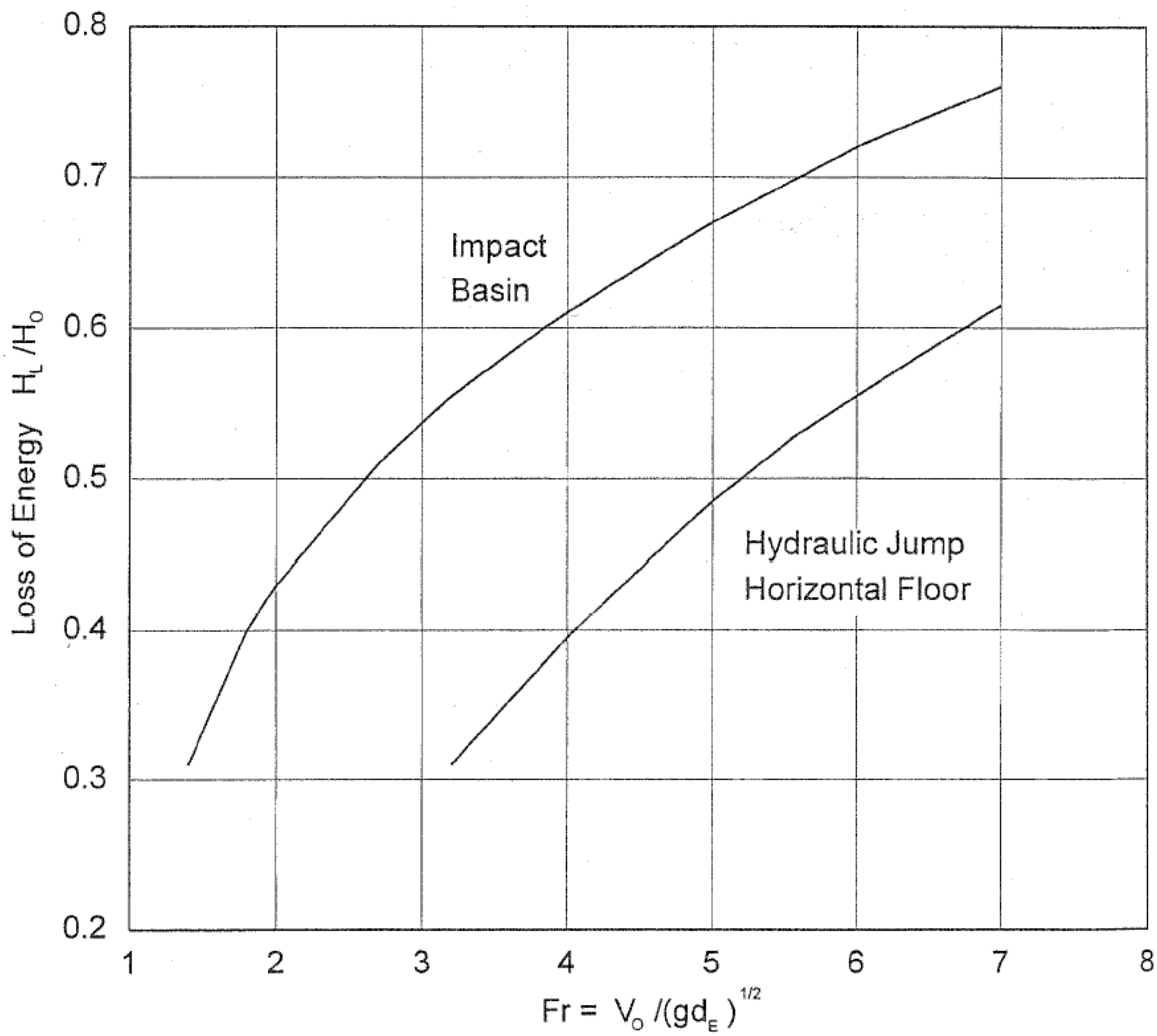


Figure 7-14   Energy Loss For USBR Type VI Dissipator

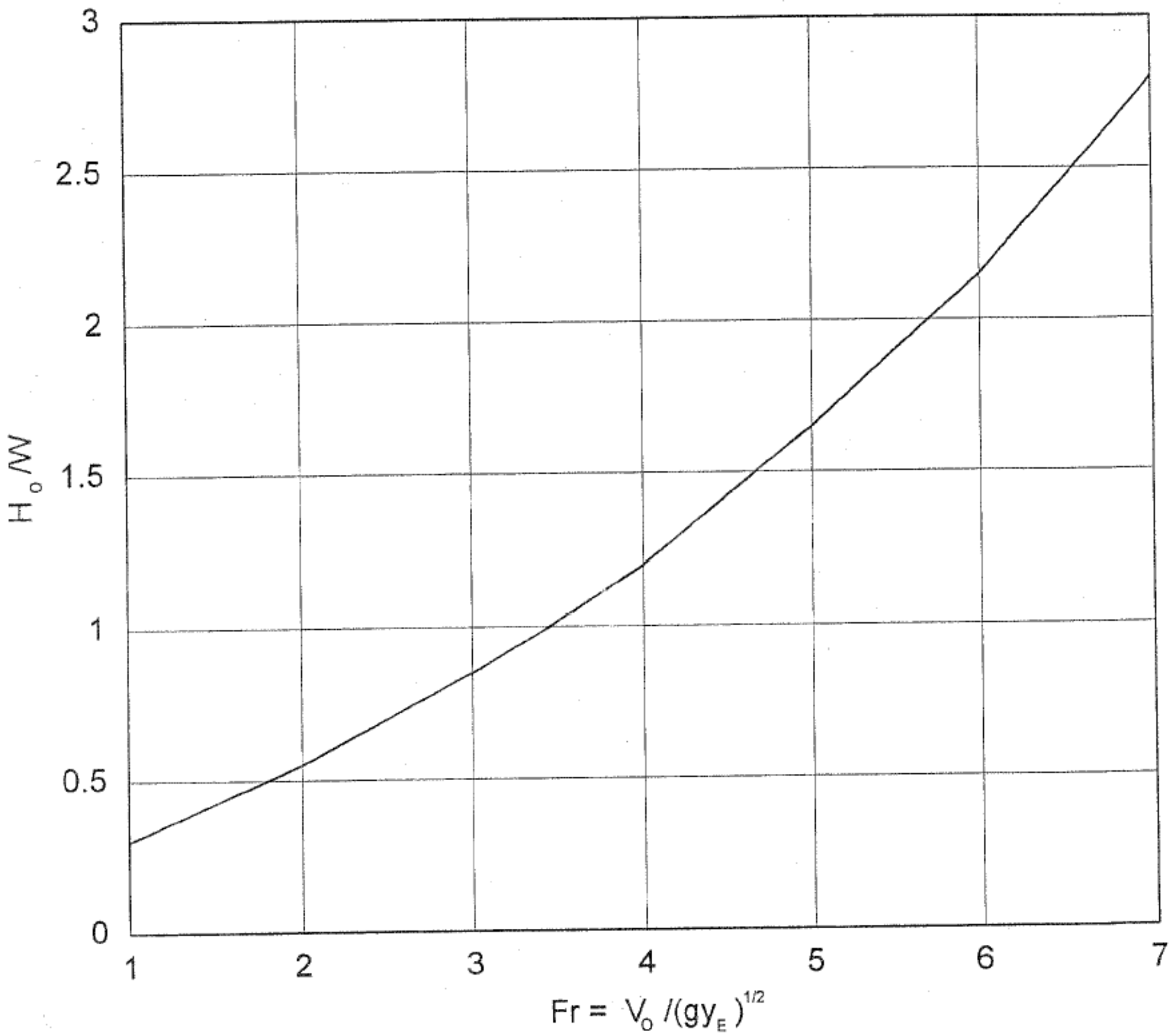
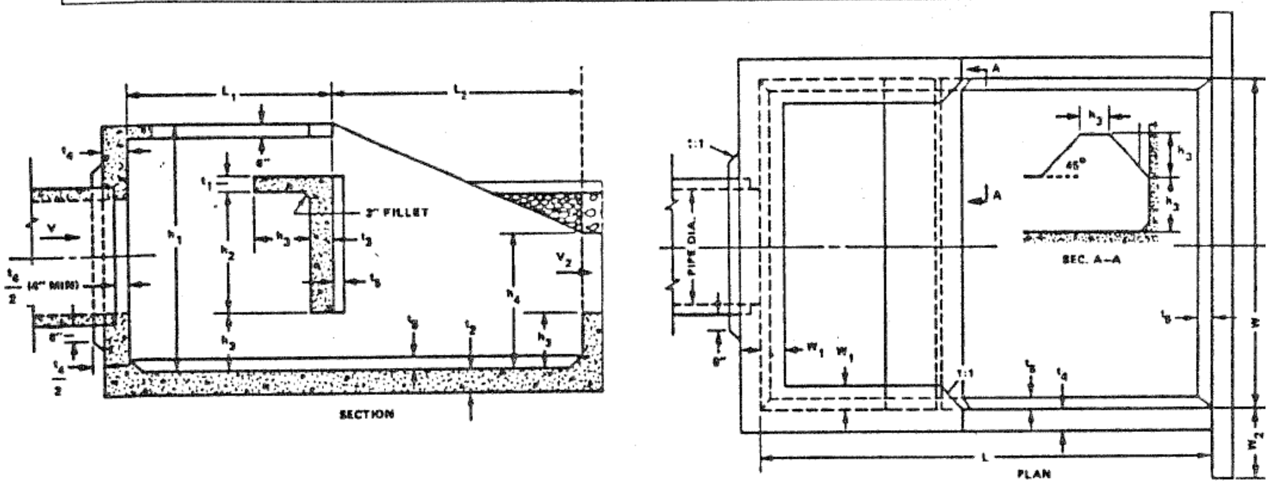


Figure 7-15 Design Curve For USBR Type VI Dissipator

USBR TYPE VI DISSIPATOR COMPUTATION FORM		
PROJECT____________________	PLAN SHEET NO._____	DATE__________
____________________________	DESIGNER__________	REVIEWER__________



CHOOSE BASIN WIDTH (W)	TRIALS		
	1	2	3
$d_e = y_e = d_o$	2.3'		
V_o	40		
$H_o = d_e + V_o 2/2g$	26.8'		
F_r	5.1		
H_o/W	1.68		
$W = H_o/(H_o/W)$	16'		

CHECK OUTLET VELOCITY (V_B)			
H_L/H_o	.67		
$H_L = (H_L/H_o) H_o$	18'		
$H_e = H_o - H_L$	8.8'		
d_B	2.3'	0.9'	0.8'
$V_B = (Q/W)/d_B$	8.1	20.8	23.4
$(H_e)_T = d_B + V_B 2/2g$	3.3'	7.6'	9.3'
IF $(H_e)_T \neq H_e$ CHOOSE ANOTHER D_B			

BASIN DIMENSIONS (FEET-INCHES)							
W	h_1	h_2	h_3	h_4	L	L_1	L_2
16	12–3	6–0	2–8	6–8	21–4	9–1	12–3
W	W_1	W_2	t_1	t_2	t_3	t_4	t_5
16	1–3	3–0	0–9	1–0	1–0	1–0	0–6

Figure 7-16 USBR Basin Type VI - Design Example

7.8.4 Computer Output

The dissipator geometry can be computed using the "Energy Dissipator" module which is available in microcomputer program HY-8, Culvert Analysis. The output of the culvert and channel input data, and computed geometry using this module are shown below.

FHWA CULVERT ANALYSIS, HY-8, VERSION 4.5

CURRENT DATE	CURRENT TIME	FILE NAME	FILE DATE

CULVERT AND CHANNEL DATA

CULVERT NO. 1	DOWNSTREAM CHANNEL
CULVERT TYPE: 4 ft CIRCULAR	CHANNEL TYPE : IRREGULAR
CULVERT LENGTH = 300 ft	BOTTOM WIDTH = 7 ft
NO. OF BARRELS = 1.0	TAILWATER DEPTH = 2.5 ft
FLOW PER BARREL = 300 cfs	TOTAL DESIGN FLOW = 300 cfs
INVERT ELEVATION = 172.5 ft	BOTTOM ELEVATION = 172.5 ft
OUTLET VELOCITY = 4.0 ft/s	NORMAL VELOCITY = 15.9 ft/s

USBR TYPE 6 DISSIPATOR — FINAL DESIGN

BASIN OUTLET VELOCITY = 2.1 ft/s

W = 16 ft	W1 = 1.3 ft	W2 = 3.0 ft
L = 21.3 ft	L1 = 9.1 ft	L2 = 12.3 ft
H1 = 12.3 ft	H2 = 6.0 ft	H3 = 2.7 ft
H4 = 6.7 ft	T1 = 0.8 ft	T2 = 1.0 ft
T3 = 1.0 ft	T4 = 1.0 ft	T5 = 0.5 ft

7.9 SAF Stilling Basin

7.9.1 Overview

The St. Anthony Falls (SAF) stilling basin uses a forced hydraulic jump to dissipate energy and:

- is based on model studies conducted by US Soil Conservation Service (SCS) at the St. Anthony Falls (SAF) Hydraulic Laboratory of the University of Minnesota;
- uses chute blocks, baffle blocks and an end sill to force the hydraulic jump and reduce jump length by about 80%;
- is recommended where Fr = 1.7 to 17.

7.9.2 Equations

Basin Width, W_B

- for box culvert W_B = B = Culvert width, ft
- for pipe, use W_B = Culvert diameter, D, ft, or

$$W_B = \frac{0.03Q}{D^{1.5}} \tag{7.7}$$

whichever is larger.

Where: Q = discharge, cfs

Flare (z:1)

Flare is optional, if used it should be flatter than 2:1.

Basin Length, L_B

$$d_j = \frac{d_1}{2}[(1 + 8Fr_1^2)^{0.5} - 1] \tag{7.8}$$

Where: d_1 = initial depth of water, ft
d_j = sequent depth of jump, ft
Fr_1 = Froude number entering basin, _ Fr

$$L_B = \frac{4.5d_j}{Fr_1^{0.76}} \tag{7.9}$$

Basin Floor

The basin floor should be depressed below the streambed enough to obtain the following depth (d_2) below the tailwater:

- For Fr_1 = 1.7 to 5.5

$$d_2 = d_j[1.1 - (\frac{Fr_1^2}{120})] \tag{7.10}$$

- For Fr_1 = 5.5 to 11

$$d_2 = 0.85d_j \tag{7.11}$$

- For Fr_1 = 11 to 17

$$d_2 = d_j[1.1 - (\frac{Fr_1^2}{800})] \tag{7.12}$$

Chute Blocks

Height, $h_1 = d_1$
Width, W_1 = Spacing, $W_1 = 0.75d_1$
Number of blocks = $N_c = W_B/2W_1$, rounded to a whole number
Adjusted $W_1 = W_2 = W_B/2N_c$
N_c includes the ½ block at each wall

Baffle Blocks

Height, $h_3 = d_1$
Width, W_3 = Spacing, $W_4 = 0.75d_1$
Basin width at baffle blocks, $W_{B2} = W_B + 2L_B/3z$
Number of blocks = $N_B = W_{B2}/2W_3$, rounded to a whole number
Adjusted $W_3 = W_4 = W_{B2}/2N_B$
Check total block width to insure that 40 to 55% of W_{B2} is occupied by block.
Staggered with chute blocks
Space at wall $\geq 0.38d_1$
Distance from chute blocks ($L_{1\text{-}3}$) = $L_B/3$

End Sill Height, $h_4 = 0.07d_j$
Sidewall Height = $d_2 + 0.33d_j$
Wingwall Flare = 45°

7.9.3 Design Procedure

The design of a St. Anthony Falls (SAF) basin consists of several steps as follows:

Step: 1 Select Basin Type
- a. Rectangular or flared.
- b. Choose flare (if needed), z:1.
- c. Determine basin width, W_B.

Step: 2 Select Depression
- a. Choose the depth d_2 to depress below the streambed, B_d.
- b. Assume $B_d = 0$ for first trial.

Step 3: Determine Input Flow
- a. d_1 and V_1, using energy equation.
- b. Froude Number, Fr_1.

Step 4: Calculate Basin Dimensions
- a. d_j (equation 7.8).
- b. L_B (equation 7.9).
- c. d_2 (equation 7.10, 7.11, or 7.12).
- d. $L_S = (d_2 - TW)/S_S$
- e. $L_T = (B_d)/S_T$ (see Figure 7-3).
- f. $L = L_T + L_B + L_S$ (see Figure 7-3).

Step 5: Review Results
- a. If $d_2 \neq (B_d - LS_o + TW)$ return to Step 2.
- b. If approximately equal, continue.

Step 6: Size Elements
- a. Chute blocks (h_1, W_1, W_2, N_c).
- b. Baffle blocks (h_3, W_3, W_4, N_B, $L_{1\text{-}3}$).
- c. End sill (h_4).
- d. Side wall height ($h_5 = d_2 + 0.33d_j$).

7.9.4 Design Example

- See Figure 7-18 for completed computation form.

Step 1: Select Basin Type
a. Use rectangular with no flare
b. Basin width, $W_B = 7$ ft

Step 2: Select Depression
Trial 1
$B_d = 8$ ft, $S_S = S_t = 1$

Step 3: Determine Input Flow
Trial 1
a. Energy equation (culvert to basin):
Culvert outlet = $B_d + d_o + V_o^2/2g = 8 + 1.8 + (32)^2/2(32.2) = 25.7$ ft
Basin floor = $0 + d_1 + V_1^2/2g$
Solve: $25.7 = d_1 + V_1^2/2g$

d_1	V_1	$d_1+V_1^2/2g$
1.5	38	24 < 25.4
1.4	41	27.5 > 25.7, Use:

b. $Fr_1 = 41/(1.4 \times 32.2)^{0.5} = 6.1$.

Step 4: Calculate Basin Dimensions
Trial 1
a. $d_j = 11.4$ ft (equation 7.8).
b. $L_B = 13.0$ ft (equation 7.9).
c. $d_2 = 9.7$ ft (equation 7.10).
d. $L_S = (d_2 - TW)/S_S = (9.7 - 2.8)/1 = 6.9$ ft.
e. $L_T = (B_d)/S_T = 8/1 = 8$ ft.
f. $L = L_T + L_B + L_S = 8 + 13 + 7 = 28$ ft.

Step 5: Review Results
Trial 1
a. If d_2 does not equal $(B_d - LS_o + TW)$, then adjust drop
$9.7 \neq (8 - 28(0.05) + 2.8) = 9.4$ ft.
b. Add 9.7 - 9.4 = 0.3 more drop and return to Step 2.

Step 2: Select Depression
Trial 2
$B_d = 8.3$ ft, $S_S = S_T = 1$

Step 3: Determine Input Flow
Trial 2
a. Energy equation (culvert to basin):
Culvert outlet = $B_d + d_o + V_o^2/2g = 8.3 + 1.8 + (32)^2/2g = 26.0$ ft
Basin floor = $0 + d_1 + V_1^2/2g$
Solve: $26.0 = d_1 + V_1^2/2g$

d_1	V_1	$d_1 + V_1^2/2g$
1.4	41	27.5 > 26.0, Use:

b. $Fr_1 = 41/(1.4 \times 32.2)^{0.5} = 6.1$.

Step 4: Calculate Basin Dimensions

Trial 2

a. $d_j = 11.4$ ft (equation 7.8).
b. $L_B = 13.0$ ft (equation 7.9).
c. $d_2 = 9.7$ ft (equation 7.10).
d. $L_S = (d_2 - TW)/S_S = (9.7 - 2.8)/1 = 6.9$ ft.
e. $L_T = (B_d)/S_T = 8.3/1 = 8.3$ ft.
f. $L = L_T + L_B + L_S = 8.3 + 13 + 7 = 28.3$ ft

Step 5: Review Results

Trial 2

b. $d_2 = 9.7 = (8.3 - 28.3(0.05) + 2.8) = 9.7$ ft. Is equal, continue.

Step 6: Size Elements

Trial 2

a. Chute blocks (h_1, W_1, W_2, N_c)
   $h_1 = d_1 = 1.4$ ft.
   $W_1 = 0.75d_1 = 1$ ft.
   $N_c = W_B/2(W_1) = 7/2(1) = 3.5$, use 4.
   Adjusted $W_1 = 7/2(4) = 0.9$ ft $= W_2$.
   Use 6 full blocks, 4 spaces and a half of block at each wall.
b. Baffle blocks (h_3, W_3, W_4, N_B, $L_{1\text{-}3}$)
   $h_3 = d_1 = 1.4$ ft.
   $W_3 = 0.75d_1 = 1$ ft.
   Use 4 blocks, and adjusted as above $W_3 = W_4 = 0.9$ ft.
   $L_{1\text{-}3} = L_B/3 = 13/3 = 4.3$ ft.
c. End sill (h_4) $= 0.07d_j = 0.07(11.4) = 0.8$ ft.
d. Side wall height (h_5) $= d_2 + 0.33d_j = 9.7 + 0.33(11.4) = 13.5$ ft.

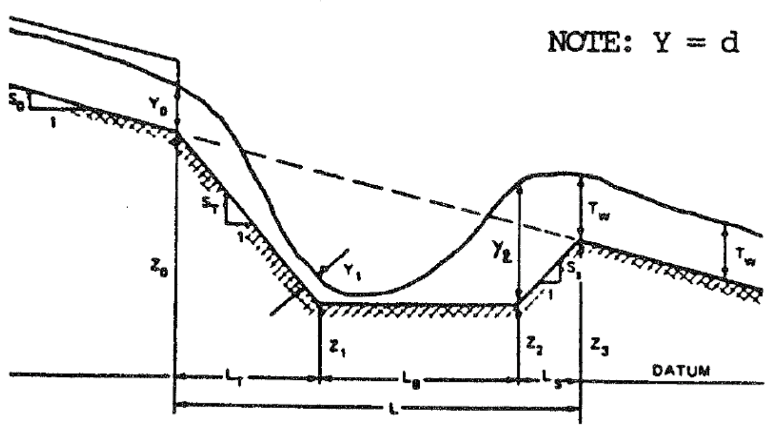
Figure 7-17 : ST. ANTHONY FALLS (SAF) BASIN

Project ____________________

Designer ____________________ Date __________

Reviewer ____________________ Date __________

SAF BASIN DESIGN VALUES	TRIAL 1	FINAL TRIAL	DIMENSIONS OF ELEMENTS	TRIAL 1	FINAL TRIAL	DIMENSIONS OF ELEMENTS	TRIAL 1	FINAL TRIAL
Type			CHUTE BLOCK			BAFFLE BLOCK		
Flare (z:1)			Height, h_1			Height, h_3		
Width, W_B			Width, W_1			Width, W_3		
Depression, B_d			Spacing, W_2			Spacing, W_4		
$S_S = S_T$			Block No., N_C			Block No., N_B		
Depth, d_o			END SILL			SIDE WALL		
Velocity, V_o			Height, h_4			Height, h_5		
$B_o = d_o+V_o^2/2g$								
Depth, d_1								
Velocity, V_1								
Fr_1								
d_j								
L_B								
d_2								
L_S								
$L_T = B_d / S_T$								
$L = L_B+L_S+L_T$								
$B_d = LS_o + TW$								



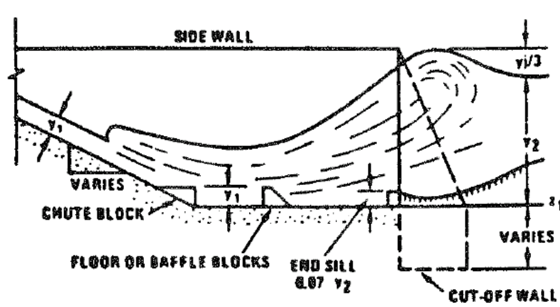


Figure 7-17 St. Anthony Falls Basin Checklist

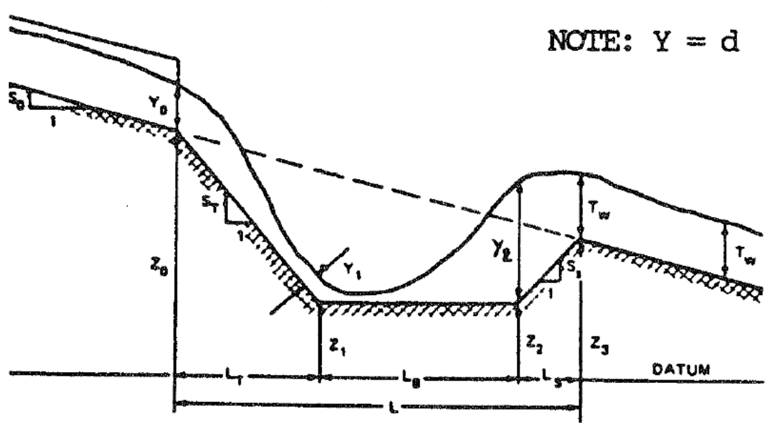
Figure 7-18 : ST. ANTHONY FALLS (SAF) BASIN

Project Example Problem

Designer ______________________ Date ________

Reviewer ______________________ Date ________

SAF BASIN DESIGN VALUES	TRIAL 1	FINAL TRIAL	DIMENSIONS OF ELEMENTS	TRIAL 1	FINAL TRIAL	DIMENSIONS OF ELEMENTS	TRIAL 1	FINAL TRIAL
Type	Rect.	Rect.	CHUTE BLOCK			BAFFLE BLOCK		
Flare (z:1)	1:1	1:1	Height, h_1	1.4	1.4	Height, h_3	1.4	1.4
Width, W_B	7	7	Width, W_1	1	0.9	Width, W_3	1	0.9
Depression, B_d	8	8.3	Spacing, W_2	0.9	0.9	Spacing, W_4	1	0.9
$S_S = S_T$	1:1	1:1	Block No., N_C	4	4	Block No., N_B	4	4
Depth, d_o	1.8	1.8	END SILL			SIDE WALL		
Velocity, V_o	32	32	Height, h_4	0.8	0.8	Height, h_5	13.5	13.5
$B_o = d_o + V_o^2/2g$	25.7	26.0						
Depth, d_1	1.4	1.4						
Velocity, V_1	41	41						
Fr_1	6.1	6.1						
d_j	11.4	11.4						
L_B	13	13						
d_2	9.7	9.7						
L_S	6.9	7.0						
$L_T = B_d / S_T$	8	8.3						
$L = L_B + L_S + L_T$	28	28.3						
$B_d = LS_o + TW$	9.4	9.7						



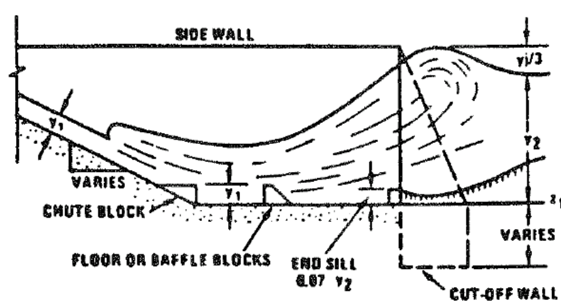


Figure 7-18 St. Anthony Falls Basin Example Problem

7.9.5 Computer Output

The dissipator geometry can be computed using the "Energy Dissipator" module which is available in microcomputer program HY-8, Culvert Analysis. The output of the culvert and channel input data, and computed geometry using this module are shown below.

FHWA CULVERT ANALYSIS, HY-8, VERSION 6.0

CURRENT DATE	CURRENT TIME	FILE NAME	FILE DATE

CULVERT AND CHANNEL DATA

CULVERT NO. 1	DOWNSTREAM CHANNEL
CULVERT TYPE: 7.0 ft × 6.0 ft BOX	CHANNEL TYPE : IRREGULAR
CULVERT LENGTH = 300.0 ft	BOTTOM WIDTH = 7.0 ft
NO. OF BARRELS = 1.0	TAILWATER DEPTH = 2.8 ft
FLOW PER BARREL = 400.0 cfs	TOTAL DESIGN FLOW = 400.0 cfs
INVERT ELEVATION = 172.5 ft	BOTTOM ELEVATION = 172.5 ft
OUTLET VELOCITY = 31.1 ft/s	NORMAL VELOCITY = 17.5 ft/s
OUTLET DEPTH = 0.616 ft	

ST. ANTHONY FALLS BASIN -- FINAL DESIGN

LB = 11.9 ft LS = 5.8 ft LT = 7.1 ft
L = 24.8 ft Y1 = 1.3 ft Y2 = 8.7 ft
Z1 = 165.5 ft Z2 = 165.4 ft Z3 = 171.3 ft
WB = 8.2 ft WB3 = 8.2 ft
------------------------------ CHUTE BLOCKS ------------------------------
H1 = 0.461 ft W1 = 0.356 ft W2 = 0.356 ft NC = 3.000
------------------------------ BAFFLE BLOCKS ----------------------------
W3 = 1.0 ft W4 = 1.0 ft NB = 4
H3 = 1.3 ft LCB = 4.0 ft
------------------------------ END SILL ----------------------------
H4 = 0.7 ft
BASIN OUTLET VELOCITY = 17.5 ft/s

Reading up to the intersection with d = 60 in, find L_a = 40 ft.

4. Apron width downstream = $d_w + 0.4\ L_a = 10 + 0.4\ (40) = 26$ ft.

5. Maximum stone diameter = $1.5\ d_{50} = 1.5\ (0.4) = 0.6$ ft.

6. Riprap depth = $1.5\ d_{max} = 1.5\ (0.6) = 0.9$ ft.

7.10 Storm Sewer Outlet End Treatment

A common failure mode of storm sewer outlets that enter perpendicularly to a main channel is erosion of the area immediately beneath the outlet of the storm sewer.

If the flow from a storm sewer outlet to a channel has velocities greater than the evasive velocity of the channel bank or bed material, some form of storm sewer outlet end treatment is required.

The energy dissipation structures described earlier in this chapter may not be applicable for dissipating energy or controlling erosion in this situation because of the main channel flows that are perpendicular to the storm sewer outlet flows.

Protection may be needed 1) below the storm sewer outlet, 2) on the channel bed, and 3) on the opposite bank. Protection of the bank below the outfall should be provided by a headwall that extends below the anticipated scour depth or by providing a riprap apron to provide protection against erosion. Protection of the channel bed can be provided by buried riprap (riprap that is flush with the channel bed). Protection of the opposite bank may be provided by riprap, vegetation or turf reinforcement.

The design of any erosion protection feature for storm sewer outlet flows must take into account the main channel flows.

Another option to dissipate energy in a storm sewer is to provide internal energy dissipation through the use of a broken back culvert. See "Hydraulic Analysis of Broken-Back Culverts" NDOR, (January 1998) for design guidance.

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CHAPTER 8

STORMWATER BEST MANAGEMENT PRACTICES

March 7, 2011

Chapter Eight - Stormwater Best Management Practices

Table Of Contents

8.1 Overview

8.1.1 Introduction

Urban runoff carries with it a wide variety of pollutants from diverse and diffuse sources. Pollutants associated with urban runoff often occur in higher concentrations than found in runoff prior to development. In addition, urban runoff can contain pollutants that are not naturally present in surface runoff from undeveloped land, such as organic pesticides, household solvents, and petroleum products. Runoff from undeveloped basins contains sediment particles, oxygen-demanding compounds, nutrients, metals, and other constituents. Once developed, pollutant loads increase because runoff volumes increase as do sources of the pollutants.

The phenomenon termed “first flush” has often been used to characterize urban runoff. First flush refers to the higher levels of initial concentrations of constituents that are washed off from the surface at the onset of a storm event. A typical pollutant concentration pattern during a storm event contains a relatively high concentration of contaminants during the first flush of runoff. However, depending on rainfall intensity, antecedent period length and conditions, deposition during the antecedent period, and surface characteristics, the affect of the first flush can be varied. After the first flush, the concentration typically drops substantially and fluctuates at a lower level for the remainder of the runoff event. A secondary “spike” in pollutant concentration can occur if a sudden burst of intense rain drives material off surfaces not completely cleaned by the initial runoff (Horner et al., 1994).

8.1.2 Structural and Nonstructural Best Management Practices (BMPs)

Studies such as the Nationwide Urban Runoff Program have documented concentrations of various pollutants in urban runoff (EPA, 1983). To reduce the concentrations and the loads of these pollutants the reach the receiving waters, a system of stormwater BMPs may be implemented. BMPs are defined as measures that function to either keep pollutants from entering stormwater or remove pollutants from stormwater. Various BMPs have been implemented throughout the United States. In general, they can be categorized as either structural or nonstructural. Structural BMPs can be thought of as constructed facilities designed to reduce runoff and/or passively treat urban stormwater runoff before it enters the receiving waters. Nonstructural BMPs consist of pollution prevention BMPs and source control BMPs. Both structural and nonstructural BMPs are used for erosion control during construction as well (UDFCD, 1992). A detailed discussion of sediment and erosion control is presented in Chapter 9.

The selection of the most appropriate BMPs for a given site or basin is largely dependent on whether development is in place or has yet to occur. In areas with existing development, nonstructural BMPs are the most cost-effective because retrofitting structural controls in a developed area can be expensive. Structural controls are more appropriate for new development and significant redevelopment, where they have been integrated into the planning of the infrastructure.

Because non-point source pollution is varied in nature and impact, no individual BMP may fit all situations. It must be tailored to fit the needs of particular sources and circumstances. An effective strategy for minimizing stormwater pollution loads is to use multiple BMPs (structural, nonstructural, and source controls). Multiple BMPs and combining BMPs in series can provide complementary water quality enhancement that minimizes pollutant loads being transported to the receiving waters.

8.2 General Water Quality Management Approach

8.2.1 General Planning and Design Guidelines

The following general planning and design guidelines for structural and nonstructural controls are recommended when developing a water quality control strategy:

- Promote natural infiltration of urban runoff by minimizing onsite impervious areas and preserving natural, broad drainageways.
- Minimize directly connected impervious areas by providing grassed or gravel buffer zones between impervious surfaces. Divert runoff from impervious areas to pervious surfaces before the flows enter surface drainageways.
- Locate structural BMPs in areas that avoid creating a nuisance and the need for increased maintenance.
- Provide multiple access to facilities to improve maintenance capabilities.

- Direct offsite stormwater flow around the onsite facilities.
- Revegetate and/or stabilize all areas disturbed by construction activities and all drainageways created as a part of the development.
- Ensure the plantings and grass cover are firmly established before the owner's obligation is released and maintenance efforts begin.

8.2.2 Effectiveness Of Management Measures

The effectiveness of many management measures was summarized in a 1992 report prepared by the Metropolitan Washington Council of Governments, entitled "A Current Assessment of Urban Best Management Practices." Some of the findings of this report include:

- Not all urban BMPs can reliably provide high levels of removal for both particulate and soluble pollutants. Effective BMPs include wet ponds, stormwater wetlands, multiple pond systems and sand filters. Infiltration BMPs are presumed to be effective in removing pollutants, but are not reliable given their poor longevity. Other BMPs, such as grassed swales, filter strips and water quality inlets, cannot provide reliable levels of pollutant removal until their basic design is significantly enhanced.
- The longevity of some BMPs is limited to such a degree that their widespread use is currently not encouraged. Of particular concern are the infiltration practices, such as basins, trenches and porous pavement. The poor longevity of these BMPs is attributable to a number of factors: lack of pretreatment, poor construction practices, application to infeasible sites, lack of regular maintenance, and in some cases, fundamental difficulties in basic design. Very often the life-spans of BMPs can be increased to acceptable lengths if local communities adopt enhanced designs and commit to strong maintenance and inspection programs.
- No single BMP option can be applied to all development situations and all BMP options require careful site assessment prior to design. Pond options are applicable to the widest range of development situations, but typically require a minimum drainage area. On the other hand, infiltration practices have very limited applications, requiring field verification of soils, water tables, slope and other factors.
- Several BMPs can have significant secondary environmental impacts, although the extent and nature of these impacts is uncertain and site-specific. Pond systems, which offer reliable pollutant removal and longevity, tend to be associated with the greatest number and strongest degree of secondary environmental impacts. Careful site assessment and design are often required to prevent stream warming, natural wetland destruction and riparian habitat modification.
- Relatively limited cost data exist to aid in the assessment of the comparative cost-effectiveness of urban BMP options. Presently, the selections of BMPs is based more on longevity, feasibility, and local design factors than on comparative cost. Maintenance costs may be significant and should be considered during the design process.
- In many cases, a systems approach to BMP design is warranted whereby multiple techniques for runoff attenuation, conveyance, pretreatment, and treatment are utilized.
- Several fundamental uncertainties still exist with respect to urban BMPs. These uncertainties include the toxicity of residuals trapped within BMPs; the interaction of groundwater and BMPs (both ponds and infiltration); and the long-term performance of urban BMPs. The USEPA has evaluated the benefits of water quality BMP's and their associated uncertainties and have determined that municipalities should encourage implementation of structural and non-structural BMP's.

Based on the above findings, it is clear how important it is to carefully plan and design BMPs on a site-specific basis. Success in applying any management practice depends on selecting the appropriate option for the control objectives, specific conditions at the site, proper implementation and maintenance.

8.3 Structural Best Management Practices

8.3.1 Pollutant Removal Mechanisms

Although runoff may contain many individual pollutants, they can, in general, be grouped into two categories: particulate and soluble. Often, pollutants such as metals and oxygen demand compounds become adsorbed or attached to particulate matter. Therefore, if the particulate matter is removed, so are the adsorbed or attached constituents.

There are four basic pollutant removal or immobilization mechanisms promoted by the BMPs described in this chapter. The following is an overview of each of them:

- Sedimentation - Particulate matter is, in part, settled out of urban runoff. Approximately 80 percent of metals in stormwater are attached or adsorbed to particles that are under 60 microns in diameter (i.e., fine silts and clays). Consequently, these particles can require long periods of time to settle out of suspension. With extended detention, however, the smaller particles can agglomerate into larger ones, thus removing a larger proportion of them through sedimentation.
- Filtering - Particulates can be removed from water by filtration. Filtration removes particles by attachment to small-diameter collectors such as sand.
- Infiltration - As surface runoff infiltrates or percolates into the ground, pollutant loads are removed or reduced in the runoff. Particulates are removed at the ground surface by filtration, and soluble contaminants can be adsorbed to the soil matrix as the runoff percolates into the ground. Soil characteristics such as permeability, cation exchange capacity, and depth to groundwater or bedrock limit the effectiveness of infiltration as a pollutant removal mechanism.
- Biological Uptake - Soluble constituents can be ingested or taken up from the water column and concentrated through bacterial action and phytoplankton growth. In addition, certain biological activities can reduce toxicity of some pollutants.

8.3.2 Structural BMP Selection

Selecting the appropriate BMP for a site depends on several factors, including:

- The permeability and type of soil underlying the BMP;
- The size of the tributary basin and the generated runoff volume in relation to the size of the BMP;
- The slopes and geometry of the site;
- The amount of base flow;
- The proximity of bedrock to the surface;
- The proximity to the seasonal high groundwater table to the surface;
- Tributary basin land uses; and
- The ability to handle high sediment loads.

8.3.3 Water Quality Control Volume

For many BMPs, combining the water quality facility with a flood control facility is practical and cost effective. Specifically, the water quality control volume (WQCV) that is recommended for control is the first half inch (0.5 inches) of runoff from the basin tributary to the BMP. For facilities that combine water quality control with flood control, the runoff from the design storms for the flood control criteria should be “stacked” on top of the water quality control volume. The water quality control volume should be detained over at least a 24-hour period, and preferably for longer.

8.3.4 Structural BMP Descriptions

This section gives information regarding the applicability, pollutant removal efficiencies, advantages, disadvantages, costs and maintenance considerations for structural BMPs that could be used within the City of Waverly. Other structural BMP’s may also be applicable for use in the City.

The structural BMPs covered in this chapter include:

- Extended Dry Detention Basins
- Constructed Wetlands
- Filter Strips and Flow Spreaders
- Infiltration Trenches
- Oil/Grit Separators
- Retention (Wet) Ponds
- Grassed Swales
- Sand Filters
- Porous Pavement
- Catch Basin Inserts

For each BMP, performance data are included to give a general idea of the pollution removal rates of different BMPs. These values are presented for comparison of BMPs only and are subject to wide variability when describing specific BMPs.

8.3.4.1 Extended Dry Detention Basins

Extended dry detention (ED) basins are designed to completely empty at some time after stormwater runoff ends. These are adaptations of the detention basins used for flood control. The primary difference is in outlet design; the extended basin uses a much smaller outlet that extends the retention time for more frequent events so that pollutant removal is facilitated. A 40-hour drain time for the WQCV is recommended to remove a significant portion of fine particulates and provide streambank erosion control (UDFCD, 1992; Schueler, 1987). The term "dry" implies that there is no significant permanent water storage (UDFCD, 1992).

Many designers encourage a two stage design in which the upper stage is dry except for infrequent large storm events and the lower stage is regularly inundated, with a volume equal to the runoff from the mean storm.

<u>General applicability and experience with technique elsewhere</u>

a. Typical Applications
   - Significant areal requirement limits use; not typically a site-based BMP.
   - Retrofitting to established developments may be very difficult due to areal requirements.
   - Extended dry detention basins can reduce peak stormwater runoff rates while trapping sediment loads, particularly when used downstream from construction sites. Sediment from such high loads will need to be removed, however. ED basins can be used to improve runoff water quality from roads, parking lots, residential, commercial and industrial areas. Typically, they are used in conjunction with other onsite BMPs.

b. Design Considerations (See Figures 8-1 through 8-3 for representative schematics)
   - Land requirement is typically 0.5 to 2.0 percent of drained area (UDFCD,1992)
   - The volume of runoff detained should be based on 0.5 inches of runoff from the tributary area.
   - Pilot channel should be constructed to minimize erosion control (alternately use turf if little low flow). Size such that any event runoff will overflow the low flow channel onto the pond floor.
   - Side slopes shall be no greater than 4:1 if mowed.
   - Inlet and outlet located to maximize flow length.
   - Design for full development upstream of control.
   - Rip-rap protection (or other suitable erosion control means) for the outlet and all inlet structures into the pond.
   - Use a water quality outlet that is capable of slowly releasing the WQCV over a 40-hour period. A perforated riser can be used in conjunction with a weir box opening above it to control for larger storm outflows. A sample outlet is illustrated in Figure 8-2. The number of perforations per row can be determined with the aid of Figure 8-3. This relationship is based on the rows being equally spaced vertically at 4 inches on center. Any other outlet that can meet the emptying time criteria should be acceptable (UDFCD,1992).
   - One foot of freeboard above peak stage for top of embankment for design storm.
   - Emergency spillway designed to pass the 100-year storm event.
   - Maintenance access (< 8 % slope and approximately 10 feet wide).
   - Trash racks, filters or other debris protection on control and anti-vortex plates.
   - Insure no outlet leakage and use anti-seep collars.
   - Benchmark for sediment removal.
   - Two stage design (top stage - dry during the mean storm, bottom stage - inundated during storms less than the mean storm event.)
   - Top stage shall have slopes between 2% and 5% and a depth of 2 to 5 feet.
   - Design as off-line pond to bypass larger flows.
   - Design as sediment settling basin for pretreatment of the larger particles.
   - Addition of a small wetland marsh or ponding area in the basin's bottom may enhance soluble pollutant removal, however storms may flush out trapped sediments and minimize this benefit.
   - The facility must also meet the criteria provided in Chapter 6 Storage Facilities.

c. Other Experiences with BMP
    - Extended dry detention basins have performed well in sediment and associated pollutant removal efficiencies. They also prevent streambank erosion. Problems noted include clogging of the outlet and detention times significantly lower than design (Galli, 1992).

Reported pollutant removal efficiencies

- Reported data

EPA,1986; Grizzard et al., 1986; Whipple and Hunter, 1981:

Suspended Solids	50-70%
Total Phosphorus	10-20%
Total Nitrogen	10-20%
Zinc	30-60%
Bacteria	50-90%
Lead	75-90%

Galli, F.J., 1992:

Lead	62%
Zinc	57%

- For soluble pollutants (e.g. phosphorous, nitrogen, zinc), the removal performance appears to be more consistent than for retention ponds or wetlands, although the latter have higher maximum removal rates. Removal rates for less soluble constituents are somewhat less than those for retention ponds or wetlands (Urbonas and Stahre, 1993).
- Due to the wide range of variability for pollutant removal, a conservative estimate near the low end of the ranges reported should be assumed.

Advantages

- Moderate to high removal of particulates and suspended heavy metals.
- Infiltration and resultant recharge to ground water is minimal compared to infiltration type BMPs, therefore the risk of direct introduction of contaminants to ground water is also minimal.

Disadvantages

a. Human Risk, Public Safety and Potential Liability

b. Environmental Risk and Implications
    - Possible habitat destruction
    - High ground water levels may inundate the basin and outlet (use retention ponds if this is the case); ground water mounding may occur with slow-draining or silt-clogged soils.
    - Thermal modification to downstream waters should be minimal (Schueler, 1987)

c. Other
    - Will likely have negative aesthetics unless a lower-stage basin is used.
    - Can become a trash dump if not maintained.
    - Potential breeding grounds for mosquitos and other insects unless a balanced habitat is established.
    - Aesthetics; must factor in debris and sediment accumulation and removal, as well as overall design integration with site.

Maintenance/monitoring/enforcement considerations

a. Reliability and Consistency over Time
    - Exfiltration will tend to decrease over time as the bottom becomes clogged with sediment; this may be a positive factor in preventing ground water contamination.

b. Routine and Non-routine Maintenance
    - Cleanup of debris and trash, pest and overgrowth control, erosion repairs, inspect for structural damage to outlets, clogging of outlet.
    - A five to ten year sediment cleanout cycle is recommended (Schueler, 1987).

c. Sustainability of Maintenance or Program Management
    - Regular maintenance and sediment cleanout are not technically difficult; long-term management should not be problematic.

