City of Concord
Technical Standards Manual

Article I
Stormwater

SECTION 1
POLICIES AND REQUIREMENTS

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LIST OF TABLES

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1.1 INTRODUCTION

1.1.1 Purpose

The intent of the Stormwater Technical Standards is to guide the planning, design, construction, operation, and maintenance of stormwater management facilities within the City of Concord (City) and all areas subject to its extraterritorial jurisdiction. Definitions, formulas, criteria, procedures, and data presented herein have been developed to support these policies. If a conflict arises between the technical data and these policies, the policies shall govern.

1.1.2 Contents

This Technical Standards addresses the following:

SECTION 1	POLICIES AND REQUIREMENTS
SECTION 2	STORMWATER RUNOFF
SECTION 3	STREET DRAINAGE
SECTION 4	STORM INLETS
SECTION 5	STORM SEWERS
SECTION 6	OPEN CHANNELS
SECTION 7	CULVERTS AND BRIDGES
SECTION 8	HYDRAULICS OF DETENTION
SECTION 9	SEDIMENT AND EROSION CONTROL
SECTION 10	WATER QUALITY
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1.1.3 Limitations

This Article is intended to establish appropriate design procedures. It is not to replace sound engineering practices nor preclude the use of procedures or information not presented herein. Information has been gathered from numerous sources and has not been presented in its entirety. It is recommended that the user obtain the original or additional reference material as appropriate. A bibliography is provided in Section 11.

1.2 ADMINISTRATIVE POLICY

These policies shall govern the planning, design, construction, operation, and maintenance of all stormwater related facilities within the City and all areas subject to its extra-territorial jurisdiction.

1.2.1 Goals and Objectives

The goals and objectives of the City of Concord Stormwater Management Program are as follows:

1. Protect human life and health.
2. Provide for the City's compliance with the Federal Clean Water Act (33 USC s1251 et seq.) and other regulations and permits conditions.
3. Improve the quality of stormwater runoff discharge to surface and groundwater.
4. Minimize private and public property damage resulting from erosion, sedimentation, and flooding.
5. Prevent new development from creating a demand for public investment in flood-control and water quality improvement works.
6. Provide an effective stormwater management system that will not result in excessive public or private monies being used for maintenance and replacement of portions of the system.
7. Facilitate the design of stormwater management systems that are consistent with good engineering practice and design and in accordance with the City's overall planning efforts and its watershed master plans.
8. Provide a mechanism that allows development of areas with minimum adverse effects to the natural environment.
9. Utilize appropriate public open space for both open space uses (parks, recreational uses, etc.) and the temporary storage and treatment of excess stormwater.
10. Encourage preservation of the drainage system in a natural and aesthetically pleasing condition as possible.
11. Encourage sustainable development.

1.2.2 Approvals

All stormwater management related plans must have written approval from the City in the form of a Stormwater Management Permit prior to bidding or beginning construction, except where specifically exempted by the City's Stormwater Quality Management and Discharge Control Ordinance (Ordinance).

1.2.3 Stormwater Management Permits

When required, permit application procedures shall be in accordance with the procedures outlined in this Article.

1.2.4 Article Revisions

Revisions to this Article will be issued in writing and can only be made by the City. Minor changes or modifications will be summarized in errata sheets and will be made available as necessary. The City reserves the right to make major changes affecting policy, criteria, methodologies, or engineering data. It shall be the responsibility of the design professional to obtain all updates. Any changes requested by users shall be accompanied by detailed comparative engineering data supporting the reasons and justification for the changes. Any changes so proposed must be in the best interest of the City.

1.2.5 Stormwater Management Permit Requirements

Stormwater management is a part of the total urban environmental system. As a requirement of the City's Stormwater National Pollutant Discharge Elimination (NPDES) Phase II Permit, the City has undertaken development of stormwater management policies that will result in improvements in the water quality present in the City's rivers and stream. Part of the City's commitment requires that all new development in the City ensure that stormwater is conveyed in an adequately designed drainage system of natural drainage ways, grass swales, storm sewers, culverts, inlets, and channels. Drainage systems must be designed, constructed, and maintained so as to provide natural infiltration, control velocity, control flooding, extend the time of concentration of stormwater runoff, and to control, to the maximum extent practicable, the export of pollutants and other impacts of development.

New developments and redevelopment that meet the criteria outlined in the City's Stormwater Ordinance must obtain a City of Concord Stormwater Management Permit prior to bidding or beginning construction or any land disturbing activity. In general the Ordinance requires a pre-approved Stormwater Management Permit for:

- All development that will exceed 20,000 square feet of cumulative impervious coverage.
- Any activity that disturbs land within a designated stream buffer area.
- Any filling or excavation of a parcel in excess of 1,000 cubic feet (cf) of material, or any filling or excavation that impacts an adjoining parcel.
- Any activity or development that will ultimately result in the disturbance of a total area of one or more acres.