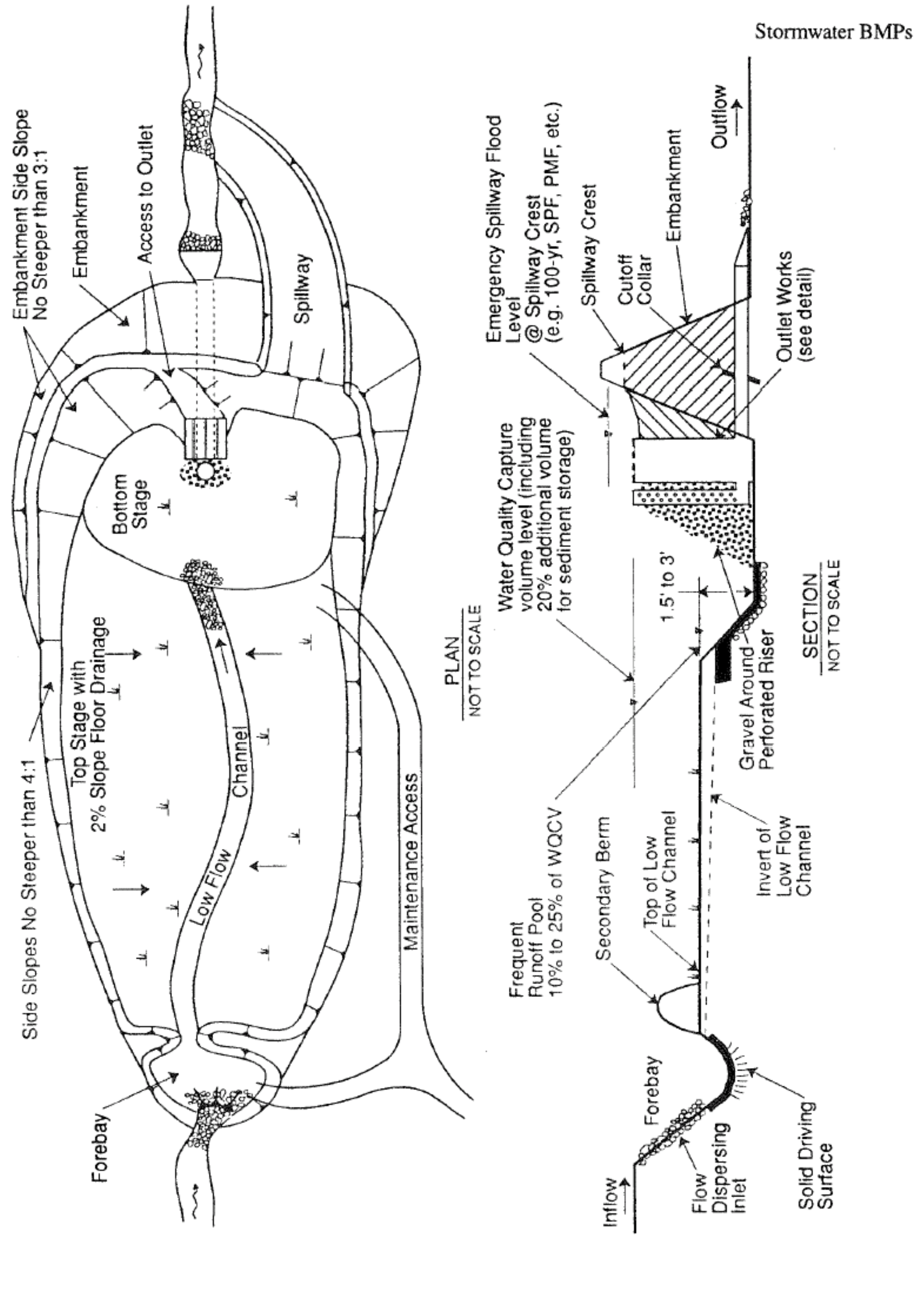
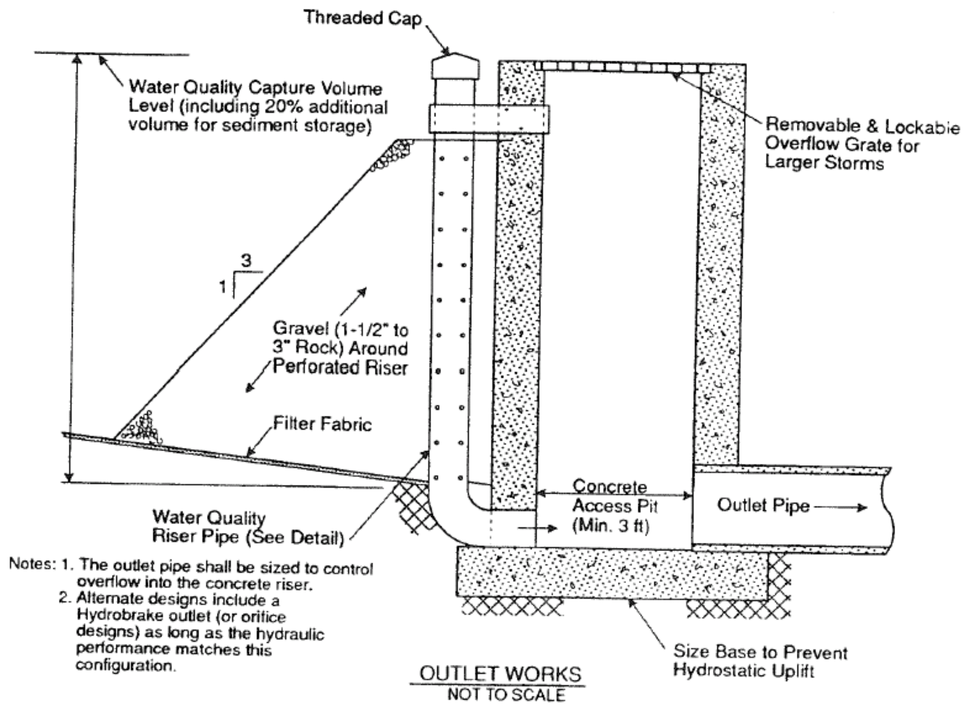
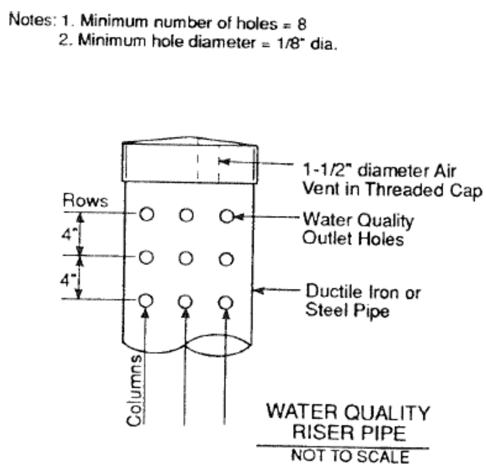


Figure 8-1 Extended Dry Detention Basin

Source: Denver Urban Drainage and Flood Control District, 1992





Maximum Number of Perforated Columns				
Riser Diameter (in.)	Hole Diameter, in.			
	1/4"	1/2"	3/4"	1"
4	8	8	--	--
6	12	12	9	--
8	16	16	12	8
10	20	20	14	10
12	24	24	18	12

Hole Diameter (in.)	Area of Hole (in.^{2})
1/8	0.013
1/4	0.049
3/8	0.110
1/2	0.196
5/8	0.307
3/4	0.442
7/8	0.601
1	0.785

Figure 8-2 Water Quality Outlet for Extended Dry Detention Basin

Source: Denver Urban Drainage and Flood Control District, 1992

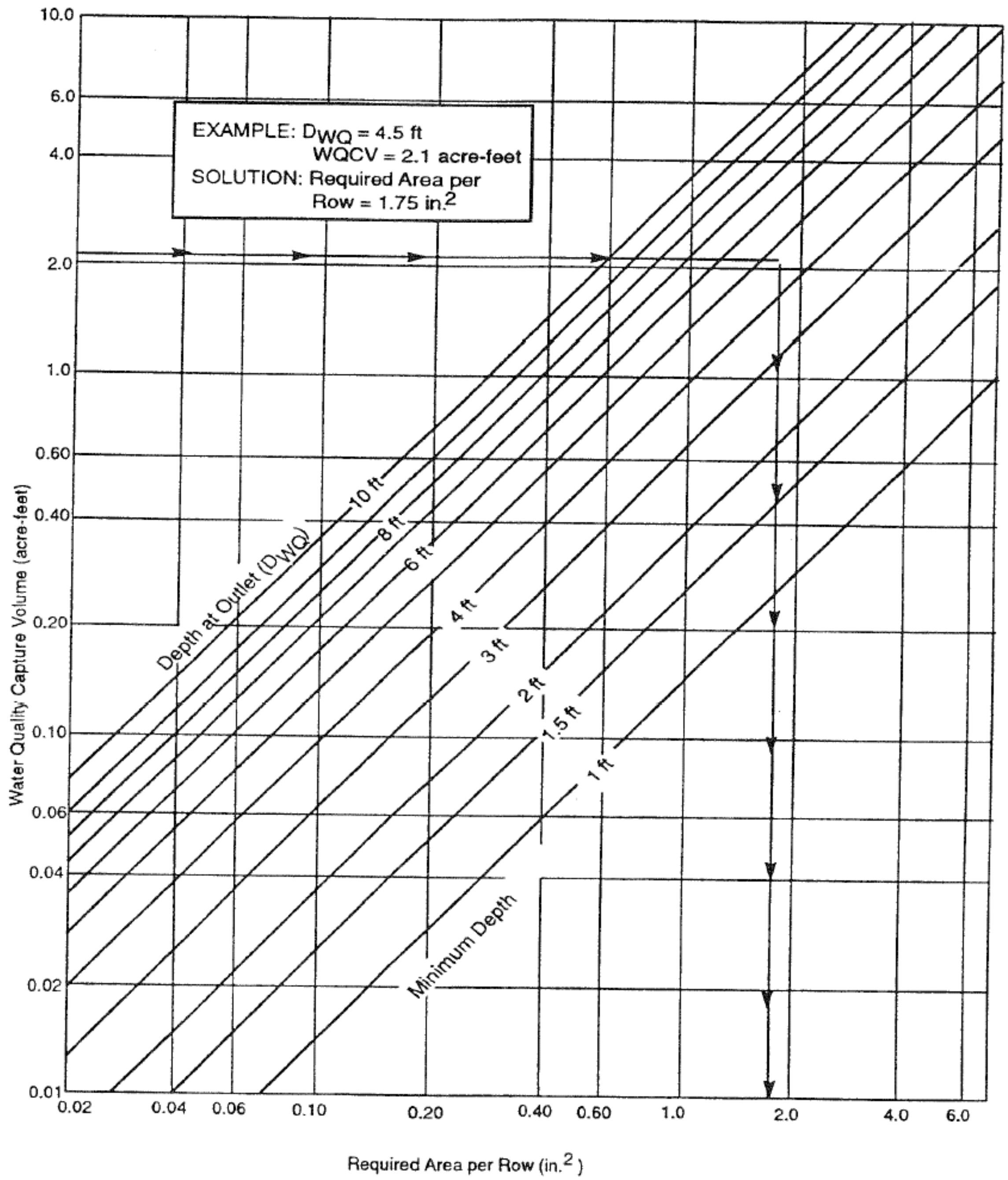


Figure 8-3 Water Quality Outlet Sizing for Extended Dry Detention Basin with 40 Hour Drain Time of the Capture Volume

Source: Denver Urban Drainage and Flood Control District, 1992

8.3.4.2 Retention (Wet) Ponds

A retention pond is designed to not completely drain as in the dry basin design. A permanent pool of water is replaced in part by stormwater during an event. For water quality purposes, the design is such that the WQCV is released over 12 to 24 hours, but the hydraulic residence time (HRT) for the permanent pool volume is two weeks or longer. Reduction of volume in the permanent pool is through evapotranspiration and infiltration only. A dry weather base flow may be required to maintain the permanent pool (UDFCD, 1992).

<u>General applicability and experience with technique elsewhere</u>

a. Typical Applications
   - Typically not a site-based BMP, but retention ponds are effective in most settings where adequate open area exists. Due to the area required, it is difficult to retrofit to a completely developed watershed.
   - Since evaporation can quickly dry up base flows, retention ponds work best in areas with low ET rates and/or non-arid climates.
   - Wet retention ponds can reduce peak stormwater runoff rates while trapping sediment loads, as well as provide some biological uptake of nutrients. They can be used downstream from construction sites, but sediment removal is difficult. They can be used to improve runoff water quality from roads, parking lots, residential, commercial and industrial areas. Typically, they are used in conjunction with other onsite BMPs.

b. Design Considerations (See Figures 8-4 through 8-6 for representative schematics)
   - Minimum length to width ratio of 3:1 (preferably wedge shaped expanding outward toward the outlet). Irregular shorelines for larger ponds provide visual variety.
   - Inlet and outlet located to maximize flow length. Use baffles to increase flow length if needed.
   - Minimum depth of permanent pool 2 to 3 feet, maximum depth of 9 to 10 feet. Average depth should be 3 to 6 feet.
   - Design for full development upstream of control.
   - Side slopes shall be no greater than 4:1.
   - Rip-rap protection (or other suitable erosion control means) for the outlet and all inlet structures into the pond. Individual boulders or baffle plates can work for this.
   - Minimum drainage area of 10 acres. Land requirement is typically 0.5 to 2.0 percent of drained area (UDFCD,1992).
   - Use a water quality outlet that is capable of slowly releasing the WQCV over a 12-hour period. A perforated riser can be used in conjunction with a weir box opening above it to control for larger storm outflows. A sample outlet is illustrated in Figure 8-5. The number of perforations per row can be determined with the aid of Figure 8-6. This relationship is based on the rows being equally spaced vertically at 4 inches on center. Any other outlet that can meet the emptying time criteria should be acceptable (UDFCD,1992).
   - Emergency drain; i.e. sluice gate, drawdown pipe; capable of draining within 24 hours.
   - Trash racks, filters, hoods or other debris control on riser.
   - Maintenance access (< 8 % slope and 10 feet wide).
   - Benchmark for sediment removal.
   - Design for multi-function as flood control and extended detention.
   - Sediment forebay for larger ponds (often designed for 5 to 15 percent of total volume). Forebay should have separate drain for de-watering. Grass biofilters for smaller ponds.
   - Incorporating a wetland design or wetland vegetation into the pond can increase contaminant removal rates. This may also encourage wildlife habitation which can help in mosquito and pest control.
   - If fast draining soils are present, a liner may be needed to sustain baseflow; conversely, if bedrock is present and needs to be excavated, construction costs may become very high.
   - The facility must also meet the criteria provided in Chapter 6 Storage Facilities.

c. Other Experiences with BMP
   - Wet retention ponds are generally more effective at removing nutrient loadings than dry basins; their use is encouraged where nutrient loads are a major contributor to water quality problems (Hartigan, 1989)
   - According to the NURP study, basins which exhibit high removal efficiencies are sized such that the mean storm displaces only about 10% of the available volume, and overflow rates (mean runoff rate/basin surface

area) are a small fraction of the median particle settling velocity (EPA, 1993). The study concluded that retention ponds are capable of providing very effective removal of pollutants in urban runoff.

Reported pollutant removal efficiencies

- Reported data

EPA,1983:

Suspended Solids	91%
Total Phosphorus	0-80%
Total Nitrogen	0-80%
Zinc	0-70%
Lead	9-95%
BOD	0-69%

Yousef, Y.A., et al. 1986:

A well-oxygenated pond with minimum organic debris appears to provide the environment for improved removal efficiencies of nutrients and selected heavy metals. Also, it was concluded that slower infiltration rates and increased mean residence time favor retention of metals within the sediments. There was no evidence of metals migration within the sediments or that a contamination hazard exists to nearby surface or ground water.

Dissolved Lead	54.5%
Particulate Lead	95.1%
Dissolved Zinc	88.3%
Particulate Zinc	96.2%
Dissolved Copper	49.7%
Particulate Copper	77.0%
Dissolved Phosphorous	90.1%
Particulate Phosphorous	11.4%
Organic Nitrogen	11.0%
Ammonia	81.6%
Nitrates + Nitrites	86.5%

Hartigan 1989:

Reported expected removal rates of 40 - 60% for phosphorous and 30 - 40% for nitrogen. Additionally, a minimum Basin area/tributary area ratio of 1% is recommended for high removal rates; 3% for poorly draining soils.

Advantages

- Cost-effective for larger tributary watersheds
- Moderate to high removal rates of many urban pollutants
- Creates wildlife habitat
- Provides recreation, aesthetics, open space areas
- More efficient sedimentation than dry basin, since outlet is above the basin bottom, leaving a 'dead zone' to trap sediment
- Infiltration and resultant recharge to ground water is minimal compared to infiltration type BMPs, therefore the risk of direct introduction of contaminants to ground water.

Disadvantages

a. Human Risk, Public Safety and Potential Liability
b. Environmental Risk and Implications
    - Attract waterfowl, which may increase downstream nutrient loading and bacteria.

- Inadequate base flow can cause very high concentrations of salts, nutrients, and algae through evaporation, resulting in significant downstream loadings from smaller events.
- Possible low DO effluent, stream warming, trophic shifts, habitat destruction, loss of upstream channels.
- Large events or low dissolved oxygen content can cause mixing or resuspension of deposited sediments, increasing turbidity and metals concentrations.

c. Other
- Higher cost than conventional stormwater detention.
- Difficult sediment removal.
- Floating litter, scum and algal blooms, odors, insects.
- Bottom of pool may need to be lined to maintain permanent pool in well-draining conditions.
- Wet retention ponds have greater storage capacity requirements than dry ED basins, resulting in higher capital costs.
- Large basins may require a dam safety permit.

<u>Maintenance/monitoring/enforcement considerations</u>

a. Reliability and Consistency over Time
- May be more efficient over time due to increased vegetation providing enhanced nutrient and metals removal rates

b. Routine and Non-routine Maintenance
- Sediment to be removed when 20% of storage volume of the facility is filled (design storage volume must account for volume lost to sediment storage).
- No woody vegetation shall be allowed on the embankment without special design provisions.
- Other vegetation over 18 inches high shall be cut unless it is part of planned landscaping.
- Debris shall be removed from blocking inlet and outlet structures and from areas of potential clogging.
- The control shall be kept structurally sound, free from erosion, and functioning as designed.
- Control of scum and algal blooms, odors, insects.
- The site should be inspected and debris removed after every major storm.

c. Sustainability of Maintenance or Program Management
- Funds must be budgeted for routine and non-routine maintenance, particularly considering the high cost of sediment removal. For this reason, public rather than private maintenance is preferred (Schueler, 1987).

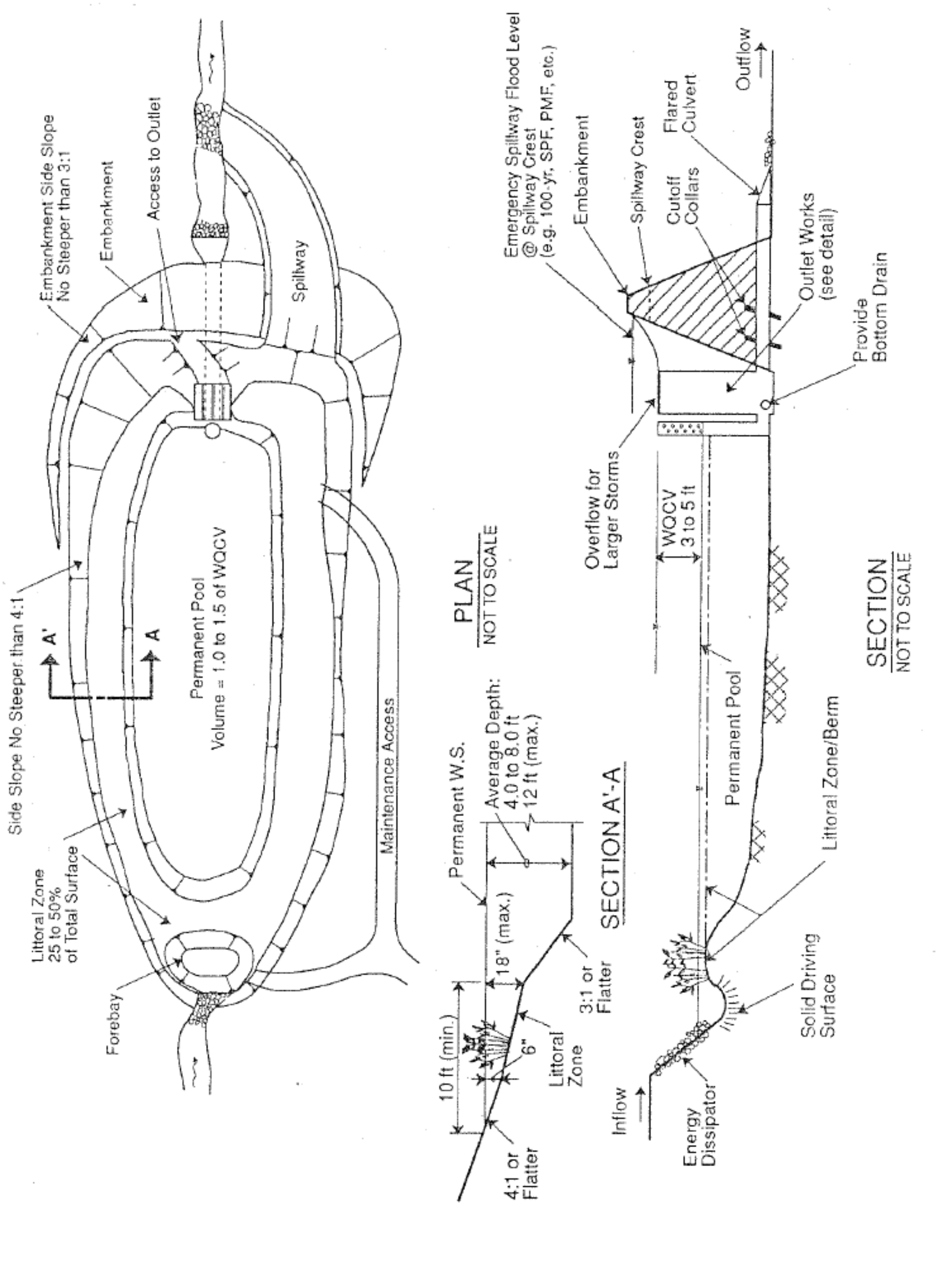
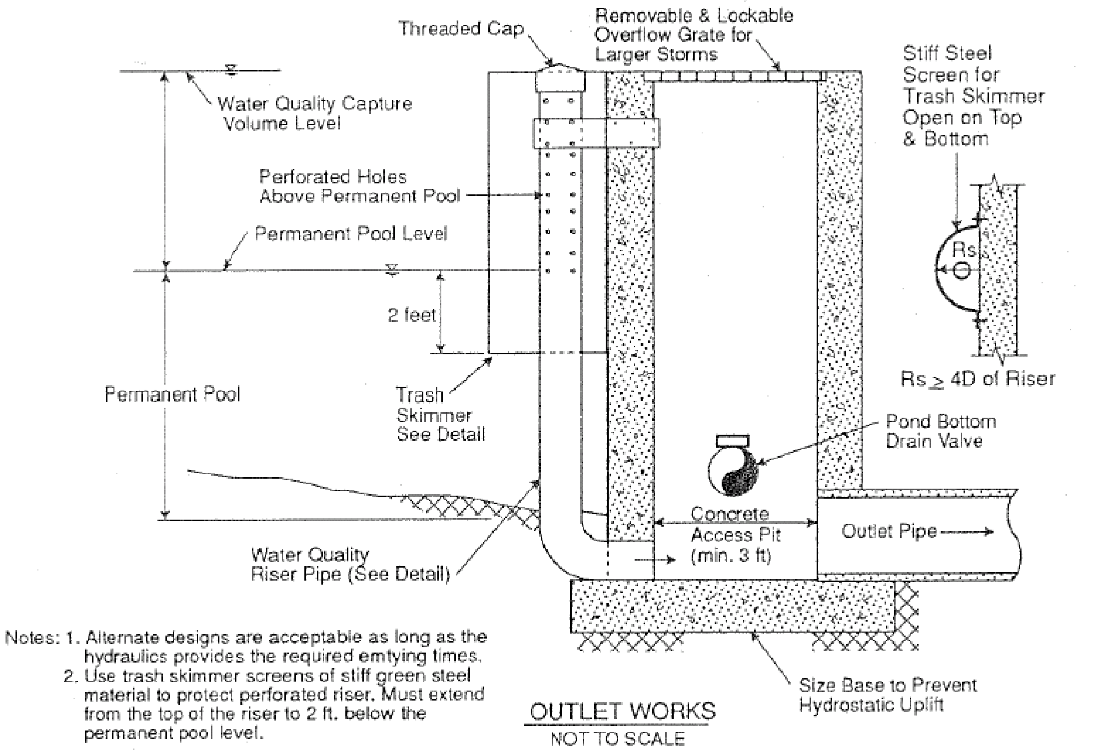
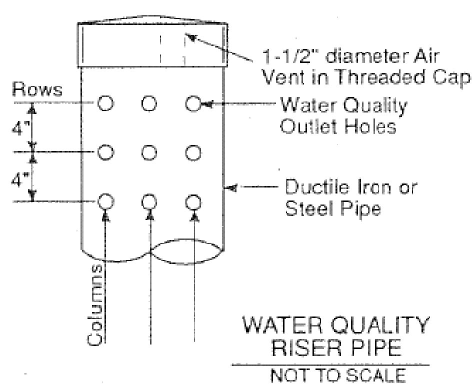


Figure 8-4 Retention (Wet) Pond

Source: Denver Urban Drainage and Flood Control District, 1992



Notes: 1. Minimum number of holes = 8
2. Minimum hole diameter = 1/8" Dia.



Maximum Number of Perforated Columns				
Riser Diameter (in.)	Hole Diameter, inches			
	1/4"	1/2"	3/4"	1"
4	8	8	--	--
6	12	12	9	--
8	16	16	12	8
10	20	20	14	10
12	24	24	18	12

Hole Diameter (in.)	Area (in. 2)
1/8	0.013
1/4	0.049
3/8	0.110
1/2	0.196
5/8	0.307
3/4	0.442
7/8	0.601
1	0.785

Figure 8-5 Water Quality Outlet for Retention Pond

Source: Denver Urban Drainage and Flood Control District, 1992

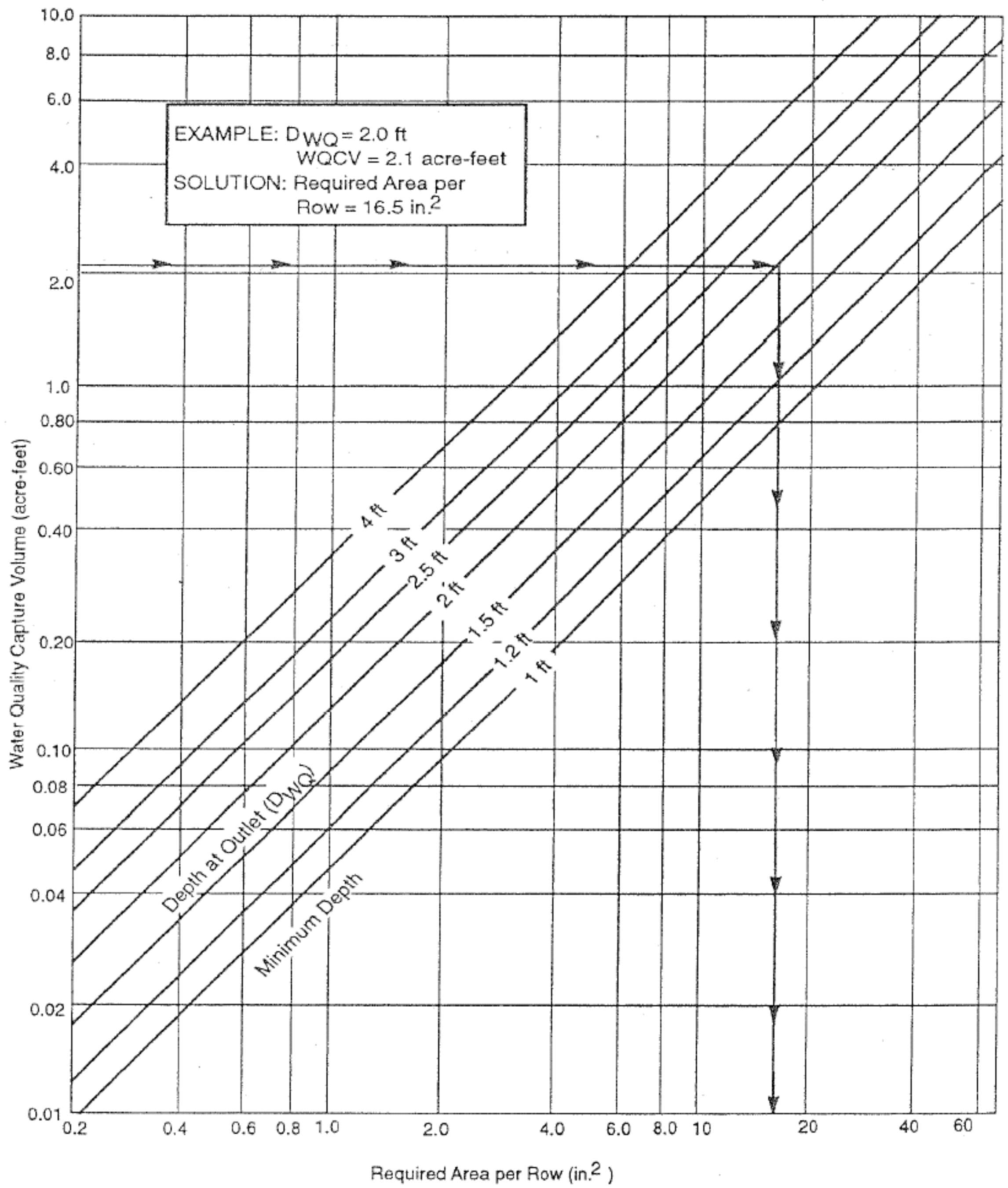


Figure 8-6 Water Quality Outlet Sizing for Retention Pond with 12 Hour Drain Time of the Capture Volume

Source: Denver Urban Drainage and Flood Control District, 1992

8.3.4.3 Constructed Wetlands

Constructed wetlands can take the form of very shallow retention ponds or wetland-bottomed channels. A perennial base flow is needed to encourage the growth of wetland species such as rushes, willows, cattails, and reeds. These slow runoff and promote settling and biological uptake. "Pocket" wetlands are typically under a tenth of an acre in size, serving developments of 10 acres or less. These are usually less reliable and efficient than larger wetlands.

<u>General applicability and experience with technique elsewhere</u>

a. Typical Applications
   - Wetland basins can be used as a follow-up BMP in a watershed, or as an onsite facility if the owner can provide sufficient water. Flood control measures may be instituted above the wetland basin.
   - Retrofitting to established developments may be very difficult due to areal requirements.
   - Arid climates or high ET rates can make maintenance of the required base flow difficult. Also, short growing seasons may inhibit vegetation growth and propagation.
   - Wetland bottom channels can be used in two ways: first, a wetland can be established in a man-made channel and can act as both a conveyance facility and a water quality enhancement facility. Perhaps the more effective option is to locate the channel downstream of a stormwater detention facility that will remove much of the sediment load. The channel then provides better quality water to the receiving water body. The detention facility should have at least a 24 hour drain period for the storm event.

b. Design Considerations (See Figures 8-7 through 8-8 for representative schematics)
   - A water budget analysis is needed to ensure the adequacy of the base flow. Also, loamy soils are needed to permit rooting of plants. A near-zero longitudinal slope is required.
   - Designed for an extended detention time of 24 hours.
   - Surface area of the wetland should account for a minimum of 3 percent of the area of the watershed draining into it.
   - The length to width ratio should be at least 2 to 1.
   - A soil depth of at least 4 inches shall be used for shallow wetland basins.
   - Approximately 75 percent of the wetland should have water depths less than 12 inches, and 25 percent of the wetland should have depths ranging from 2 to 3 feet. Of the 75 percent of the wetland that should be 12 inches deep or less, it is recommended that approximately 25 percent range from 6 inches deep to 12 inches deep, and that the remaining 50 percent be 6 inches or less in depth.
   - The deeper area of the wetland should include the outlet structure so outflow from the basin is not interfered with by sediment buildup.
   - A forebay should be established at the pond inflow points to capture larger sediments and be 4 to 6 feet deep. Direct maintenance access to the forebay should be provided with access 15 feet wide minimum and 5:1 slope maximum. Sediment depth markers should be provided.
   - If high water velocity is a potential problem, some type of energy dissipation device should be installed.
   - The designer should maximize use of pre- and post-grading pondscaping design to create both horizontal and vertical diversity and habitat.
   - A minimum of 3 aggressive wetland species (obligate wetland species) of vegetation should be planted 2 feet on center within the area of wetland that contains approximately 6 inches of water or less.
   - Three additional wetland species (facultative wetland species) of vegetation should be planted in clumps of 5 in saturated soil outside of the obligate wetland area with a spacing of 3 feet on center.
   - Wetland mulch, if used, should be spread over the high marsh area and adjacent wet zones (-6 to +6 inches of depth) to depths of 3 to 6 inches.
   - A minimum 25-foot buffer, for all but pocket wetlands, should be established and planted with riparian and upland vegetation (50-foot buffer if wildlife habitat value required in design).
   - Surrounding slopes should be stabilized by planting in order to trap sediments and some pollutants and prevent them from entering the wetland.
   - A written maintenance plan should be provided and adequate provision made for ongoing inspection and maintenance, with more intense activity for the first three years after construction.
   - The wetland should be maintained to prevent loss of area of ponded water available for emergent vegetation due to sedimentation and/or accumulation of plant material.

- To minimize maintenance as much as possible, it is recommended that wetland basins be installed on stabilized watersheds and not be used for sediment control.
- Complex topography can be maintained by bioengineering methods (such as fascines) or straw bales and geotextile rolls.
- It is recommended that the frequently flooded zone surrounding the wetland be located within approximately 10 to 20 feet from the edge of the permanent pool.
- The wetland should be designed to allow slow percolation of the runoff through the substrate (add a layer of clay for porous substrates).
- The depth of the forebay should be in excess of 3 feet and contain approximately 10 percent of the total volume of the normal pool.

c. Other Experiences with BMP
- Wetlands for storm water treatment have been used for 10 to 15 years. Estimates for removal rates vary widely in the literature, probably due to a lack of data that would produce design protocols. Some higher removal rates may be the result of testing in experimental wetlands and from wastewater treatment sites which have much higher concentrations of BOD and nutrients.
- Pollutant removal efficiencies appear to vary greatly depending on design and environment.

<u>Reported pollutant removal efficiencies</u>

- Reported data

USGS, 1986 (based on 13 sampled runoff events in Orlando, Florida):

Suspended Solids	40-94%
Total Nitrogen	0-21%
Zinc	(-29)-82%
Lead	27-94%
BOD	18%

Lakatos and McNemer, 1987:

Total Phosphorus	(-4)-90%

Wright Water Engineers, 1991:

Manganese	36-77%
Suspended Solids	29-92% (71% average on eight projects)

- Claims of high removal rates of nutrients from stormwater are not substantiated by data; these claims may be based on data from experimental wetlands and from wastewater removal rates, where influent concentrations are much higher.
- Nitrification and Denitrification are dependent on many variables, with detention time perhaps the most significant; Shaver (1994) recommends 14 days.

<u>Advantages</u>

- Aesthetics, wildlife habitat, erosion control, pollutant removal

<u>Disadvantages</u>

a. Environmental Risk and Implications
- Possible stream warming, natural wetlands alteration
- Salts and scum may accumulate and be flushed out with a major storm event.
- Possible breeding ground for pests, mosquitos. However, the Maryland study (Galli, 1992) found no mosquito larvae at any of nine sites surveyed, and there is evidence that this is the norm for constructed wetlands.
- Effectiveness at removing nitrogen and some forms of phosphates is questionable.

b. Other
   - Need for periodic sediment removal to maintain proper distribution of growth zones and water movement.

Maintenance/monitoring/enforcement considerations

a. Reliability and Consistency over Time
   - Difficult to determine, but with proper design and maintenance the wetland should perform well for an indefinite period of time.
b. Routine and Non-routine Maintenance
   - Proper depth and spatial distribution of growth zones must be maintained
   - Remove unwanted vegetation, debris and litter, accumulated sediment and organic muck.
c. Sustainability of Maintenance or Program Management
   - Maintenance is generally greatest during the first three years in order to establish vegetation.

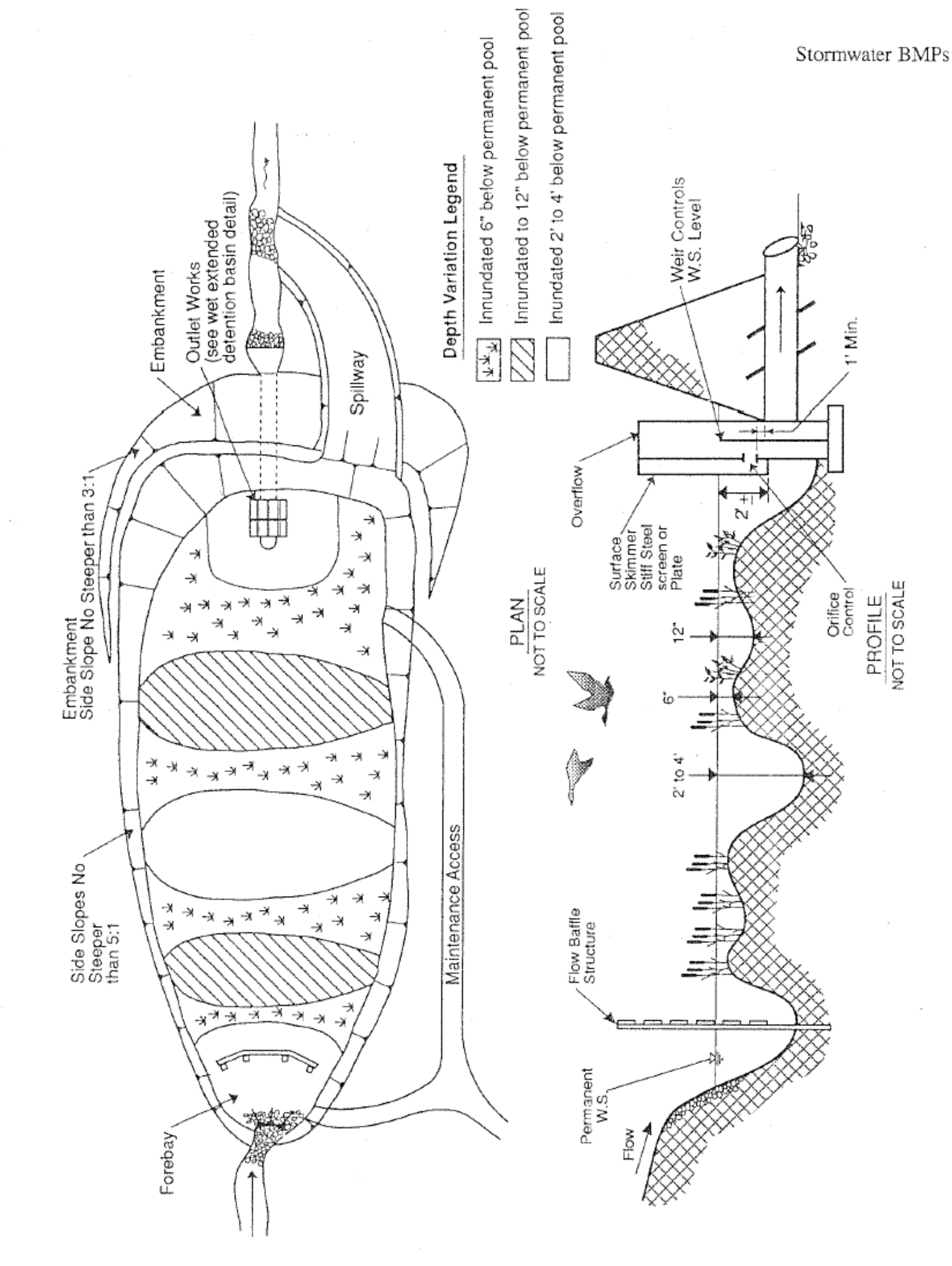
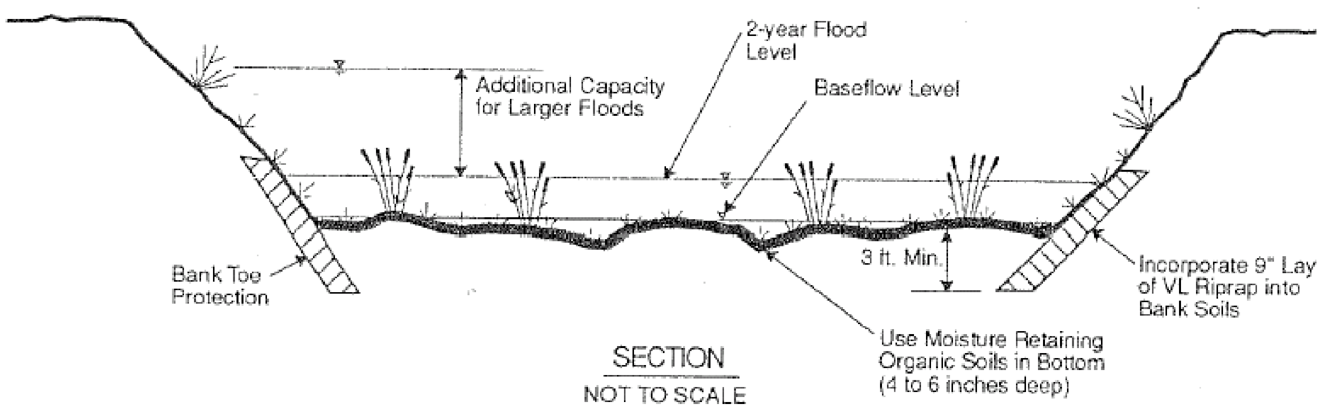


Figure 8-7 Plan and Profile of Wetland Pond

Source: Denver Urban Drainage and Flood Control District, 1992



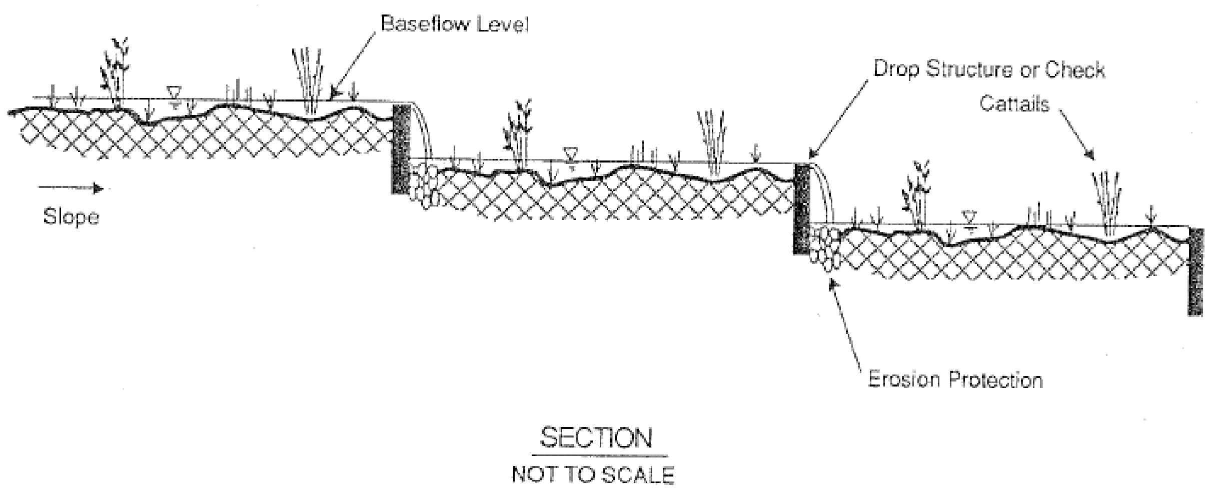


Figure 8-8 Plan and Section of Wetland Channel

Source: Denver Urban Drainage and Flood Control District, 1992

8.3.4.4 Grassed Swales

Grassed swales are densely vegetated drainageways with low-pitched sideslopes that collect and slowly convey runoff. The emphasis is on slow, shallow flow that encourages sedimentation and discourages erosion. They are set lower than the surrounding ground level, allowing runoff to enter the swales over grassy, shallow banks. Check dams may be used in conjunction with the swales to further slow the runoff. If base flow is present, wetland vegetation may also develop.

<u>General applicability and experience with technique elsewhere</u>

a. Typical Applications
   - Swales are often used to collect overland flow from impervious areas such as parking lots, buildings and roadways, as well as semi-previous areas such as grass filter strips and residential yards. They are often presented as an option to curb-and-gutter systems in order to reduce peak flow rates and reduce pollutant loading downstream. A follow-up BMP will be required to enhance water quality.

b. Design Considerations (See Figures 8-8 for representative schematic)
   - Generally well adapted for sites with ground slopes up to 3 or 4 percent, and not over 6 percent. The longitudinal slope of the swale should be less than 1 percent.
   - Limited to runoff velocities less than 1.5 to 2.5 ft/s.
   - Maximum design flow depth to be approximately 3 foot.
   - Swale cross-section should have side slopes of 4:1 (h:v) or flatter.
   - Underlying soils should have a high permeability.
   - Swale area should be tilled before grass cover is established.
   - Dense cover of a water tolerant, erosion resistant grass should be established over swale area.
   - As a BMP, grassed swales are best suited to residential or institutional areas where percentage of impervious area is relatively small.
   - Check dams can be installed in swales to promote additional infiltration. Recommended method is to sink a railroad tie halfway into the swale. Riprap stone should be placed on the downstream side to prevent erosion.
   - The NURP study concluded that adequate residence times are key to significant pollutant removal, although parameters were not determined.

c. Other Experiences with BMP
   - A New Hampshire NURP project showed heavy metal reductions of approximately 50% and COD, nitrate, and ammonia reductions around 25%.
   - Primarily an infiltration practice, so that soluble pollutants may be directed to the ground water.
   - Removal efficiencies vary widely; the reasons for this are not well understood.

<u>Reported pollutant removal efficiencies</u>

- Reported data

Whalen and Callum, 1988:

TSS	80%

Oakland, P.H., 1983:

Cadmium, total	56%
Cadmium, dissolved	42%
Zinc, dissolved	47%
Copper, dissolved	53%
Lead, dissolved	63%

Schueler, et al., 1992 (for low gradient swales with check dams):

TSS	60-80%
Total Phosphorus	20-40%
Total Nitrogen	20-40%
BOD	20-40%
Metals	60-80%

- The key to pollutant removal may be soils with high infiltration rates and flow velocities of less than 0.5 ft/sec. (Urbonas and Stahre, 1993). This may be inappropriate for areas with high ground water tables.
- Filtration, adsorption, and ion exchange may occur in the underlying soils, reducing the potential for ground water contamination.

<u>Advantages</u>

- Aesthetics.
- Effective in reducing runoff in small storm events where other BMPs are less effective.
- Can be used to limit the extent of directly connected impervious areas.

<u>Disadvantages</u>

a. Human Risk, Public Safety and Potential Liability
   - Potential for soggy yards, mosquito breeding, and more right-of-way requirement than for equivalent storm sewers.
b. Environmental Risk and Implications
   - Particularly with small storm events, the primary removal mechanism is infiltration; in areas of high ground water vulnerability, this may not be a good option.
c. Other
   - Effectiveness is limited by infiltration capacity of soils; conversely, well-draining soils may direct polluted runoff directly to ground water.

<u>Maintenance/monitoring/enforcement considerations</u>

a. Reliability and Consistency over Time
   - Dependent on proper design and maintenance
b. Routine and Non-routine Maintenance
   - Routine maintenance; grass must be mowed, some litter removal, sediment removal to maintain channel flow capacity.
   - Non-routine maintenance: replacement of damaged grass.
c. Sustainability of Maintenance or Program Management
   - Maintenance must be included in the budget to insure proper operation.

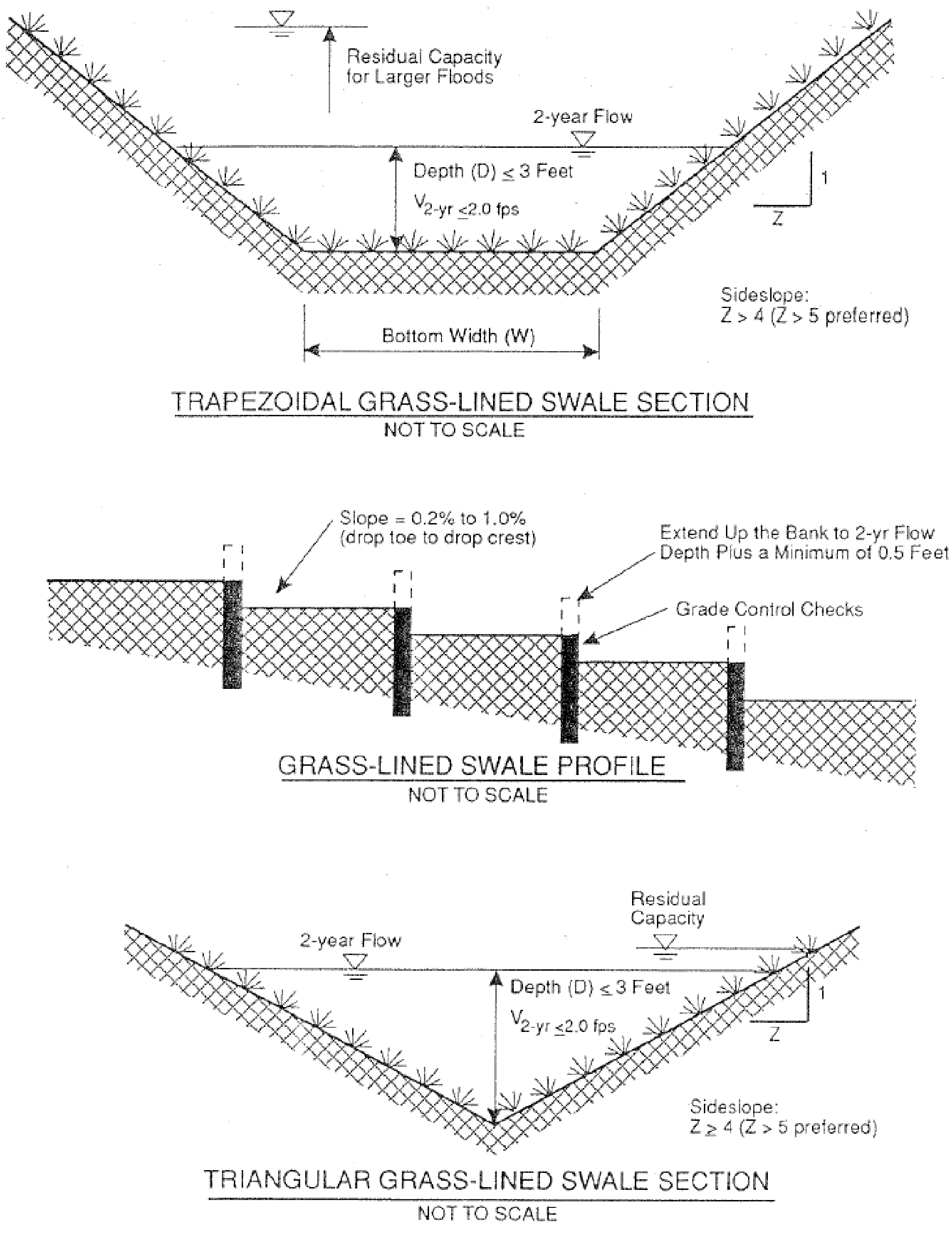


Figure 8-9 Profile and Sections of a Grassed-Lined Swale

Source: Denver Urban Drainage and Flood Control District, 1992

8.3.4.5 Filter Strips And Flow Spreaders

Filter strips are vegetated areas designed to accept sheet flow provided by flow spreaders which accept flow from an upstream development. Vegetation may take the form of grasses, meadows, forests, etc. The primary mechanisms for pollutant removal are filtration, infiltration, and settling.

<u>General applicability and experience with technique elsewhere</u>

a. Typical Applications
   - Filter strips can be used in residential and commercial sites, adjacent to impervious areas. Effectiveness depends on evenly distributed sheet flow, and limited drainage area and runoff volume. For grass filter strips, the environment must support turf-forming grasses. They have limited effectiveness in pollutant removal, and follow-up structural BMPs will still be required.

b. Design Considerations (See Figure 8-10 for representative schematic)
   - The proper function of the flow spreader is key to the performance of the filter strip. If flow is allowed to concentrate, the bulk of the filter strip will be ineffective for pollutant removal and flow reduction. This will also result in erosion over time.
   - Flow spreaders and filter strips should be limited to drainage areas of 5 acres or less.
   - Channel grade for the last 20 feet of the dike or diversion entering the level spreader should be less than or equal to 1% and designed to provide a smooth transition into spreader.
   - Grade of level spreader should be 0%.
   - Depth of level spreader as measured from the lip should be at least 6 inches.
   - Recommended length, width, and depth of flow spreader are presented in the following table:

Design Flow (cfs)	Entrance Width (ft)	Depth (ft)	End Width (ft)	Length (ft)
0 - 10	10	0.5	3	10
10 - 20	16	0.6	3	20
20 - 30	24	0.7	3	30

   - Level spreader lip should be constructed on undisturbed soil (not fill material) to uniform height and zero grade over length of the spreader.
   - Released runoff to outlet onto undisturbed stabilized areas in sheet flow and not allowed to reconcentrate below the structure.
   - Slope (S_o) of filter strip from level spreader should not exceed 10 percent.
   - The design width of the filter strip (W_G) should be the greater of the following: $W_G \geq 10$ feet or $W_G \geq 0.2L_l$, where L_l is the length of flow path of the sheet flow over the upstream impervious surface.
   - Spreader lip to be protected with erosion resistant material, such as fiberglass matting or a rigid non-erodible material for higher flows, to prevent erosion and allow vegetation to be established.
   - Wooded filter strips are preferred to gravel strips.

c. Other Experiences with BMP
   - A Maryland study (Galli, 1992) of six grass filter systems showed that all filters were showing deterioration and decreased performance 1.3 to 2.6 years after installation. Low grass height was cited as the primary cause of decreased performance; the recommendation was that grass should be left as high as possible between mowings. Also, the invasion of annual grasses and weeds that experience seasonal die-back can greatly reduce filtering performance. It was concluded that all filters would fail within three years due to erosion from high runoff rates unless substantial repairs were made.

<u>Reported pollutant removal efficiencies</u>

- Filter strips must accept stormwater runoff as overland sheet flow in order to effectively filter suspended materials out of the overland flow.
- The removal of soluble pollutants is low because the degree of infiltration provided is generally very small.
- Removals of nutrients and oxygen demand decrease as the amount of clay in the soil increases.
- Reported data

20-Foot Wide Grassed Filter Strip (Taken from Schueler, 1987):

Pollutant	Removal Rate (%)
Total Phosphorus	10
Lead	30
BOD	10
Sediment	30
Total Nitrogen	10
COD	10
Copper	30
Zinc	30

100-Foot Wide Grassed Filter Strip (Taken from Schueler, 1987):

Pollutant	Removal Rate (%)
Total Phosphorus	50
Lead	90
BOD	70
Sediment	90
Total Nitrogen	50
COD	70
Copper	90
Zinc	90

Advantages

- Aesthetics of open, green space.
- Low cost, since developments are typically required to have open space in their plans
- Grasses and shrubs or trees provide wildlife habitat.

Disadvantages

a. Human Risk, Public Safety and Potential Liability
   - Minimal
b. Environmental Risk and Implications
   - The primary flood-control mechanism is infiltration; in areas of high ground water vulnerability, this may not be a good option.
c. Other
   - On unstable slopes, soils or vegetation, rills and gullies may develop that negate the usefulness of the strips.
   - Excessive pedestrian or vehicle traffic may damage the vegetation and soils structure. The planting of shrubs and trees can help eliminate both of these disadvantages.
   - Inadequate maintenance of vegetation may result in partially denuded soils with predictable results in erosion, runoff quality and volume.

Maintenance/monitoring/enforcement considerations

a. Reliability and Consistency over Time
   - The key is proper design and maintenance.
b. Routine and Non-routine Maintenance
   - Routine maintenance consists of standard turf maintenance.
   - Non-routine maintenance consists of turf replacement, soils replacement, and regrading.

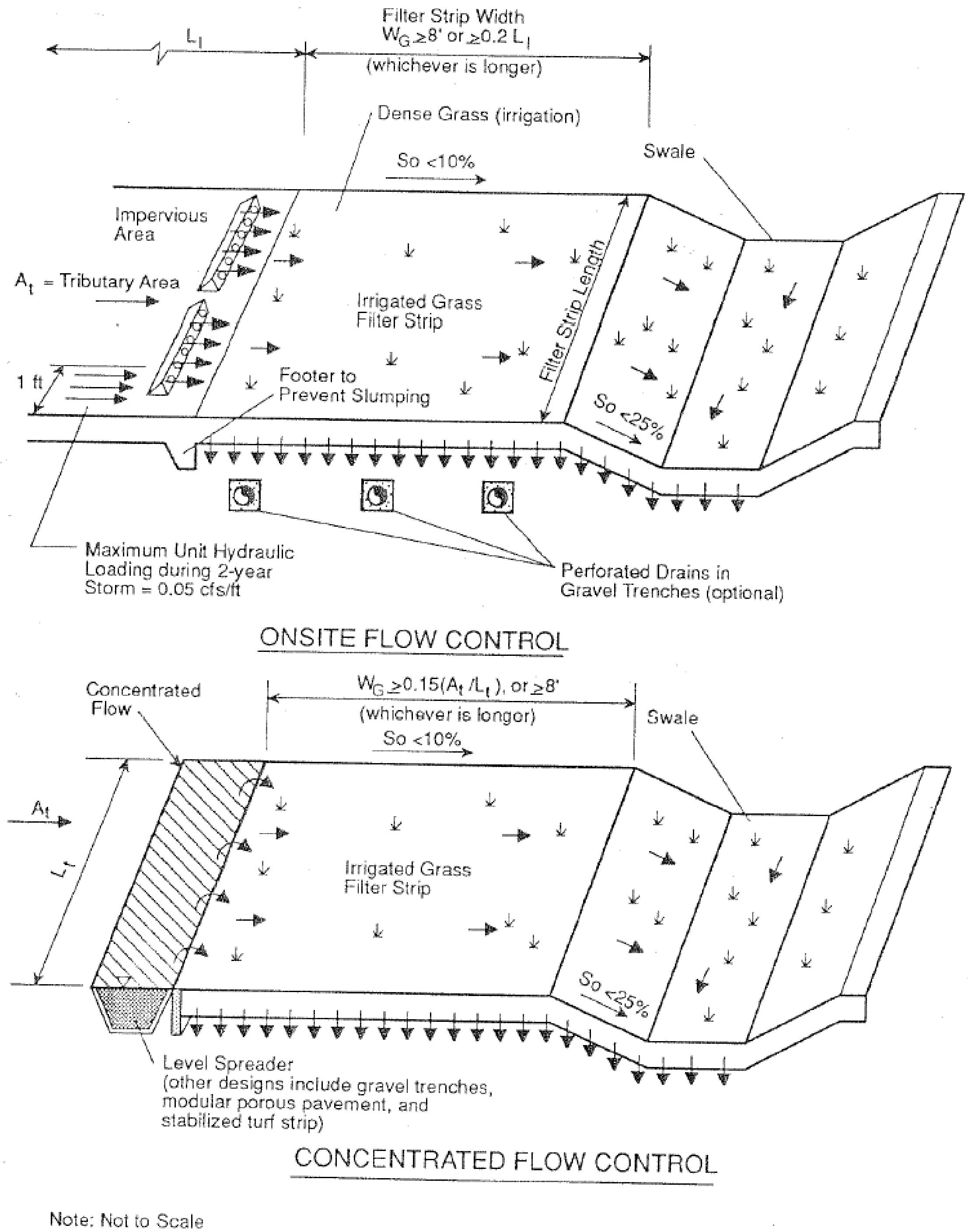


Figure 8-10 Onsite and Offsite Applications of Grass Filter Strips

Source: Denver Urban Drainage and Flood Control District, 1992

8.3.4.6 Sand Filters

In its simplest form, a sand filter is a self-contained bed of sand into which the first flush of runoff is diverted. The water is filtered as it passes through the sand, much like a slow sand filtration system for drinking water supply. The effluent is typically collected with perforated pipe and discharged to a stream or channel.

Sand filters are often placed at the outlet of detention basins to improve effluent water quality. Enhanced sand filters use layers of peat, limestone, and/or topsoil to improve removal rates. Sand trench systems are used to treat parking lot runoff, and these include the Austin sand filter and the Shaver design.

A new modification of the sand filter concept is the biofiltration pond. Using a media which has a cation exchange capacity of at least 10 milli-equivalents per 100 grams will improve metals capture (WEI, 1994). Although sand is still the predominant media of choice, clays and other compounds may be included to attain high pollutant removal rates while still providing ample drainage for the design storm event. This can typically be accomplished using a gradation of filter media, decreasing in size with depth.

General applicability and experience with technique elsewhere

a. Typical Applications
  - Sand filters have been successfully used in diverse applications for small (less than 10 acres) tributary areas (Debo, 1994).
  - Recommended for "ultra-urban" areas where area is limited and runoff is poor quality; not recommended for new construction sites.
  - Most sand filters are limited to an impervious tributary area of 5 to 10 acres. Follow-up sand filters, placed at the outlet of detention basins, may treat tributary areas in excess of 100 acres (Urbonas and Ruzzo, 1986).

b. Design Considerations (See Figure 8-11 for representative schematic)
  - Inlet structure should be designed to spread the flow uniformly across the surface of the filter media.
  - Riprap or other dissipation devices should be installed to prevent gouging of the sand media and to promote uniform flow.
  - Final sand bed depth should be at least 18 inches.
  - Underdrain pipes should consist of main collector pipes and perforated lateral branch pipes.
  - The underdrain piping should be reinforced to withstand the weight of the overburden.
  - Internal diameters of lateral branch pipes should be 4 inches or greater and perforations should be 3/8 inch.
  - Maximum spacing between rows of perforations should not exceed 6 inches.
  - All piping should be schedule 40 polyvinyl chloride or greater strength.
  - Minimum grade of piping should be 1% slope.
  - Access for cleaning all underdrain piping should be provided.
  - A presettling basin and/or biofiltration swale is recommended to pretreat runoff discharging to the sand filter.
  - A maximum spacing of 10 feet between lateral underdrain pipes is recommended.
  - The primary purpose of the sand filter is to improve stormwater quality; they have a limited ability to reduce peak flows.
  - The retrofitting of sand filters has been performed in several applications (Schueler et al. 1992).

c. Other Experiences with BMP
  - Of the nearly 1000 sand filters installed since the early '80s in the Austin, Texas area, the vast majority are working to design specifications and very few have failed (Schueler et al., 1992)

Reported pollutant removal efficiencies

- Reported data:

Pollutant	Removal Rate (%)
Total Phosphorus	65
Lead	50-70
BOD	60
Sediment	85
Total Nitrogen	50
Zinc	60-80
COD	80
Bacteria	50-60

- Sand/peat beds have higher removal effectiveness due to adsorptive properties of peat.
- Designs incorporating vegetative cover on the filter bed increase nutrient removal.
- Pretreatment (sedimentation or oil and grease removal) will enhance the performance of the filter and will decrease the maintenance frequency required to maintain effective performance.
- The sand filter does not rely on infiltration to remove peak stormwater flows or improve effluent quality. At the same time it does appear to have good removal rates of most pollutants (with the possible exception of nitrogen), with the potential to increase removal efficiencies through the addition of other media such as peat, clays, limestone, and grass cover. The use of sand with high iron content also may improve efficiencies.

Advantages

- Since infiltration is not a significant mechanism, ground water protection is maximized.
- This BMP has a proven performance record in a variety of applications.

Disadvantages

a. Human Risk, Public Safety and Potential Liability
   - Basin should be grated to prevent unauthorized entry.
b. Environmental Risk and Implications
   - Since the removed sand has been demonstrated to be non-toxic, and there is no evidence that resuspension of contaminated sediment is a problem, there is little concern for environmental problems with this BMP.
c. Other
   - Larger sand filters with no vegetative cover may be unattractive; the surface can be extremely unattractive and some have caused odor problems.
   - Sand filters are primarily for stormwater quality mitigation, not quantity or peak flow mitigation.

Maintenance/monitoring/enforcement considerations

a. Routine and Non-routine Maintenance
   - The primary routine maintenance is debris removal and scraping of the upper sand layer. This is mostly manual work.
   - Non-routine maintenance includes resanding (replacement of the sand) after enough sand has been removed that significant breakthrough occurs. In the case of the Shaver design in which a sedimentation basin is included, this must be cleaned out when the basin loses its holding capacity.

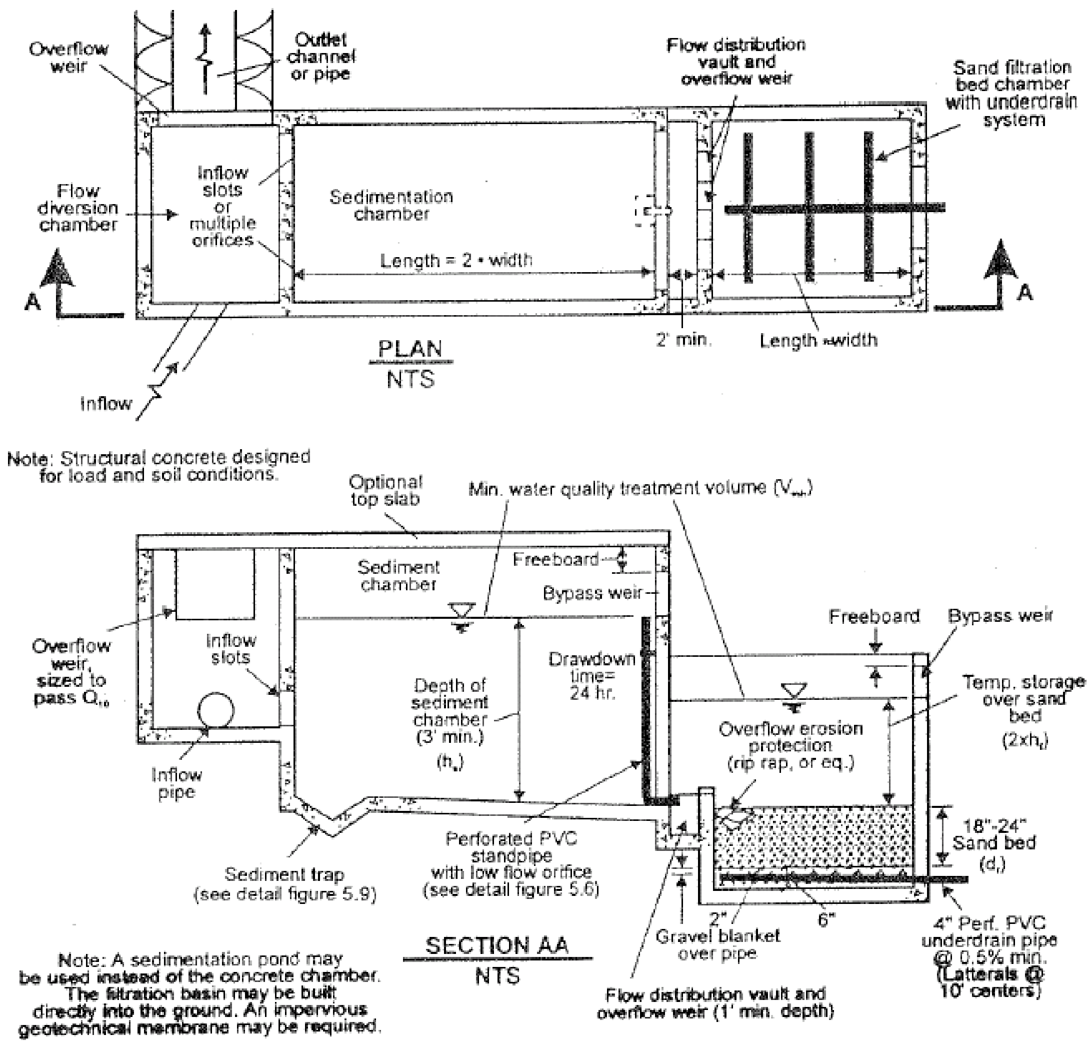


Figure 8-11 Sand Filtration Basin

Source: Center for Watershed Protection, 1996

8.3.4.7 Infiltration Trenches

Infiltration trenches can be generally described as a part of an open ditch that encourages rapid infiltration of runoff to the ground water through the placement of materials with high hydraulic conductivities. The trench is basically an excavated area within the ditch into which clean gravels are placed. The ditch should provide for slow flow rates to allow settling of suspended solids as well as the opportunity for substantial infiltration of the total intercepted flow.

<u>General applicability and experience with technique elsewhere</u>

a. Typical Applications
   - As an infiltration type BMP, use should be limited to those areas where ground water levels are well below the bottom of the trench and there is significant retention time in the soils before reaching ground water.
   - Commonly, infiltration trenches are sized to intercept and dispose of runoff from a specific design storm (typically 2-year storms).

b. Design Considerations (See Figure 8-12 for representative schematic)
   - Use in drainage areas less than 15 acres.
   - Soils that are suitable for infiltration systems are silt loam, loam, sandy loam, loamy sand, and sand.
   - Soils that have a 30 percent or greater clay content are not suitable for infiltration trenches.
   - The soil infiltration rate should be between 0.5 and 2.4 inches per hour.
   - The use of infiltration systems on fill is not allowed due to the possibility of creating an unstable subgrade.
   - A minimum of 3 feet between the bottom of the infiltration trench and the groundwater table is recommended.
   - Site slope should be less than 20 percent and trench should be horizontal.
   - The proximity of building foundations should be at least 10 feet upgrade.
   - A minimum of 100 feet should be maintained from water supply wells when the runoff is from industrial or commercial areas.
   - Water quality infiltration trenches should be preceded by a pretreatment BMP.
   - The aggregate material for the trench should consist of a clean aggregate with a maximum diameter of 3 inches and a minimum diameter of 1.5 inches.
   - The aggregate should be graded such that there will be few aggregates smaller than the selected size. For design purposes, void space for these aggregates may be assumed to be in the range of 30 percent to 40 percent.
   - The aggregate should be completely surrounded with an engineering filter fabric. If the trench has an aggregate surface, filter fabric should surround all aggregate fill material except for the top one foot.
   - Bypass larger flows.
   - To reduce clogging of the trench with sediments, a sump pit or a filter strip and flow spreader should be used to treat water entering the ditch.
   - Since infiltration is the primary mechanism for pollutant removal from runoff, the infiltration trench could actually impair ground water quality in fast-draining soils. Some biological uptake of nutrients may occur in well-vegetated ditches, and removal of sediments will remove some associated heavy metals.
   - The most important aspect to the potential for success of an infiltration trench is ground water levels. If the trench is easily inundated by high ground water levels or ground water mounding due to infiltration of runoff, the trench will simply become an open channel. Thus, the trench can fail in two modes; high infiltration rates to ground water that is near the base of the trench, or low infiltration rates due to poor draining soils or clogging of the trench with sediments.
   - Infiltration trenches work well for residential lots, commercial areas, parking lots, and open space areas.
   - Infiltration systems should not be constructed until all construction areas draining to them are fully stabilized.
   - An analysis should be made to determine any possible adverse effects of seepage zones when there are nearby building foundations, basements, roads, parking lots, or sloping sites.

c. Other Experiences with BMP
   - In a Maryland study (Galli, 1992) of 38 infiltration trenches, losses of infiltration capacity were caused by high water tables, poorly draining soils, and inadequately sized filter strips.
   - As mentioned previously, the term 'infiltration trench' implies that runoff water is intercepted and directed to the ground water. Unless other BMPs are included to remove pollutants before the runoff enters the trench,

ground water quality may be compromised. However, Urbonas and Stahre (1993) state that data available so far shows that ground water quality does not degrade noticeably due to infiltration of stormwater from residential and many types of commercial developments. Filtration, adsorption, and ion exchange may occur in the underlying soils.

Reported pollutant removal efficiencies

- Removal rates have been estimated by Schueler (1987) using assumed efficiencies and modeling. These show very high removal rates for TSS, Nitrogen, Phosphorous, Zinc and BOD.

Advantages

- The combination of water conveyance, runoff reduction, lowering of peak flows, and pollutant removal make this an effective BMP.

Disadvantages

a. Human Risk, Public Safety and Potential Liability
   - Minimal
b. Environmental Risk and Implications
   - The use of infiltration as the primary pollutant reduction mechanism may increase ground water contamination by highly soluble contaminants in fast-draining soils and/or high water level conditions.
c. Other
   - If a trench becomes clogged with sediments, it simply stops functioning. The gravel must be removed and replaced with clean gravel, and it may be necessary to remove soils lining the trench which have also become clogged. The Maryland study (Galli, 1992) gave the following results for 38 trenches averaging 2.4 years old (maximum 5.1 years):

     Operating as designed?

Pre-treatment type	Yes	No	Unknown
Sump Pit	48.4%	42.0%	9.6%
Grass Filter	42.9%	57.1%	0.0%
TOTAL	47.4%	44.7%	7.9%

   - Conversely, a state survey of infiltration devices in Maryland in 1986 showed 80% of the infiltration trenches working as designed (Clement and Pensyl, 1987). These results are questionable, however, as 50% of the trenches had no observation wells to determine if there was standing water under the gravel. Such trenches were reported as operating properly even though this primary evaluation criteria could not be determined.

Maintenance/monitoring/enforcement considerations

a. Reliability and Consistency over Time
   - If non-routine maintenance is performed correctly, there should be little degradation in performance
b. Routine and Non-routine Maintenance
   - Routine maintenance includes debris and litter removal and control of overgrown vegetation.
   - Non-routine maintenance involves a clogged trench which requires complete removal and replacement of the gravel as well as surrounding clogged native soils. This can be greatly reduced by proper design and routine maintenance.
c. Sustainability of Maintenance or Program Management
   - If the trench becomes fully clogged, complete rehabilitation may cost as much as initial construction; if funding is private, the trench may go unrepaired.

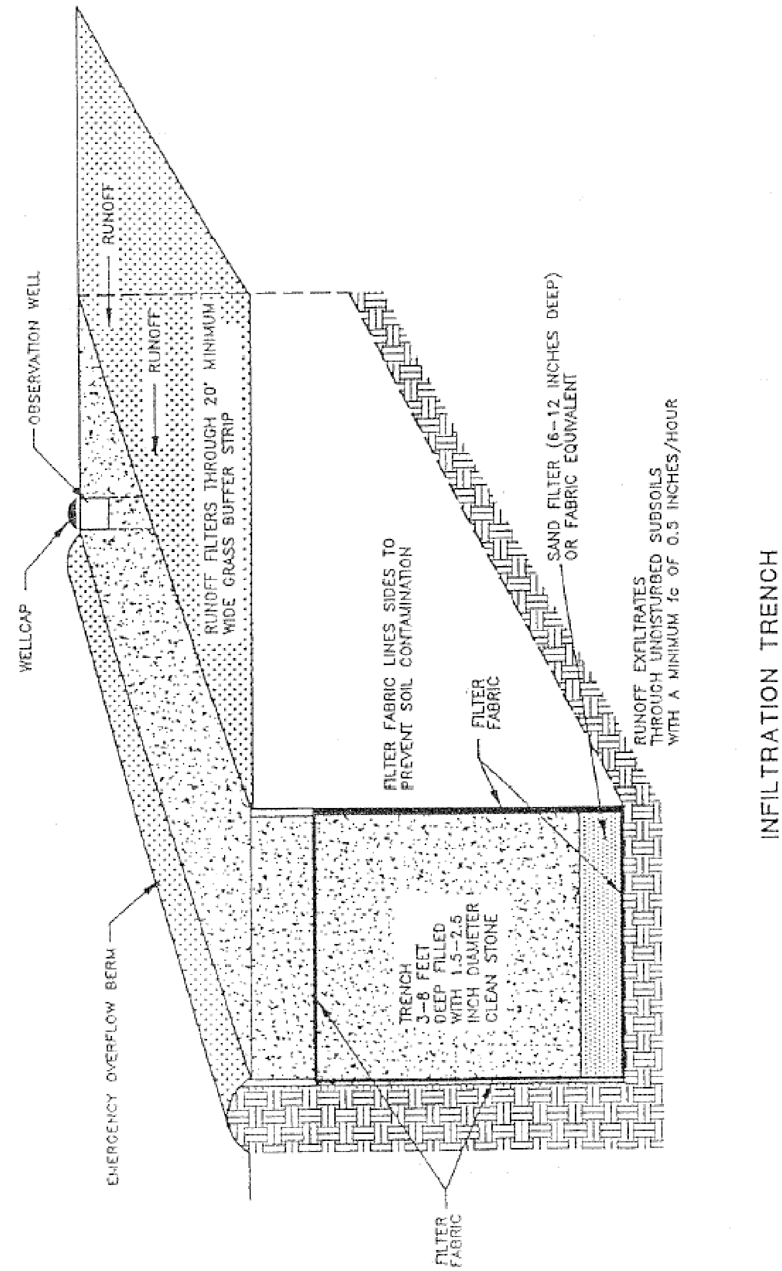


Figure 8-12 Infiltration Trench

Source: Stormwater Management Manual For The Puget Sound Basin

8.3.4.8 Porous Pavement

There are two forms of porous pavement: modular block, which is made porous through its structure, and poured-in-place concrete or asphalt which is porous due to the mix of the materials.

Modular block porous pavement consists of perforated concrete slab units underlain with gravel. The surface perforations are filled with coarse sand or sandy turf. It is used in low traffic areas to accommodate vehicles while facilitating stormwater runoff at the source. It should be placed in a concrete grid that restricts horizontal movement of infiltrated water through the underlying gravels.

Poured-in-place porous concrete or asphalt is generally placed over a substantial layer of granular base (Urbonas and Stahre, 1993). The pavement is similar to conventional materials, except for the elimination of sand and fines from the mix.

If infiltration to ground water is not desired, a liner may be used along with perforated pipe and a flow regulator to slowly drain the water away over a 6 to 12 hour period.

<u>General applicability and experience with technique elsewhere</u>

a. Typical Applications
   - This is exclusively an on-site BMP that should never be used for treating water with high sediment loads. This is particularly true for porous concrete or asphalt, which are primarily designed to remove pollutants deposited on the pavements from the atmosphere (Schueler, 1987)
   - Modular block pavement is applicable to low traffic zones and permeable upper soils with ground water no less than 4 feet from the gravel bedding.
   - The use of porous concrete or asphalt is not well-received in colder climates where freeze-thaw cycles may fracture the pavement. Despite this, it has been found that properly designed systems are not damaged by such processes (Debo, 1994).

b. Design Considerations (See Figures 8-13 and 8-14 for representative schematics)
   - Either form of porous pavement must be limited to low traffic areas with limited deposition of clays and fines which could clog the pavement.
   - As infiltration is the primary mechanism of pollutant removal, areas with high ground water vulnerability may not be good choices for this BMP.
   - Large soil surface areas are needed for maximum exfiltration and pollutant removal.
   - Soil infiltration rate should be greater than 0.27 inches per hour and clay content less than 30 percent.
   - Only feasible on sites with gentle slopes (less than 5%).
   - Design infiltration rate should be equal to ½ of the infiltration rate determined from soil textural analysis.
   - Minimum of 3 feet between stone reservoir level and seasonally high water table.
   - Should not be constructed over fill soils.
   - Vegetative strip or diversion berm required to protect pavement area from off-site runoff before, during, and after construction.
   - If porous pavement areas receive runoff from off-site areas, a pretreatment facility should be constructed to remove oil, grit, and sediments before entering the porous pavement.
   - Dry subgrade should be covered with engineering filter fabric such as Mirafi #14N or equal on bottom and sides.
   - Pavement section consisting of 4 layers as shown on Figure 8-9.
   - Stone should be clean, washed, stone meeting roadway standards.
   - Reservoir base course should consist of 1" to 3" crushed stone aggregate compacted lightly at the depth required to achieve design storage.
   - Filter courses to be ½" crushed stone aggregate at a 1" to 2" depth.
   - Surface course should be laid in 1 lift at the design depth with compaction done while the surface is cool enough to resist a 10-ton roller. Only 1 or 2 passes are required.
   - After final rolling, no vehicular traffic should be permitted on pavement until cooling and hardening, a minimum of 1 day.
   - Stone reservoir should be designed to completely drain within a maximum of 2 to 3 days after design storm event.
   - The porous pavement site should be posted with signs indicating the nature of the surface and warning against resurfacing, using abrasive, and parking heavy equipment.

- An observation well should be installed on the downslope end of the porous pavement area to monitor runoff clearance rates. The observation well should consist of perforated PVC pipe, 4 to 6 inches in diameter, constructed flush with the ground. The top of the observation well should be capped to discourage vandalism and tampering.
- Limited in application to parking lots, service roads, emergency and utility access lanes, and other low traffic areas.
- Limited to sites between 1/4 acre and 10 acres.
- Should not be constructed near groundwater drinking supplies.
- Heavy equipment should be prevented from compacting the underlying soils before and during construction.

c. Other Experiences with BMP
- Modular block BMPs have been in use since the mid-1970's. Field data is lacking to quantify long-term performance as an infiltration device, but anecdotal evidence indicates it is reliable. Pratt (1990) found that if excessive sediment deposition is controlled, modular paved surfaces can function for at least 15 years.
- Anecdotal experience indicates that unless careful cleaning with vacuum cleaners is performed on a frequent basis, the pavement will seal within 1-3 years. Ultimately, it will seal anyway and cannot be repaired; the only option appears to be replacement. (Urbonas and Stahre, 1993). Porous pavement sites have one of the highest failure rates of any BMP. At the same time, when working properly it can be a very cost-effective BMP for commercial sites.
- As infiltration is a primary mechanism, the potential for pollutant discharge to ground water, particularly soluble pollutants, is significant. Additionally, there is concern that hydrocarbons may be leached from asphalt material, thereby increasing the contaminant load.

Reported pollutant removal efficiencies

- Reported data

  Representative long-term pollutant removal rate for porous pavement sites designed for the 2-year storm are as follows (Debo, 1995):

Pollutant	Removal Rate (%)
Total Phosphorus	65
Lead	98
Sediment	95
Total Nitrogen	85
COD	82
Zinc	99

- Suspended sediment and associated metals, oil and grease removal may be high, as long as the pavement remains porous. Removal rates estimates vary from 0 to 95 percent. Soluble constituent removal is likely lower, depending on the materials used. Filtration, adsorption, and ion exchange may occur in the underlying soils. With good drainage, soluble constituents are likely to show low removal efficiencies (UDFCD, 1992).

Advantages

- Low maintenance for modular block pavement.
- Slows and reduces runoff, reducing the need for expensive detention facilities.

Disadvantages

a. Environmental Risk and Implications
- Fast draining soils can encourage ground water pollution from soluble metals and other pollutants.
- Risk can vary from very minor to great, depending on how well the system is functioning

b. Other
- Large silt and sand loads (e.g. from construction sites) may accelerate the clogging of the pavement pores, requiring expensive removal of sediments.

- Porous concrete or asphalt tends to seal in 1-3 years unless vacuum cleaning is done frequently; even then, it will eventually seal. The need for vacuum cleaning makes this option more expensive for routine maintenance.

<u>Maintenance/monitoring/enforcement considerations</u>

a. Reliability and Consistency over Time
    - All porous pavement designs will degrade in performance over time, with careful maintenance only incrementally increasing its operational lifespan.

b. Routine and Non-routine Maintenance
    - Maintenance is minimal for modular block except when the surface becomes clogged. This will require expensive non-routine maintenance in the form of removing the blocks and the underlying clogged gravels. Routine (quarterly) vacuum sweeping and high pressure water washing of porous asphalt is required to prevent clogging. Non-routine maintenance consists of complete replacement, and may be required in as little as one year's time.
    - When turf is used with modular block, lawn care maintenance is needed.
    - Sand or ash should not be applied to porous pavement.
    - Spot clogging of the porous pavement layer can be relieved by drilling 1/4" holes through the porous asphalt layer every few feet.

c. Sustainability of Maintenance or Program Management
    - The obvious limitation is the need for expensive non-routine repairs or replacement. If privately owned, this expense may preclude necessary work. If publicly owned, there may be insufficient funds budgeted for maintenance.

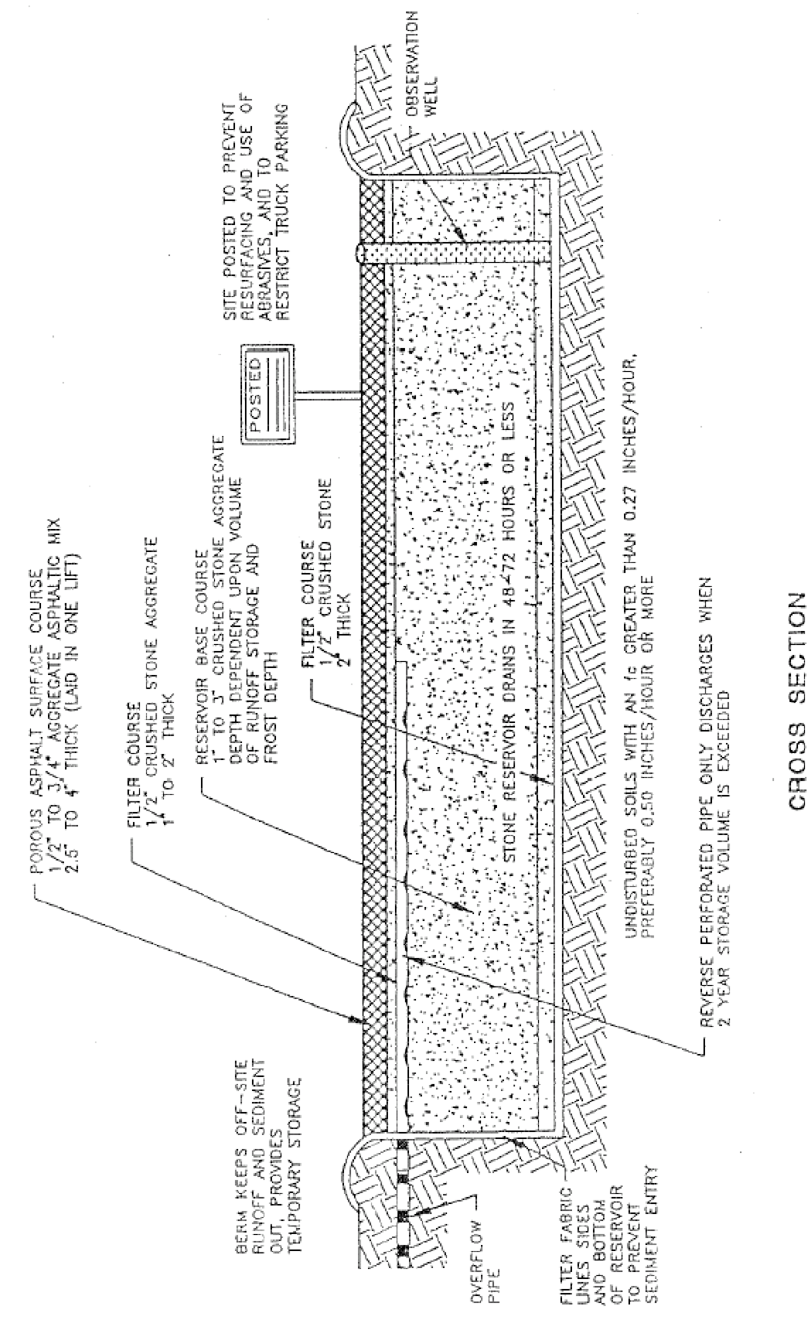


Figure 8-13 Design Schematic For Porous Pavement

Source: Controlling Urban Runoff

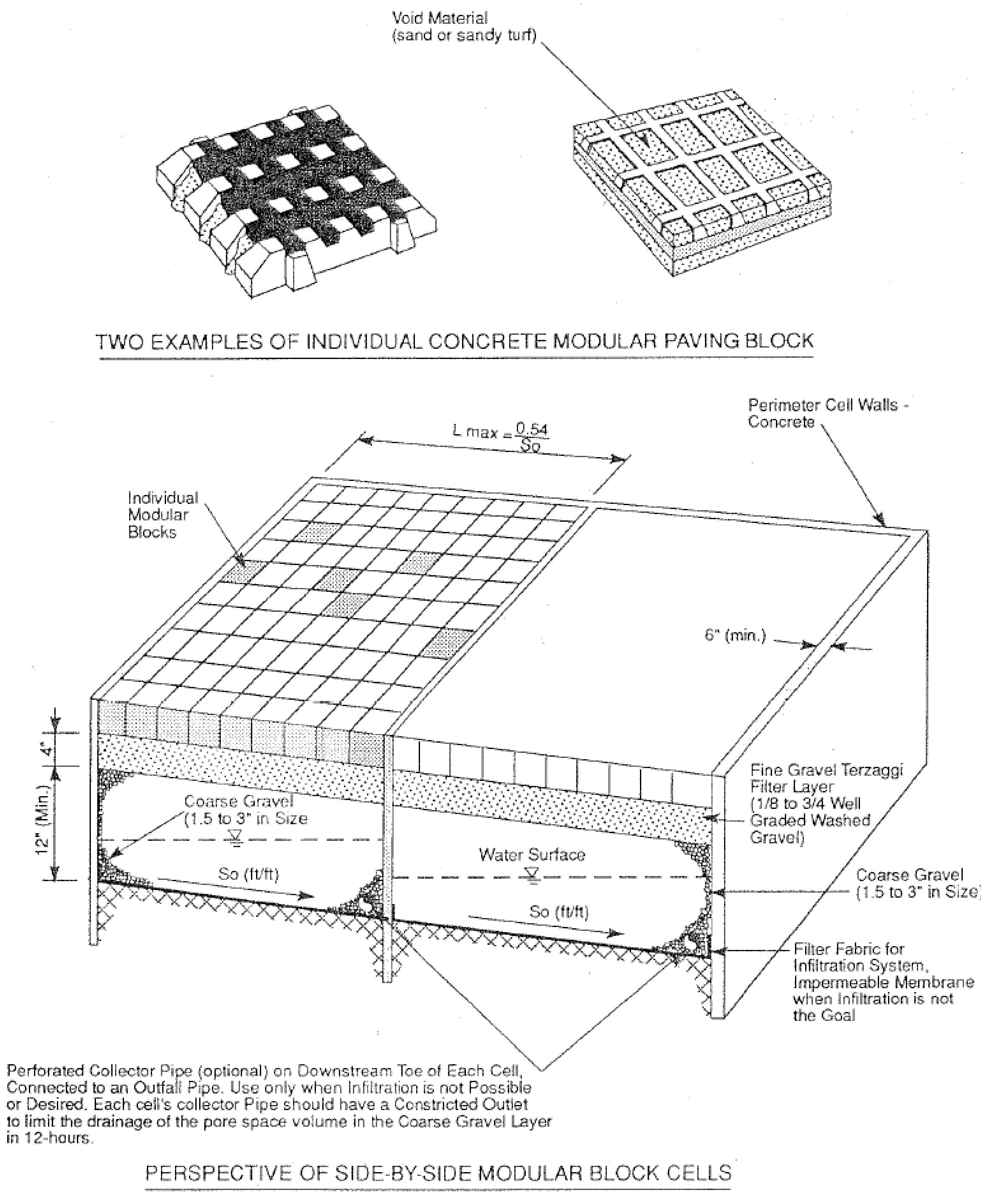


Figure 8-14 Design Schematic For Modular Block Porous Pavement

Source: Urban Drainage and Flood Control District, 1992

8.3.4.9 Oil/Grit Separators

Also known as a water quality inlet, an oil and grit separator is a three-stage underground retention system designed to remove heavy particulates and hydrocarbons from runoff. The first chamber allows for sedimentation. The second chamber has an inverted elbow for an outlet, such that oil is trapped at the surface. The third chamber directs the water out.

General applicability and experience with technique elsewhere

a. Typical Applications
   - This BMP was originally designed for industrial applications, rater than urban storm water applications. When translating to a storm water BMP, two problems arise: (1) an expectation of removal of pollutants other than oil and grit is created; and (2) widely varied flows can overwhelm and make ineffective a BMP that was designed for steady low flows and not "flashy" high flows.
   - The most effective use of this BMP is in capturing runoff from small, high density sites where high concentrations of oils in runoff are expected.
   - Oil/Grit separators are most frequently used in highly urbanized areas where other BMPs cannot be used due to space limitations.

b. Design Considerations (See Figure 8-15 for representative schematic)
   - Tributary area is usually limited to two acres or less of mostly impervious surfaces. This is primarily a water quality rather than quantity mitigation BMP.
   - Separator should be structurally sound and designed for acceptable traffic loadings where subject to traffic loadings.
   - Separator should be designed to be water tight.
   - Volume of separator should be at least 400 cubic feet per acre tributary to the facility (first two chambers).
   - Forebay or first chamber should be designed to collect floatables and larger settleable solids. Its surface area should not be less than 20 square feet per 10,000 square feet of drainage area.
   - Oil absorbent pads, oil skimmers, or other approved methods for removing accumulated oil should be provided.
   - Separator pool should be at least 4 feet deep.
   - Manholes should be provided to each chamber to provide access for cleaning.
   - Separator to be located close to the source before pollutants are conveyed to storm sewers or other BMPs.
   - Use only on sites of less than one acre.
   - Provide perforated covers as trash racks on orifices leading from first to second chamber.

c. Other Experiences with BMP
   - Experience demonstrates that these have limited pollutant removal ability with resuspension of trapped particulates common. Since residues tend to be toxic, disposal is a problem. As a result, there are no current clean-out and disposal procedures.

Reported pollutant removal efficiencies

- Pollutant removal ability is limited. This is due to the lack of clean-out and disposal procedures and the tendency for trapped sediments to resuspend. One study (Shepp et al. 1992) showed that the depth of trapped sediments in over 120 separators was less than two inches. More than eighty percent of the sediments were coarse-grained grit and organic matter. Additionally, the amount of trapped sediment did not correlate with age, indicating that resuspension is a common failure mode. Sediments that were trapped were very oily in nature.
- The positive aspect to this BMP is that examination of the sediments shows that the adsorbed pollutants match those found in receiving water bodies, indicating that the right target pollutants are being addressed.
- Three chamber oil and grit devices may remove from 60 to 80 percent of the hydrocarbons found in parking lot and street runoff.
- Three chamber oil and grit devices may also remove a small portion of the suspended sediment and trace metal loads.

Advantages

- Can be used in highly urbanized areas where other BMPs cannot be used.
- Trapping of floatable trash and debris and possible reduction of hydrocarbon loadings from impervious areas.
- They do not rely on infiltration, so that direct input of runoff into the ground water is unlikely.

Disadvantages

a. Human Risk, Public Safety and Potential Liability
   - The trapped sediments are highly toxic (organics)
b. Environmental Risk and Implications
   - Trapped sediments are highly toxic and cannot be easily disposed of, resulting in the generation of a toxic waste.
   - Large storm events can cause resuspension of trapped solids, resulting in a pulse of very poor quality effluent.
c. Other
   - The lack of a practical disposal method for the toxic sediments results in improper maintenance that causes failure of the system.

Maintenance/monitoring/enforcement considerations

a. Reliability and Consistency over Time
   - Pollutant removal performance likely drops off very quickly after a few months.
b. Routine and Non-routine Maintenance
   - Sediments are toxic and cannot be landfilled; therefore, maintenance involves only the removal of floatables.
   - Cleaning on a quarterly basis should be a minimum schedule with more intense land uses such as gas stations requiring cleaning as often as monthly.
   - Cleaning should include pumping out waste water and grit and having the water processed to remove oils and metals.

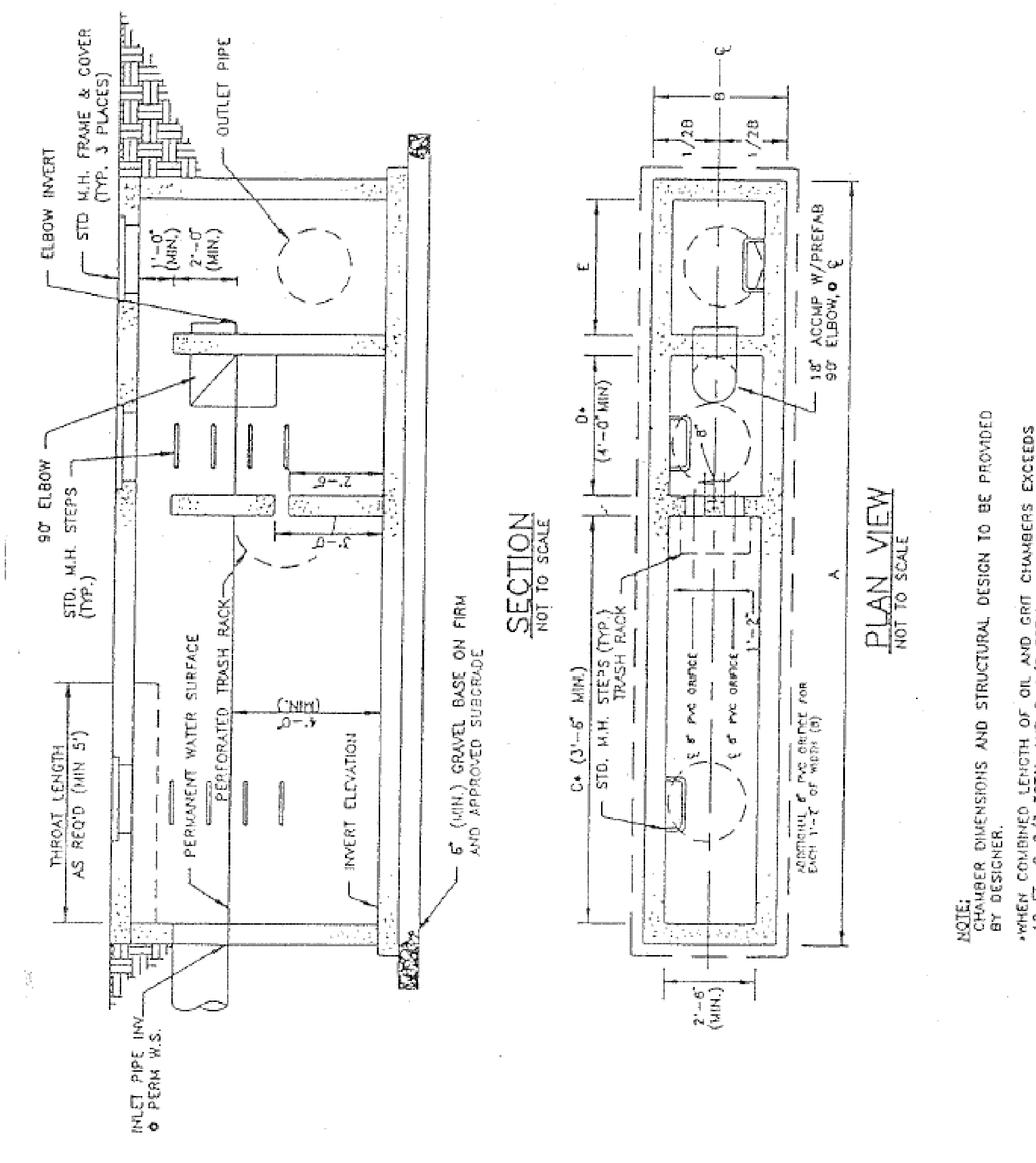


Figure 8-15 Oil/Grit Separator

Source: Maryland Department Of The Environment Sediment And Stormwater Administration

8.3.4.10 Grate Inlet Inserts

Grate inlet inserts are a newer type of oil/grit separator consisting of an insert that fits inside a standard grate inlet. Normally the inserts are made of a stainless steel, aluminum or cast iron framework which sits on the lip of the inlet grate frame and hangs down into the catch basin inlet chamber. One or more trays of filtration media are placed into the framework. The top screen or tray is usually a sediment trap. The flow enters the top of the filtration tray and filters through. Filtering media can be made of activated charcoal (for pesticides, fertilizer and metals removal), reconstituted wood fiber (primarily for oil and grease) or household fiberglass insulation. Excess flow beyond the capacity of the media bed or due to media clogging is routed over the sides of the tray(s) and out through the bottom or side of the framework. The capacity of the overflow is designed to equal or exceed the capacity of the grate.

One or more trays of filtering media, sometimes of different types, are then placed either stacked or in a rack below the sediment trap and screen. The media can be disposed in a manner similar to oil and grit chamber sediment though it may need to be tested once to see if it is a hazardous waste.

<u>General applicability and experience with technique elsewhere</u>

a. Typical Applications
   - These can be used in most places where catch basins are installed.
   - It appears to be an ideal application for retrofitting such areas as parking lots, gas stations, vehicle maintenance areas, "dirty" neighborhoods or industrial areas, etc.

b. Design Considerations
   - Several companies produce such inserts, or they can be fabricated from common materials. The materials which make up the framework and the trays should be highly resistant to corrosion, easy to install manually and fit standard inlets.

<u>Reported pollutant removal efficiencies</u>

- Pollutant removal rate information is limited to a few installations, some bench tests and visual inspections. But it appears to be quite high for oil and grease and metals (above 80%-90%) (Debo, 1994).

<u>Advantages</u>

- It is easy to install (may take as little as 15 minutes), relatively inexpensive, requires no construction or modifications of existing catch basins, easy to maintain by property owners, and is targeted toward the major pollutants from these areas.

<u>Disadvantages</u>

a. Human Risk, Public Safety and Potential Liability
   - Similar to conventional catch basins

b. Environmental Risk and Implications
   - Difficult to quantify, but should be significantly improved over conventional basins.

<u>Maintenance/monitoring/enforcement considerations</u>

a. Reliability and Consistency over Time
   - Unknown, but proper maintenance should provide a reasonable lifetime vs. costs. There is some question concerning the chemical integrity and longevity of the fiberglass in harsher environments.

b. Routine and Non-routine Maintenance
   - Maintenance requirements include inspecting the flow integrity of the system and replacement of the filtration media. Quarterly replacement is a good starting estimate though the installations should be checked after wet periods and periodically.

c. Sustainability of Maintenance or Program Management
   - Routine maintenance is required and must be built into the cost estimate for the system.

8.3.4.11 Cluster Development

Communities have repeatedly found that property adjacent to protected wetlands, floodplains, shorelines and forests constitutes an excellent location for development (U.S. EPA, 1995). One of the drawbacks to developing these locations is that the very amenities that were the selling points of the sale are often removed, or destroyed in the development process. Many communities and home owners associations which allow the destruction of these one functioning watershed components are later having to pay thousands of dollars to repair and maintain facilities that had once been naturally provided to control pollutant removal and flood control. A site design practice known as “cluster”or open space development, preserves these natural resources or “environmentally sensitive areas” to function and protect the area that is being developed. Under this practice housing still consists of the same number of single family housing dwellings or multi-family housing units or a combination of both. Cluster development creates protected open space by consolidating the buildings onto smaller lots adjacent to the preserved amenities which increase the value of the lots in comparison to subdivisions that have removed these natural areas. In addition the property in a cluster development maintains a higher resale value and does not require the same amount of infrastructure as that of a conventionally designed subdivision.

Figure 8-16 Conventional Development

Figure 8-17 Cluster Development

Some of the typical savings associate with the using the cluster vs the conventional development include the following:

- reduced lineal ft. of curb and gutter roadway
- reduced lineal ft. of sidewalks
- reduced lineal ft. of storm drain system
- reduced lineal ft. of sanitary sewer
- reduced grading
- reduced vegetative maintenance
- reduced lawn water usage
- reduced phosphorus and nitrogen loads
- reduced pervious surfaces
- increased ground water infiltration recharge

8.3.4.12 Conservation Easement

The use of a cluster or open space development is often planned in conjunction with a conservation easement to protect “environmentally sensitive areas” such as wetlands, floodplains, shorelines and wooded areas. A conservation easement is a set of restrictions a landowner voluntarily places on his or her property in order to preserve its conservation values. The conservation values of the property and the restrictions created to preserve those values, along with the rights reserved by the landowner, are detailed in a legal document known as a conservation easement. This document is filed with the governing entity in which the easement is bounded by at the time of filing. The easement is then reviewed by several departments for conformance with the Comprehensive Plan, zoning issues, future construction projects, etc..., an outline of this process is available upon request from the Planning Department

A conservation easement is conveyed to a government agency or nonprofit conservation organization qualified to hold and enforce easements. Most conservation easements are perpetual. They apply to the current owner and all future landowners, permanently protecting the property. Each conservation easement is unique, specifically tailored to the particular land being protected as well as to the particular situation of the landowner.

<u>What restrictions are included in a conservation easement?</u>

First, conservation values are defined and then restrictions are created to protect those values. Restrictions may apply to all of a landowner's property or to only a portion of it. Typically, easements address subdivision, commercial or industrial uses, mining, construction of buildings or roads, utilities, disturbance of the vegetation or topography and any activities on the property that might interfere with the conservation purpose for the easement.

For example, an easement preserving rare woodland habitat may require that the property be left entirely in its natural state, prohibiting all development. Or, to protect a lake or stream, an easement may allow limited inland construction of buildings or trails while restricting such activities along the more fragile shoreline. Some easements may permit continued farming or limited timbering. Others may provide for enhancement of wildlife habitat or restoration of native prairie.