1.2.6 Financial Responsibility

City policy requires that developers pay for all stormwater management facilities that directly serve their development. These facilities shall be adequate to meet the requirements for control and treatment of runoff for each of the design storm events. Property owners are required to operate and maintain such facilities to ensure their proper function and prevent them from becoming a public nuisance.

1.2.7 Design Requirements

The design criteria presented within this Article represent good engineering practice and should be utilized in the preparation of stormwater management plans. The criteria are not intended to be an iron-clad set of rules within which the developer and the design professional must work; they are intended to establish guidelines, standards, and methods for sound planning and design. Alternative methods of design should be submitted to the City for consideration.

The design criteria shall be revised and updated as necessary to reflect advances in the field of urban drainage engineering and urban water resource management.

The City and design professionals will utilize the Article in the planning of new facilities and in their review of proposed work by developers, private parties, and other governmental agencies.

The strict application of this Article, in the overall planning of new developments, is practical and economical. In the planning of drainage improvements and the designation of floodplains for built-up areas, the use of the criteria and standards herein may be adjusted as determined by the City.

1.3 DESIGN POLICY

1.3.1 Design Storm Frequencies

Every area shall be evaluated for two distinct stormwater management purposes: maintaining effective drainage systems and maintaining and improving water quality. Design for the more frequent storm events such as the 1- and 10-year storms serve both drainage and water quality protection purposes. Design for the larger events (25, 50, and 100-year storms) is focused on preventing damage and protecting health and safety.

Table 1-1, on the next page, lists the design storm frequencies for the various components of the stormwater management systems. Section 2 Stormwater Runoff, of this Article, provides the design storm precipitation for the City associated with the design storm frequencies.

TABLE 1-1
DESIGN STORM FREQUENCIES

Description	Design Storm Frequency
Piped Storm Sewer Systems	10-year, 24-hour
Drainage Ditches	10-year, 24-hour
Culverts and Bridges:	
Alley or Local Street	25-year, 24-hour
Collector	25-year, 24-hour
Thoroughfare	50-year, 24-hour
Freeway	100-year, 24-hour
Water Quality BMPs:	
Swales and Other Minor Facilities	1-year, 24-hour and 10-year, 24-hour
Infiltration Basins and Filters	1-year, 24-hour and 10-year, 24-hour
Wet and Dry Detention Basins	1-year, 24-hour and 10-year, 24-hour
Flood Control Detention Basins	According to most restrictive downstream condition, see Section 1.3.4
Detention Basin Emergency Spillways	Per NC Dam Safety Standards
Major Channels and Drainageways	100-year, 24-hour

Major Storm Provisions (100-year Return Frequency)

Provisions shall be made to prevent major property damage and loss of life for the storm runoff expected to have a one percent chance of occurring in any single year.

1.3.2 Hydrologic Analysis

The determination of runoff magnitude shall be accomplished using the Rational Method, the Soil Conservation Service (SCS) Unit Hydrograph (UH) methods as presented in TR55 and SWMM, a Kinematic Wave Method, or other computer modeling techniques and acceptable methods as approved by the City. Guidelines and limitations on application of several methods are presented in Section 2 Stormwater Runoff.

Computer Models

Approval by the City must be obtained before using hydrologic and hydraulic models other than those identified in this Article. The most recent, or other approved, version of the chosen model must be used.

1.3.3 Maximum Permissible Flooding

It is desirable to minimize the use of streets as a conveyance for storm runoff. However, streets are significant and important in urban drainage and can be used for storm runoff up to reasonable limits, recognizing that the primary purpose of streets is for traffic. All new conveyance systems shall be sized to convey the design storm event. Streets may be used to convey local runoff to drain inlets. Limits of the use of streets for conveying storm runoff shall be governed by the design criteria in Table 1-2.

TABLE 1-2
MINOR STORM RUNOFF ALLOWABLE STREET USE

Street Classification	Maximum Pavement Encroachment
Alley and Local Street	Flow may spread to crown of street. Velocities may not exceed 8 feet per second. No curb overtopping.
Collector	Flow spread must leave at least one half of a travel lane in each direction free of water. Velocities may not exceed 8 feet per second. No curb overtopping.
Thoroughfare	Flow spread must leave at least one lane in each direction free of water. Velocities may not exceed 8 feet per second. No curb overtopping.
Freeway	No encroachment is allowed on any travel lanes.

When the above maximum encroachment is reached, measures to reduce the flow, additional storm inlets, or other measures are required. Stormwater runoff conveyed through streets must be controlled and treated to the Maximum Extent Practicable before it is discharged to the City's municipal storm sewer system or receiving waters.