<u>What are the effects of a conservation easement on a landowner's property rights?</u>

A landowner retains all rights to the property not specifically restricted or relinquished by the easement. The landowner still owns the land and has the right to use it for any purpose that is consistent with the easement, to sell, to transfer or to leave it through a will. Typically, landowners also retain the right to restrict public access.

<u>Financial Benefits</u>

Conservation easements may reduce a landowner's tax obligations in number of ways:

- Income Taxes: As with other charitable contributions, the donation of a conservation easement under certain circumstances may allow the landowner to claim a federal income tax deduction for the value of the easement.
- Estate taxes: A gift of a conservation easement may also reduce federal estate taxes, making this an effective way to transfer land to the next generation with its natural features intact.
- Property Taxes: An easement that reduces the value of the land may result in lowered annual property taxes.

NOTE: The rules governing all of these potential tax savings are complex and require the advice of professional advisors.

8.3.4.13 Stream Buffers

Buffers can be applied to new development by establishing specific preservation areas and sustaining management through easements or community associations. For existing developed areas, an easement may be needed from adjoining landowners.

In many regions of the country, the benefits of buffers are amplified if they are managed in a forested condition. In some settings, buffers can remove pollutants traveling in storm water or ground water. Shoreline and stream buffers situated in flat soils have been found to be effective in removing sediment, nutrients, and bacteria from storm water runoff and septic system effluent in a wide variety of rural and agricultural settings.

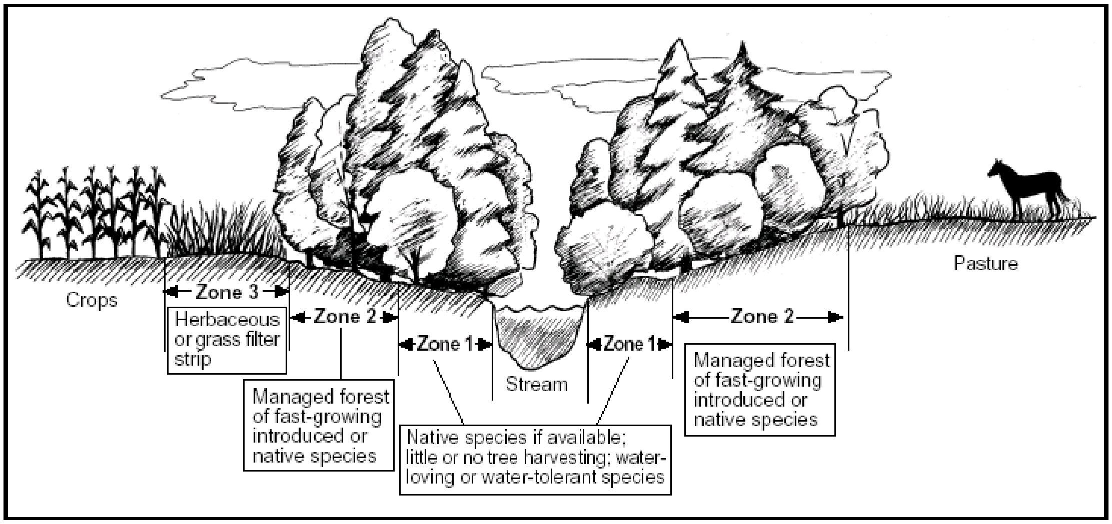


Figure 8-18 Stream Buffer Schematic

Siting and Design Considerations

There are ten key criteria to consider when establishing a stream buffer:

- Minimum total buffer width
- Three-zone buffer system
- Mature forest as a vegetative target
- Conditions for buffer expansion or contraction
- Physical delineation requirements
- Conditions where buffer can be crossed
- Integrating storm water and storm water management within the buffer
- Buffer limit review
- Buffer education, inspection, and enforcement
- Buffer flexibility

In general, a minimum base width of at least 100 feet is recommended to provide adequate stream protection. The three-zone buffer system, consisting of inner, middle, and outer zones, is an effective technique for establishing a buffer. The zones are distinguished by function, width, vegetative target, and allowable uses. The inner zone protects physical and ecological integrity and is a minimum of 25 feet plus wetland and critical habitats. The vegetative target consists of mature forest, and allowable uses are very restricted (flood controls, utility right-of-ways, footpaths, etc.).

The middle zone provides distance between upland development and the inner zone and is typically 50 to 100 feet, depending on stream order, slope, and 100-year floodplain. The vegetative target for this zone is managed forest, and usage is restricted to some recreational uses, some storm water BMPs, and bike paths. The outer zone functions to prevent encroachment and filter backyard runoff. The width is at least 25 feet and, while forest is encouraged, turfgrass can be a vegetative target. Uses for the outer zone are unrestricted and can include lawn, garden, compost, yard wastes, and most storm water BMPs.

For optimal storm water treatment, the following buffer designs are recommended. The buffer should be composed of three lateral zones: a storm water depression area that leads to a grass filter strip that in turn leads to a forested buffer. The storm water depression is designed to capture and store storm water during smaller storm events and bypass larger stormflows directly into a channel. The captured runoff within the storm water depression can then be spread across a grass filter designed for sheetflow conditions for the water quality storm. The grass filter then discharges into a wider forest buffer designed to have zero discharge of surface runoff to the stream (i.e., full infiltration of sheetflow).

Stream buffers must be highly engineered in order to satisfy these demanding hydrologic and hydraulic conditions. In particular, simple structures are needed to store, split, and spread surface runoff within the storm water depression area. Although past efforts to engineer urban stream buffers were plagued by hydraulic failures and maintenance problems, recent experience with similar bioretention areas has been much more positive (Claytor and Schueler, 1996). Consequently, it may be useful to consider elements of bioretention design for the first zone of an urban stream buffer (shallow ponding depths, partial underdrains, drop inlet bypass, etc).

Maintenance Considerations

An effective buffer management plan should include establishment, management, and distinctions of allowable and unallowable uses in the buffer zones. Buffer boundaries should be well defined and visible before, during, and after construction. Without clear signs or markers defining the buffer, boundaries become invisible to local governments, contractors, and residents. Buffers designed to capture storm water runoff from urban areas will require more maintenance if the first zone is designated as a bioretention or other engineered depression area.

Effectiveness

The pollutant removal effectiveness of buffers depends on the design of the buffer; while water pollution hazard setbacks are designed to prevent possible contamination from neighboring land uses, they are not designed for pollutant removal during a storm. With vegetated buffers, some pollutant removal studies have shown that they range widely in effectiveness.

Cost Considerations

One way to relieve some of the significant financial hardships for developers is to provide flexibility through buffer averaging. Buffer averaging allows developers to narrow the buffer width at some points if the average width of the buffer and the overall buffer area meet the minimum criteria. Variances can also be granted if the developer or landowner can demonstrate severe economic hardship or unique circumstances that make compliance with the buffer ordinance difficult.

8.4 Nonstructural Best Management Practices

Previous sections of this chapter presented the details of structural best management practices and their use within the municipal drainage system. The other major category of BMPs include the many nonstructural or source control practices that can be used for pollution prevention and control of pollutants. In most cases it is much easier and less costly to prevent the pollutants from entering the drainage system than trying to control pollutants with structural BMPs. Thus within the "treatment train" concept, the nonstructural BMPs should be the first line of defense in protecting the receiving stream within the municipality. If used properly, the nonstructural BMPs can be very effective in controlling pollutants and greatly reduce the need for structural BMPs. In addition, nonstructural BMPs tend to be less costly, easier to design and implement and easier to maintain than structural BMPs. The following is a brief discussion of some nonstructural BMPs that can be used in the Waverly area.

8.4.1 Public Education/Participation

Public education/participation is not so much a best management practice as it is a method by which to implement BMPs. Public education/participation are vital components of many of the individual source control BMPs. A public education and participation plan provides the municipality with a strategy for educating its employees, the public, and businesses about the importance of protecting stormwater from improper use, storage, and disposal of pollutants. It is important that residents become aware that a variety of hazardous products are used in the home and that their improper use and disposal can pollute stormwater and groundwater supplies. Businesses, particularly smaller ones that may not be regulated by Federal, State, or local regulations, must be informed of ways to reduce their potential to pollute stormwater.

The public education and participation plan should be based on four objectives:
- promote a clear identification and understanding of the problem and the solutions,
- identify responsible parties and efforts to date,
- promote community ownership of the problems and the solutions, and
- integrate public feedback into program implementation.

Target audiences include:
- Political - elected officials, chambers of commerce, and heads of departments, agencies, and commissions;
- Technical - municipal department and agency staffs, State agencies;
- Business - commercial and industrial, including trade associations;
- Community Groups - fraternal, ethnic, hobby, horticulture, senior citizen, and service;
- Environmental;
- General Public/Residential;
- Schools/Youth Groups;
- Media - print and electronic, and
- Pollutant-defined - groups of individuals defined by the specific pollutant(s) they discharge (e.g., used motor oil, pesticides)

For these target audiences the activities within the public education/participation plan can include surveys, presentations, school activities, development of working committees, development of literature and media campaigns, workshops, etc. All of these activities can be an important part of controlling local stormwater management problems.

8.4.2 Land Use Planning/Management

This BMP presents an important opportunity to reduce the pollutants in stormwater runoff by using a comprehensive planning process to control or prevent certain land use activities in areas where water quality is sensitive to development. It is applicable to all types of land use and represents one of the most effective pollution prevention practices. Subdivision regulations, zoning ordinances, preliminary plan reviews and detailed plan reviews, are tools that may be used to mitigate stormwater contamination in newly developing areas. Also, master planning, cluster development, terracing and buffers are ways to use land use planning as a BMP in the normal design for subdivisions and other urban developments. These are planning tools that municipal agencies can use to require conditions of approval or establish improvement/construction standards to meet the water quality objectives within specific watersheds.

An impervious cover limitation is one of the more effective land use management tools, since nationwide research has consistently documented increases in pollution loads with increases in impervious cover.

In addition, directly connected impervious areas should be kept to a minimum. This is especially important for large impervious areas such as parking lots and highways and it can also be effective for small impervious areas such as roof drainage. Minimization of impervious cover within a development is encouraged.

8.4.3 Material Use Controls

There are three major BMPs included in this category:
1. Housekeeping Practices
2. Safer Alternative Products
3. Pesticide/Fertilizer Use

In housekeeping practices, the goal is to promote efficient and safe practices such as storage, use, cleanup, and disposal, when handling potentially harmful materials such as fertilizers, pesticides, cleaning solutions, paint products, automotive products, and swimming pool chemicals. Alternatives exist for most product classes including fertilizers, pesticides, cleaning solutions, and automotive and paint products, and thus the use of less harmful products should be encouraged.

Pesticides and fertilizers have become an important component of land use and maintenance for municipalities, commercial land uses and residential land owners. Any usage of pesticides and fertilizers increases the potential for stormwater pollution. BMPs for pesticides and fertilizers include education in their use, control runoff from affected areas, control times when they are used, provide proper disposal areas, etc.

For the general public, public education provides information on such items as stormwater pollution and the beneficial effects of proper disposal on water quality; reading product labels; safer alternative products; safe storage, handling, and disposal of hazardous products; list of local agencies; and emergency phone numbers. This information can be provided through brochures or booklets that can be made available at a variety of places including municipal offices, public information fairs, and places where such products are sold. Education should also be developed for municipal employees and commercial and industrial establishments.

8.4.4 Material Exposure Controls

Material storage control is used to prevent or reduce the discharge of pollutants to stormwater from material delivery and storage by minimizing the storage of hazardous materials onsite, storing materials in a designated area, installing secondary containment, conducting regular inspections, and training employees and subcontractors.

8.4.5 Material Disposal And Recycling

There are three major BMPs included in this category:

1. Storm Drain System Signs
2. Household Hazardous Waste Collection
3. Used Oil Collection

Stenciling of the storm drain system (inlets, catch basins, channels, and creeks) with prohibitive language/graphic icons discourages the illegal dumping of unwanted materials.

Household hazardous wastes are defined as waste materials which are typically found in homes or similar sources, which exhibit characteristics such as: corrosivity, ignitability, reactivity, and/or toxicity, or are listed as hazardous materials by the EPA.

Used oil recycling is a responsible alternative to improper disposal practices such as dumping oil in the sanitary sewer or storm drain system, applying oil to roads for dust control, placing used oil and filters in the trash for disposal to landfill, or simply pouring used oil on the ground.

Storm drain system signs act as highly visible source controls that are typically stenciled directly adjacent to storm drain inlets. The signs contain brief statements that discourage the dumping of improper materials into the storm drain system. Graphical icons, either illustrating anti-dumping symbols or images of receiving water fauna, are effective supplements to the anti-dumping message. The intent of such a storm drain system stenciling program is to enhance public awareness of the pollutant effect on local receiving waters from stormwater runoff and also to discourage individual's habitual waste disposal actions (e.g., automotive fluids and landscaping wastes). An important aspect of a stenciling program is the distribution of informational flyers that educate the neighborhood (business or residential) about stormwater pollution, the storm drain system, and the watershed, and that provides information on alternatives such as recycling, household hazardous waste disposal, and safer products.

While it is generally recognized that the potential exists for hazardous household materials to come in contact with stormwater runoff, it is unclear at present how significant this source of contamination is. As such, it is difficult to quantify the benefits to water quality from household hazardous waste collection programs. However, such programs are a preventative, rather than curative measure, and may reduce the need for more elaborate treatment controls. Programs can be a combination of permanent collection centers, mobile collection centers, curbside collection, recycling, reuse, and source reduction. Public education is extremely important in implementing this BMP.

8.4.6 Spill Prevention And Cleanup

There are two major BMPs included in this category:

1. Vehicle Spill Control
2. Aboveground Tank Spill Control

The purpose of a vehicle spill control program is to prevent or reduce the discharge of pollutants to stormwater from vehicle leaks and spills by reducing the chance for spills by preventive maintenance, stopping the source of spills, containing and cleaning up spills, properly disposing of spill materials, and training employees. It is also very important to respond to spills quickly and effectively.

Aboveground tank spill control programs prevent or reduce the discharge of pollutants to stormwater by installing safeguards against accidental releases, installing secondary containment, conducting regular inspections, and training employees in standard operating procedures and spill cleanup techniques.

Accidental releases of materials from aboveground liquid storage tanks present the potential for contaminating stormwater with many different pollutants. Materials spilled, leaked, or lost from tanks may accumulate in soils or on impervious surfaces and be carried away by stormwater runoff.

Proper handling and storage of materials is very important and should include proper labeling; development of storage and handling procedures, secondary containment procedures, spill response procedures; and adequate training and education for those involved with this BMP.

8.4.7 Dumping Controls

This BMP addresses the implementation of measures to detect, correct, and enforce against illegal dumping of pollutants on streets and into the storm drain system, streams, and creeks. Substances illegally dumped on streets and into the storm drain system and creeks include paints, used oil and other automotive fluids, construction debris, chemicals, fresh concrete, leaves, grass clippings, and pet wastes. All of these wastes can cause stormwater and receiving water quality problems as well as clog the storm sewer system itself. Increased coordination with Lancaster County Health Department efforts would be useful.

8.4.8 Connection Controls

There are three major BMPs included in this category:

1. Illicit Connection Prevention
2. Illicit Connection Detection and Removal
3. Leaking Sanitary Sewer Control

Illicit connection protection tries to prevent unwarranted physical connections to the storm drain system from sanitary sewers, floor drains, etc., through regulation, regular inspection, testing, and education. In addition, programs include implementation control procedures for detection and removal of illegal connections from the storm drain conveyance system. Procedures include field screening, follow-up testing, and complaint investigation.

Leaking sanitary sewer control includes implementing control procedures for identifying, repairing, and remediating infiltration, inflow, and wet weather overflows from sanitary sewers into the storm drain conveyance system. Procedures include field screening, testing, and complaint investigation.

Illegal connections can occur in new as well as existing developments. Improper connections in areas of new development can be prevented through inspection and other verification techniques. The first measure to prevention is to make sure that existing municipal building and plumbing codes prohibit any unwarranted, non-permitted physical connections to the storm drain system. Building and plumbing code inspectors, in addition to new land development project inspectors, must visually inspect to ensure that illegal connections are not being physically tied to the storm conveyance system. Proper documentation and record keeping is essential to the function of such inspections. Documentation helps catalog the storm drain system and is required by Federal regulations. Visual inspection, however, is not a very reliable means of verifying the status of new physical connections and their final destination. Continued monitoring throughout the entire development phase would be necessary to guarantee the new physical connections between the sanitary sewers and storm drains had been prevented through the inspection process.

Public education programs will also aid in the monitoring of illegal connections and leaking sanitary sewers by making individuals aware of evidence of unwarranted discharges to the storm drain system. A community hotline for reporting such evidence can greatly supplement the stormwater department's field screening efforts.

8.4.9 Street/Storm Drain Maintenance

There are seven major BMPs included in this category:

1. Roadway Cleaning
2. Catch Basin Cleaning
3. Vegetation Controls
4. Storm Drain Flushing
5. Roadway/Bridge Maintenance
6. Detention/Infiltration Device Maintenance
7. Drainage Channel/Creek Maintenance

Roadway cleaning may help reduce the discharge of pollutants to stormwater from street surfaces by conducting cleaning on a regular basis. However, cleaning often removes the larger sizes of pollutants but not the smaller sizes.

Most pollutants accumulate within three feet of the curb which is where the roadway cleaning should be concentrated. Catch basin cleaning on a regular basis also helps reduce pollutants in the storm drain system, reduces high pollutant concentrations during the first flush of storms, prevents clogging of the downstream conveyance system and restores the catch basins' sediment trapping capacity.

Vegetation control typically involves a combination of chemical (herbicide) application and mechanical methods. Mechanical vegetation control includes leaving existing vegetation, cutting less frequently, handcutting, planting low maintenance vegetation, mulching, collecting and properly disposing of clippings and cuttings, and educating employees.

Storm drains can be "flushed" with water to suspend and remove deposited materials. Flushing is particularly beneficial for storm drain pipes with grades too flat to be self-cleansing. Flushing helps ensure pipes convey design flow and removes pollutants from the storm drain system. However, flushing will only push the pollutants into downstream receiving waters unless the discharge from the flushing is captured and removed from the drainage system.

Proper maintenance and siltation removal is required on both a routine and corrective basis to promote effective stormwater pollutant removal efficiency for wet and dry detention ponds and infiltration devices. Also, regularly removing illegally dumped items and material from storm drainage channels and creeks will reduce pollutant levels.

8.4.10 Permanent Erosion Control

There are three major BMPs included in this category:

1. Erosion Control - Permanent Vegetation
2. Erosion Control - Flow Control
3. Erosion Control - Channel Stabilization

Vegetation is a highly effective method for providing long term, cost effective erosion protection for a wide variety of conditions. It is primarily used to protect the soil surface from the impact of rain and the energy of the wind. Vegetation is also effective in reducing the velocity and sediment load in runoff sheet flow.

Channel stabilization addresses the problem of erosion due to concentrated flows. Concentrated flows occur in channels, swales, creeks, rivers and other water courses in which a substantial drainage area drains into a central point. Overland sheet flow begins to collect and concentrate in the form of rills and gullies after overland flow length of as little as 100 feet. Erosion due to concentrated flow is typically extensive, causing large soil loss, undermining foundations and decreasing the flow capacity of watercourses.

Proper selection of ground cover is dependent on the type of soil, the time of year of planting, and the anticipated conditions that the ground cover will be subjected. In addition, mulching is a form of erosion protection which is commonly used in conjunction with establishment of vegetation. It typically improves infiltration of water, reduces, retards erosion and helps establish plants in disturbed areas.

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CHAPTER 9

EROSION AND SEDIMENT CONTROL

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Chapter Nine - Erosion And Sediment Control

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9.1 Purpose And Scope

This chapter provides criteria for measures that should be taken for construction site stormwater discharges to meet the requirements of the Federal Clean Water Act, and the Nebraska Environmental Protection Act. Through implementation of the guidelines in this chapter, including development of a Stormwater Pollution Prevention Plan (SWPPP), adverse water quality impacts associated with erosion and sedimentation can be prevented or minimized. Section 9.2 provides an overview of the Fundamentals of the Erosion Process. Requirements for Construction Activity SWPPPs are in Section 9.3. The remainder of the this chapter embodies a range of guidelines, criteria and alternatives for meeting the preparation and implementation requirements of the SWPPP. Section 9.4 covers Best Management Practice (BMP) selection, and Section 9.5 addresses BMP design.

The guidelines in this section were developed by various local, state and federal agencies. The Lower Platte South Natural Resources District (NRD)'s (1994) *Erosion and Sediment Control and Stormwater Management Manual* and the EPA's (2007) *Developing Your Stormwater Pollution Prevention Plan: A Guide for Construction Sites* were key sources of information for these guidelines and can be referenced for more detail on topics such as specifications for particular erosion and sediment control measures and other SWPPP guidelines.

9.2 Fundamentals of the Erosion Process

Soil erosion is the process by which the land's surface is worn away by the action of wind, water, ice and gravity. Natural, or geologic, and is a tremendous factor in creating the earth as we know it today. Except for some cases of shoreline and stream erosion, natural erosion occurs at a very slow and uniform rate and remains a vital factor in maintaining environmental balance. Human activities accelerate the erosion process by loosening and pulverizing soil, making it more susceptible to detachment by natural forces. Accelerated soil erosion is what is most commonly dealt with in the built environment. Accelerated soil erosion is the removal of the surface of the land through the combined action of human activities and the natural processes at a rate greater than would occur because of the natural processes alone

9.2.1 Erosion Types

Water-generated erosion is unquestionably the most severe type of erosion, particularly in developing areas; it is, therefore, the problem to which this chapter is primarily addressed. Soil erosion by water involves the detachment of particles from the soil mass, transportation by surface runoff, and eventual deposition. Soil particles are detached by the impact of rainfall and the shear force of runoff. Transportation of soil particles is primarily by channelized runoff, although raindrop splash causes some net downslope movement and increases the erosive capability of unchannelized overland flow. Runoff occurs when the rainfall intensity is greater than the soil infiltration rate. Once runoff begins, the quantity and size of material transported is a function of runoff velocity and turbulence. Water-generated erosion can be broken down into the following types:

1. <u>Raindrop erosion</u> is the first effect of a rainstorm on the soil. Raindrop impact dislodges soil particles and splashes them into the air. These detached particles are then vulnerable to the next type of erosion.

2. <u>Sheet erosion</u> is the erosion caused by the shallow flow of water as it runs off the land. These very shallow moving sheets of water are seldom the detaching agent, but the flow transports soil particles which have been detached by raindrop impact and splash. The shallow surface flow rarely moves as a uniform sheet for more than a few feet on land surfaces before concentrating in surface irregularities.

3. <u>Rill erosion</u> is the erosion which develops as the shallow surface flow begins to concentrate in the low spots of the irregular contours of the surface. As the flow changes from the shallow sheet flow to deeper flow in these low areas, the velocity and turbulence of flow increase. The energy of this concentrated flow is able to both detach and transport soil materials. This action begins to cut small channels of its own. Rills are small but well-defined channels which are at most only a few inches in depth. They are easily obliterated by harrowing or other surface treatments.

4. Gully erosion occurs as the flow in rills comes together in larger and larger channels. The major difference between gully and rill erosion is a matter of magnitude. Gullies are too large to be repaired with conventional tillage equipment and usually require heavy equipment and special techniques for stabilization.

5. Channel erosion occurs as the volume and velocity of flow causes movement of the stream bed and bank materials. Urban development, typified by removing existing vegetation, increasing the amount of impervious areas and paving tributaries, drastically changes the volume and velocity of flow within a stream, destroying the equilibrium of the stream and causing channel erosion to begin. Common points where erosion occurs are at stream bends and at constrictions, such as those where bridges cross a stream. Erosion may also begin at the point where a storm drain or culvert discharges into a stream. Repair of eroded streambanks is difficult and costly.

Figure 9-1 illustrates the five stages of erosion.

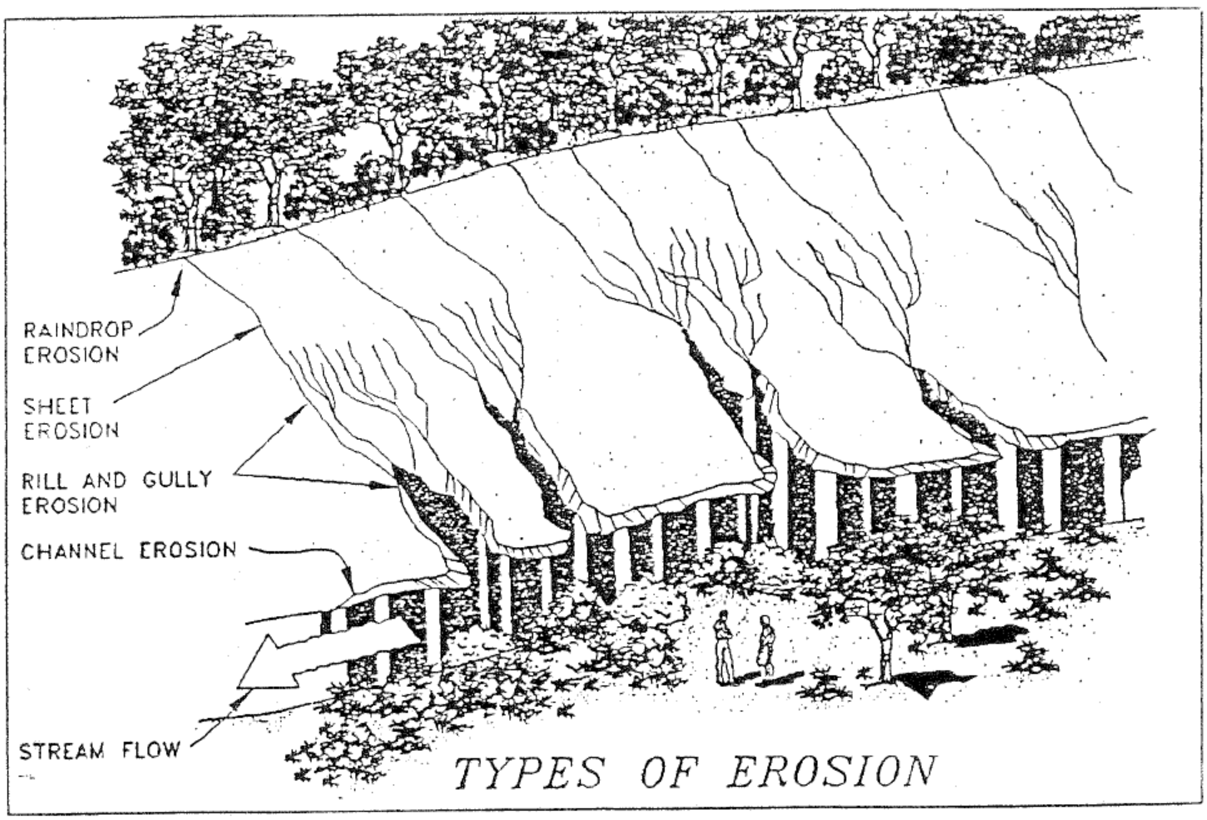


Figure 9-1 Types of Erosion

Source: LPSNRD 1994.

9.2.2 Factors Influencing Soil Erosion

The erosion potential of any area is determined by four interrelated principal factors: the characteristics of its soil, its vegetative cover, its topography and its climate. Each of these factors is discussed separately below.

Soil characteristics which influence the potential for erosion by rainfall and runoff are those properties which affect the infiltration capacity of a soil and the resistance of the soil to detachment and being carried away by falling or flowing water. The following four characteristics are important in determining soil erodibility:

- Soil texture (particle size and gradation): Soil texture refers to the sizes and proportions of the particles making up a particular soil. Sand, silt, and clay are the three major classes of soil particles. Soils high in sand content are said to be coarse-textured. Because water readily infiltrates into sandy soils, the runoff, and consequently the erosion potential, is relatively low. Soils with a high content of silts and clays are said to be fine-textured or heavy. Clay, because of its stickiness, binds soil particles together and makes a soil resistant to erosion. However, once the fine particles are eroded by heavy rain or fast flowing water, they will travel great distances before settling. Even with the sediment control measures described in this manual, it is extremely difficult to remove clay particles from flowing water. Typically, particles of clay and fine silt will settle in a large, calm water body, such as a bay, lake, or reservoir, at the bottom of a watershed. Thus, silty and clayey soils are frequently the worst water polluters. Soils that are high in silt and fine sand and low in clay and organic matter are generally the most erodible. Well-drained sandy and rocky soils are the least erodible.

- Percentage of organic matter: Organic matter consists of plant and animal litter in various stages of decomposition. Organic matter improves soil structure and increases permeability, water-holding capacity, and soil fertility. Organic matter in an undisturbed soil or in a mulch covering a disturbed site reduces runoff and, consequently, erosion potential. Mulch on the surface also reduces the erosive impact of raindrops.

- Soil structure: Soil structure is the arrangement of soil particles into aggregates. A granular structure is the most desirable one. Soil structure affects the soil's ability to absorb water. When the soil surface is compacted or crusted, water tends to run off rather than infiltrate. Erosion hazard increases with increased runoff. Loose, granular soils absorb and retain water, which reduces runoff and encourages plant growth.

- Soil permeability: Soil permeability refers to the ability of the soil to allow air and water to move through the soil. Soil texture and structure and organic matter all contribute to permeability. Soils with high permeability produce less runoff at a lower rate than soils with low permeability, which minimizes erosion potential. The higher water content of a permeable soil is favorable for plant growth, although it may reduce slope stability in some situations.

Vegetative cover plays an extremely important role in controlling erosion, providing the following benefits:

1. Shielding the soil surface from raindrop impact
2. Providing root systems that hold soil particles in place
3. Maintaining the soil's capacity to absorb water
4. Slowing the velocity of runoff
5. Removing subsurface water between rainfalls through the process of evapotranspiration

By limiting and staging the removal of existing vegetation and by decreasing the area and duration of exposure, soil erosion and sedimentation can be significantly reduced during construction. Special consideration should be given to the maintenance of existing vegetative cover on areas of high erosion potential such as moderately to highly erodible soils, steep slopes, drainageways, and the banks of streams.

Topography, including the size, shape, and slope characteristics of a watershed, influences the amount and rate of runoff. As both slope length and gradient increase, the rate of runoff increases and the potential for erosion is magnified. The shape of a slope also has a major bearing on erosion potential. The base of a slope is more susceptible to erosion than the top because runoff has more momentum and is more concentrated as it approaches the base. Slope orientation can also be a factor in determining erosion potential. For example, a slope that faces south and contains droughty soils may have such poor growing conditions that vegetative cover can be difficult to re-establish. Conversely, northern exposures tend to be cooler and more moist, but they also receive less sun, which results in slower plant growth.

Climate characteristics, such as precipitation patterns and temperature, influence runoff and susceptibility of soils to erosion. The frequency, intensity, and duration of rainfall are fundamental factors in determining the amounts of runoff produced in a given area. As both the volume and velocity of runoff increase, the capacity of runoff to detach and transport soil particles also increases. Where storms are frequent, intense, or of long duration, erosion risks are high. Seasonal changes in temperature, as well as variations in rainfall, help to define the high erosion risk period of the year. When precipitation falls as snow, no erosion will take place. However, when the temperature rises, melting snow adds to runoff, increasing erosion hazards. When the ground is still partially frozen, its absorptive capacity is reduced. Frozen soils are relatively erosion resistant; however, soils with high moisture content are subject to uplift action and are usually very easily eroded upon thawing (LPSNRD, 1994).

9.3 SWPPP Requirements for Construction Activity

Construction activity is the disturbance of one (1) acre or more of land area and less than one acre if part of a common plan of development or sale, more particularly defined in Section 28.01.030 of the Regulations for Construction Site Discharges. Prior to Construction Activity, a permit application must be submitted in the form of a Notice of Intent (NOI) to the Nebraska Department of Environmental Quality (NDEQ). For sites greater than five (5) acres, the forms are also required by the state to be submitted to the Nebraska Department of Environmental Quality for approval. The NOI must include a Construction Activity Stormwater Pollution Prevention Plan (SWPPP) with the information identified in the Drainage Criteria Manual. The SWPPP must identify the appropriate Best Management Practices (BMPs) to be implemented to control erosion, sedimentation, and pollutants, such as those described in Sections 9.4, 9.5, 9.3.6, 9.3.7 and 9.3.8.

The Construction Activity SWPPP must be prepared and signed by a qualified individual such as a Professional Engineer, Landscape Architect, and/or Certified Professional in Erosion and Sediment Control (CPESC). If review comments are not received by the permittee within 7 calendar days after receipt of application by the NDEQ, the application will be deemed authorized. Prior to actual initiation of the construction activity, the applicant must submit to the NDEQ a Notice of Start of construction. Once the construction is complete in accordance with the design standards, the applicant must submit to the NDEQ a Notice of Termination.

In preparing the Construction Activity SWPPP, individuals should review this section and those that follow. Specifically, those preparing plans should be familiar SWPPP requirements (Section 9.3), as well as the selection and design of BMPs (Sections 9.4-9.5) and the fundamentals of the erosion process (Section 9.2).

9.3.1 Summary of Required SWPPP Items for Construction Activity

The following is a summary of required SWPPP items for Construction Activity to be prepared in accordance with sections 9.3-9.6 of this chapter.

Narrative

1. Project Description - Briefly describes the nature and purpose of the construction activity, and the area (acres) to be disturbed.

2. Existing site conditions - A description of the existing topography, vegetation and drainage.

3. Adjacent areas - A description of neighboring areas such as streams, lakes, residential areas, roads, etc., which might be affected by the construction activity.

4. Off-site areas - Describe any off-site construction activities that will occur (including borrow sites, waste or surplus areas, etc.). Will any other areas be disturbed?

5. Soils - A brief description of the soils on the site giving such information as soil name, erodibility, permeability, depth, texture and soil structure.

6. Critical areas - A description of areas on the site which have potentially serious erosion problems (steep slopes, channels, etc.).

7. Erosion and sediment control measures - A description of the methods which will be used to control erosion and sedimentation on the site. (Controls must meet the minimum specified requirements as found in Section 9.5 of this manual).

8. Permanent Stabilization - A brief description, including specifications, of how the site will be stabilized after construction is completed.

9. Stormwater runoff and management - Will the developed site cause an increase in peak runoff rates? Will the increase in runoff cause flooding or channel degradation downstream? Describe the strategy to control stormwater runoff.

10. Spill prevention & response plan - When developing a spill prevention plan, include, at a minimum, the following:
    - Note the locations of chemical storage areas, storm drains, tributary drainage areas, surface water bodies on or near the site, and measures to stop spills from leaving the site.
    - Specify how to notify the appropriate authorities to request assistance.
    - Describe the procedures for immediate cleanup for spills and proper disposal.
    - Identify personnel responsible for implementing the plan in the event of a spill.

Site Plan

1. Vicinity map - A small map locating the site in relation to the surrounding area. Include any landmarks which might assist in locating the site.

2. Indicate north - The direction of north in relation to the site.

3. Limits of clearing and grading - Areas which are to be cleared and graded.

4. Existing contours - The existing contours of the site.

5. Final contours - Changes to the existing contours, including final drainage patterns.

6. Existing vegetation - The existing tree lines, grassed areas, or unique vegetation.

7. Soils - The boundaries of different soil types.

8. Existing drainage patterns - The dividing lines and the direction of flow for the different drainage areas. Include the size (acreage) of each drainage area.

9. Critical erosion areas - Areas with potentially serious erosion problems.

10. Site development - Show all improvements such as buildings, parking lots, access roads, utility roads, etc.

11. Location of practices - The locations of erosion and sediment controls and stormwater management practices used on the site. Use the standard symbols and abbreviations as noted in Sections 9.5.

12. Off-site areas - Identify any off-site construction activities (borrow sites, waste sites, etc.). Show location of erosion controls. (Is there sufficient information to assure adequate protection and stabilization?)

Details

1. Detailed drawings - Enlarged, dimensioned drawings of such key features as sediment basin risers, energy dissipators, and waterway cross-sections.

2. Detailed specifications - Specifications for specific items such as seeding mix and planting schedule, filter fabric size, rock gradations, etc.

3. Construction sequencing - Specifications for the sequence of construction operations describing the relationship between the implementation and maintenance of sediment controls, including permanent and temporary stabilization and the various stages or phases of earth disturbance and construction.

4. Maintenance program - A description of inspection schedules, spare materials needed, stockpile locations, instructions for sediment removal and disposal, and for repair of damaged structures should be provided. A clear

statement defining maintenance responsibility should also be included.

Calculations

1. Calculations and assumptions - Provide data for design storm used to size pipes, channels, sediment basins and traps. Include calculations for pre- and post-development runoff as well as any other calculations necessary to support drainage, erosion and sediment, and stormwater management systems.

9.3.2 General Information for Construction Activity SWPPPs

A SWPPP is more than just a sediment & erosion control plan. It is a comprehensive, written document that describes the pollution prevention practices and activities that will be used during each phase of construction. It includes descriptions of the site and of each major phase of the planned activity, the roles and responsibilities of contractors, and the inspection schedules and logs. It is also a place to document changes and modifications to the construction plans and associate stormwater pollution prevention activities.

1. The SWPPP must be implemented either prior to or concurrent with the initiation of construction activity. SWPPP activities must be maintained throughout the period construction activities are ongoing until final site stabilization is achieved. A current and updated copy of the SWPPP must be available on-site at all times that work is being performed. Persons and/or subcontractors responsible for carrying out duties pursuant to the SWPPP must be properly trained and informed of their responsibilities.

2. The SWPPP shall be dynamic. If deficiencies in the plan arise during the course of the project, or differing site conditions warrant, the applicant must implement effective corrective actions that may require modification of the SWPPP.

3. NDEQ may require modification of the SWPPP:
   - If it is not effective in minimizing erosion or the release of storm water pollutants from the site;
   - If more effective procedures are available and practical;
   - If previous experience has shown the control methods specified have proven to be inadequate in similar circumstances; or
   - To meet basin specific Nebraska Department of Environmental Quality water quality requirements or goals.
   - To correspond to changes in the development plan for the site.
   - In the event of repetitive failure to adequately maintain practices.

9.3.3 General Requirements for Construction Activity SWPPPs

The SWPPP lays out the steps and techniques required to reduce pollutants in stormwater runoff leaving the construction site. Therefore, proper development and implementation of the SWPPP is a crucial aspect of permit compliance. First and foremost, the SWPPP must be developed and implemented consistent with the requirements of the NPDES Construction General Permit, and City standards and ordinances regarding erosion and sediment control.

1. The SWPPP is to identify all pollution sources that could come into contact with stormwater leaving the site. It describes the BMP's utilized to reduce pollutants in the construction site's stormwater discharges, and it includes written records of the site inspections and the follow-up maintenance that is performed.

2. At a minimum the following must be provided in the SWPPP:
   - Pollutant sources
   - Description of contents Cover/title page
   - Project/SWPPP contact information
   - Site and activity description, including a site map
   - Pre and post-development runoff coefficients
   - Identification of potential controls to reduce pollutants
   - Spill prevention & response plan
   - Maintenance/inspection procedures
   - Records of inspections and follow-up maintenance of BMP's

- SWPPP amendments
- SWPPP Certification

3. The Construction Activity SWPPP must be prepared and signed by a qualified individual such as a Professional Engineer, Landscape Architect, and/or Certified Professional in Erosion and Sediment Control (CPESC). If review comments are not received by the permittee within seven (7) calendar days after receipt of application by NDEQ, the application will be deemed authorized. If the SWPPP has been denied, it may be revised and resubmitted for approval. Prior to actual initiation of the construction activity, the applicant must submit to NDEQ a Notice of Start of construction. Once the construction is complete in accordance with the design standards, the applicant must submit to NDEQ a Notice of Termination.

4. The SWPPP must include placement of the following statement. "The undersigned certifies this plan has been designed in accordance with the terms of the interlocal agreement for NPDES compliance."

9.3.4 Common SWPPP Objectives

For a SWPPP to be effective, it must be developed in the project planning stage and effectively applied during construction. In most cases, the most practical method of controlling erosion and the associated production and transport of sediment includes a combination of limited time of soil exposure and judicious selection of erosion control practices and sediment trapping facilities. The SWPPP should be prepared to meet the following objectives:

1. Minimize the extent and the duration of soil exposure. The duration of soil exposure can be minimized through construction phasing, prompt revegetation and mulching. Grading should be completed as soon as possible and followed by permanent revegetation. As cut slopes are made and as fill slopes are brought up to grade, these areas should be revegetated. Minimizing grading of large or critical areas during the seasons of maximum erosion potential (April through September) reduces the risk of erosion.

2. Apply erosion control practices to prevent excessive sediment production. Keep soil covered to the extent practicable with temporary or permanent vegetation or mulch. Special grading methods such as roughening a slope on the contour or tracking with a cleated dozer may be used. Other practices include diversion structures to divert surface runoff from exposed soils and grade stabilization structures to control surface water. "Gross" erosion in the form of gullies must be prevented by these water control devices.

3. Apply perimeter sediment control practices to protect the disturbed area from off-site runoff and to prevent sedimentation damage to areas below the construction site. This principle relates to using practices that effectively isolate the construction site from surrounding properties, and especially to controlling sediment once it is produced and preventing its transport from the site. Generally, sediment can be retained by two methods: (a) filtering runoff as it flows through an area and (b) impounding the sediment-laden runoff for a period of time so that the soil particles settle out. Diversions, dikes, sediment traps, vegetative and structural sediment control measures can be used to control sediment. These measures may be temporary or permanent, depending on whether they will remain in use after construction is complete. The best way to control sediment, however, is to prevent erosion.

4. Keep runoff velocities low and retain runoff on the site. The removal of existing vegetative cover and the resulting increase in impermeable surface area during construction will increase both the volume and velocity of runoff. These increases must be taken into account when providing for erosion control. Keeping slope lengths short and gradients low, and preserving natural vegetative cover can keep stormwater velocities low and limit erosion hazards. Runoff from the development should be safely conveyed to a stable outlet using storm drains, diversions, stable waterways or similar measures. Conveyance systems should be designed to withstand the velocities of projected peak discharges. These facilities should be operational as soon as possible.

5. Stabilize disturbed areas as soon as practicable, but in no case more than 14 days after final grade has been attained. Permanent structures, temporary or permanent vegetation, mulch, stabilizing emulsions, or a combination of these measures should be employed as quickly as possible after the land is disturbed. Temporary vegetation and mulches and other control materials can be most effective when it is not practical to establish permanent vegetation or until permanent vegetation is established. Such temporary measures should be employed

as soon as practicable, but in no case more than 14 days after rough grading is completed if a delay is anticipated in obtaining finished grade. The finished slope of a cut or fill should designed to be stable and easily maintained. Stabilize roadways, parking areas and paved areas with a gravel sub-base whenever possible.

6. Implement a thorough maintenance and follow-up program. This last principle is vital to the success of the six other principles. A site cannot be effectively controlled without thorough, periodic checks of the erosion and sediment control practices. These practices must be maintained just as construction equipment must be maintained and material checked and inventoried. An example of applying this principle would be to start a routine "end of day check" to make sure that all control practices are working properly.

9.3.5 SWPPP Development - Site Assessment and Planning

The following section describes five critical steps in the SWPPP development process that will help provide a good foundation for the SWPPP.

1. Assess the site and proposed project. The SWPPP should describe the undeveloped site and identify features of the land that can be incorporated into the final plan and natural resources that should be protected.

    a. Visit the site: The people responsible for site design drafting the SWPPP should conduct a thorough walk-through of the entire construction site to assess site-specific conditions such as soil types, drainage patterns, existing vegetation, and topography. Avoid copying SWPPPs from other projects to save time and money. Each construction site is unique, and visiting the site is the only way to create a SWPPP that addresses the unique conditions at that site.

    b. Assess Existing Construction Site Conditions: Assess the existing conditions at the construction site, including topography, drainage and soil type. This assessment is the foundation for building the SWPPP and for developing the final site plan. In this assessment, use or create a topographic drawing that:
        - Indicates how stormwater currently drains from the site, and identify the location of discharge points or areas
        - Identifies slopes and slope lengths. The topographic features of the site are a major factor affecting erosion from the site
        - Identifies soil type(s) and any highly erodible soils and the soil’s infiltration capacity
        - Identifies any past soil contamination at the site
        - Identifies natural features, including trees, streams, wetlands, slopes and other features to be protected

        In most cases, the site designer can compile all this information on a digitized drawing that can then be adapted to show the planned construction activity, the phases of construction, and the final site plan.

    c. Identify Receiving Waters, Storm Drains, and Other Stormwater Conveyance Systems: The SWPPP should clearly identify the receiving waters and stormwater systems through which stormwater from the site could flow. If the site’s stormwater flows into a municipal drain system, the plan designer will need to determine the ultimate destination of that system’s discharge. If the site’s stormwater runs off to areas not connected to the storm drain system, the designer should consider the land’s topography and then identify the waterbodies that it could reach.

    d. Describe The Construction Project: The SWPPP should contain a brief description of the construction activity, including:
        - Project type or function (i.e. low-density residential, industrial center, street widening)
        - Project location, including latitude and longitude, and section-township-range
        - Estimated project start and end dates
        - Sequence and timing of activities that will disturb soils at the site
        - Size of the project
        - Estimated total area expected to be disturbed by excavation, grading, or other construction activities, including dedicated off-site borrow and fill areas
        - Runoff Coefficient before and after construction
        - Soil types

- Describe and identify the location of other potential sources of stormwater contamination, such as asphalt and concrete plants, paint and concrete washout areas, etc.

e. Identify Pollutants and Pollution Sources: Identify the pollutants and sources that are likely to be found on the site. Sediment is the main pollution of concern, but other pollutants may be found, usually in substantially smaller amounts, in stormwater runoff from construction sites. These can include nutrients, heavy metals, organic compounds, pesticides, oil and grease, bacteria and viruses, trash and debris, and other chemicals. After identifying the pollutants and sources, be as specific as possible in the SWPPP about the BMPs that will use to address them.

2. Identify Approaches to Protect Natural Resources. The SWPPP should describe methods to be utilized to protect and preserve any streams, wetlands, ponds, or other waterbodies that are on the property or immediately adjoining it. Riparian areas around headwater streams are especially important to the overall health of the entire river system. Contact the Nebraska Department of Environmental Quality to determine if any impaired waters designation has been placed on any adjacent streams, rivers, or waterbodies. A permittee might be subject to additional requirements to protect these waterbodies.

   Wetland areas, including bogs marshes and sloughs, maybe found in areas adjacent to rivers, streams and lakes, but may also be found in isolated places far from other surface waters. Many types of wetlands, especially saline wetlands, are protected under the Clean Water Act and construction activities in and around these areas may require an additional permit from the U.S. Army Corps of Engineers. Construction site operators should make every effort to preserve wetlands and must follow local, state, and federal requirements before disturbing them or the areas around them.

3. Assess Whether There Are Endangered Plant or Animal Species in the Area. The Federal Endangered Species Act protects endangered and threatened species and their critical habitat areas. In developing the assessment of the site, determine whether listed endangered species are on or near the property. Critical habitat areas are often designated to support the continued existence of listed species. The SWPPP designer will also need to determine whether critical habitat areas have been designated in the vicinity of the project. Contact local offices of the U.S. Fish and Wildlife Service (FWS), or the Nebraska Game and Parks Service. For more information and to locate lists for the State of Nebraska, visit www.epa.gov/npdes/endangeredspecies.

4. Assess Whether There Are Historic Sites that Require Protection. The National Historic Preservation Act applies to construction activities. As with endangered species, some permits may specifically require the SWPPP designer to assess the potential impact of the stormwater discharges on historic properties. However, whether or not this is listed as a condition for permit coverage, the National Historic Preservation Act and any applicable State laws apply to the project. Contact the State Historic Preservation Officer for the Nebraska State Historical Society at 402-471-3100 for more information.

5. Develop Site Maps. The final step in the site evaluation process is to document the results of the site assessment and the planned phases of construction activity on a detailed site map or maps. This includes developing site maps showing planned construction activities and stormwater practices for the various major stages of construction, protected areas, natural features, slopes, erodible soils, nearby water bodies, permanent stormwater controls, and so on. The permittee must keep the SWPPP and the site maps up-to-date to reflect changes at the site during the construction process.

   a. Location Maps: A general location map is required on the SWPPP, and is helpful to identify nearby, but not adjacent water bodies in proximity to other properties.

   b. Site Maps: The detailed construction site maps should show the entire site and identify a number of features at the site related to construction activities and stormwater management practices. Site maps should show the construction activities and stormwater management practices for each major phase of construction (i.e. initial grading, infrastructure, construction, and stabilization). The site maps should legibly identify the following features:

- Stormwater flow and discharges. Indicate flow direction(s) and approximate slopes after grading activities, as well as locations of discharges to surface waters or municipal storm drain systems.
- Areas and features to be protected. Include wetlands, nearby streams, coastal waters, mature trees and natural vegetation, steep slopes, highly erodible soils, etc.
- Disturbed areas. Indicate locations and timing of soil disturbing activities (i.e. grading). Mark clearing limits
- BMPs. Identify locations of structural and non-structural BMPs identified in the SWPPP, as well as post-construction stormwater BMPs. Erosion & sediment control BMPs are described in section 9.5 of this chapter.
- Areas of stabilization. Identify locations where stabilization practices are expected to occur. Mark areas where final stabilization has been accomplished.
- Other areas and roads. Indicate locations of material, waste, borrow, or equipment.

c. Develop and keep up-to-date site maps showing non-structural BMPs that change frequently in location as the work on a construction site progresses. The permit requires that the permittee keep the SWPPP up-to-date, so mark up the site map with the current location of these BMPs. Indicate the current location of the following:
   - Portable toilets
   - Material storage areas
   - Vehicle and equipment fueling and maintenance areas
   - Concrete washouts
   - Paint washouts
   - Dumpsters or other trash and debris containers
   - Spill kits
   - Stockpiles
   - Any other non-structural non-stormwater management BMPs
   - Any temporarily removed structural BMPs
   - Any changes to the structural BMPs

d. If a marked-up site map is too full to be easily read, the SWPPP designer should date and fold it, put it in the SWPPP for documentation, and start a new one. That way, there is a good hard copy record of what has occurred on-site.

9.3.6 Requirements for the Building Phase of Development

Any person who engages in construction activity is responsible for compliance with this chapter and all applicable terms and conditions of the Permit and SWPPP as it relates to the building phase of development. The following information shall be included on the application for building permit and be submitted to the Director of Building and Safety.

a. The legal decription and permit number for the Construction Activity SWPPP;

b. The location of the property where the building phase of development is to occur; and

c. A certification that the building phase of development for the property described on the application for building permit will be conducted in conformance with Chapter 28.01 and the Construction Activity SWPPP.

9.3.7 SWPPP Erosion And Sediment Control Requirements

1. The applicant must incorporate erosion and sediment control practices into the SWPPP and implement said practices at all locations undergoing construction activity. The erosion and sediment control practices utilized must consider site specific variables including slope, soil types, the size of the project, the duration of construction activities, the proximity of perennial and seasonal streams, and the existence of impounded waters downstream of the project. The controls utilized may vary from site-to-site, but the controls used must be effective in minimizing erosion and sediment release from the site, and in protecting the water quality in the receiving stream or water body.

2. The existence of downstream lakes or other impounded water increases water quality concerns relative to sediment release. In these instances, more stringent erosion and sediment controls may need to be implemented.

3. The applicant must upgrade the erosion and sediment control practices utilized in the SWPPP and implement additional controls, if existing controls prove inadequate in minimizing erosion and sediment releases, or in protecting the water quality of the receiving stream or water body. The applicant must comply with NDEQ requests to implement additional controls to minimize erosion and sediment releases, and to protect receiving water bodies.

4. Physical erosion and sediment control practices incorporated into the SWPPP must comply with the requirements of the Nebraska Department of Environmental Quality.

5. All SWPPPs submitted for approval must include the following statement, "Unless otherwise indicated, all vegetative and structural erosion and sediment control practices and stormwater management practices will be constructed and maintained according to the minimum standards and specifications of the City of Waverly Drainage Criteria Manual.

6. All of the following principles must be considered for inclusion in the SWPPP.

   a. Minimize disturbed area and protect natural features and soil. By carefully delineating and controlling the area that will be disturbed by grading or construction activities, the SWPPP designer can greatly reduce the potential for soil erosion and stormwater pollution problems. Limit disturbed areas to only those necessary for the construction of the project. Natural vegetation is the best and cheapest erosion control BMP. When possible, vegetative strips must be maintained on the down gradient perimeter of sites, and adjacent to waterways and drainage ways that are within the site. Temporary or permanent seeding must be established as soon as possible after grading and clearing activities are completed, and during interim periods on areas that are not being actively worked.

   b. Phase construction activity. By scheduling or sequencing the construction work and concentrating it in certain areas, the SWPPP designer can minimize the amount of soil that is exposed to the elements at any given time. Limiting the area of disturbance to places where construction activities are underway and stabilizing them as quickly as possible can be one of the most effective BMPs.

   c. Control stormwater flowing onto and through the project. Plan for any potential stormwater flows coming onto the project area from upstream locations, and divert (and slow) flows to prevent erosion. Likewise, the volume and velocity of onsite stormwater runoff should be controlled to minimize erosion. Stabilization measures must be applied to earthen structures such as dams, dikes and diversions immediately after installation.

      An example of a BMP for controlling run-on would be:
      - Diversion Dikes or berms

   d. Stabilize soils properly. Where construction activities have temporarily or permanently ceased, the area must be temporarily or permanently stabilized as soon as practicable, but in no case more than 14 days. All SWPPP plans submitted for approval must include placement of the following statement, "following soil disturbance, permanent or temporary stabilization must be completed as soon as practicable, but in no case more than 14 days to the surface of all perimeter sediment controls, topsoil stockpiles, and any other disturbed or graded areas on the project site which are not being used for material storage, or on which actual earth moving activities are not being performed." In subdivisions, this permanent or temporary stabilization must be maintained until development commences on street work, utility work on individual lots within the subdivision.

      Temporary measures are necessary when an area of a site is disturbed but where activities in that area are not completed or until permanent BMPs are established. Topsoil stockpiles should also be protected to minimize any erosion from these areas. Silt fence and other sediment control measures are NOT stabilization measures.

Temporary cover BMPs include:
- seeding
- mulches
- bonded fiber matrices (hydroseeding/mulching)
- blankets and mats
- the use of soil binders

Permanent-cover BMPs include:
- permanent seeding and planting
- sodding
- channel stabilization
- vegetative buffer strips

e. Protect slopes. Protect all slopes with appropriate erosion controls. Steeper slopes, slopes with highly erodible soils, or long slopes require a more complex combination of controls. Cut and fill slopes must be designed and constructed in a manner that will minimize erosion. Slopes that are found to be eroding excessively within one year of permanent stabilization must be provided with additional slope stabilization measures until the problem is corrected.

   Examples of BMPs for slope stabilization include:
   - erosion control blankets
   - turf reinforcement mats
   - bonded fiber matrices (hydroseeding/mulching)

   Silt fence, straw wattles, or compost socks may also be used to help control erosion on moderate to shallow slopes and should be installed on level contours spaced at 10 to 20-foot intervals. The SWPPP designer can also use diversion dikes and berms to keep stormwater off slopes. Concentrated runoff must not flow down cut or fill slopes unless contained within an adequate temporary or permanent channel, flume or slope drain structure.

f. Protect storm drain inlets. Protect all inlets that could receive stormwater from the project until final stabilization of the site has been achieved. Install inlet protection before soil-disturbing activities begin, if possible. Maintenance throughout the construction process is important. Storm drain inlet protection should be used not only for storm drains within the active construction project, but also for storm drains outside the project area that might receive stormwater discharges from the project. If there are storm drains on private property that could receive stormwater runoff from the project, coordinate with the owners of that property to ensure proper inlet protection.

g. Establish perimeter controls. Maintain natural areas and supplement them with perimeter sediment controls to help stop sediment from leaving the site. Install controls on the downslope perimeter of the project (it is most often not necessary to surround the entire site with silt fence). Sediment barriers can be used to protect stream buffers, riparian areas, wetlands, adjacent public right-of-way, and neighboring private properties. They are effective only in small areas and should not be used in areas of concentrated flow. Sediment basins and traps, perimeter dikes, sediment barriers and other measures intended to trap sediment must be constructed as a first step in any land disturbing activity and must be made functional before upslope land disturbance takes place.

h. Retain sediment on-site and control dewatering practices. When sediment retention is from a larger area is required, consider using a sediment trap or basin. These practices detain sediment-laden runoff for a period of time, allowing sediment to settle before runoff is discharged. Proper design and maintenance are essential to ensure that these practices are effective. Where a large sediment basin is not practical, use smaller sediment basins and traps(or both) where feasible. At a minimum, use silt fences, vegetative buffer strips, or equivalent sediment controls for all down-gradient boundaries (and for those side-slope boundaries deemed appropriate for individual site conditions).

   Dewatering practices as used to remove groundwater or accumulated rain water from excavated areas.

Pump muddy water from these areas to a temporary or permanent sedimentation basin or to an area completely enclosed by silt fence or other sediment retention device in a flat vegetated area where discharges can infiltrate into the ground. Never discharge muddy water into storm drains, streams, lakes or wetlands unless sediment has been removed before discharge.

i. Establish stabilized construction exits. Vehicles entering or leaving the site have the potential to track significant amounts of sediment onto streets. Identify and clearly mark one or two locations where vehicles will enter and exit the site and focus stabilizing measures at those locations. Construction exits are commonly made with crushed rock. They can be further be stabilized using stone pads or concrete. No system is perfect, so sweeping/vacuuming the street regularly completes this BMP.

j. Stabilize channels and watercourses. When work in a live watercourse is performed, precautions must be taken to minimize encroachment, control sediment transport and stabilize the work area to the greatest extent possible during construction. Nonerodible material shall be used for the construction of causeways and cofferdams. Earthen fill may be used for these structures if armored by nonerodible cover materials. When live watercourse must be crossed by construction vehicles more than twice in any six month period, a temporary stream crossing constructed of nonerodible material must be provided. The bed and banks of a watercourse must be stabilized immediately after work in the watercourse is completed.

9.3.8 Good Housekeeping BMPs

Construction projects generate large amounts of building-related waste, which can end up polluting stormwater runoff if not properly managed. The suite of BMPs that are described in the SWPPP must include pollution prevention practices that are designed to prevent contamination of stormwater from a wide range of materials and wastes at the site. The five (5) principles described in this section are designed to help the SWPPP designer identify the pollution prevention practices that should be described in the SWPPP and implement at the site.

1. Provide for waste management.

    a. Design proper management procedures and practices to prevent or reduce the discharge of pollutants to stormwater from solid or liquid wastes that will be generated at the site. Practices such as trash disposal, recycling, proper material handling, and cleanup measures can reduce the potential for stormwater runoff to pick up construction site wastes and discharge them to surface waters.

    b. Provide well-maintained and properly located toilet facilities. Provide for regular inspections, service, and disposal. Locate portable toilet facilities at least 20 feet away from storm drain inlets, and at least 10 feet back from the edge of curb and gutter conveyance systems.

2. Establish proper building material handling and staging areas.

    a. The SWPPP must include comprehensive handling and management procedures for building materials, especially those that are hazardous and toxic. Paints, solvents, pesticides, fuels and oils, other hazardous materials or any building materials that have the potential to contaminate stormwater should be stored indoors or under cover whenever possible, or in areas with secondary containment. Secondary containment prevents a spill from spreading across the site and include dikes, berms, curbing, or other containment methods. Secondary containment systems should also ensure protection of groundwater.

    b. Designate staging areas for activities such as fueling vehicles, mixing paints, plaster, mortar, etc. Designated staging areas will help monitor the use of materials and to clean up any spills. Training employees and subcontractors is essential to the success of this pollution prevention principle.

3. Designate washout areas.

    a. All concrete contractors and any subcontractors installing concrete must be required to use designated and marked concrete washout areas on the permitted construction site. Designate specific washout areas and design facilities to handle anticipated washout with water.

b. Washout areas must also be provided for paint and stucco operations. Because washout areas can be a source of pollutants from leaks or spills, it is required that they be located at least 50 yards away from storm drains and watercourses.

c. Regular inspection & maintenance are important for these BMPs. If there is evidence that contractors are dumping materials or into drainage facilities, of if the washout areas are not being used regularly, the SWPPP designer must consider posting additional signage, relocating the facilities to more convenient locations, or providing training to workers and contractors.

4. Establish proper equipment/vehicle fueling and maintenance practices

   a. If off-site fueling and maintenance is not feasible, create an on-site fueling and maintenance area that is clean and dry. The on-site fueling area should have a spill kit, and staff should know how to use it. If possible, conduct vehicle fueling and maintenance activities in a covered area; outdoor vehicle maintenance is a potentially significant source of stormwater pollution, Significant maintenance on vehicles and equipment should be conducted off-site.

   b. Clearly designate vehicle/equipment service areas away from drainage facilities and watercourses to prevent stormwater run-on and runoff.

5. Develop a spill prevention and response plan.

   a. A Spill Prevention and Response Plan is required for the SWPPP that addresses fueling, maintenance, or storage areas on the site. The plan must comply with the requirements of NDEQ Title 126, Chapter 18 - Rules and Regulations Pertaining to the Management of Wastes. If the permittee knows, or has reason to believe, that oil or hazardous substances were released at the facility and could enter Waters of the State or any of the outfall discharges authorized by the permit, the permittee must immediately notify the Nebraska Department of Environmental Quality of a release of oil or hazardous substances. The contact number during office hours is 402-471-2186. When NDEQ cannot be contacted, the permittee must report to the Nebraska State Patrol for referral to the NDEQ Emergency Response Team at telephone number 402-471-4545.

   b. The plan should clearly identify ways to reduce the chance of spills, stop the source of spills, contain and clean up spills, dispose of materials contaminated by spills, and train personnel responsible for spill prevention and response. The plan should also specify material handling procedures and storage requirements and ensure that clear and concise spill cleanup procedure are provided and posted for areas in which spills may potentially occur.

   c. When developing a spill prevention plan, include, at a minimum, the following:
      - Note the locations of chemical storage areas, storm drains, tributary drainage areas, surface water bodies on or near the site, and measures to stop spills from leaving the site.
      - Specify how to notify the appropriate authorities to request assistance.
      - Describe the procedures for immediate cleanup for spills and proper disposal.
      - Identify personnel responsible for implementing the plan in the event of a spill.

9.3.9 SWPPP Inspection, Maintenance and Enforcement Procedures

<u>Inspection & Maintenance Requirements</u>

1. All SWPPP plans submitted for approval must include placement of the following statement, "All sediment and erosion control practices will be inspected at least once every seven calendar days and after any storm event of greater than 0.5 inches of precipitation during any 24-hour period by responsible personnel. Any necessary repairs or cleanup to maintain the effectiveness of the best management practices must be made prior to the next storm event whenever practicable. If implementation before the next storm event is impracticable, the situation must be documented in the SWPPP and alternative BMPs must be implemented as soon as possible."

2. Inspections should be conducted by qualified personnel who are knowledgeable in the principles and practices of erosion & sediment control. Qualified personnel should possess the technical skills to assess conditions at the construction site that could impact stormwater quality, and assess the effectiveness of any erosion & sediment control measures selected.

3. A log of these inspections must be retained with the SWPPP, along with photographs or other supporting information. Any deficiencies must be noted in a report if the inspection and include any action taken to correct the deficiency. Inspection reports and follow-up documentation regarding violations and associated corrective actions must be submitted to the City of Waverly upon request.

4. Inspection reports must include the following information:
   a. Inspectors name
   b. Inspection date
   c. Weather information for the period since the last inspection including a best estimate of the beginning of each storm, its duration, approximate amount of rainfall for each storm, and whether any discharges occurred.
   d. Findings of the inspections
   e. Any corrective actions taken (including dates, times, and party completing maintenance activities)
   f. Documentation of changes made to the SWPPP
   g. Monitoring results, if requested

5. Record keeping: The permittee must keep copies of the SWPPP, inspection records, copies of all reports required by the permit, and records of all data used to complete the NOI to be covered by the permit for a period of at least three (3) years from the date that permit coverage expires or is terminated. Records should include:
   a. A copy of the SWPPP, with any modifications
   b. A copy of the NOI and Notice of Termination (NOT) and any stormwater-related correspondence with federal, state, and local regulatory authorities
   c. Inspection forms, including the date, place, and time of BMP inspections
   d. Names of inspector(s)
   e. The date, time, exact location, and a characterization of significant observations, including spills and leaks
   f. Records of any non-stormwater discharges
   g. BMP maintenance and corrective actions taken at the site (Corrective Action Log)
   h. Any documentation and correspondence related to endangered species and historic preservation requirements
   i. Date(s) when major land disturbing (i.e. clearing, grading, and excavating) activities occur in an area
   j. Date(s) when construction activities are either temporarily of permanently ceased in an area
   k. Date(s) when an area is either temporarily or permanently stabilized

Enforcement Procedures

1. By submittal of the SWPPP for approval, the applicant certifies the right of NDEQ to conduct on-site inspections at any time.

2. Pursuant to the Waverly Municipal Code, it is unlawful for any person to:
   a. Engage in construction activity without a permit
   b. Violate any term or condition of the permit
   c. Violate any term or condition of the SWPPP
   d. Make any false statement, representations, or certification in a document submitted to or requested by the City or the Lower Platte South Natural Resources District on behalf of the City, or
   e. Violate any provision of Chapter 28.01

   Violations will be addressed by the City of Waverly.

3. Upon completion of the grading, implementation of the SWPPP and permanent, final stabilization of the site, the landowner/land developer must submit a "Notice of Termination of Construction Activity" to NDEQ.

4. A final inspection will be made by the City for full and final compliance.

9.4 Best Management Practice (BMP) Selection

This section provides a decision-making process that can be used to select best management practices (BMPs) to control erosion and sedimentation. It also provides principles for the selection of BMPs for "good housekeeping" on a construction site.

9.4.1 Steps in Selection of Control Measures

Step 1: Identify Control Method(s) - On any construction site the objective in erosion and sediment control is to prevent off-site sedimentation damage. Three basic methods are used to control sediment transport from construction sites: runoff control, soil stabilization, and sediment control. Controlling erosion (runoff control and soil stabilization) should be the first line of defense. Controlling erosion is effective for small disturbed areas such as single lots or small areas of a development that do not drain to a sediment trapping facility. Sediment trapping facilities should be used on large developments where mass grading is planned, where it is impossible or impractical to control erosion, and where sediment particles are relatively large. Runoff control and soil stabilization should be used together where soil properties and topography of the site make the design of sediment trapping facilities impractical. Cost-effective erosion and sediment control typically includes a combination of vegetative and structural erosion and sedimentation control measures.

Step 2: Identify Problem Areas - Potential erosion and sediment control problem areas should be identified. Areas where erosion is to be controlled will usually fall into categories of slopes, graded areas or drainage ways. Slopes include graded rights-of-way, stockpile areas, and all cut and fill slopes. Graded areas include all stripped areas other than slopes. Drainage ways are areas where concentrations of water flow naturally or artificially, and the potential for gully erosion is high.

Step 3: Identify Required Strategy - The third step in erosion and sediment control planning is to develop a strategy that can be taken to solve the problem. For example, if there is a cut slope to be protected from erosion, the strategies may include protecting the ground surface, diverting water from the slope or shortening the slope. Any combination of the above can be used. If no rainfall except that which falls on the slope has the potential to cause erosion and if the slope is relatively short, protecting the soil surface is often all that is required to solve the problem.

Step 4: Select Specific Control Measures - The final step in erosion and sediment control planning can be accomplished by selecting and adapting specific control measures that accomplish the strategy developed in Step 3. Items to consider when selecting a final best management practice are as follows:

- **Acceptance** - Look at environmental compatibility, institutional acceptance and visual impact.

- **Cost** - Consider material cost, add-ons, installation and preparation costs. (See the LPSNRD (1994) Manual of Erosion and Sediment Control and Stormwater Management for detailed examples on conducting benefit-cost analysis.)

- **Effectiveness** - Compare effectiveness of different BMP's. Use manufacturer specs to compare engineering properties. BMP technology has improved dramatically; it is important to be familiar with new, effective techniques & products for effective erosion & sediment control.

- **Installation** - Consider ease of installation and durability once installed.

- **Vegetation** - Consider compatibility of BMP to foster vegetation.

- **Operation** - Consider maintenance requirements for the various BMPs, and care for establishing vegetation.

9.5 Best Management Practice Design

9.5.1 Introduction

This section provides a discussion of commonly used erosion and sediment control practices with specific emphasis on use limitations, design details and construction specifications. Please note that this section provides an overview of the more common BMPs.

9.5.2 Vegetated Buffer/Filter Strip

Vegetated buffers are areas of either natural or established vegetation that are to protect water quality of neighboring areas. As shown in Figure 9-2, vegetated buffer/filter strips reduce the velocity of storm water runoff, provide an area for the runoff to permeate the soil, contributes to ground water recharge, and acts as a filter to catch sediment. The reduction on velocity also helps to prevent soil erosion. With proper design and maintenance they can be viewed as a landscape amenity.

Design Detailing

- Soils should not be compacted
- Slopes should be less than 5 percent
- Buffer with should be determined after careful consideration of slope, vegetation, soils, depth of impermeable layers, runoff sediment characteristics, types and quanity of storm water pollutants, and annual rainfall.
- Buffer widths should increase as the slope increases.
- Zones of vegetation (native vegetation in particular), including grasses, deciduous and evergreen shrubs and trees should be intermixed.
- In areas where flows are concentrated and velocities are high, vegetated buffer/.filter strips should be combined with other structural or nonstructural BMPs as a pretreatment.

Maintenance

Keeping vegetation healthy in a vegetated buffer/filter strip requires routine maintenance, which (depending on species, soil types, and climate conditions) can include weed and pest control, mowing, fertilizing, irrigating , and pruning. Inspection and maintenance are most important when buffers are first installed. Once established, vegetated buffer/filter strips do not require much maintenance beyond the routine procedures. Inspections should be conducted after any heavy rainfall and at least once a year. Inspections should be focused on encroachment, gully erosion, density of vegetation, any evidence of concentrate flows through the area or any foot or vehicle traffic that might have caused damage.

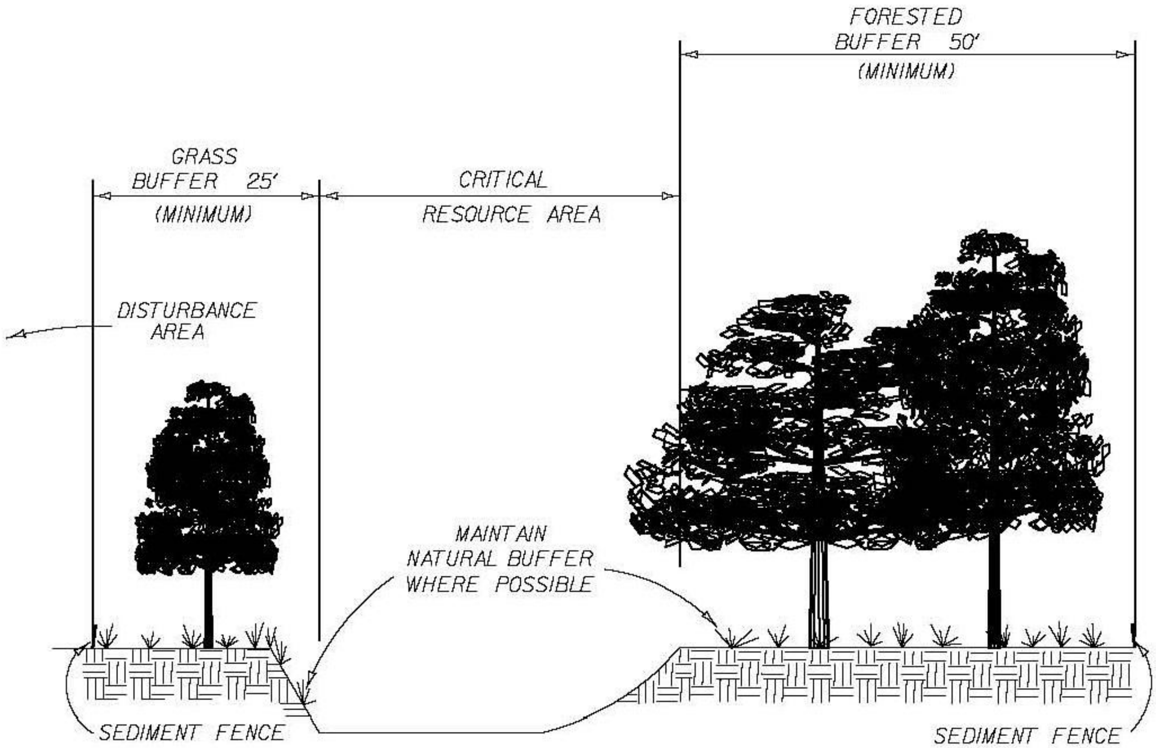


Figure 9-2: Vegetated Filter / Buffer Strip

9.5.3 Silt Fence

Silt fence is a temporary sediment barrier consisting of a synthetic fabric stretched across and attached to supporting posts and entrenched or sliced in place. Figures 9-3 through 9-6 provides an example illustrations of installation and placement of silt fence. Silt fences can be used in the following applications:

- for intercepting and detaining small amounts of sediment from disturbed areas during construction operations in order to prevent sediment from leaving the construction site,
- for decreasing the velocity of sheet flows
- in high-risk areas, such as those adjacent to streams, wetlands, reservoirs, lawns, etc.,
- in short lengths at the toe of fill where ground slopes toward the fill,
- behind curb and gutter to prevent silting of the pavement.

Prior to start of construction, silt fence placement should be designed by a qualified professional. Plans and specifications should be referred to by field personnel throughout the construction process.

<u>Use Limitations</u>

- If the size of the drainage areas is more than 1/4-acre per 100 feet of silt fence length, a different sediment and erosion control strategy should be investigated. The maximum gradient behind the barrier should be no more than 50% (2H:1V).
- Under no circumstances should silt fences be constructed in live streams or in swales or ditch lines where flows are likely to exceed 1 cubic foot per second.
- On steep slopes, care should be given to placing alignment of fence perpendicular to the general direction of the flow.

Design Detailing

- Drainage Area: Limited to 1/4 acre per 100 feet of fence. Area is further restricted by slope steepness as shown in Table 9-1:

Table 9-1: Typical Land Slope and Distance for Silt Fence

Land slope (%)	Maximum Slope Distance* above fence (Feet)
Less than 2	150
2 to 5	100

*Follow manufacturer's recommendations for proper placing.

- Location: Fence should be built on a nearly level grade and at least 6 feet from the toe of the slope to provide a broad shallow sediment pool.
- Length: Maximum of 100 Feet. Runs of silt fence should always be designed on the contour for maximum sediment retention. Flare ends of the fence uphill to temporarily impound water. Proper placement and location of silt fence is further explained in Figures 9.4-9.7

Construction Guidelines

The following guidelines apply to silt fence materials:

- Support Posts: A minimum weight of 1.0 lb./linear foot steel 'T' posts or 2-inch square wood stakes, buried or driven to a length of 18-24 inches.
- Support Post Spacing: Post spacing should be a maximum of 6 feet.
- Synthetic Geotextile Fabric: Conforming to specifications in the table below and containing ultraviolet light inhibitors and stabilizers. Minimum design life of 6 months. The following minimum specifications are:
  - Filtering Efficiency Test -75% (ASTM 5141)
  - Tensile Strength at 20% (max) Elongation - 30 lb./linear inch (ASTM 4632)
  - High Strength - 50 lb./linear inch (ASTM4632)
  - Flow Rate - 0.2 gal./sq. ft./minute (ASTM 5141)
  - Ultraviolet Radiation Stability - 90% (ASTM-G-26)

  Note: Properties are reduced by 50% after 6 months.
- The height of a silt fence shall not exceed 3 feet (higher fences may impound volumes of water sufficient to cause failure of the structure).
- When joints are necessary, filter cloth shall be spliced together only at a support post, with a minimum 6-inch overlap, and securely sealed.
- A trench shall be excavated approximately 4 inches wide and 8 inches deep along the line of posts and upslope from the barrier.
- The trench must be backfilled and the soil compacted on both sides of the trench and over the filter fabric.
- Silt fences should be removed when they have served their useful purpose, but not before the upslope area has been permanently stabilized.

9.5.4 Silt Fence Installation Machines

Silt fence installation machines insert a narrow custom-shaped blade at least 10 inches into the ground and simultaneously pull silt fence fabric into the small opening created as the blade is pulled through the ground.

Construction Guidelines

When silt fence is used in a stormwater pollution prevention plan, a silt fence installation machine should be used to place the silt fence. By slicing the fence into the ground and compacting soil over the imbedded fence, a sturdy structure is created that can trap sediment more efficiently.

See Figures 9.4 - 9.7 for installation guidelines.

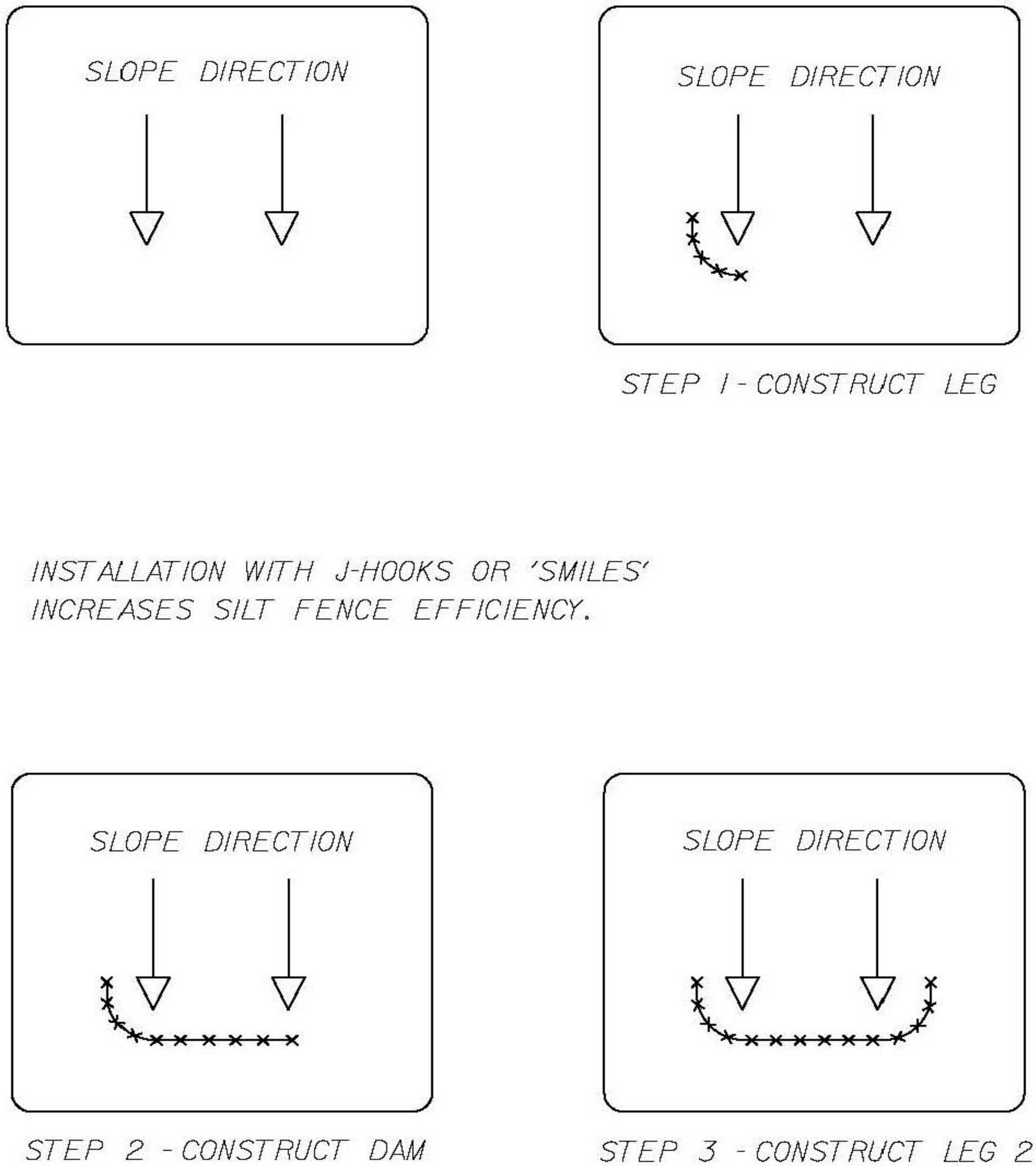


Figure 9-3: Silt Fence Placement - One Slope

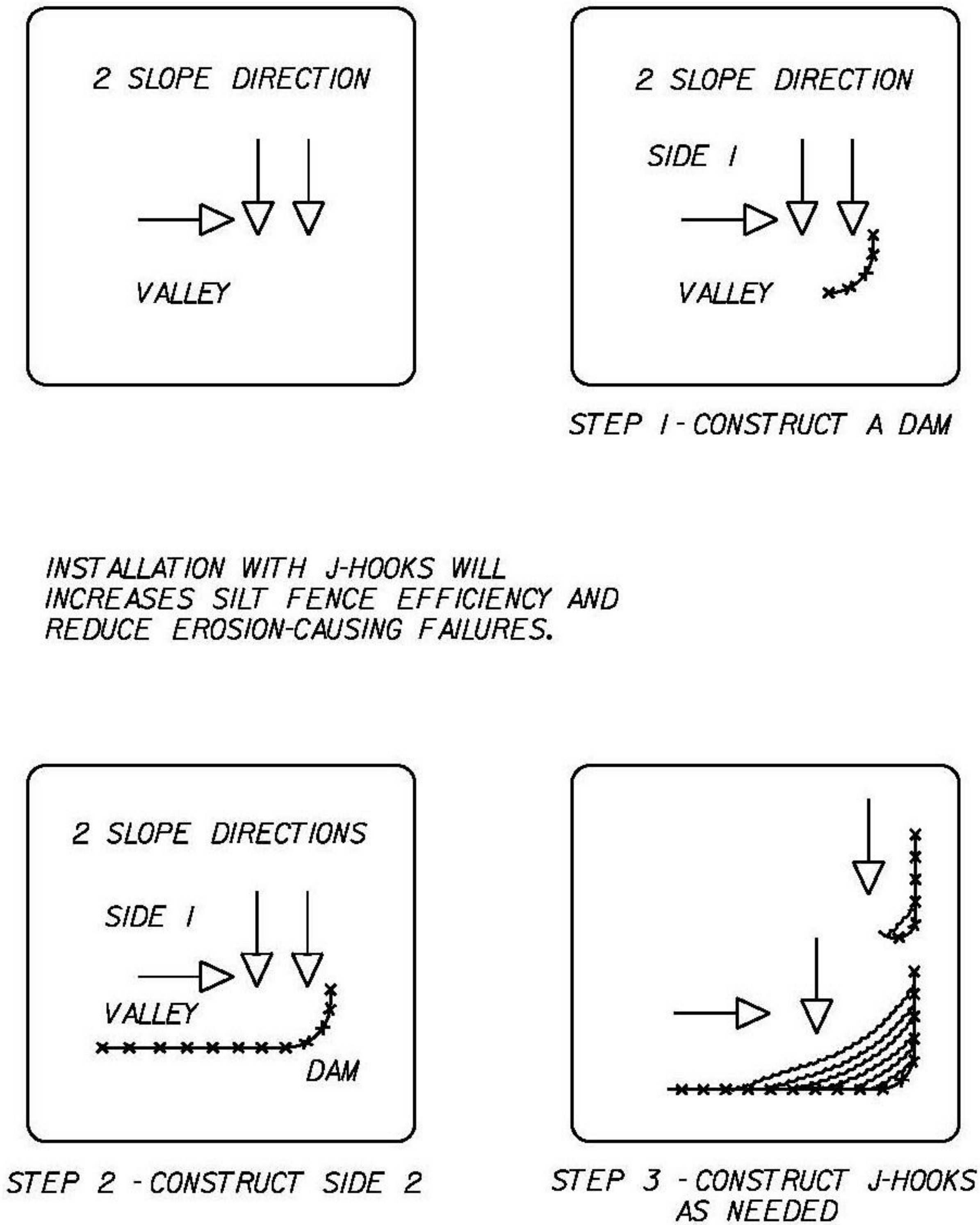


Figure 9-4: Silt Fence Placement - Two Slopes

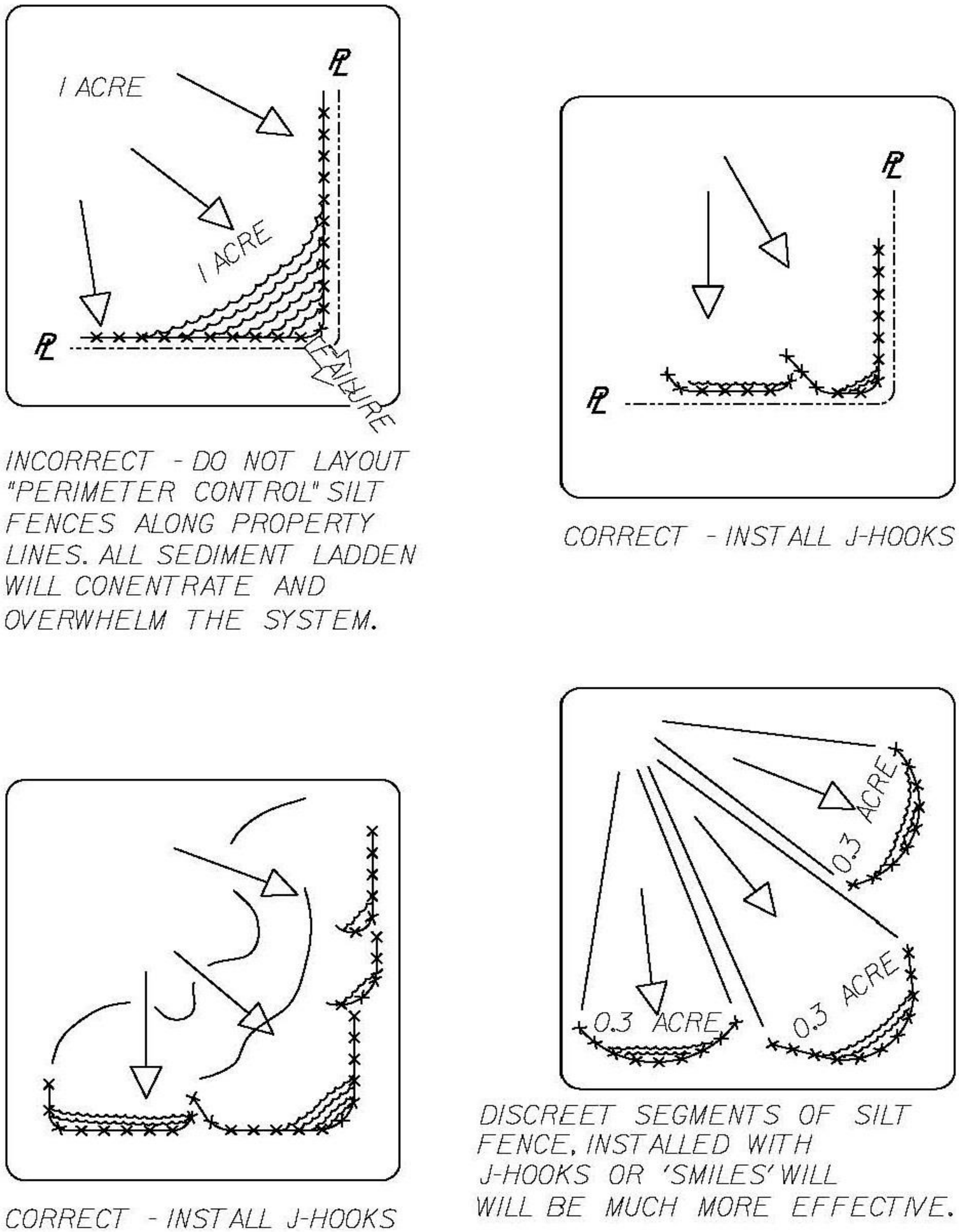


Figure 9-5: Silt Fence Placement - Perimeter Control

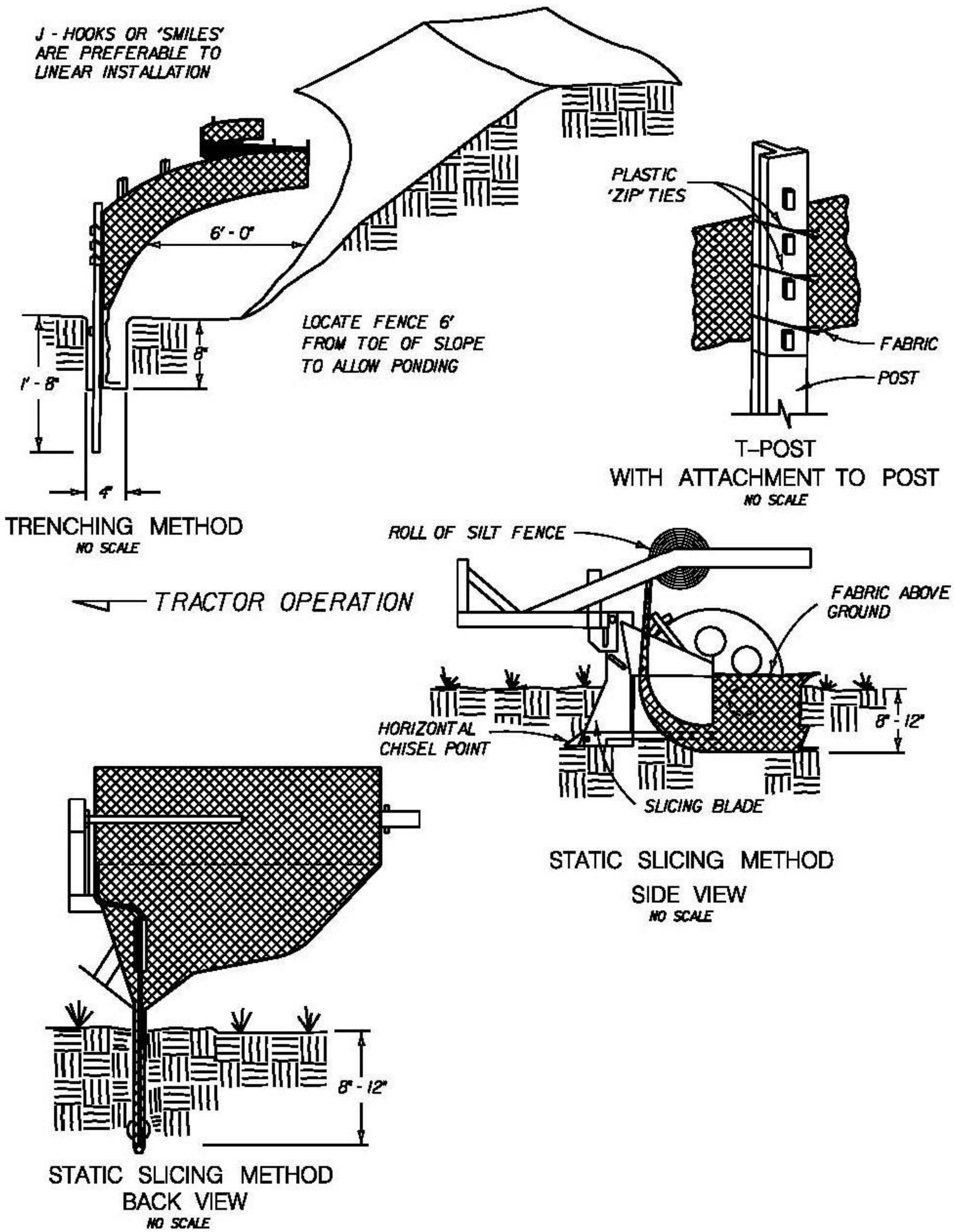


Figure 9-6: Silt Fence Installation / Slicing Method

9.5.5 Wattle Barriers

Wattle Barriers are elongated tubes of compacted straw and or other fibers that are installed along contours or at the base of slopes to help reduce soil erosion and retain sediment. They function by shortening slope lengths; reducing runoff water velocity thus trapping dislodged soil particles. They can work as check dams to prevent sheet, rill, and gully erosion.

Design Detailing

- Wattle Barriers are designed for low surface flow and not be place in the path of high waterflow.
- Wattles should be placed on contours with a slight downward angle at the end of the row to prevent ponding at the mid-section.
- Wattles should be installed in a shallow trench with the running lengths abutted firmly to ensure no leakage at the abutments.
- When the Wattle is installed behind a sidewalk or curb, Wattle does not have to be placed in a trench.
- Vertical spacing for slope installation shall be determined by the site conditions.

Slope	Spacing
1:1	10 Feet apart
2:1	20 Feet apart
3:1	30 Feet apart
4:1	40 Feet apart

- Use wooden stakes or willow cuttings. Wood stakes will eventually bio-degrade and willow cuttings have the potential to grow and provide additional stabilization. Stakes are to be driven through the middle of the wattle leaving 2 - 3 inches protruding above the wattle.

Maintenance

The Wattle Barriers shall be inspected after installation to insure that they are trenched in and that no gaps exist underneath or between adjacent ends and must be inspected after significant rainfall event. Any rills, gullies, and undercutting on the upslope side must be repaired. Sediment must be removed when it reaches one third of the height. Wattle Barriers must be removed and/or replaced as required to adapt to changing conditions.

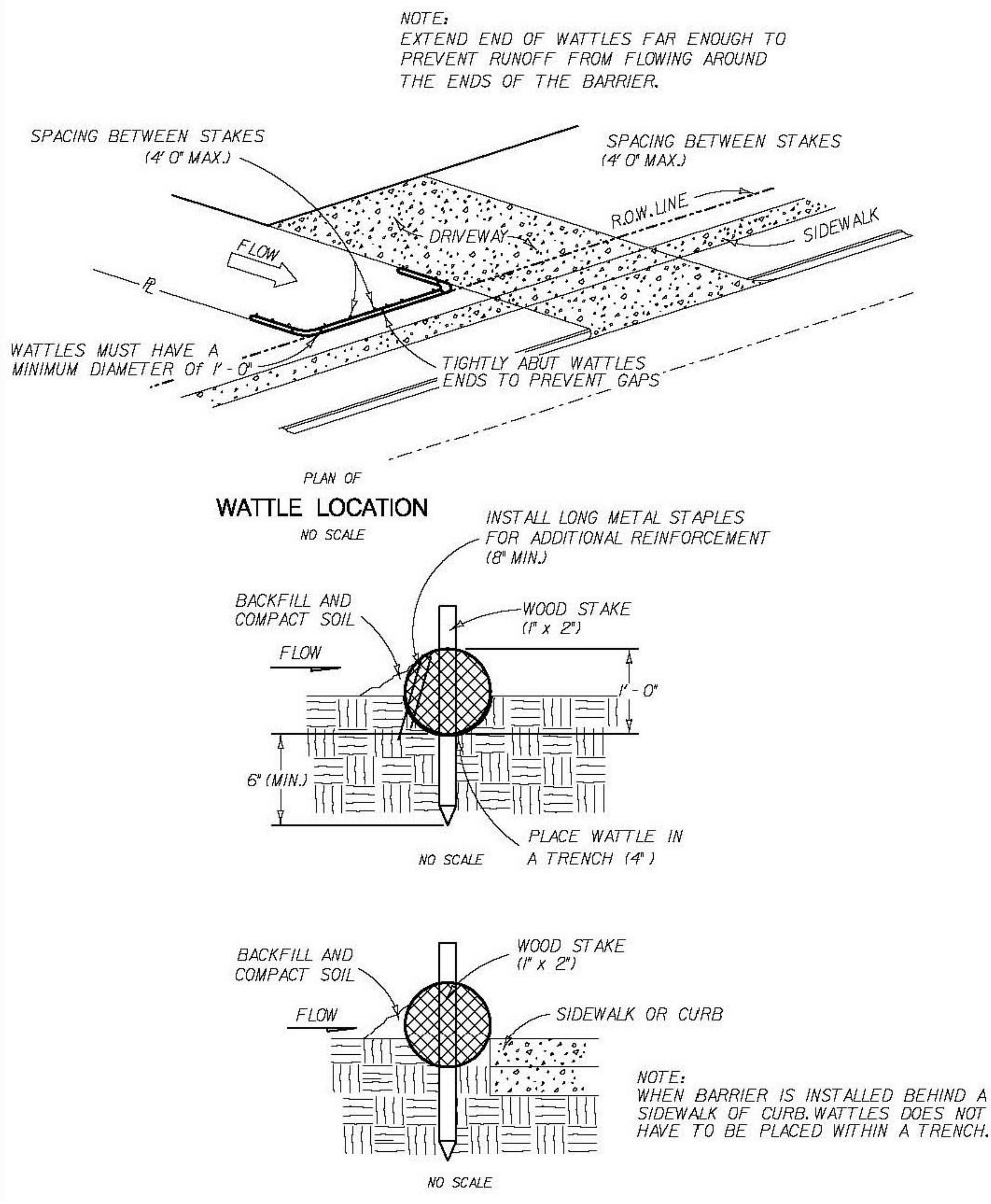


Figure 9-7: Wattle Barrier Installation

9.5.6 Compost Berms

Compost berms are contoured runoff and erosion filtration methods usually used for steeper slopes with high erosive potential. The berm allows runoff water to penetrate it and continue to flow while filtering sediment and pollutants from the water. It also slows the flow down, allowing soil particles to settle out. Compost berms work well when the slope exceeds 4:1.

<u>Design Detailing</u>

- Berm size may vary based on slope severity, larger berms are recommended for steeper slopes.
- Compost berms are typically contoured to the base of the slope.
- Berms may be windrow or trapezoidal (allows maximum water penetration) in shape
- Windrow shaped berms should be between 1 to 2 feet high and 2.5 to 4 feet wide.
- Trapezoidal berms should be approximately 2 feet high, 2 to 3 feet wide at the top and at least 4 feet wide at the base
- Placed on uncompacted or bare soil.
- Vegetation or compost blankets may be used in front or above the berms but never under them.
- Can be seeded at time of installation for additional filtering capacity.
- Never construct compost berms in runoff channels, ditches or gullies.

<u>Maintenance</u>

Accumulated sediment should be removed, or a new berm installed, when it reaches approximately one-third of the berm height. If concentrated flows are bypassing or breaching the berm, it must be expanded, enlarged or augmented with additional erosion and sediment control practices. Dimensions of the berm must be maintained. Any damage should be repaired immediately.

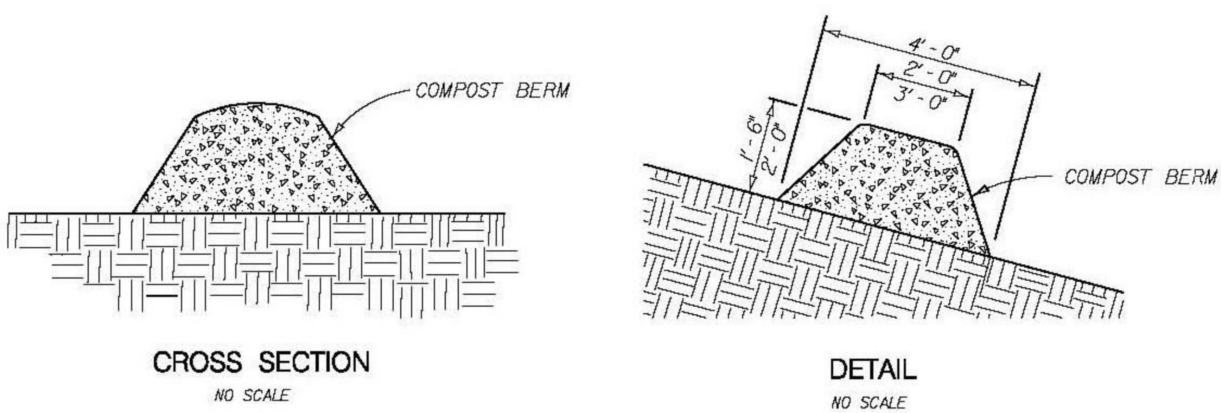


Figure 9-8: Compost Berm Detail

9.5.7 Storm Drain Inlet Protection

Storm drainage inlet protection is a sediment filter or an excavated impounding area around a storm drain drop inlet or curb inlet. Its purpose is to prevent sediment from entering storm drainage systems prior to permanent stabilization of the disturbed area. Different types of storm drain inlet protection are shown in Figures 9-9 through 9-14.

Conditions Where Practice Applies

This practice shall be used where the drainage area to an inlet is disturbed, it is not possible to temporarily divert the storm drain outfall into a trapping device and watertight blocking of the inlets is not advisable. It is not to be used in place of sediment trapping devices. This may be used in conjunction with storm drain diversion to help prevent siltation of pipes installed with low slope angle. There are eight specific types of storm drain inlet protection practices that vary according to their function, location, drainage area and availability of materials:

1. Silt Fence Drop-Inlet Protection
2. Block and Gravel Drop-Inlet Sediment Filter
3. Gravel Curb Inlet Sediment Filter
4. Curb Inlet Protection with Weir
5. Block and Gravel Curb Inlet Sediment Filter

Design Detailing

- The drainage area shall be no greater than 1 acre.
- The inlet protection device must be constructed in a manner that will facilitate cleanout and disposal of trapped sediment and minimize interference with construction activities.
- The inlet protection device must be constructed in such a manner that any resultant ponding of stormwater will not cause excessive inconvenience or damage to adjacent areas of structures.
- Design criteria more specific to each particular inlet protection device will be found within this specification.
- For the inlet protection devices which utilize stone as the chief ponding/filtering medium, a range of stone sizes is offered, 3/4" to 1-1/2" clean stone can be used. The designer should attempt to get the greatest amount of filtering action possible (by using smaller size stone), while not creating significant ponding problems.
- In all designs which utilize stone with a wire mesh support as a filtering mechanism, the stone can be completely wrapped with the wire mesh to improve stability and provide easier cleaning.
- Filter fabric may be added to any of the devices which utilize coarse aggregate stone to significantly enhance sediment removal. The fabric should be secured between the stone and the inlet (on wire mesh if present). As a result of the significant increase in filter efficiency provided by the fabric, a larger size of stone aggregate (1-1/2" to 2-1/2"), may be utilized with such a configuration. The larger stone will help keep larger sediment masses from clogging the fabric. Notably, significant ponding may occur at the inlet if the filter cloth is utilized in this manner.

Construction Guidelines

1. Silt Fence-Type II Grate Inlet Protection
   a. Silt fence shall be cut from a continuous roll to avoid joints.
   b. Stakes should be steel ‘T’ posts with a minimum length of three feet. Stakes shall be spaced evenly around the perimeter of the inlet a maximum of three feet apart and securely driven into the ground a minimum of 18-24 inches deep.
   c. Place the bottom 8 inches of the fabric in a trench and backfill the trench with 8 inches of compacted soil.
   d. Fasten fabric securely by plastic “zip” ties to the stakes and frame. Joints must be overlapped to the next stake.
   e. It may be necessary to build a temporary dike on the downslope side of the structure to prevent bypass flow.
   f. Remove sediment from the pool area as necessary or when it reaches half the height of the fabric, with care not to undercut or damage the fabric.

2. Block and Gravel Drop-Inlet Sediment Filter
    a. Place concrete blocks lengthwise on their sides in a single row around the perimeter of the inlet, with the ends of adjacent blocks abutting. The height of the barrier can be varied, depending on the design needs, by stacking combinations of 4", 8" and 12" wide blocks. The barrier shall be at least 12 inches high and no greater than 24 inches high.
    b. Wire mesh shall be placed over the outside vertical face of the concrete blocks to prevent stone from being washed through the holes in the blocks. Wire mesh with ½" opening shall be used.
    c. Stone shall be piled against the wire to the top of the block barrier.
    d. If the stone filter becomes clogged with sediment so that it no longer adequately performs its function, the stone must be pulled away from the blocks, cleaned and replaced.

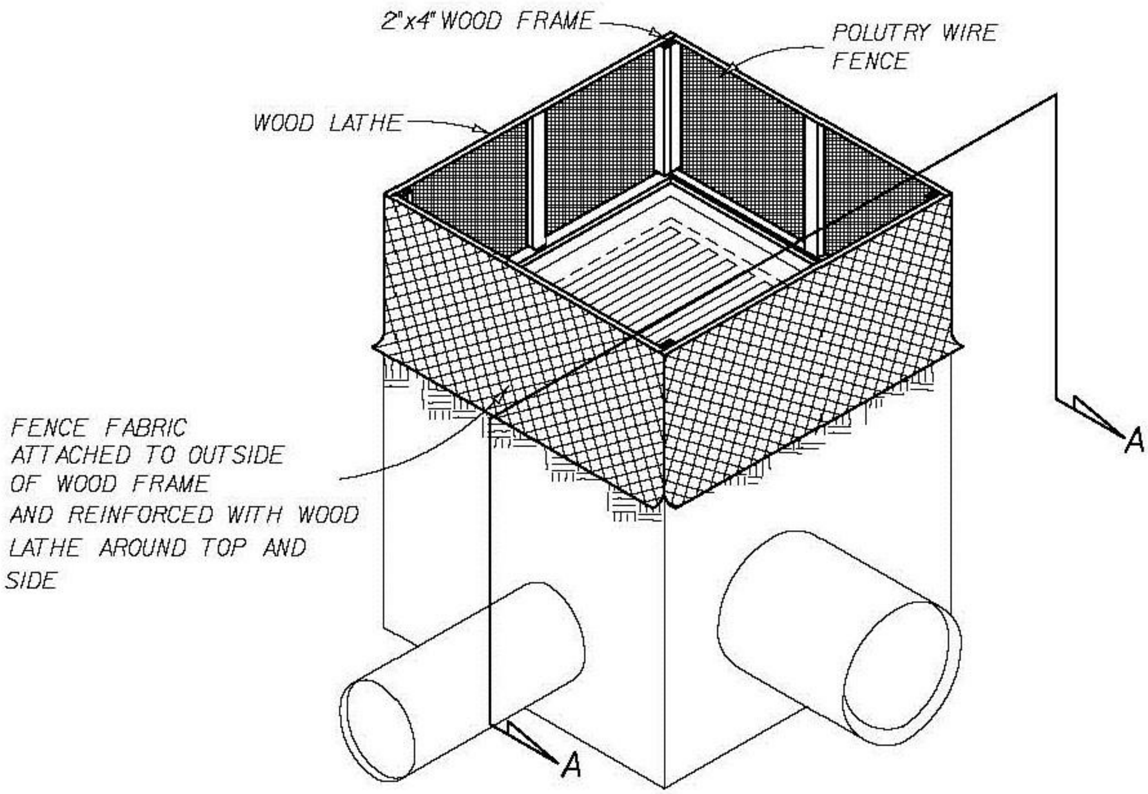
3. Gravel Curb Inlet Sediment Filter
    a. Wire mesh with ½" openings shall be placed over the curb inlet opening so that at least 12 inches of wire extends across the inlet cover and at least 12 inches of wire extends across the concrete gutter from the inlet opening.
    b. Stone shall be piled against the wire so as to anchor it against the gutter and inlet cover and to cover the inlet opening completely.
    c. If the stone filter becomes clogged with sediment so that it no longer adequately performs its function, the stone must be pulled away from the inlet, cleaned and replaced.

4. Curb Inlet Protection with Weir
    a. Attach a continuous piece of wire mesh (30" minimum width x inlet throat length plus 4 feet) to the 2" x 4' wooden weir (with a total length of inlet throat length plus 2 feet). Wood shall be exterior type construction lumber.
    b. Place a piece of filter fabric the same dimensions as the wire mesh over the wire mesh and securely attach to the 2" x 4" weir.
    c. Securely nail the 2" x 4" weir to the 9" long vertical spacers which are to be located between the weir and inlet face at a maximum 6 foot spacing.
    d. Place the assembly against the inlet throat and nail 2 foot minimum lengths of 2" x 4" lumber to the top of the weir at spacer locations. These anchors shall extend across the inlet tops and be held in place by sandbags or alternate weight.
    e. The assembly shall be placed so that the end spacers are a minimum of 1 foot beyond both ends of the throat opening.
    f. Form the wire mesh and filter fabric to the concrete gutter and against the face of the curb on both sides of the inlet. Place coarse aggregate over the wire mesh and filter fabric in such a manner as to prevent water from entering the inlet under or around the filter cloth.
    g. This type of protection mush must be inspected frequently and the filter cloth and stone replaced when clogged with sediment.
    h. Assure that storm flow does not bypass inlet by installing temporary earth dikes directing flow into inlet.

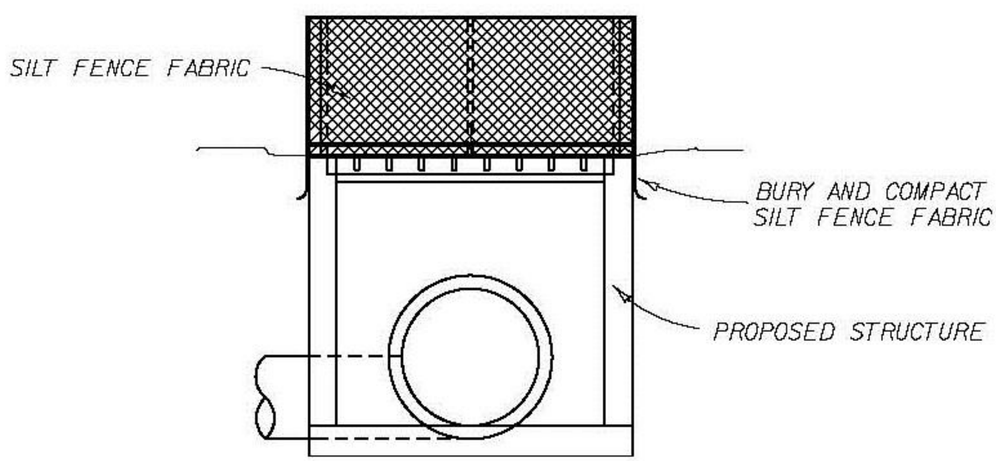
5. Block and Gravel Curb Inlet Sediment Filter
    a. Two concrete blocks shall be placed on their sides abutting the curb at either side of the inlet opening. A 2"x 4" stud shall be cut and placed through the outer holes of each spacer block to help keep the front blocks in place.
    b. Concrete blocks shall be placed on their sides across the front of the inlet and abutting the spacer blocks.
    c. Wire mesh with ½" openings shall be placed over the outside vertical face of the concrete blocks to prevent stone from being washed through the holes in the blocks.
    d. Coarse aggregate shall be piled against the wire to the top of the barrier.
    e. If the stone filter becomes clogged with sediment so that it no longer adequately performs its function, the stone must be pulled away from the blocks, cleaned and replaced.

<u>Maintenance</u>

1. Structures must be inspected after each rain and repairs made as necessary.
2. Structures shall be removed and the area stabilized when the remaining drainage area has been properly stabilized



TOP View
NO SCALE



SECTION A–A
NO SCALE

Figure 9-9: Type I Grate Inlet Protection

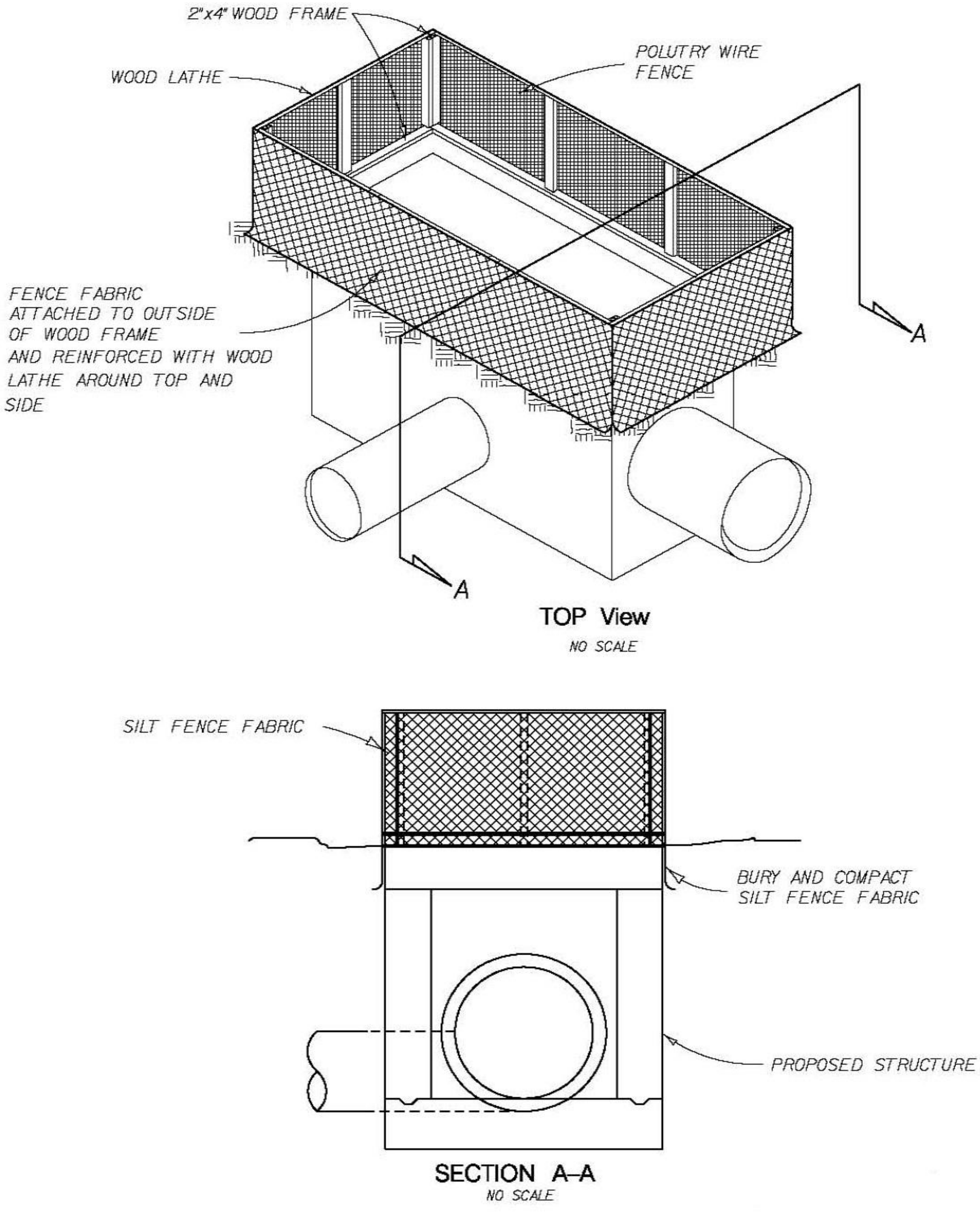
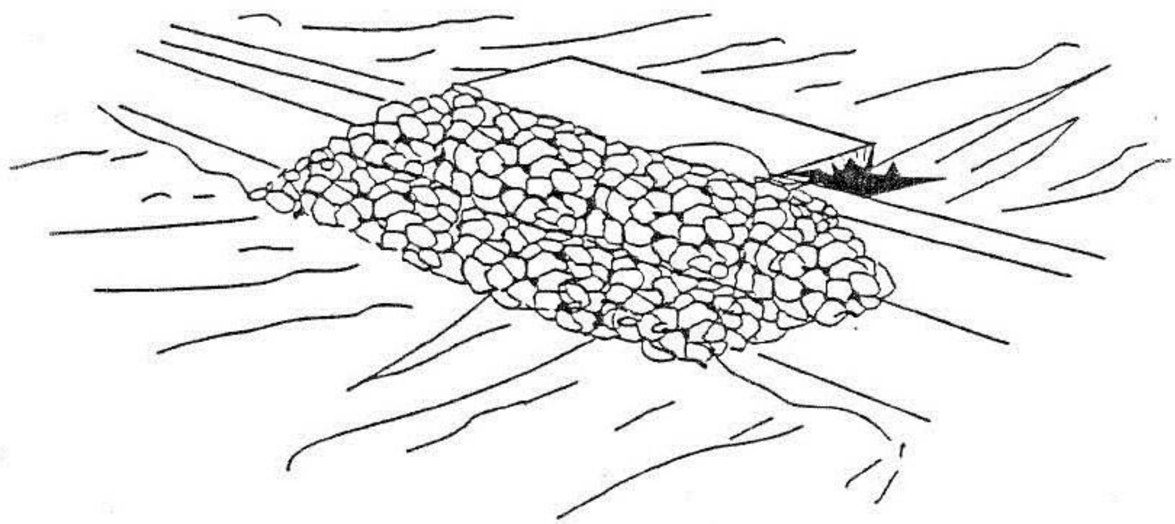
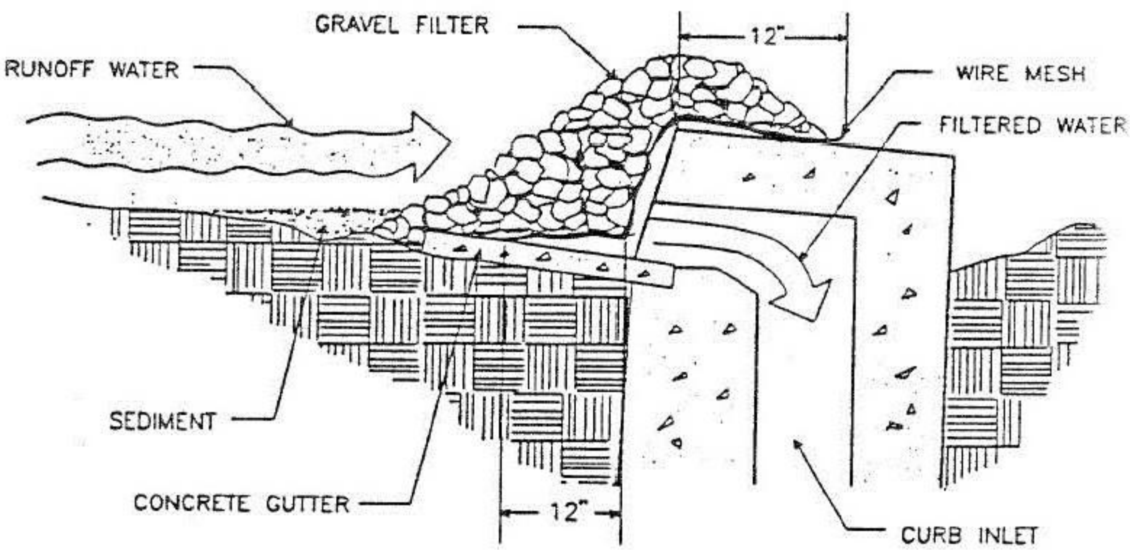


Figure 9-10: Type II Grate Inlet Protection



GRAVEL CURB INLET SEDIMENT FILTER



FILTER CROSS SECTION

SPECIFIC APPLICATION

THIS METHOD OF INLET PROTECTION IS APPLICABLE AT CURB INLETS WHERE PONDING IN FRONT OF THE STRUCTURE IS NOT LIKELY TO CAUSE INCONVENIENCE OR DAMAGE TO ADJACENT STRUCTURES AND UNPROTECTED AREAS.

Figure 9-11: Gravel Curb Inlet Filter

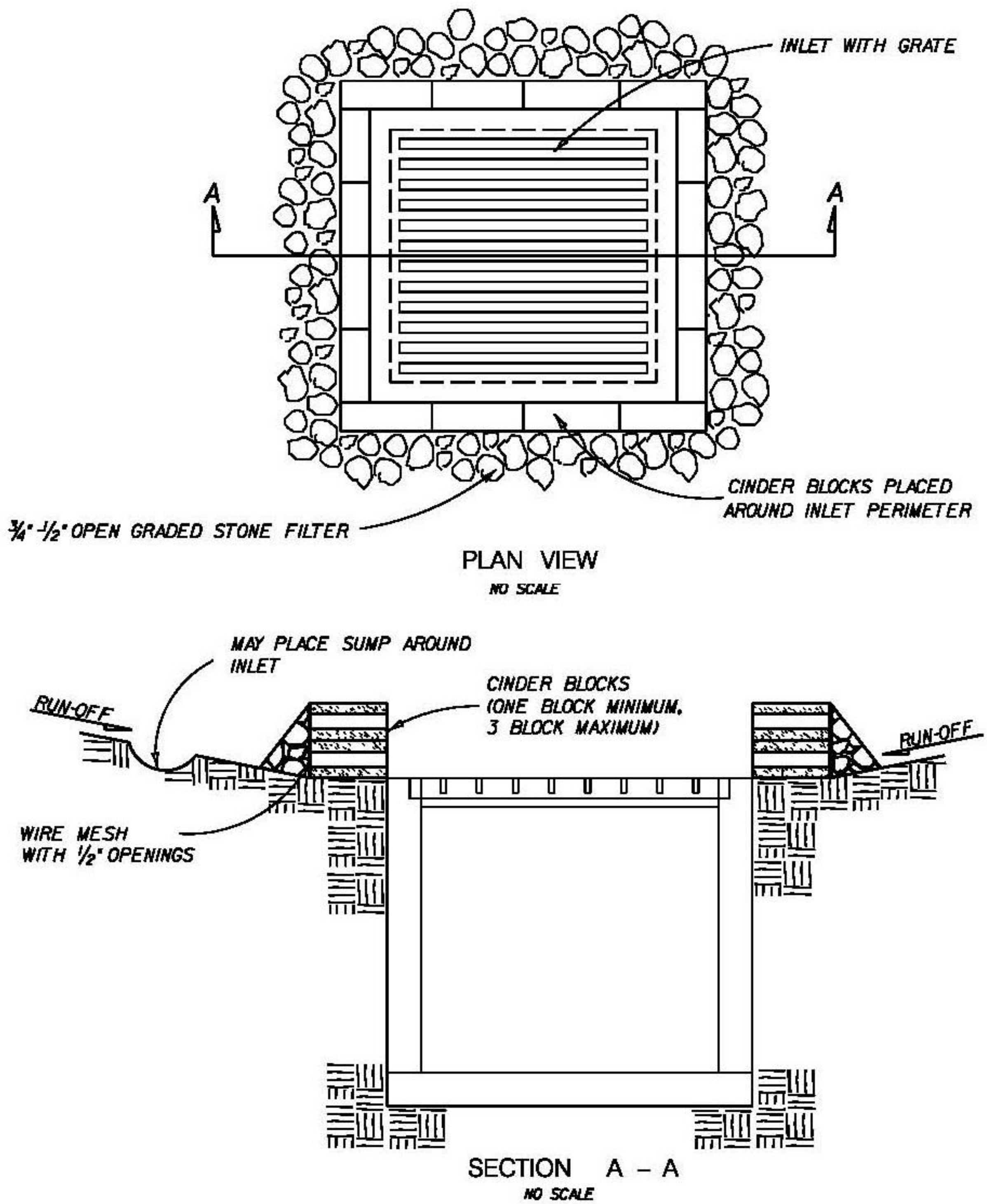


Figure 9-12: Cinder Block and Stone Grate Inlet Protection

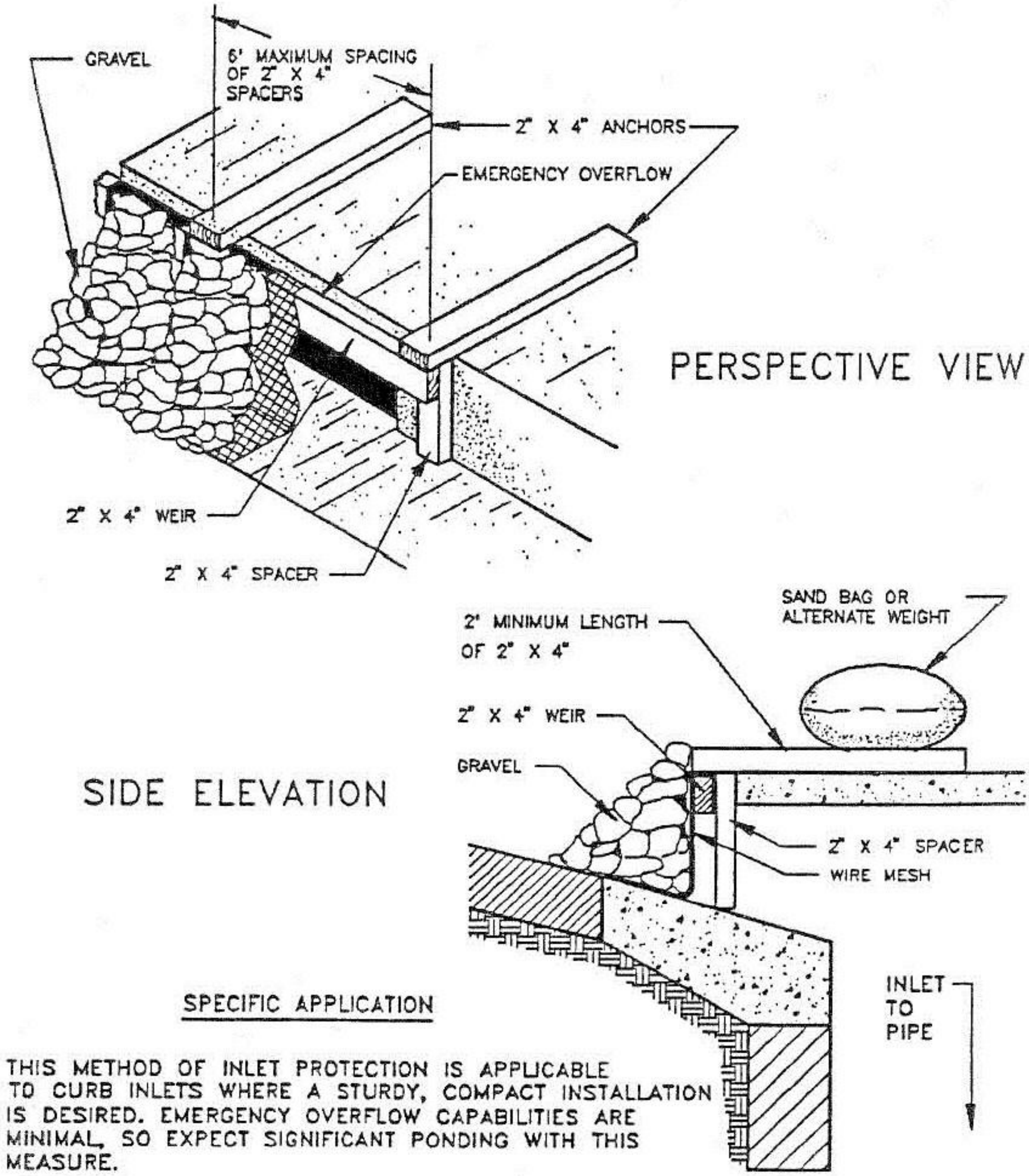
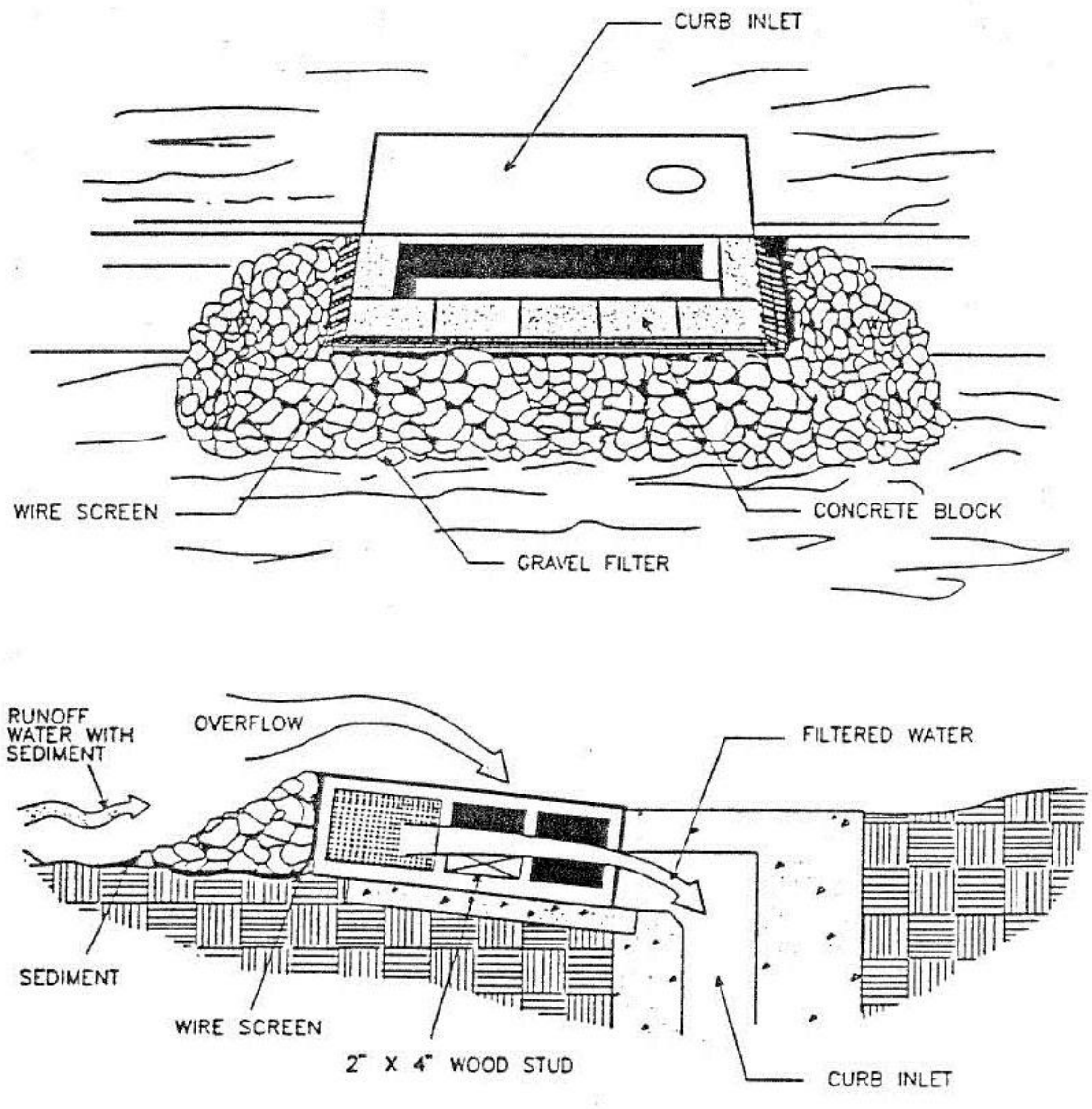


Figure 9-13: Curb Inlet Protection with 2"x4" Wooden Weir



SPECIAL APPLICATION

THIS METHOD OF INLET PROTECTION IS APPLICABLE AT CURB INLETS WHERE AN OVERFLOW CAPABILITY IS NECESSARY TO PREVENT EXCESSIVE PONDING IN FRONT OF THE STRUCTURE.

Figure 9-14: Block and Gravel Curb Inlet Sediment Filter

9.5.8 Outlet Protection

The outlets of pipes and structurally lined channels are points of critical erosion potential. To prevent scour at stormwater outlets, a flow transition structure is needed which will absorb the initial impact of the flow and reduce the flow velocity to a level which will not erode the receiving channel or area.

The most commonly used device for outlet protection is a structurally lined apron. These aprons are generally lined with riprap, grouted riprap or concrete. Where flow is excessive for the economical use of an apron, excavated stilling basins or other alternative structures may be used (Figures 9-15 through 9-17). Examples of other aprons described in Chapter 7 are shown in Figure 9-15.

Design Detailing

Table 9-2 gives the permissible velocity recommendations for the determination of outlet protection needs. Additional design detailing can be found in Chapter 7 of this manual or in the following sources.

- Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14, U.S. Department of Transportation, Federal Highway Administration.
- Hydraulic Design of Stilling Basins and Energy Dissipators, Engineering Monograph No. 25, U.S. Department of the Interior, Bureau of Reclamation.

Table 9-2 Permissible Velocities For Grass And Earth Lined Channels

Grass-Lined Channels

Channel Slope	Lining	Permissible Velocity*
0.5%	Bermuda grass	6 ft/s
	Reed canarygrass	5 ft/s
	Tall fescue	5 ft/s
	Kentucky bluegrass	5 ft/s
	Grass-legume mixture	4 ft/s
	Red fescue	4 ft/s
	Redtop	4 ft/s
	Sericea lespedeza	4 ft/s
	Annual lespedeza	4 ft/s
5-10%	Bermudagrass	5 ft/s
	Reed canarygrass	4 ft/s
	Tall fescue	4 ft/s
	Kentucky bluegrass	4 ft/s
	Grass-legume mixture	3 ft/s
>10%	Bermudagrass	4 ft/s
	Reed canarygrass	3 ft/s
	Tall fescue	3 ft/s
	Kentucky bluegrass	3 ft/s

Earth Linings

Soil Types	Permissible Velocity[1]
Fine Sand (noncolloidal)	2.5 ft/s
Sandy Loam (noncolloidal)	2.5 ft/s
Silt Loam (noncolloidal)	3.0 ft/s
Ordinary Firm Loam	3.5 ft/s
Fine Gravel	5.0 ft/s
Stiff Clay (very colloidal)	5.0 ft/s
Graded, Loam to Cobbles (noncolloidal)	5.0 ft/s
Graded, Silt to Cobbles (colloidal)	5.5 ft/s
Alluvial Silts (noncolloidal)	3.5 ft/s
Alluvial Silts (colloidal)	5.0 ft/s
Coarse Gravel (noncolloidal)	6.0 ft/s
Cobbles and Shingles	5.5 ft/s
Shales and Hard Pans	6.0 ft/s

[1]For highly erodible soils, decrease permissible velocities by 25%

*Source: Soil and Water Conservation Engineering, Schwab, et. al. and American Society of Civil Engineers

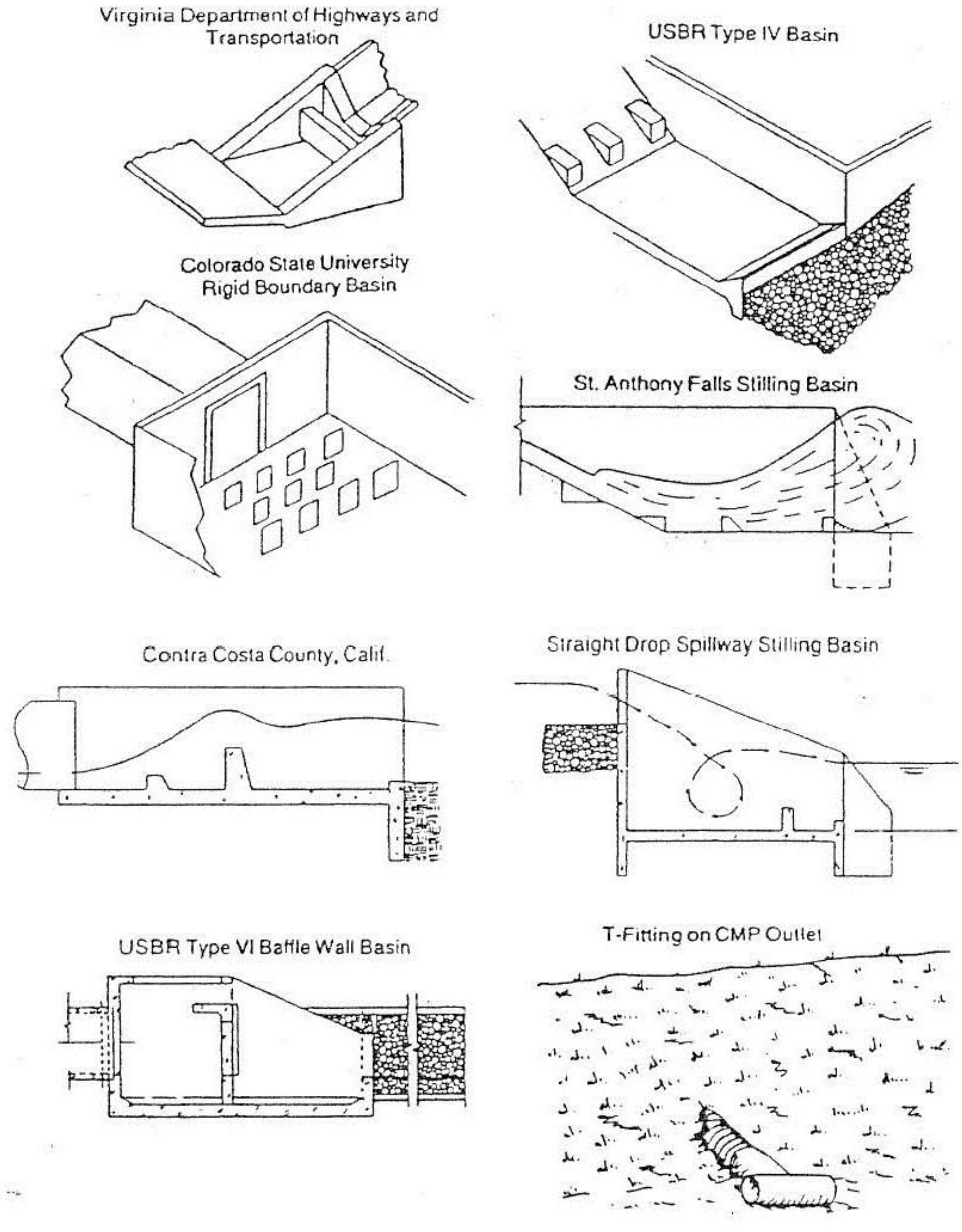


Figure 9-15: Structures Used For Outlet Protection by Dissipating Energy

Source: North Carolina Erosion and Sediment Control Manual, 1988.

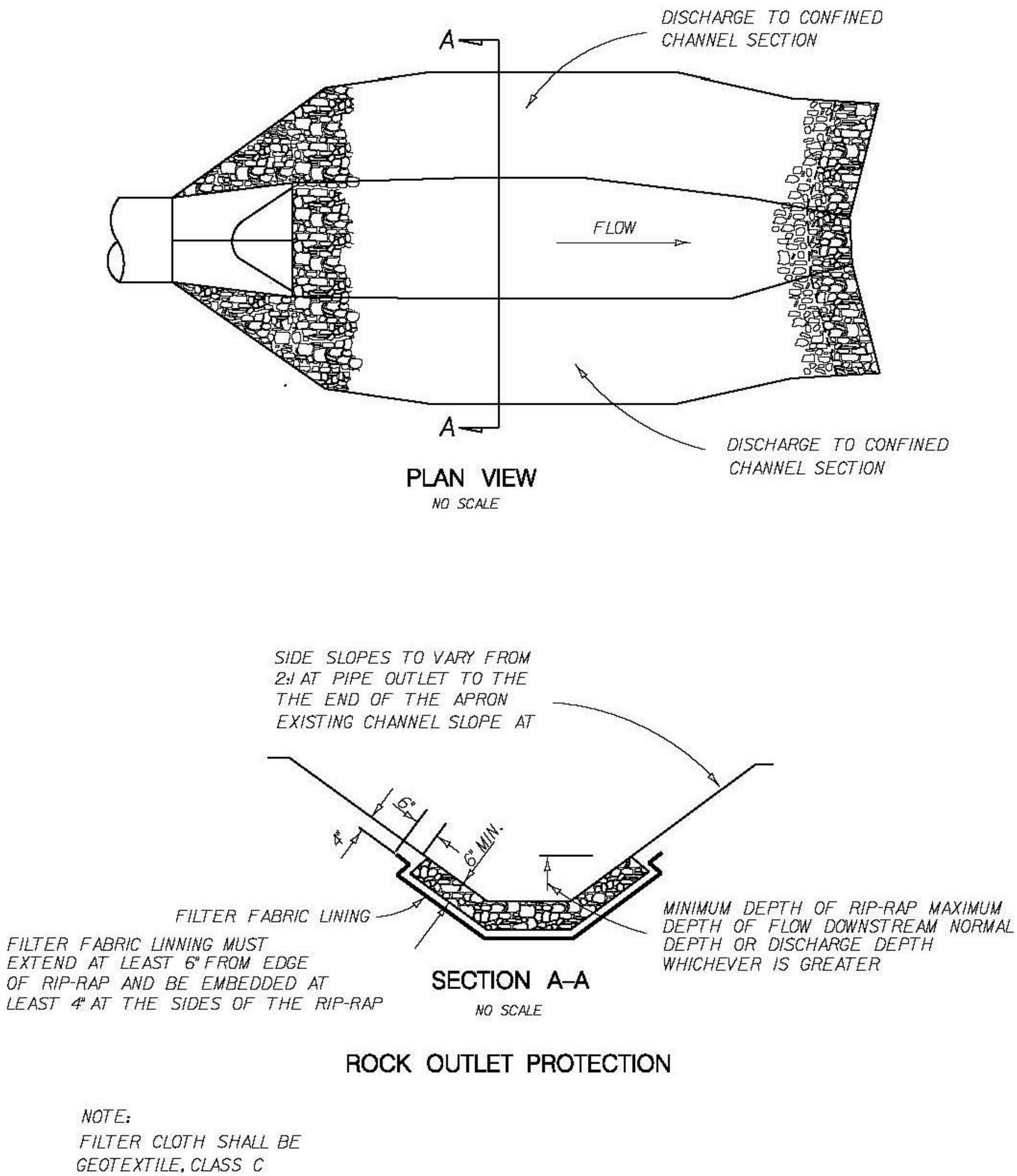
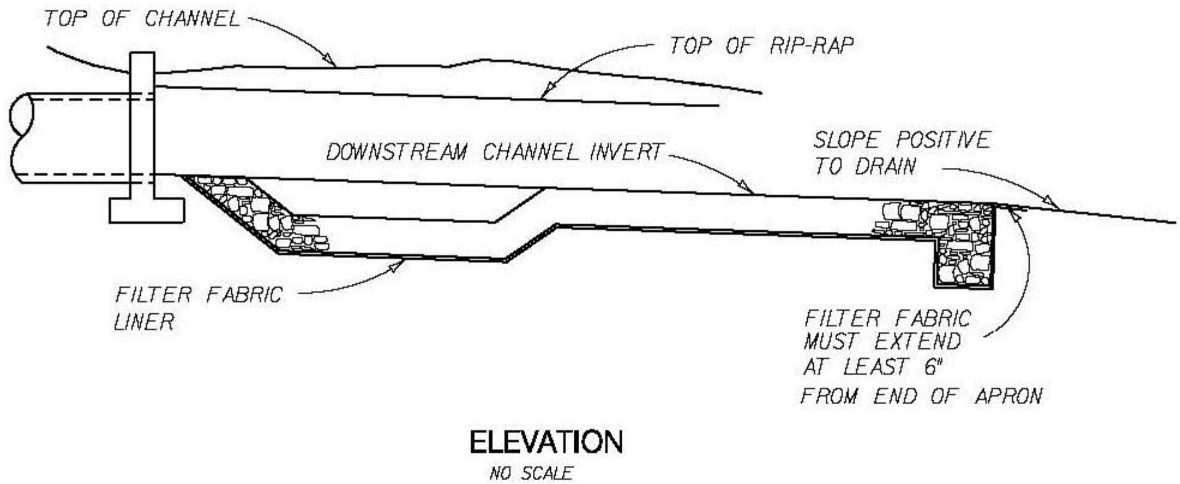


Figure 9-16: Outlet Protection



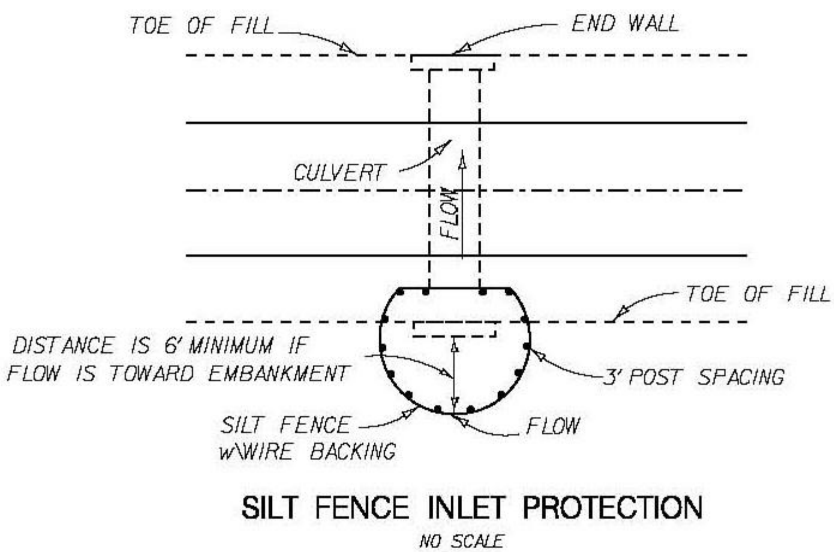


Figure 9-17: Rock Outlet and Culvert Inlet Protection

9.5.9 Scour Protection Mat

Scour Protection Mat combines vegetation with modern structural measures to mechanically protect the soil from scour and erosion until the shear forces have dissipated.

Design Detailing

- Prepare the area downstream of the outlet
- Create an area as wide and level as possible.
- Remove any saturated soils, hard clods, and replace with fertile and workable soils.
- Do not fill in low areas where it can be easily eroded. Compact the soil to enable a proper seed bed.
- Level the discharge area to minimize the potential of water concentrating in one area.
- Grade the soil to the final elevation and level to the floor elevation.
- Grade and level the discharge channel within the construction limits.
- Avoid impact and waterfall erosion.
- The bottom of the channel should be at least the width of the outlet. When using pipe it is recommended to have the channel width 5 times the diameter, with a minimum of 3 times the diameter.
- Install sod perpendicular to the flow. Install sod up the sides of the channel to half the height of the outlet and at least twice the distance of the transition mat.
- Use TRMs only in low flow, low volume outlets.
- If install TRM, sod must be mowed or trimmed to less that 2 inches.
- Install TRM perpendicular to the flow, instal in a stair step fashion.
- Center the mat layout with the discharge point and parallel to the flow direction.
- Anchor Installation:
  a. Install at least 5 anchors per mat, 3-2-3 configuration.
  b. Thread plastic cable through mat. Do not cut cable until the desired anchor depth has been achieved.
  c. Drive the anchors at least 12 inches or as deep as necessary to achieve effective anchoring.
  d. Cut the cable approximately 6 inches above mat
  e. Install the plastic washer and a one-way stop over the washer, pull the cable backward while pushing down on the stop. This will create a snug fit.

Maintenance

No formal maintenance is required once vegetation has been established.

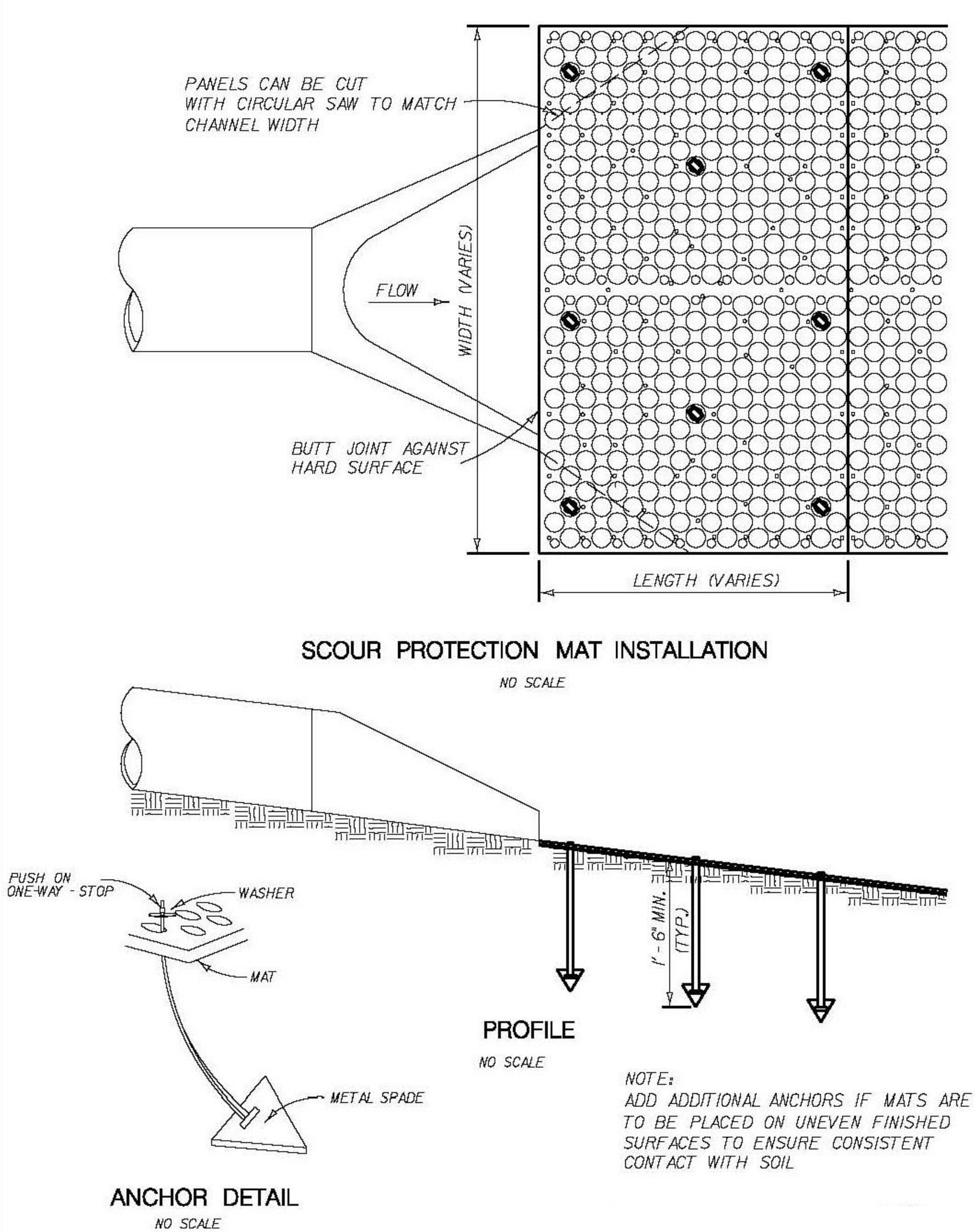


Figure 9-18: Scour Protection Mat

Source: Scourstop, Erosion Tech Inc.

9.5.10 Diversions

A diversion is a channel constructed across a slope with a supporting ridge on the lower side for the purpose of reducing the slope length and intercepting and diverting stormwater runoff to stabilized outlets at non-erosive velocities. Diversions are used where:

- runoff from higher areas may damage property, cause erosion, or interfere with the establishment of vegetation on lower areas;
- surface and/or shallow subsurface flow is damaging upland slopes; or
- slope length needs reduction to minimize soil loss.

Figure 9-19 and 9-20 illustrate the use of diversions.

Design Detailing

- In most instances, diversions are constructed using a standard design or sized for site flow conditions.

- Location - Diversion location should be determined by considering outlet conditions, topography, land use, soil type, length of slope, seepage planes (where seepage is a problem) and the development layout.

- Capacity - The diversion channel must have a minimum capacity to carry the runoff expected from a minimum of 2-year frequency storm with a freeboard of at least 0.3 foot. Diversions designed to protect homes, schools, industrial buildings, roads, parking lots and comparable high-risk areas and those designed to function in connection with other structures must have sufficient capacity to carry peak runoff expected from a storm frequency consistent with the hazard involved.

- Channel Design - The diversion channel may be parabolic, trapezoidal or V-shaped.

- Ridge Design - The supporting ridge cross-section must meet the following criteria:
    - The side slopes must be no steeper than (2H:1V).
    - The width at the design water elevation must be a minimum of 4 feet.
    - The minimum freeboard shall be 0.3 feet-

Construction Guidelines

- Outlet - Diversions must have adequate outlets which will convey concentrated runoff without erosion.

- Stabilization - Unless otherwise stabilized, the ridge and channel must be seeded and mulched within 15 days of installation. Disturbed areas draining into the diversion must be seeded and mulched prior to or at the same time the diversion is constructed.

- All trees, brush, stumps, obstructions and other objectionable material must be removed and disposed of so as not to interfere with the proper functioning of the diversion.

- The diversion shall be excavated or shaped to line, grade and cross-section as required to meet the criteria specified, and be free of irregularities which will impede flow.

- Fills shall be compacted as needed to prevent unequal settlement that would cause damage in the completed diversion.

- All earth removed and not needed in construction shall be spread or disposed of so that it will not interfere with the functioning of the diversion.

- Permanent stabilization of disturbed areas must be done in accordance with the applicable standards and specification.

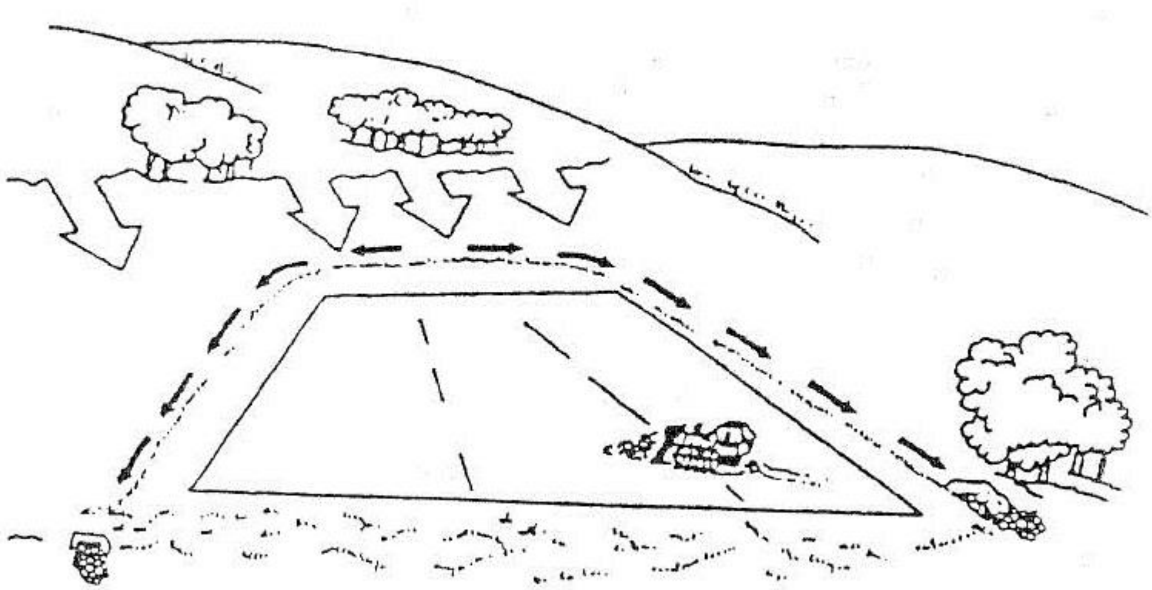


Figure 9-19: Use of Perimeter Dikes as Diversions

Source: North Carolina Erosion and Sediment Control Manual, 1988.

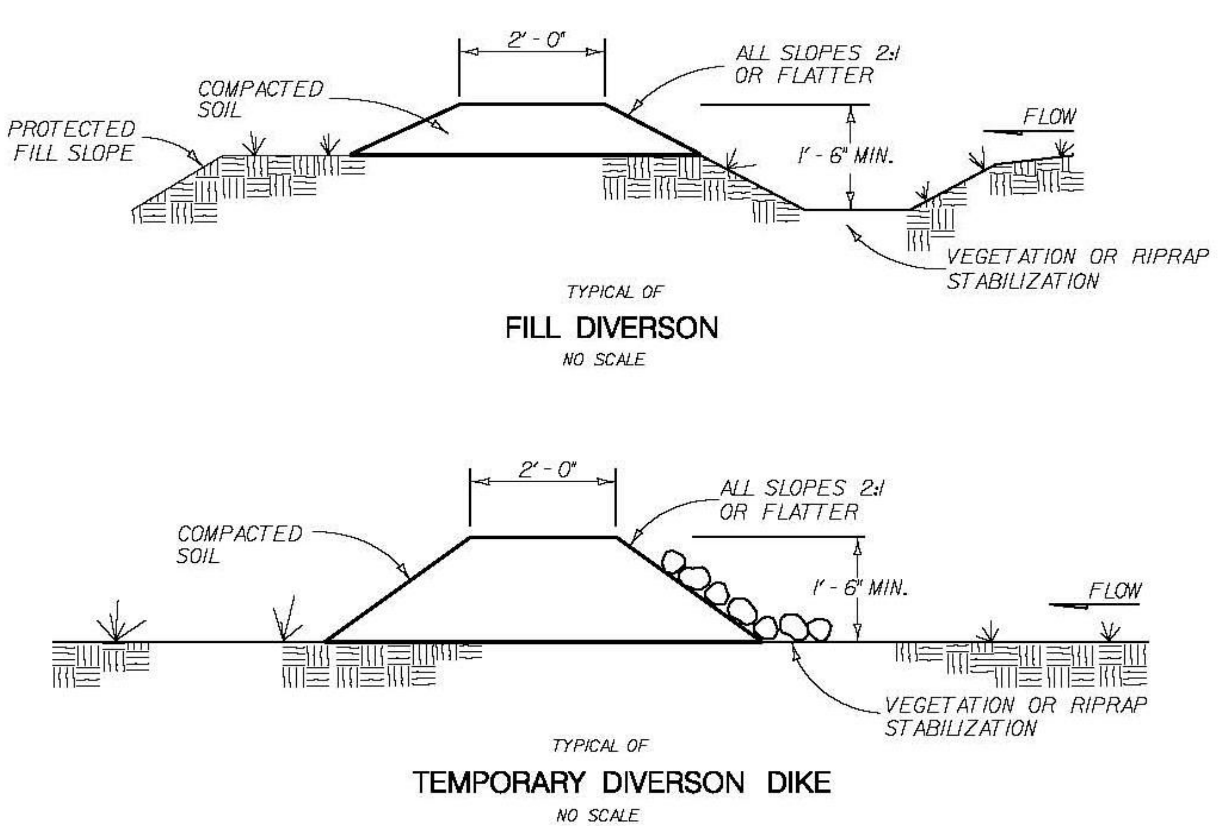


Figure 9-20: Temporary Diversion Dike

9.5.11 Check Dams

Check dams are small temporary dams constructed across a swale or drainage ditch for the purpose of reducing the velocity of concentrated stormwater flows, thereby reducing erosion of the swale or ditch. Check dams also trap small amounts of sediment generated in the ditch itself; however, these are not sediment trapping practices and should not be used as such. Figures 9-21 through 9-23 illustrates various examples of check dams.

Some specific applications include the following:
- Temporary ditches or swales which, because of their short length of service, cannot receive a non-erodible lining but still need some protection to reduce erosion.
- Permanent ditches or swales which for some reason cannot receive a permanent non-erodible lining for an extended period of time.
- Temporary or permanent ditches or swales which need protection during the establishment of grass linings.

<u>Use Limitations</u>

- Use limited to small open channels which drain 10 acres or less.
- Should not be used in an active stream.
- Should not to be used where high flows or high velocities are expected.
- In locating the check dam, consideration should be given to the effects and the reach of the impounded water and sediment.
- Storm flows across a deteriorated check dam can result in the loss of the structure and the washout of the accumulated sediment.

<u>Design Detailing</u>

The drainage area of the ditch or swale being protected should not exceed 10 acres. The maximum height of the check dam should be in accordance with Figure 9-21. The center of the check dam must be at least 6 inches lower than the outer edges. If used in combination, the maximum spacing between the dams should be such that the toe of the upstream dam is at the same elevation as the top of the downstream dam

<u>Construction Guidelines</u>

- Stone check dams should be constructed of 2- to 3-inch stone. Hand or mechanical placement will be necessary to achieve complete coverage of the ditch or swale and to insure that the center of the dam is lower than the edges.
- Log check dams may be constructed of 4- to 6-inch logs salvaged from clearing operations on site, if possible. The logs should be embedded into the soil at least 18 inches. The 6-inch lower height required at the center can be achieved either by careful placement of the logs or by cutting the logs after they are in place.
- Logs and/or brush should be placed on the downstream side of the dam to prevent scour during high flows.
- Sediment Removal - Although this practice is not intended to be used primarily for sediment trapping, some sediment will accumulate behind the check dams. Sediment should be removed from behind the check dams when it has accumulated to one half of the original height of the dam.
- Removal - Check dams should be removed when their useful life has been completed. In temporary ditches and swales, check dams should be removed and the ditch filled in when they are no longer needed. In permanent structures, check dams should be removed when a permanent lining can be installed. In the case of grass-lined ditches, check dams should be removed when the grass has matured sufficiently to protect the ditch or swale. The area beneath the check dams should be seeded and mulched immediately after they are removed.

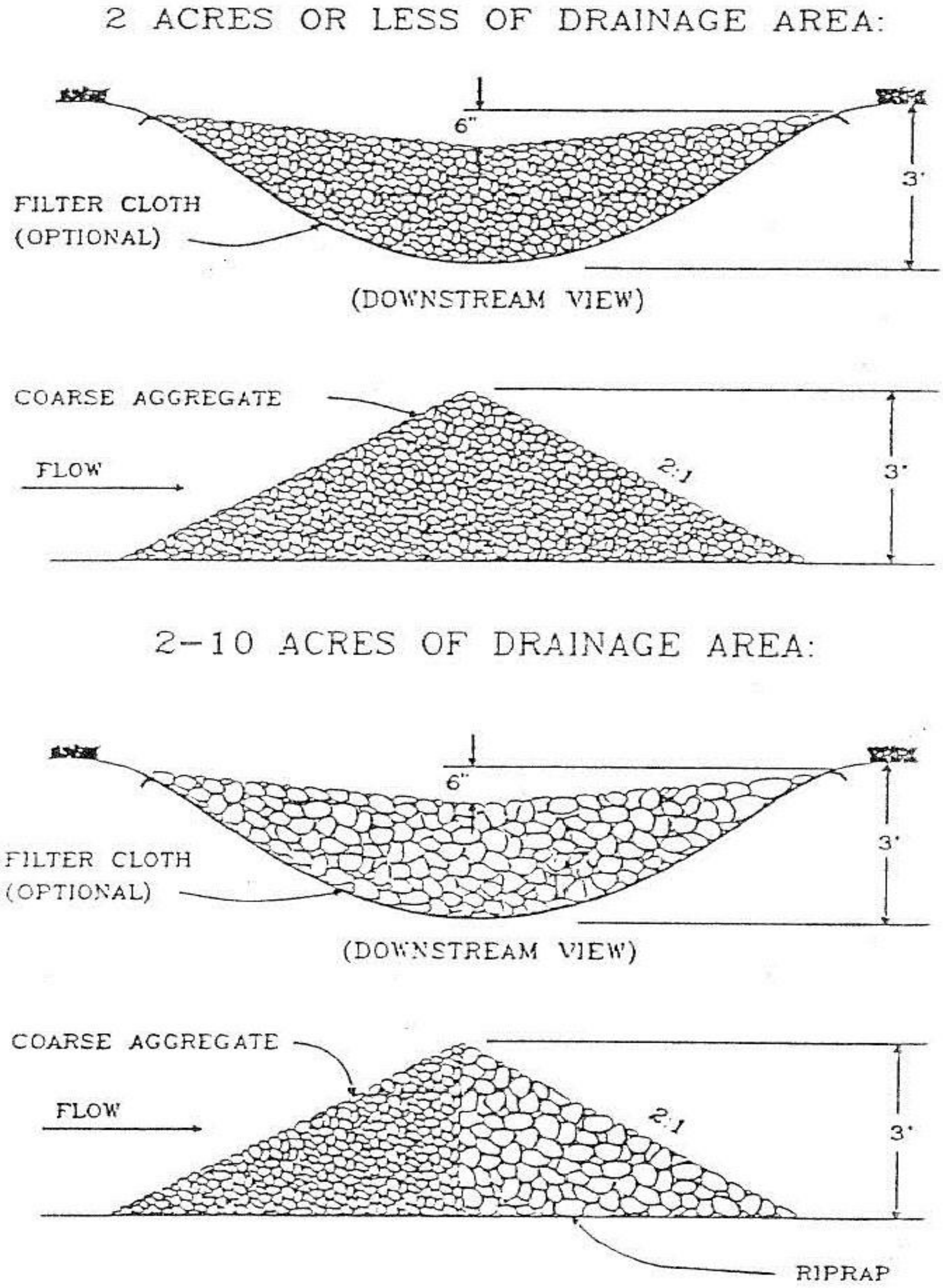


Figure 9-21: Rock Check Dams

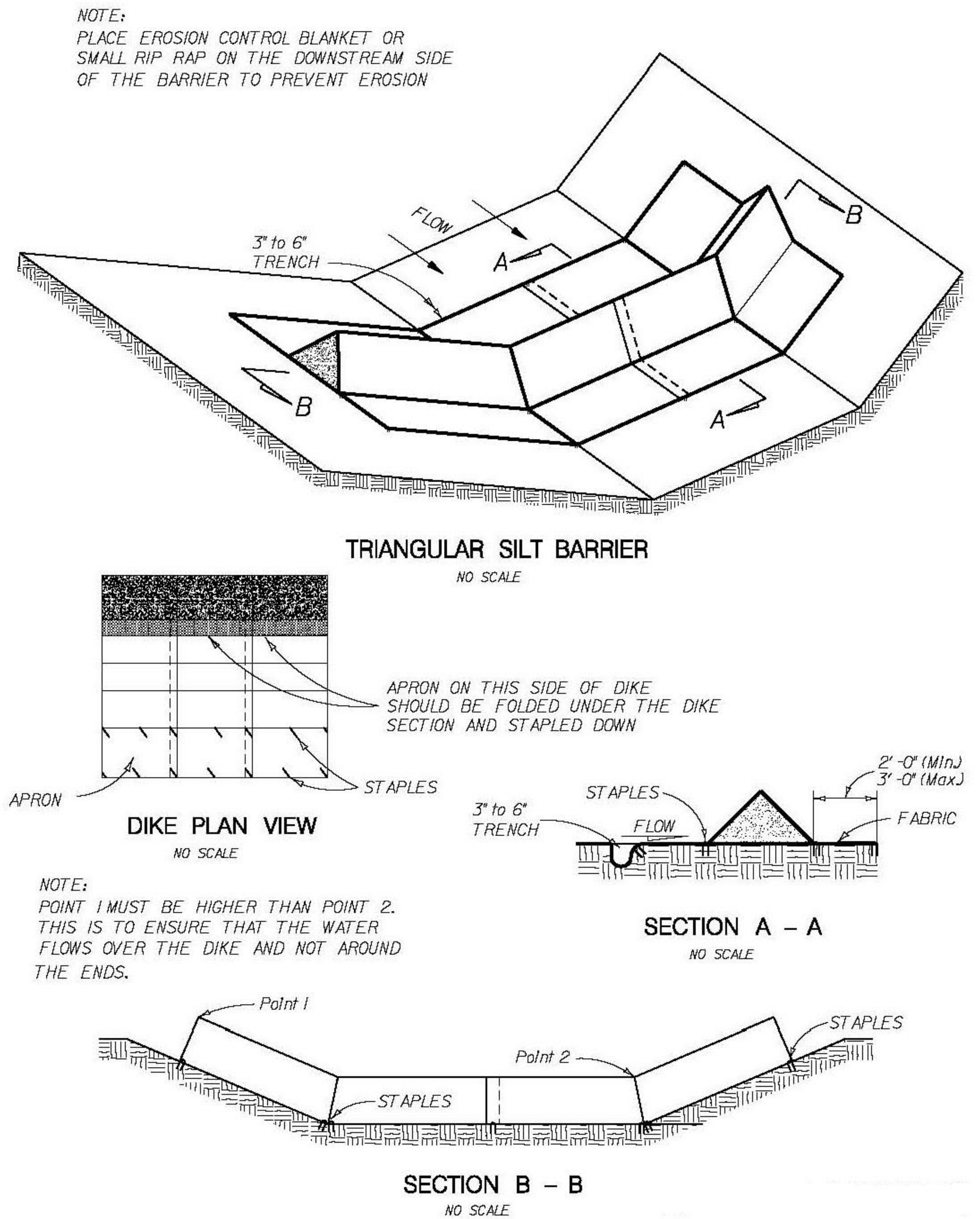


Figure 9-22: Triangular Sediment Barrier

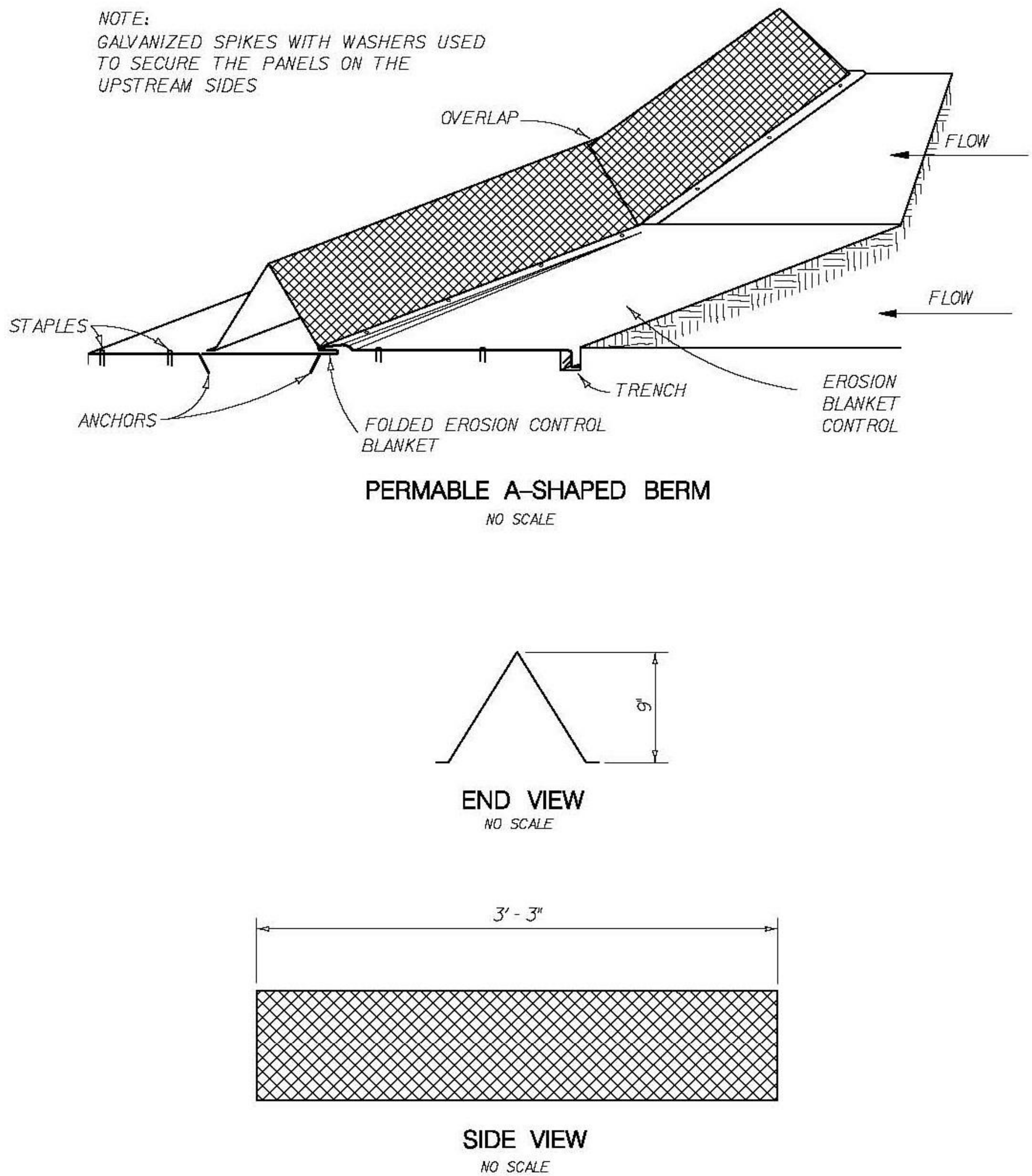


Figure 9-23: Permeable A-Shaped Berm

9.5.12 Construction Entrance

A construction entrance is a stabilized stone pad with a filter fabric underliner located at any point where vehicular traffic will be entering or leaving a construction site to or from a public right-of-way, street, alley, sidewalk or parking area. Its purpose is to reduce or eliminate the tracking of sediment onto public rights-of-way or streets. It should be used wherever traffic will be leaving a construction site and move directly onto a public road or other paved area. A construction entrance schematic is shown in Figure 9-24.

Design Detailing

Aggregate Size: Use 2 to 3 1/2-inch stone, or reclaimed or recycled concrete equivalent.
Thickness: Not less than 6 inches.
Entrance Dimension: 12 foot minimum width and must extend the full width of the vehicular ingress and egress area. 24 foot minimum width if there is only one access to the site. Length must be as required but not less than 70 feet.
All sediment must be prevented from entering storm drains, ditches, or watercourses.
Filter Cloth: To be placed on the entire area to be covered with aggregate. The filter cloth shall be woven or nonwoven fabric, inert to commonly encountered chemicals, hydro-carbons, mildew, rot-resistant, and conform as a minimum to the fabric properties shown in Table 9-3:

Table 9-3 Properties of Filter Cloth

Fabric Properties[1]	Light Duty Entrance[2]	Heavy Duty Entrance[3]	Test Method
Grab Tensile Strength (lbs.)	180	250	ASTM D4632
Elongation @ Failure (%)	50	60	ASTM D4632
Mullen Burst Strength (psi)	250	380	ASTM D3786
Puncture Strength (lbs.)	90	125	ASTM D4833
Apparent Opening Size (mm)	.20	.20	ASTM D4751
Aggregate Depth (in.)	6	10	ASTM D4751

[1] Fabrics not meeting these specifications may be used only when design procedure and supporting documentation are supplied to determine aggregate depth and fabric strength.
[2] Light Duty Entrance shall be defined as sites that have been graded to subgrade and where most travel would be single axle vehicles and an occasional multi-axle truck. Examples of fabrics which can be used are: Trevira Spunbond 1125, Synthetic Industries 701, Polyfelt TS650, or equivalent.
[3] Heavy Duty Entrance shall be defined as sites with only rough grading and where most travel would be multi-axle vehicles. Examples of fabrics which can be used are: Trevira Spunbond 1135, Synthetic Industries 1001, Polyfelt TS750 or equivalent.

Construction Guidelines

1. The area of the entrance must be excavated a minimum of 3 inches and must be cleared of all vegetation, roots, and other objectionable material. The filter fabric underliner will then be placed the full width and length of the entrance.

2. Following the installation of the filter cloth, the stone shall be placed to the specified dimensions. If wash racks are used, they must be installed according to the manufacturer’s specifications. Any drainage facilities required because of the washing shall be constructed according to specifications.

3. All surface water flowing or diverted towards construction entrances must be piped across the entrance. If piping is impractical, a mountable berm with 5:1 slopes will be permitted.

4. When washing is required, it must be done on a area stabilized with stone and which drains into an approved sediment trapping device.

<u>Maintenance</u>

The entrance must be maintained in a condition which will prevent tracking or flow of sediment onto public rights-of-way. This may require periodic top dressing with additional stone or the washing and reworking of existing stone as conditions demand and repair and/or cleanout of any structures used to trap sediment. All materials spilled, dropped, washed, or tracked from vehicles onto roadways or into storm drains must be removed within a reasonable period of time. If the City or the Lower Platte South Natural Resources District determines that the condition constitutes an immediate nuisance and hazard to public safety, the City shall issue a written notice abate and remove said hazard within twenty-four hours. The use of water trucks to remove materials dropped, washed, or tracked onto roadways will not be permitted under any circumstances.

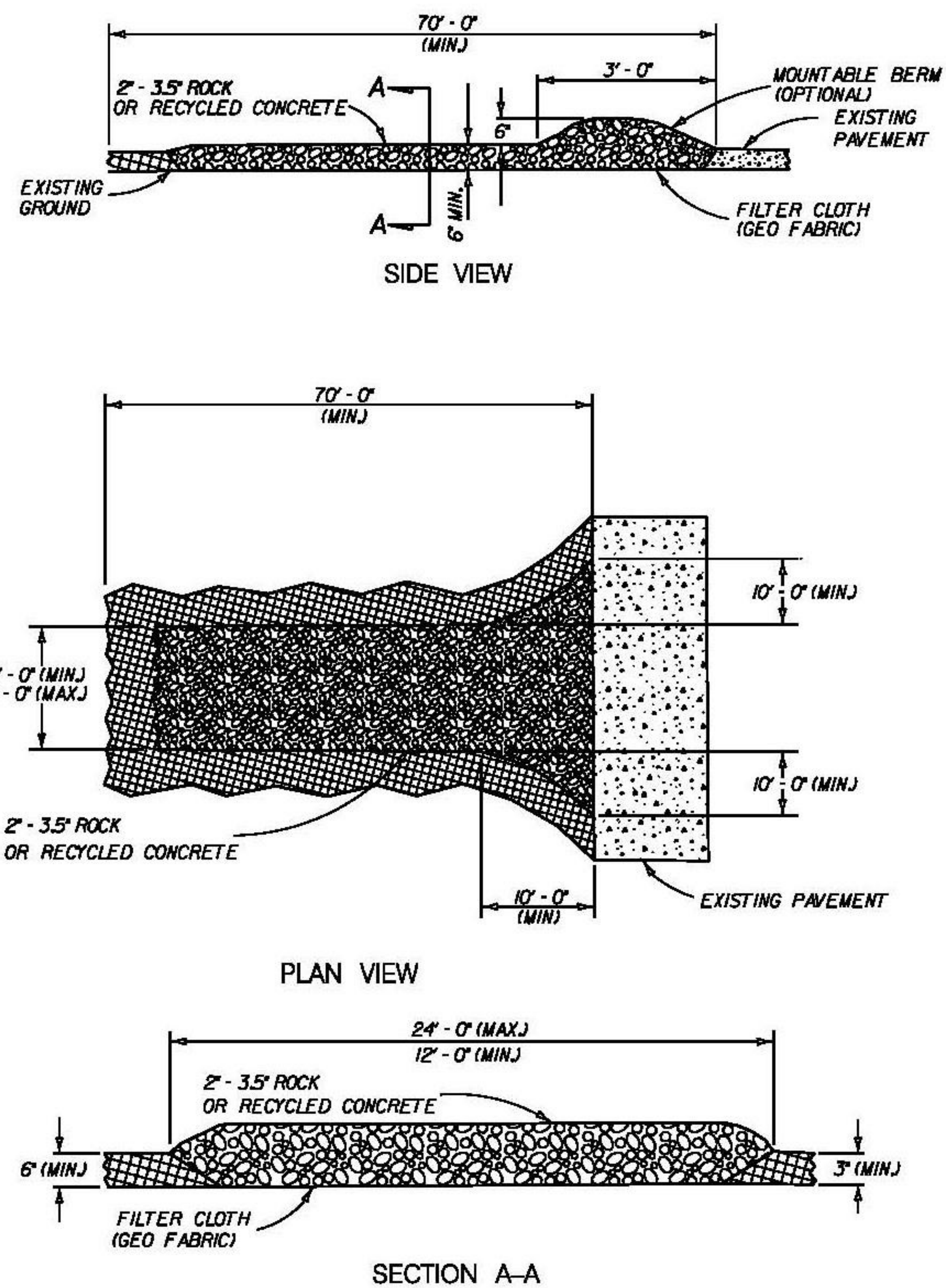


Figure 9-24: Stone Construction Entrance

9.5.13 Temporary Culvert Stream Crossing

A temporary vehicular stream crossing is a temporary structural span installed across a flowing watercourse for use by construction traffic. Structures may include bridges, round pipes, pipe arches, or oval pipes. Its purpose is to provide a means for construction traffic to cross flowing streams without damaging the channel or banks and to keep sediment generated by construction traffic out of the watercourse.

It is generally applicable to flowing streams with drainage areas less than 1 square mile. Structures which must handle flow from larger drainage areas should be designed by methods which more accurately define the actual hydrologic and hydraulic parameters which will affect the functioning of the structure. Temporary culvert crossings are presented in Figure 9-25.

Design Detailing

1. Where culverts are installed, 2" coarse aggregate or larger will be used to form the crossing. The depth of stone cover over the culvert shall be equal to one half the diameter of the culvert or 12 inches, whichever is greater. To protect the sides of the stone from erosion, riprap shall be used.
2. If the structure will remain in place for up to 14 days, the culvert must be large enough to convey the flow from a 2-year frequency storm without appreciably altering the stream flow characteristics. See Table 9-3 for aid in selecting an appropriate culvert size. If the structure will remain in place 14 days to one year, the culvert must be large enough to convey the flow from a 10-year frequency storm. In this case, the hydrologic calculation and subsequent culvert size must be done for the specific watershed characteristics. If the structure must remain in place over 1 year, it must be designed as a permanent measure by a qualified professional.
3. Multiple culverts may be used in place of one large culvert if they have the equivalent capacity of the larger one. The minimum sized culvert that may be used is 18 inches.
4. All culverts must be strong enough to support the maximum expected load.
5. The length of the culvert must be adequate to extend the full width of the crossing, including side slopes.
6. The slope of the culvert must be at least 0.25 inches per foot.

Construction Guidelines

1. Clearing and excavation of the stream bed and banks shall be kept to a minimum.
2. The invert elevation of the culvert must be installed on the natural streambed grade.
3. Filter cloth shall be placed on the streambed and streambanks prior to placement of the pipe culvert(s) and aggregate. The filter cloth shall cover the streambed and extend a minimum of six inches and a maximum of one foot beyond the end of the culvert and bedding material. Filter cloth reduces settlement and improves crossing stability.
4. The culvert(s) must extend a minimum of one foot beyond the upstream and downstream toe of the aggregate placed around the culvert. In no case shall the culvert exceed 40 feet in length.
5. The culvert(s) must be covered with a minimum of one foot of aggregate. If multiple culverts are used, they must be separated by at least 12 inches of compacted aggregate fill. At a minimum, the bedding and fill material used in the construction of the temporary access culvert crossings must be 2" coarse aggregate.
6. When the crossing has served its purpose, all structures including culverts, bedding and filter cloth materials will be removed. Removal of the structure and clean-up of the area shall be accomplished without construction equipment working in the waterway channel.
7. Upon removal of the structure, the stream must immediately be shaped to its original cross-section and properly stabilized.

Maintenance

Structures must be inspected after every rainfall and at least once a week, whether it has rained or not, and all damages repaired immediately.

Table 9-4 Pipe Diameter (inches) for Stream Crossing

Drainage Area (acres)	Average Slope of Watershed 1%	4%	8%	16%
1-25	24	24	30	30
26-50	24	30	36	36
51-100	30	36	42	48
101-150	30	42	48	48
151-200	36	42	48	54
301-350	42	48	60	60
351-400	42	54	60	60
451-500	42	54	60	72
501-550	48	60	60	72
551-600	48	60	60	72
601-640	48	60	72	72

Note: Table is based on USDA-SCS Graphical Peak Discharge Method for 2-year frequency storm event, CN = 65; Rainfall depth = 3.5 inches

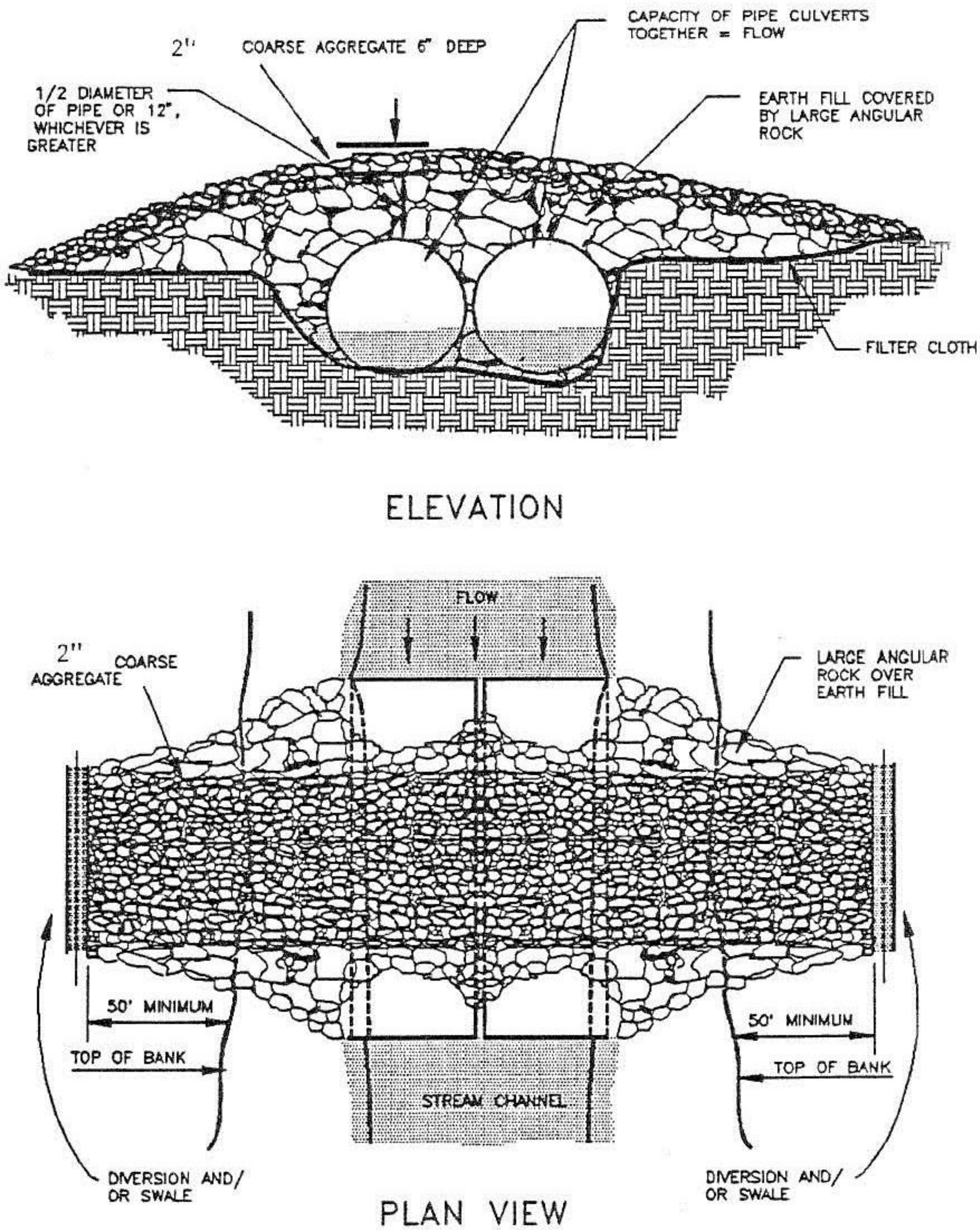


Figure 9-25: Temporary Culvert Crossing

9.5.14 Temporary Sediment Trap

A temporary sediment trap is a temporary ponding area formed by constructing an earthen embankment with a stone outlet. Its purpose is to detain sediment-laden runoff from small disturbed areas long enough to allow the majority of the sediment to settle out. It should be used below disturbed areas where the total contributing area is less than 3 acres and where the sediment trap will be used no longer than 18 months. Figure 9-26 shows a temporary sediment trap.

Design Details

- Sediment traps should be used only for small drainage areas. If the contributing drainage area is 3 acres or greater, use a temporary sediment basin.

- Sediment traps, along with other perimeter controls intended to trap sediment, must be constructed as a first step in any land-disturbing activity and must be made functional before upslope land disturbance takes place.

- The sediment trap must have an initial storage volume of 134 cubic yards per acre of drainage area, half of which shall be in the form of a permanent pool or wet storage to provide a stable settling medium. The remaining half shall be in the form of a drawdown or dry storage which will provide extended settling time during less frequent, larger storm events. The volume of the wet storage shall be measured from the low point of the excavated area to the base of the stone outlet structure. The volume of the dry storage shall be measured from the base of the stone outlet to the crest of the stone outlet. Sediment must be removed from the basin when the volume of the wet storage is reduced by one-half.

  a. For a sediment trap, the wet storage volume may be approximated as follows:
     $V1 = 0.85 \times A1 \times D1$ where
     V1 = The wet storage volume in cubic feet.
     A1 = The surface area of the flooded area at the base of the stone outlet in square feet.
     D1 = The maximum depth in feet, measured from the low point in the trap to the base of the stone outlet.
  b. For a sediment trap, the dry storage volume may be approximated as follows:
     $V2 = (A1 + A2) / 2 \times D2$ where
     V2 = The dry storage volume in cubic feet.
     A1 = The surface area of the flooded area at the base of the stone outlet in square feet.
     A2 = The surface area of the flooded area at the crest of the stone outlet in square feet.
     D2 = The depth in feet, measured from the base of the stone outlet to the crest of the stone outlet.

- The designer should seek to provide a storage area which has a minimum 2:1 length to width ratio (measured from point of maximum runoff introduction to outlet (See Table 9-5).

- Side slopes of excavated areas should be no steeper than 1:1. The maximum depth of excavation within the wet storage area should be 4 feet to facilitate clean-out and for site safety considerations.

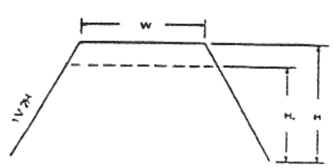
- The outlet for the sediment trap shall consist of a stone section of the embankment located at the low point in the basin. A combination of coarse aggregate and riprap shall be used to provide for filtering/detention, as well as outlet stability. The coarse aggregate shall be 3/4 - 1-1/2 inch clean stone (smaller stone sizes will enhance filter efficiency) and riprap shall be NDOR specifications Type A or B filter cloth. Riprap protection shall be placed at the stone-soil interface to act as a "separator". The minimum length of the outlet must be 6 feet times the number of acres comprising the total area draining to the trap. The crest of the stone outlet must be at least 1 foot below the top of the embankment to ensure that the flow will travel over the stone and not the embankment.
- The maximum height of the sediment trap embankment must be 5 feet as measured from the base of the stone outlet. Minimum top widths (W) and outlet heights (Ho) for various embankment heights (H) are shown on the accompanying diagram. Side slopes for the embankment must be 2:1 or flatter.
- Sediment traps must be removed after the contributing drainage area is stabilized. Plans should show how the site of the sediment trap is to be graded and stabilized after removal.

Construction Guidelines

- The area under the embankment shall be cleared, grubbed and stripped of any vegetation and root mat.
- Fill material for the embankment must be free of roots or other woody vegetation, organic material, large stones, and other objectionable material. The embankment should be compacted in 6 inch layers by traversing with construction equipment.
- The earthen embankment must be seeded with temporary or permanent vegetation immediately after installation.
- Construction operations must be carried out in such a manner that erosion and water pollution are minimized.
- The structure will be removed and the area stabilized when the upslope drainage area has been stabilized.
- All cut and fill slopes must be 2:1 or flatter (except for the excavated wet storage area which may be at a maximum 1:1 grade).

Maintenance

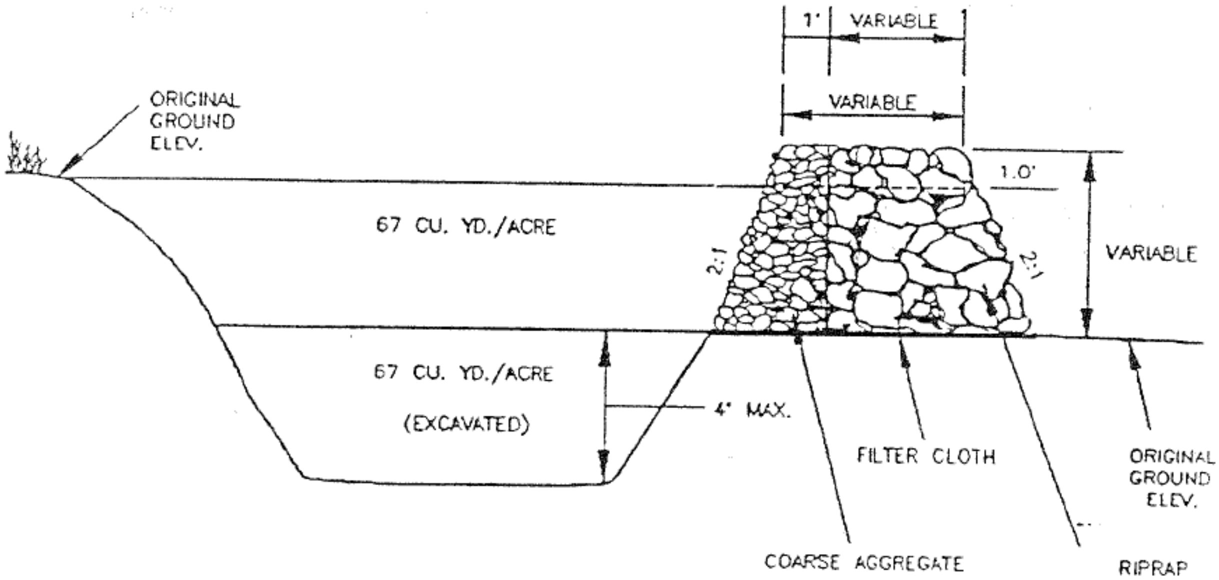
- Sediment must be removed and the trap restored to its original dimensions when the sediment has accumulated to one half the design volume of the wet storage. Sediment removal from the basin must be deposited in a suitable area and in such a manner that it will not erode and cause sedimentation problems.
- Filter stone shall be regularly checked to ensure that filtration performance is maintained. Stone choked with sediment must be removed and cleaned or replaced.
- The structure should be checked regularly to ensure that it is structurally sound and has not been damaged by erosion or construction equipment. The height of the stone outlet should be checked to ensure that its center is at least 1 foot below the top of the embankment.



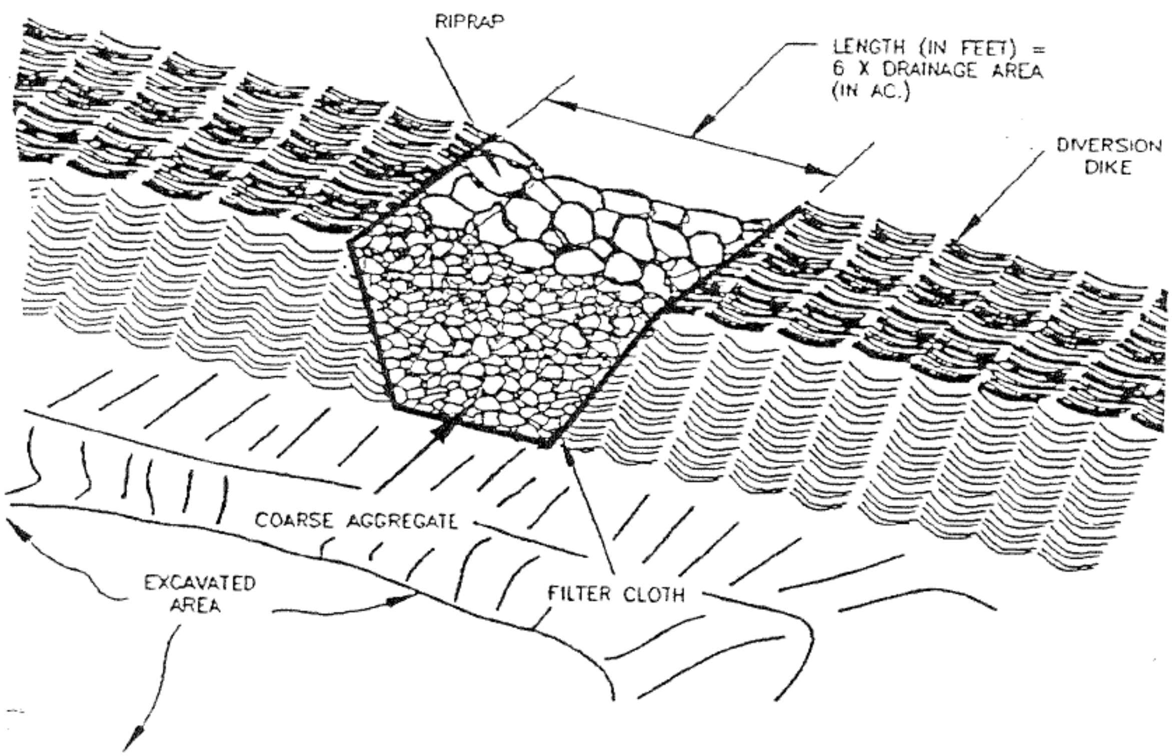
H	H_o	W
1.5	0.5	2.0
2.0	1.0	2.0
2.5	1.5	2.5
3.0	2.0	2.5
3.5	2.5	3.0
4.0	3.0	3.0
4.5	3.5	4.0
5.0	4.0	4.5

Table 9-5 Minimum Top Width (W) Required For Sediment Trap Embankments According To Height Of Embankment (Feet)

Source: Modified From Virginia Erosion & Sediment Control Handbook, 1980



CROSS SECTION OF OUTLET



OUTLET (PERSPECTIVE VIEW)

Figure 9-26: Temporary Sediment Trap

9.5.15 Temporary Sediment Basin

Temporary sediment basins are storage areas provided to detain sediment-laden runoff from disturbed areas long enough for the majority of the sediment to settle out. The facility is a temporary basin with a controlled stormwater release structure, formed by constructing an embankment of compacted soil across a drainageway.

Use Limitations

- Drainage Area and Topography - Temporary sediment basins can be used below disturbed areas generally greater than 5 acres. Sufficient space and appropriate topography for the construction of a temporary impoundment are necessary.

- Longevity - These structures are limited to a useful life of 18 months unless they are designed as permanent ponds by a qualified professional engineer.

- Effectiveness - Sediment basins are at best only 70-80% effective in trapping sediment which flows into them. Therefore, they should be used in conjunction with erosion control practices such as temporary seeding, mulching, diversion dikes, etc., to reduce the amount of sediment flowing into the basin.

- Location - To improve the effectiveness of the basin, it should be located to intercept the largest possible amount of runoff from the disturbed area. The best locations are generally low areas and natural drainageways below disturbed areas. Drainage into the basin can be improved by the use of diversion dikes and ditches. The basin must not be located in a live stream but should be located to trap sediment-laden runoff before it enters the stream. The basin should not be located where its failure would result in the loss of life or interruption of use of public utilities or roads.

- Multiple Use - Sediment basins may be designed as permanent structures to remain in place after construction is completed. Wherever these structures are to become permanent, or if they exceed the size limitations of the design criteria, they must be designed as permanent ponds by a qualified professional engineer.

Design Detailing

- Maximum Drainage Area - Unless the structure is designed as a permanent pond by a professional engineer, the maximum allowable drainage area into the basin shall be 150 acres.

- Maximum storage volume and embankment height - Refer to Department of Water Resources regulations.

- Basin Capacity - The design capacity of the basin must be at least 134 cubic yards per acre of drainage area, measured from the bottom of the basin to the crest of the principal spillway (riser pipe). Sediment should be removed from the basin when the volume of the basin has been reduced to 67 cubic yards per acre of drainage area. In no case shall the sediment cleanout level be higher than 1 foot below the top of the riser. The elevation of the sediment cleanout level should be calculated and clearly marked on the riser. A series of small basins has proven to be more effective in some instances than one large basin and may be better adaptable to a particular site.

- Basin Surface Area - Sediment trapping efficiency is primarily a function of sediment particle size and the ratio of basin surface area to inflow rate. Therefore, design the basin to have a large surface area for its volume.

- Design Life - Sediment basins with an expected life greater than 18 months must be designed as permanent structures. In these cases, the structure must be designed by a qualified professional engineer experienced in the design of dams.

- Basin Shape - To improve sediment trapping efficiency of the basin, the effective flow length must be twice the effective flow width. This basin shape may be attained by properly selecting the site of the basin, by excavation, or by the use of baffles.

- Embankment Cross-Section - The embankment must have a minimum top width of 8 feet. The side slopes must be 2H:1V or flatter. The embankment may have a maximum height of 10 feet if the side slopes are 2H:1V. If the side slopes are 2.5H:1V or flatter, the embankment may have a maximum height of 15 feet.

- Spillway design - The outlets for the basin may consist of a combination of principal and emergency spillways or a principal spillway alone. In either case, the outlet(s) must pass the peak runoff expected from the drainage area for a 10-year storm without damage to the embankment of the basin. Runoff computations must be based upon the soil cover conditions which are expected to prevail during the life of the basin. To increase the efficiency of the basin, the spillway(s) can be designed to maintain a permanent pool of water.

- Principal Spillway - The principal spillway must consist of a solid (non-perforated), vertical pipe or box of corrugated metal or reinforced concrete joined by a watertight connection to a horizontal pipe (barrel) extending through the embankment and outletting beyond the downstream toe of the fill. If the principal spillway is used in conjunction with an emergency spillway, the principal spillway must have a minimum capacity of 0.2 cfs per acre of drainage area when the water surface is at the crest of the emergency spillway. If no emergency spillway is used, the principal spillway must be designed to pass the entire peak flow expected from a 10-year storm.

  - Design Elevations - If the principal spillway is used in conjunction with an emergency spillway, the crest of the principal spillway must be a minimum of 1 foot below the crest of the emergency spillway. If no emergency spillway is used, the crest of the principal spillway must be a minimum of 3 feet below the top of the embankment. In either case, a minimum freeboard of 1 foot must be provided between the design high water and the top of the embankment.

  - Anti-Vortex Device and Trash Rack - A trash rack must be attached to the top of the principal spillway to prevent floating debris from being carried out of the basin. An anti-vortex device should be considered to improve flow into the spillway.

  - Dewatering - As a minimum, provisions must be made to dewater the basin down to the sediment cleanout elevation. This can be accomplished by providing dewatering in the spillway structure. Dewatering holes must be no larger than 4 inches in diameter. A stone filter will be required around the spillway structure to prevent loss of stored sediment.

  - Base - The base of the principal spillway must be firmly anchored to prevent its floating. If the riser of the spillway is greater than 10 feet in height, computations must be done to determine the anchoring requirements. As a minimum, a factor of safety of 1.25 must be used (downward forces = 1.25 × upward forces).

  - Barrel - The barrel of the principal spillway, which extends through the embankment, must be designed to carry the flow provided by the riser of the principal spillway with the water level at the crest of the emergency spillway. The connection between the riser and the barrel must be watertight. The outlet of the barrel must be protected to prevent erosion or scour of downstream areas.

  - Anti-Seep Collars - If the pond is not provided with means for releasing the stored runoff between inflow storms, anti-seep collars must be used on the barrel of the principal spillway within the normal saturation zone of the embankment to increase the seepage length by at least 10%, if either of the following two conditions is met:

    1. the settled height of the embankment exceeds 10 feet, or
    2. the embankment has a low silt-clay content (Unified Soil Classes SM or GM).

    Anti-seep collars must be installed within the saturated zone. The maximum spacing between collars must be 14 times the projection of the collar above the barrel. Collars must not be closer than 2 feet to a pipe joint. Collars should be placed sufficiently far apart to allow space for hauling and compacting equipment. Connections between the collars and the barrel must be watertight. Figure 9-27 illustrates anti-seep collars.

- Emergency Spillway - The emergency spillway must consist of an open channel constructed adjacent to the embankment over undisturbed material (not fill). Figure 9-28 illustrates emergency spillways.

    Capacity - The emergency spillway must be designed to carry the peak rate of runoff expected from a 10-year storm, less any reduction due to the flow through the principal spillway.

    Design Elevations - The design high water through the emergency spillway must be at least 1 foot below the top of the embankment. The crest of the emergency spillway channel must be at least 1 foot above the crest of the principal spillway.

    Location - The channel must be located so as to avoid sharp turns or bends. The channel must return the flow of water to a defined channel downstream from the embankment.

    Maximum Velocities - The maximum allowable velocity in the emergency spillway channel will depend upon the type of lining used. See Chapter 5 for allowable velocities.

Construction Guidelines

- Site Preparation - Areas under the embankment and any structural works shall be cleared, grubbed and stripped of topsoil to remove trees, vegetation, roots, or other objectionable material. In order to facilitate cleanout and restoration, the pool area (measured at the top of the principal spillway) will be cleared of all brush and trees.

- Cutoff Trench - When a cutoff trench is specified it shall be excavated along the centerline of the dam. The minimum depth shall be 2 feet. The cutoff trench must extend up both abutments to the riser crest elevation. The minimum bottom width shall be 4 feet, but wide enough to permit operation of compaction equipment. The side slopes must be no steeper than 1H:1V. Compaction requirements shall be the same as those for the roadway embankment. The trench shall be drained during the backfilling-compacting operations.

- Principal Spillway - The riser of the principal spillway must be securely attached to the barrel by a watertight connection. The barrel and riser must be placed on a firm compacted soil foundation. The base of the riser shall be firmly anchored according to design criteria to prevent its floating. Pervious materials such as sand, gravel or crushed stone must not be used as backfill around the barrel or anti-seep collars. Fill material must be placed around the pipe in 4-inch layers and compacted by hand at least to the same density as the embankment. A minimum of 2 feet of fill must be hand-compacted over the barrel before crossing it with construction equipment.

- Emergency Spillway - Design elevations, widths, entrance and exit channel slopes are critical to the successful operation of the spillway and should be adhered to closely during construction.

- Embankment - The fill material shall be taken from approved borrow areas. It must be clean mineral soil, free of roots, woody vegetation, oversized stones, rocks, or other objectionable material. Areas on which fill is to be placed must be scarified prior to the placement of fill. Fill material will be placed in 6- to 8-inch continuous layers over the entire length of the fill. Compaction shall be obtained by routing the hauling equipment over the fill so that the entire surface of the fill is traversed by at least one wheel or tread track of the equipment, or by using a compactor.

- Vegetative Stabilization - The embankment and emergency spillway of the sediment basin must be stabilized with temporary vegetation.

- Erosion and Sediment Control - The construction of the sediment basin shall be carried out in a manner such that it does not result in any undue sediment problems downstream.

- Safety - All state and local requirements must be met concerning fencing and signs warning the public of the hazards of soft sediment and flood waters.

Note: For a detailed discussion of design procedures and specifications for temporary sediment basins, see LPSNRD, 1994.

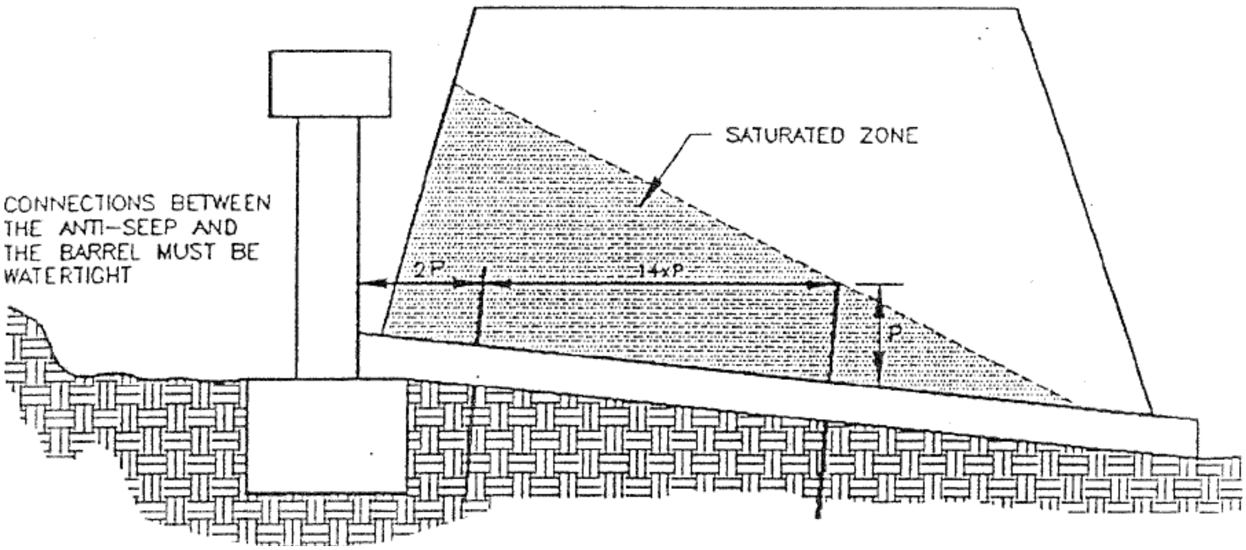


Figure 9-27: Anti-Seep Collars

Source: LPSNRD, 1994.

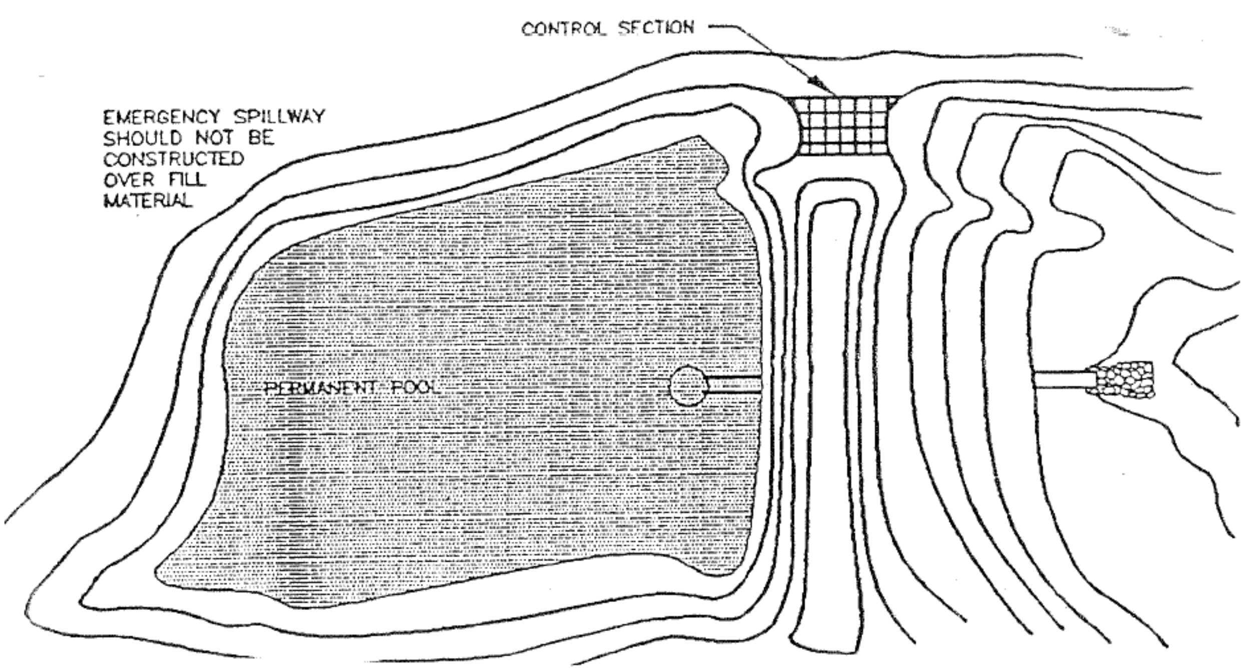


Figure 9-28: Emergency Spillway

Source: LPSNRD, 1994.

9.5.16 Temporary Seeding

Temporary seeding is the establishment of a temporary vegetative cover on disturbed areas by seeding with appropriate rapidly growing annual or perennial plants. Its purpose is to reduce erosion and sedimentation by stabilizing disturbed areas that will not be brought to final grade for a period of thirty days or more, reduce damage from sediment and runoff to downstream or off-site areas, and to provide protection to bare soils exposed during construction until permanent vegetation or other erosion control measures can be established.

It should be used on exposed soil surfaces. Such areas include denuded areas, soil stockpiles, dikes, dams, sides of sediment basins, temporary roadbanks, etc. A permanent vegetative cover must be applied to areas that will be left dormant for a period of more than 1 year.

Construction Guidelines

1. Prior to seeding, install all necessary erosion control practices such as dikes, waterways, and basins.
2. Provide proper shaping of the area to be seeded in a manner such that seedbed preparation and seeding operations can be carried out.
3. Seedbed Preparation:
   a. If the area has been recently loosened or disturbed, no further roughening is required. When the area is compacted, crusted or hardened, the soil surface shall be loosened by discing, raking, harrowing, or other acceptable means. Seedbed preparation should not be undertaken when excessively wet conditions exist. Seedbed shall be prepared to a depth of approximately 3 inches.
   b. If the soil being seeded is fertile topsoil, fertilizer is not required. However, if subsoil is to be seeded, it will most likely be deficient in nutrients required for seed germination and growth. 450 lbs./acre of 10-20-20 fertilizer should be used, and it is essential that this fertilizer be incorporated into the top 2-4 inches of soil during seedbed preparation. Soils which are highly acidic should be limed.
4. Seeding:
   a. Certified seed must be used on all temporary seedings. Select plants appropriate to the season and site conditions from those listed in Table 9-6:

Table 9-6 Guidelines for Temporary Seeding

Time of Year	Species	Seeding Rate
March 15 - May 15	Spring Oats	2 bu./AC.
	Barley	2 bu./AC.
	Perennial Ryegrass	30-40 lbs./AC.
	Orchard Grass	20-25 lbs./AC.
May 16 - July 15	Grain Sorghum (drilled)	10-20 lbs./AC.
	Forage Sorghum (drilled)	10-20 lbs./AC.
	Hybrid Sundangrass	20-30 lbs./AC.
July 16 - October 15	Spring Oats	2 bu./AC.
	Barley	2 bu./AC.
August 16 - October 15	Winter Wheat	1.5 bu./AC.
	Winter Rye	1.5 bu./AC.
October 15 - March 15	No planting, use mulches	

   b. Seed should be evenly applied with a drill, cultipacker seeder, or hydroseeder. For cyclone spreaders, the rate should be twice that of drill seeding. Small grains shall be planted no more than 1-1/2 inches deep and grasses no more than ½" deep.
5. When seedings are made on critical sites or adverse soil conditions, mulch material will be applied immediately after seeding. Seedings made during optimum seeding dates and with favorable soils on very flat areas may not need to be mulched.

Maintenance

Areas which fail to establish vegetative cover adequate to prevent rill erosion will be re-seeded as soon as such areas are identified. Control weeds by mowing.

9.5.17 Permanent Seeding

Permanent vegetation is the establishment of perennial vegetative cover on disturbed areas by planting seed. Its purpose is to reduce erosion and sediment yield from disturbed areas, to permanently stabilize disturbed areas in a manner that is economical, adaptable to site conditions, and allows selection of the most appropriate plant materials, to improve wildlife habitat and to enhance natural beauty. It may be used on disturbed areas where permanent, long-lived vegetative cover is needed to stabilize the soil and rough-graded areas which will not be brought to final grade for a year or more.

Construction Guidelines

1. Prior to seeding, install all necessary erosion control practices such as dikes, waterways, and basins.

2. Provide proper shaping of the area to be seeded in a manner such that seedbed preparation and seeding operations can be carried out.

3. Soil conditions needed for the establishment and maintenance of permanent seeding must be as follows:
   a. Enough fine-grained material to maintain adequate moisture and nutrient supply.
   b. Sufficient pore space to permit root penetration. A bulk density of 1.2 to 1.5 indicates that sufficient pore space is present. A fine granular or crumb-like structure is also favorable.
   c. Sufficient depth of soil to provide an adequate root zone. The depth to rock or impermeable layers such as hardpans shall be 12 inches or more, except on slopes steeper that 2:1 where the addition of soil is not feasible.
   d. A favorable pH range for plant growth. If the soil is so acidic that a pH range of 6.0-7.0 cannot be attained by addition of pH-modifying materials, then the soil is considered an unsuitable environment for plant roots and further soil modification would be required.
   e. Freedom from toxic amounts of materials harmful to plant growth.
   f. Freedom from excessive quantities of roots, branches, large stones, large clods of earth, or trash of any kind. Clods and stones may be left on the slopes steeper than 3:1 if they do not significantly impede good seed soil contact.

   If any of the above criteria cannot be met, then topsoil must be applied.

4. Seedbed Preparation:
   a. Flat areas and slopes up to 3:1 grade shall be loose and friable to a depth of at least 3 inches. The top layer of soil shall be loosened by raking, discing or other acceptable means before seeding.
   b. Slopes steeper than 3:1 must have the top 1-3 inches of soil loose and friable before seeding.
   c. When the area is compacted, crusted or hardened, the soil surface shall be loosened by discing, raking, harrowing, or other acceptable means. Seedbed preparation should not be undertaken when excessively wet conditions exist.
   d. Soil amendments must be applied according to the recommendations of a soil test.

5. Seeding:
   a. Mixtures for permanent plantings will contain a mixture of two or more species. A single species may be used on some residential or recreational areas.
   b. Certified “Blue Tagged” seed will be used on all permanent seedings. Permanent seedings shall have a minimum of 90 PLS/s.f.
   c. Seed should be evenly applied in two directions with a cyclone spreader, drill, cultipacker seeder, or hydroseeder on a firm, moist seedbed. Maximum seeding depth shall be 1/4" on clayey soils and ½" on sandy soils, when using other than hydroseeder method of application.

d. If hydroseeding is used and the seed and fertilizer is mixed, they will be mixed on-site and the seeding shall be immediate without interruption.
e. Cool-season dominant mixtures shall be applied August 15 - May 30. Warm-season dominant mixtures shall be applied October 1 - June 15.
f. A protective cover crop of annual plants may be seeded for erosion protection until establish of the permanent vegetation. Cover crop planting may be done in conjunction with permanent seeding or immediately after permanent seeding has taken place. Select cover crop plants appropriate to the season and site conditions from those listed in Table 9-7.
g. All permanent seedings must be mulched immediately upon completion of seed application.

Table 9-7 Guidelines for Selecting Cover Crop Plants

Time of Year	Species	Seeding Rate
March 15 - May 15	Spring Oats	2 bu./AC.
May 16 - July 15	Grain Sorghum (drilled)	10-20 lbs./AC.
	Forage Sorghum (drilled)	10-20 lbs./AC.
	Hybrid Sundangrass	20-30 lbs./AC.
July 16 - October 15	Spring Oats	2 bu./AC.
	Winter Wheat	1.5 bu./AC.
	Rye	1.5 bu./AC.
October 15 - March 15	No planting, use mulches	

Maintenance

1. In general, a stand of vegetation cannot be determined to be fully established until it has been maintained for one full year after planting.
2. New seedings shall be supplied with adequate moisture. Supply water as needed, especially late in the Spring season. Water applications shall be controlled to prevent excessive runoff.
3. Inspect all seeded areas for failures and make necessary repairs, replacements, and re-seedings within the planting season, if possible.
   a. If stand is inadequate for erosion control, overseed and fertilize using half of the rates originally specified. Fertilizer should be a low/no-phosphorus mix.
   b. If stand is 60% damaged, re-establish following seedbed and seeding recommendations.
   c. If stand has less than 40% cover, re-evaluate for use of soil amenities.

9.5.18 Hydroseeding/Hydro-Mulching

Hydro- mulching/Hydro- seeding is a grass planting process. The process begins by mixing mulch, seed, tackifier, fertilizer, and water into a tank of a hydro-mulching machine. The material is often called a slurry. Once applied to the soil, the material enhances initial growth. Figure 9-29 shows an installation detail for hydroseeding/hydromulching

Design Detailing

- Prior to hydro-mulching and hydro-seeding, install any needed erosion and sediment control practices.
- Complete required shaping of area in such a manner that the operations can be carried out.
- Seed must be incorporated as needed except where seed is to be applied as part of the process.

Maintenance

All mulches and soil covering should be inspected periodically and after each rain event to check for erosion. Where erosion is observed in mulched areas, additional mulch should be applied.

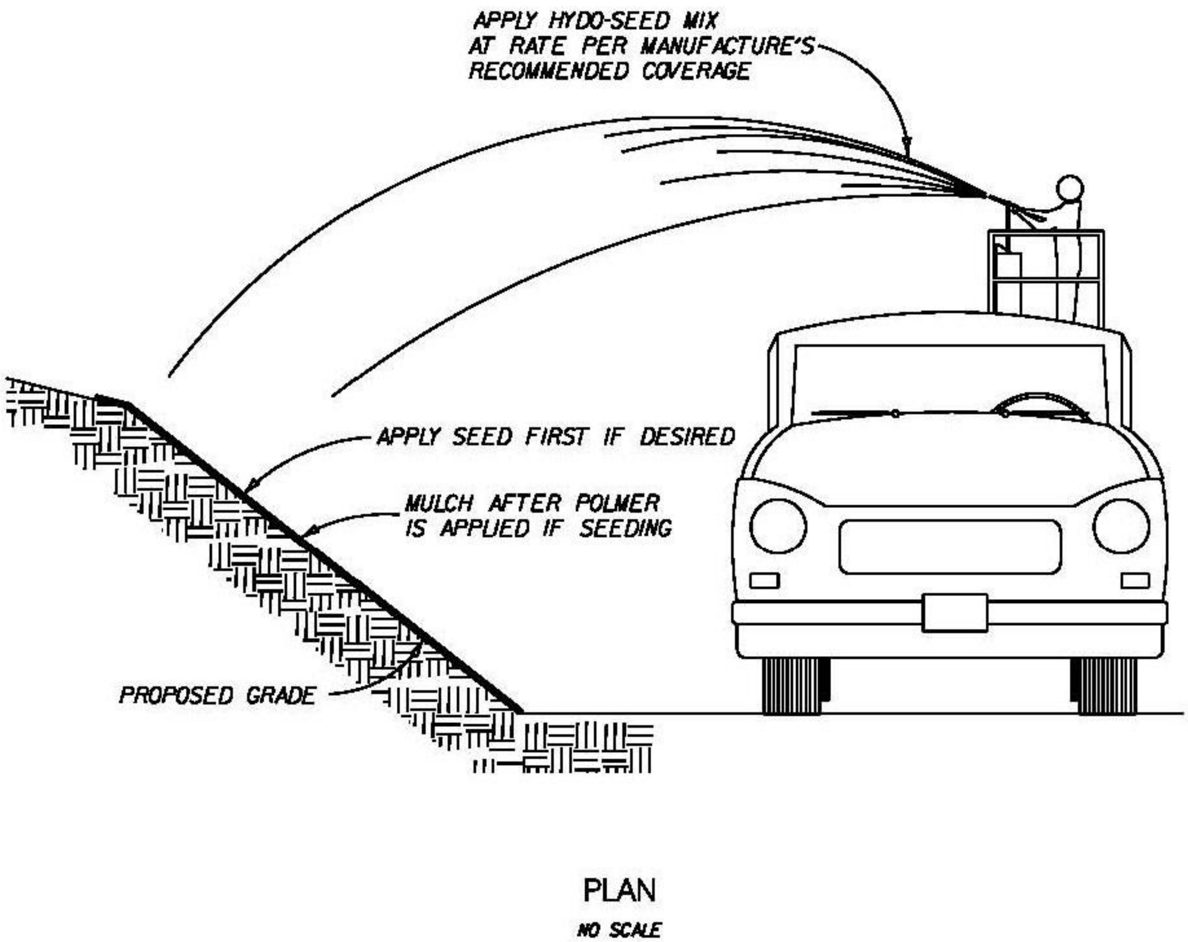


Figure 9-29: Hydroseed / Hydromulch / Polymer Application

9.5.19 Slope Tracking

Slope tracking is the technique used for surface roughening or scarification by means of mechanical equipment. Slope Tracking creates grooves that are perpendicular to the slope. The primary functions for Slope Tracking are to reduce erosion potential by decreasing runoff velocities, trap sediment, increase the chances for water infiltration, and aid in the establishment of vegetative cover.

Figure 9-30 shows an installation detail for slope tracking.

Design Detailing

When Slope Tracking is used for a surface roughing technique, is shall be done by operating tracked machinery up and down the slope to leave horizontal depressions in the soil. As few of passes of the machinery should be made to minimize compaction. If the slope steepness is greater than 3:1, Slope Tracking is not recommended. Immediately seed and mulch roughened areas to obtain optimum seed germination and growth.

Maintenance

Areas need to be inspected after each storm event, particularly after events that are greater than .05 inches. Regular inspections of slopes will indicate where additional erosion and sediment controls are needed. If rills appear they should be filled, graded again, and re-seeded immediately.

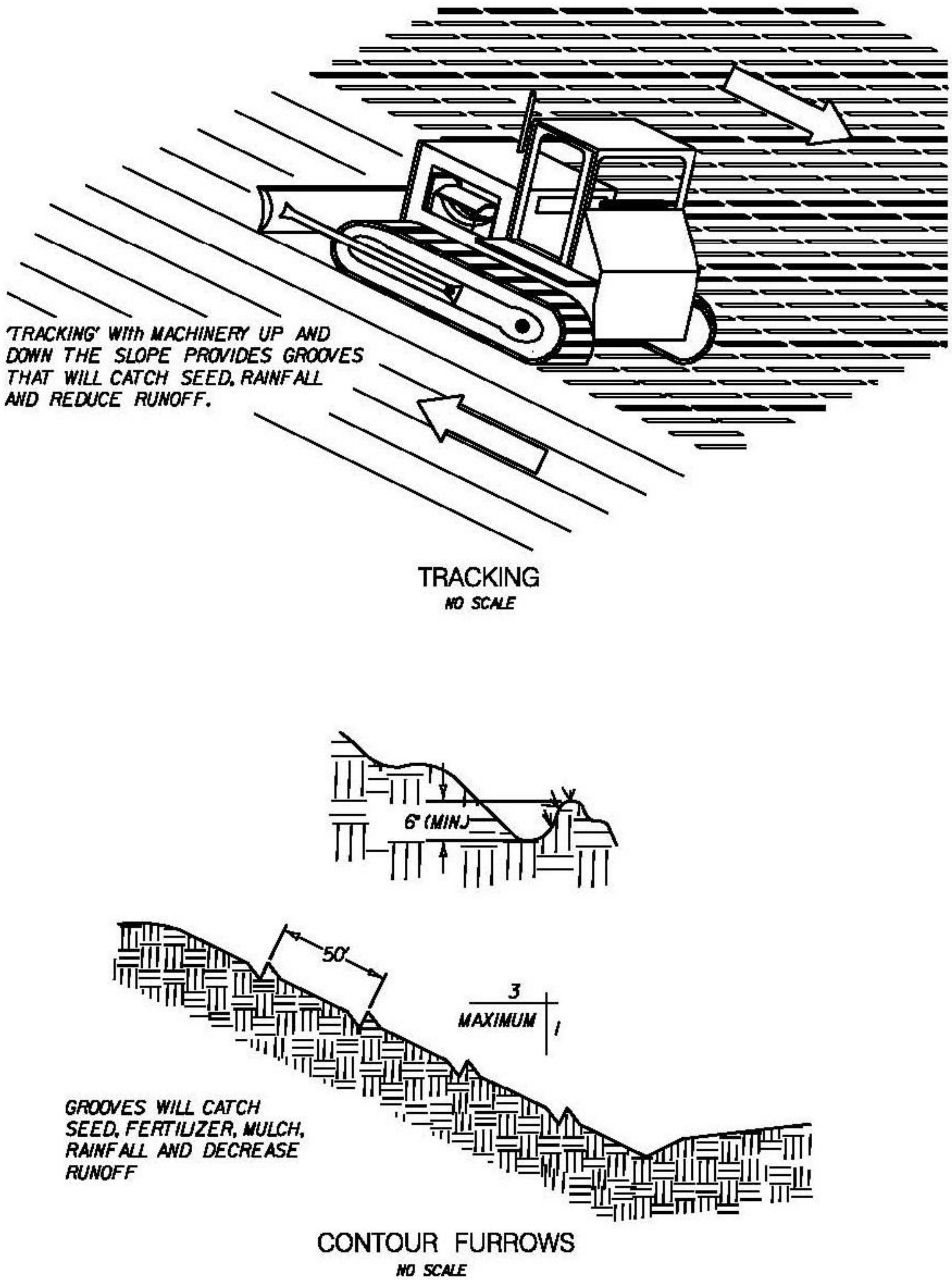


Figure 9-30: Slope Tracking / Contour Furrows

9.5.20 Mulching

Mulching is the application of plant residues or other suitable materials to the soil surface. Its purpose is to prevent erosion by protecting the soil surface from raindrop impact and reducing the velocity of overland flow. Mulch helps foster the growth of vegetation by increasing available moisture and providing insulation against extreme heat and cold. Mulching can be used at anytime where protection of the soil surface is desired. Mulch can be used in conjunction with seedings to establish vegetation, or by itself to provide temporary protection of the soil surface.

Construction Guidelines

1. Site Preparation:
   a. Prior to mulching, install any needed erosion and sediment control practices such as diversions, grade stabilization structures, berms, dikes, grassed waterways and sediment basins.
   b. Complete required shaping of area in a manner such that mulching operations can be carried out.
   c. Soil amendments shall be incorporated and surface roughening accomplished as needed. Seed must be applied prior to mulching except where seed is to be applied as part of a hydroseeder slurry containing fiber mulch or where seed is to be applied following an organic mulch spread during winter months.

2. Materials:
   a. Organic materials may be used in any area where mulch is required. Select mulch material based on site requirements, availability of materials, and availability of labor and equipment (see Table 9-7).
   b. Mulch materials must be spread uniformly by hand or machine. When spreading straw mulch by hand, divide the area to be mulched into approximately 1,000 s.f. sections and place 70-90 lbs. (two bales) of straw in each section to facilitate uniform distribution.

3. Anchoring Mulch:

   Mulch must be anchored immediately to minimize loss by wind and water. This may be done by one of the following methods (listed by preference) depending upon the size of area, erosion hazard and cost.

   a. Mulch Anchoring Tool and Tracking - A mulch anchoring tool is a tractor drawn implement designed to punch and anchor mulch into the top two inches of soil. This practice offers maximum erosion control but is limited to flatter slopes where equipment can operate safely. "Tracking" is the process of cutting mulch into the soil using a bulldozer or other equipment that runs on cleated tracks. Tracking is used primarily on slopes 3:1 or steeper. This practice should be done on the contour whenever possible, except tracking which should be done up and down the slope with cleat marks running across the slope.
   b. Mulch Nettings - Staple lightweight biodegradable paper, plastic or cotton netting over the mulch according to manufacturer's recommendations.
   c. Liquid Mulch Binders - Application of liquid mulch binders and tackifiers should be heavier at edges, in valleys, and at crests of banks and other areas where the mulch has a greater potential to be moved by wind or water. All other areas should have a uniform application of binder. Binders may be applied after the mulch is spread or may be sprayed into the mulch as it is being blown onto the soil. The use of synthetic binders is the preferred method of mulch binding. Apply at rates recommended by the manufacturer.
   d. Wood Cellulose Fiber - The fiber binder shall be applied at a net dry weight of 750 lbs./AC. The wood cellulose fiber shall be mixed with water, and the mixture must contain a maximum of 50 lbs. Of wood cellulose fiber per 100 gallons of water.
   e. Peg and Twine - Drive 8 to 10 inch wooden pegs to within 2 to 3 inches of the soil surface every 4 feet in all directions. Stakes may be driven before or after applying mulch. Secure mulch to the soil surface by stretching twine between pegs in a criss-cross within a square pattern. Secure twine around each peg.

Maintenance

All mulches and soil coverings should be inspected periodically and after each rainstorm to check for erosion. Where erosion is observed in mulched areas, additional mulch should be applied. Nets and mats should be inspected after rainstorms for dislocation or failure. If washouts or breakage occur, re-install netting or matting as necessary after repairing damage to the slope or ditch. Inspections should take place until grasses are firmly established. Where mulch is used in conjunction with ornamental plantings, inspect periodically throughout the year to determine if mulch is maintaining coverage of the soil surface; repair as needed.

Table 9-8 Organic Mulch Materials and Application Rates

	Rates		
Mulches	Per Acre	Per 1000 sq. ft.	Notes
Straw or Hay	1 ½ - 2 tons (Minimum 2 tons for winter cover)	70 - 90 lbs.	Free from weeds and coarse matter. Must be anchored. Spread with mulch blower or by hand.
Fiber Mulch	Minimum 1500 lbs.	35 lbs.	Do not use as mulch for winter cover or during hot, dry periods. * Apply as slurry.
Corn Stalks	4 - 6 tons	185 - 275 lbs.	Cut or shredded in 4-6" lengths. Air-dried. Do not use if fine turf areas. Apply with mulch blower or by hand.
Wood Chips	4 - 6 tons	185 - 275 lbs.	Free of coarse matter. Air-dried. Treat with 12 lbs nitrogen per ton. Do not use in fine turf areas. Apply with mulch blower, chip handler, or by hand.
Bark Chips or Shredded Bark	50 - 70 cu. yds.	1 - 2 cu. yds.	Free of coarse matter. Air-dried. Do not use in fine turf areas. Apply with mulch blower, chip handler, or by hand.

* When fiber mulch is only available mulch during periods when straw should be used, apply at a minimum rate of 2000 lbs./ac. or 45 lbs./1000 sq.ft.

9.5.21 Rolled Erosion Control Products

Rolled erosion control products are protective covering netting, blankets or turf reinforcement mats (TRMs) installed on a prepared planing area of a steep slope, channel, or shoreline. They aid in controlling erosion on critical areas by absorbing the energy from raindrop impacts and providing a microclimate which protects young vegetation and promotes its establishment. TRMs are also used to raise the maximum permissible velocity and shear stress of turf grass stands in channelized areas by enabling the turf to resist the forces of erosion during storm events.

Design Details

Netting, blankets, and TRMs will aid in controlling erosion on slopes steeper than 8 percent and of highly erodible soils by providing a protective cover made of straw, jute, wood or other organic plant fiber with cotton string or polypropylene netting to hold the product in a flat form. Netting can be used alone over blown straw as an alternative to crimping or use of a tackifier.

These products can be used on short, steep slopes where erosion hazard is high and planting is likely to be too slow in providing adequate protective cover; in vegetated channels where the design velocity and shear stress of design flow exceed allowable on streambanks where moving water is likely to wash out new plantings; or in areas where the forces of wind prevent standard mulching practices from remaining in place until vegetation becomes established.

Rolled erosion control products provide protection from raindrop impact and offer additional soil stabilization on prepared planting areas. TRMs also raise the maximum permissible velocity and shear stress of turfgrass stands in channelized areas by reinforcing the vegetation to resist the forces of erosion during storm events.

Before installation of these products, the area should be final graded to a smooth and uniform surface, free of debris. Topsoil should be incorporated if needed. Seed and fertilize as shown on the plan. The erosion control netting, blankets, and mats should be installed in accordance with the manufacturer's recommendations and specifications. All products should be anchored following the manufacturer's recommended stapling pattern for each specific application.

Some important factors in the choice of netting, blanket, or TRM are soil conditions, steepness of slope, length of slope, type and duration of protection required to establish desired vegetation, and probable sheer stress. Consult the manufacturer's product specifications to determine the correct product for each specific application required.

Construction Guidelines

Rolled erosion control blankets and mats can be applied to problem areas to supplement vegetation in its initial establishment and to provide a safe and more natural conveyance for high velocity stormwater runoff. They are used in many applications where a structural lining would previously have been required. Care must be taken to choose the blanket or matting which is most appropriate for the specific needs of a project. Two general types of blankets and mats are discussed within this section. However, with the abundance of soil stabilization products available today, it is impossible to cover all the advantages, disadvantages, and specifications of all manufactured blankets and mats. Therefore, there is no substitute for a thorough understanding of the manufacturer's recommendations and a site visit by a designer or plan reviewer to verify a product's appropriateness.

Blankets should be used to help establish vegetation on previously disturbed slopes of 3H:1V or steeper. Since the materials which compose the soil stabilization blankets will deteriorate over time, they should be used in permanent conveyance channels with the realization that resistance to erosion will ultimately be based on the type of vegetation planted and the existing soil characteristics. During the establishment of vegetation, blankets should not be subjected to velocities greater than 4 feet per second.

Blankets provide the following benefits in vegetative stabilization when properly applied:

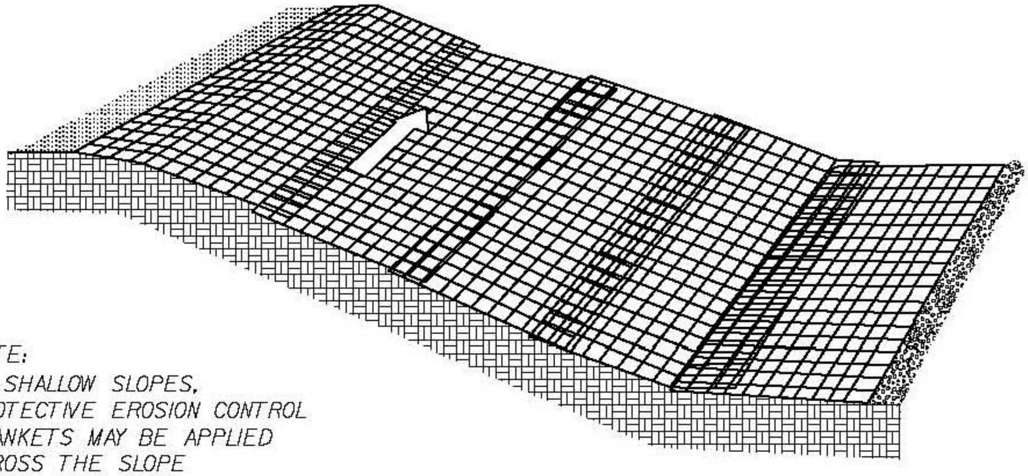
1. Protection of the seed and soil from raindrop impact and subsequent displacement.
2. Thermal consistency and moisture retention for seedbed area.
3. Stronger and faster germination of grasses and legumes.
4. Planing off excess stormwater runoff.
5. Prevention of sloughing of topsoil added to steeper slopes.

TRMs consists of a non-degradable, three-dimensional polypropylene structure which may also have coconut or other organic fiber layers within it so long as the non-degradable portion of the blanket will withstand design velocities and shear stresses after the organic fibers degrade. The matting becomes entangled and penetrated by roots forming continuous anchorage for surface growth and promoting enhanced energy dissipation. They should be used on slopes 2H:1V or steeper, and in stormwater conveyance channels.

In addition to those benefits noted for blankets, TRMs provide the following benefits for vegetative stabilization and when replacing concrete and riprap channel linings:

1. Cause sediment to drop out of stormwater and fill matrix with fine soils which become the growth medium for the development of roots.
2. Act with the vegetative root system to form an erosion resistant cover, which resists hydraulic lift and shear forces when embedded in the soil within stormwater channels.

Since TRMs are non-degradable, they can be used in permanent conveyance channels to withstand higher velocities and shear stresses than would normally be allowable with only soil and vegetation. Permissible velocities and shear stresses for TRM for reinforced grass-lined channels range from 10 - 20 fps and 6 - 10 psf respectively.



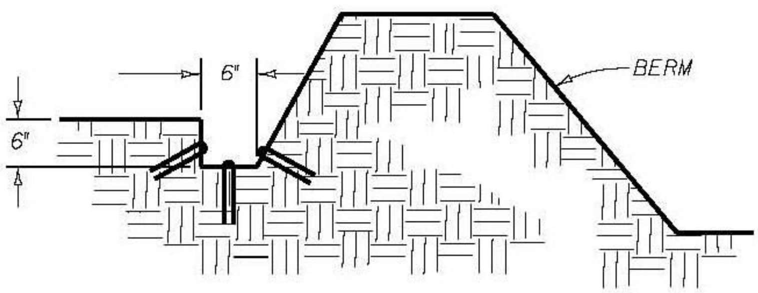


Figure 9-31a: Rolled Erosion Control Products - Shallow Slope Application

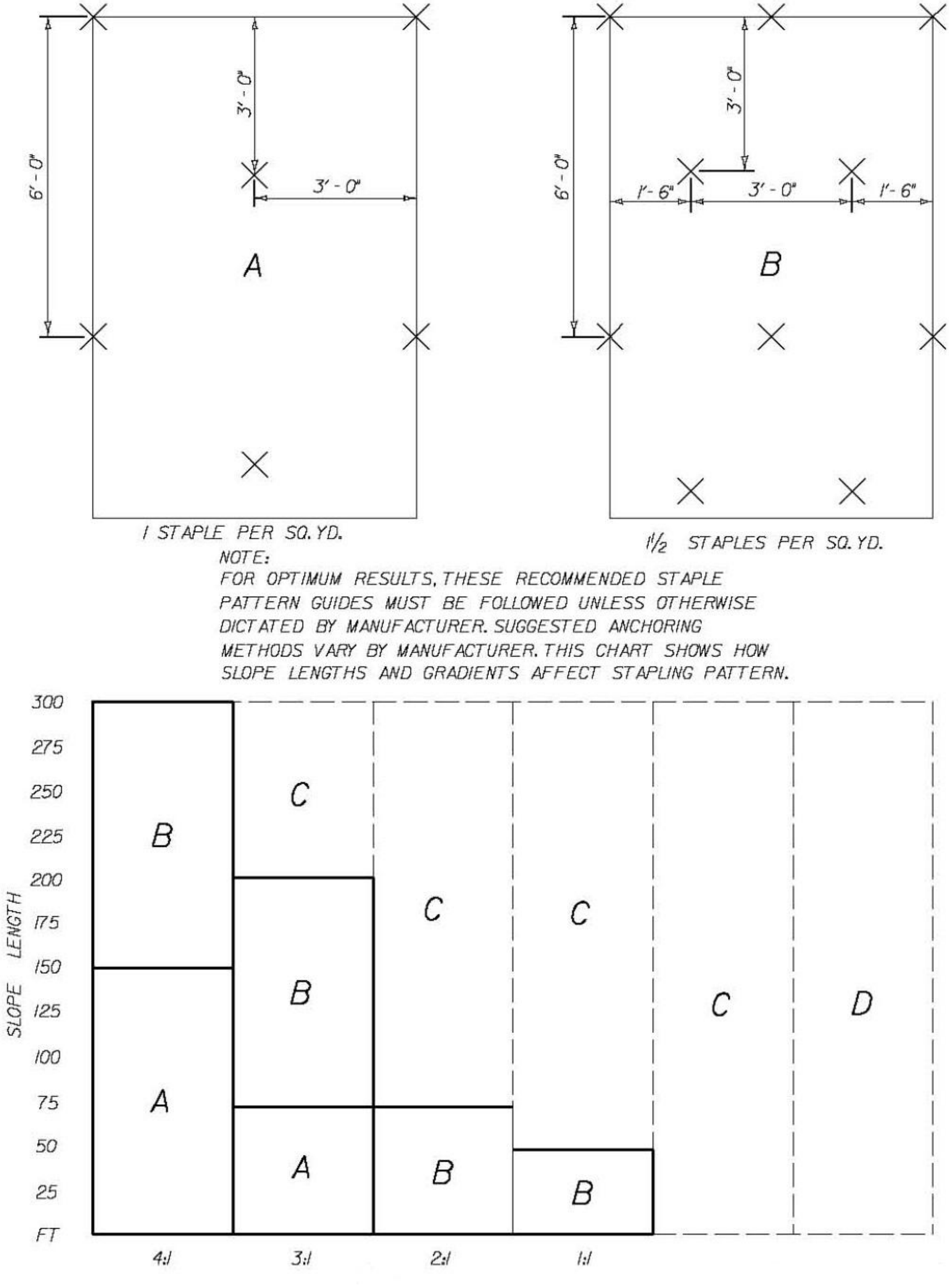


Figure 9-31b: Rolled Erosion Control Products - Steep Slope / Ditch Application

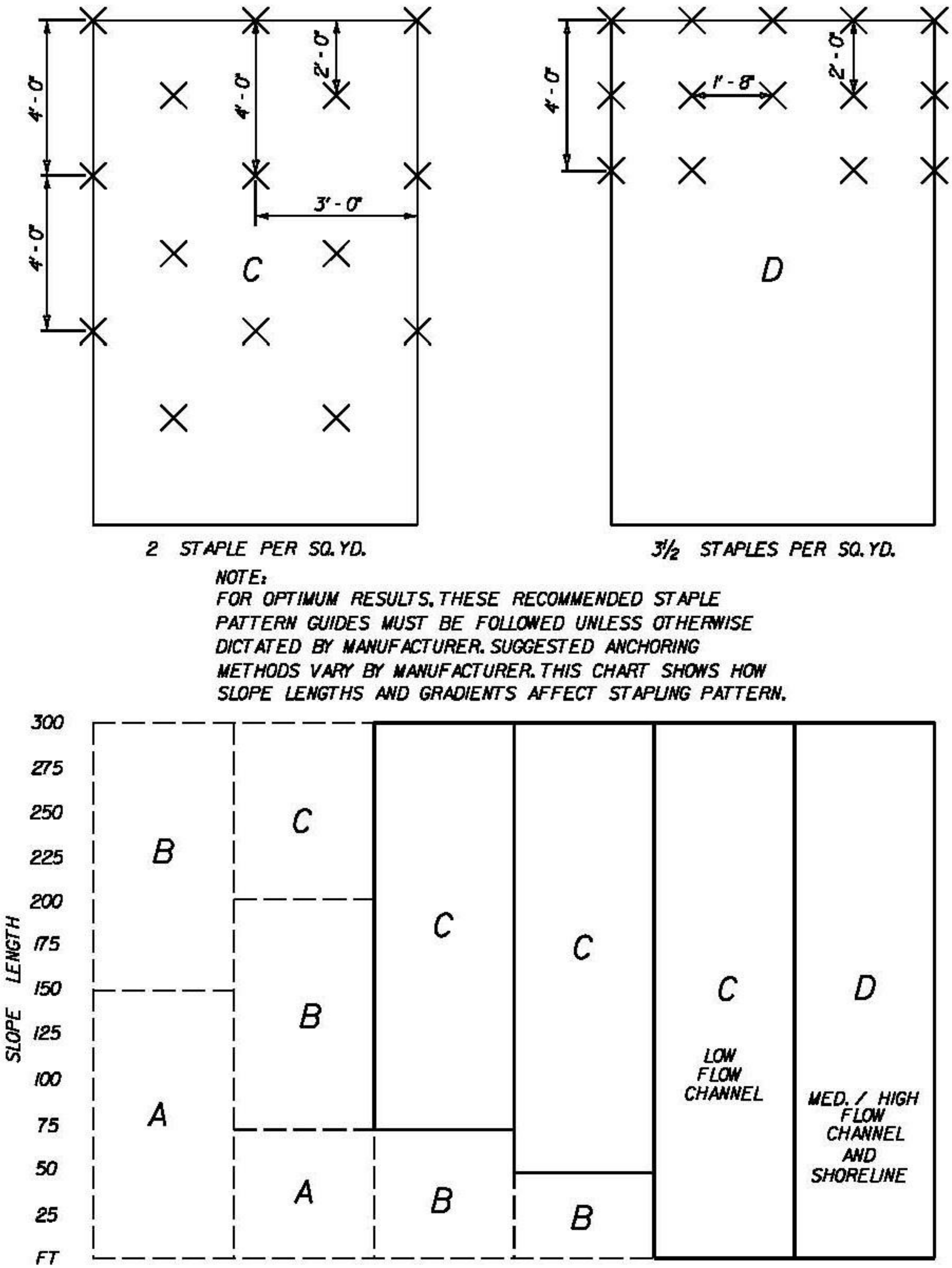


Figure 9-31c: Staple Pattern Parameters for Control Type C & D

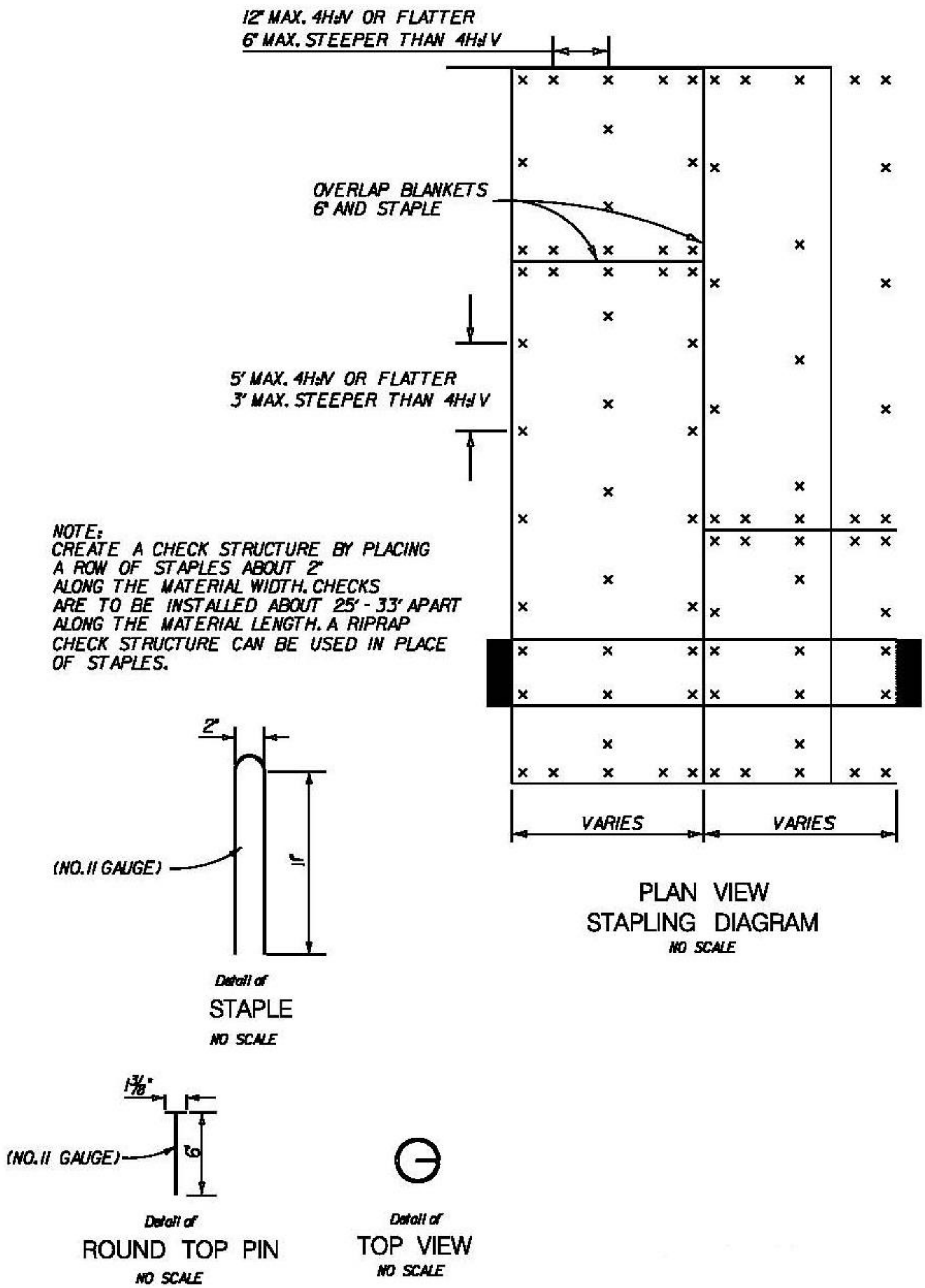


Figure 9-32: Rolled Erosion Control Products Staple Details

9.5.22 Other Best Management Practices

The LPSNRD 1994 Manual of Erosion and Sediment Control and Stormwater Management Standards contains detailed specifications and drawings for a number of different erosion and sediment control measures. The following structural control measures are not included herein for brevity, but they are included in the LPSNRD (1994) manual and may be useful in specialized applications.

- Safety Fence
- Brush Barrier
- Temporary Slope Drain
- Road Stabilization
- Utility Stream Crossing (e.g. diversion channel crossing, flume pipe crossing, coffer dam crossing)
- Dewatering Structure (e.g. portable sediment tank, filter box, straw bale/silt fence pit)
- Temporary Fill Diversion
- Turbidity Curtain
- Dust Control
- Surface Roughening
- Lot Benching
- Paved Flumes and Energy Dissipators
- Subsurface Drain
- Structural Streambank Stabilization (e.g. gabions, deflectors, log cribbing, grid pavers)
- Grade Stabilization Structure
- Infiltration Basin
- Infiltration Trench
- Detention Pond
- Extended Detention Pond
- Stormwater Conveyance Channel
- Vegetative Stream Bank Stabilization
- Topsoiling
- Sodding
- Permanent Diversion
- Vegetated Swale
- Temporary Right-of-Way Diversion

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