Planning and design objectives for public streets shall be based upon the limiting criteria in Table 1-3.

TABLE 1-3
MAJOR STORM RUNOFF ALLOWABLE STREET INUNDATION

Street Classification	Allowable Depth and Inundated Areas
Alley and Local Street	The depth of water over the gutter flow line shall not exceed 12 inches. Velocities shall not exceed 8 feet per second.
Major Collector and Both Thoroughfares	The depth of water at the street crown shall not exceed 6 inches in order to allow operation of emergency vehicles. The depth of water over the gutter flow line shall not exceed 12 inches. Velocities shall not exceed 8 feet per second.
Freeway	No inundation is allowed.

The allowable flow across streets shall be limited within the criteria shown in Table 1-4.

TABLE 1-4
MAJOR STORM RUNOFF ALLOWABLE CROSS STREET FLOW

Street Classification	Maximum Cross Flow Depth
Alley and Local Street	12 inches
Collector and Thoroughfare	6 inches or less over crown
Freeway	None

1.3.4 Stormwater Detention

Stormwater detention is required where necessary to limit flows such that the post-development peak discharge rate of flow does not exceed the pre-development rate for the 1-year, 24-hour and the 10-year, 24-hour storm events. This requirement may be waived when the development is part of a larger master plan approved by the City and which contains higher or lower discharge limitations for a particular area. Runoff volume drawdown time shall be a minimum of 24 hours, but not more than 120 hours.

Stormwater detention facilities shall be designed to achieve an 85 percent reduction (by weight) in the export of total suspended solids on an average annual basis. All detention facilities shall be designed to control and treat, at a minimum, the difference in runoff from pre- and post development conditions from the area draining to the facility.

Wherever reasonably acceptable from a social standpoint, parks and other open space may be used for short-term detention of storm runoff to create drainage benefits.

Maintenance of detention facilities entails the removal of debris and sediment. Without proper maintenance, a detention facility will become unsightly, a social liability, and eventually ineffective for water detention and treatment. Detention facilities will not be approved unless adequate maintenance can be provided. A maintenance plan and agreement must be filed with the City to ensure that the owner of the facilities understands the required frequency and type of maintenance necessary for proper operation of such facilities. Easements in favor of the City must be provided for emergency maintenance by the City when a property owner defaults on the maintenance agreement. Emergency maintenance performed or directed by the City shall be completed at the cost of the owner of the detention facility. Property owners are responsible for the maintenance and upkeep of the easement area. No permanent structures or other impediments to access shall be constructed within the area of easement.

Allowable Release Rates for Future Development

The allowable release rates from future developments shall not exceed the pre-development discharge rate for the specified design storms except as elsewhere provided for in this Article.

1.3.5 Natural Drainageways

The use of naturally occurring channels and drainageways is desired. Major consideration must be given to the floodplains and open space requirements for the area.

Natural drainageways within an urbanizing area are too often deepened, straightened, lined, and put underground. A community loses a natural asset when this happens. Channelizing a natural waterway usually speeds up the flow, causing greater downstream peaks and higher drainage costs downstream. Therefore, alternatives which include new or reconstructed drainage channels should be carefully weighed against the positive environmental and financial considerations of maintaining a natural drainageway.

Drainageways having slow flow, grassy bottoms and sides, and wide water surfaces provide significant water quality benefits and storage capacity. This storage is beneficial in that it reduces downstream pollutant loadings and runoff peaks. This reduces measures needed downstream to offset the impacts of development.

The depth of flow in the receiving stream must be taken into consideration for backwater computations for both the design and the major storm event, when sizing channel cross sections.

Significant changes to natural drainageways shall not be allowed unless computations show that changes will not adversely impact the channel or adjacent downstream development or have an adverse impact upon water quality.

A permanent maintenance access corridor easement shall be required with all constructed drainage channels. This easement shall provide a minimum access width of 20 feet from the top of bank on each side of the channel unless otherwise designated by the City. A wider easement will be required for River/Stream Overlay Districts and Stream Buffers on all intermittent and perennial streams as set forth by Article 4 of the Concord Development Ordinance and by the City's Stormwater Ordinance.

1.3.6 Sediment and Erosion Control

All new developments during construction shall be required to provide interim erosion and sedimentation control facilities to prevent the discharge of material into established drainageways or receiving bodies of water. Developments resulting in disturbance of 1 acre or more must provide the City's Stormwater Administrator with an approved Cabarrus County Erosion and Sediment Control Plan.

The City reserves the right to inspect for compliance with an approved Erosion and Sediment Control Plan and to take, or cause the developer to take, immediate corrective actions of any violations of such a plan.

1.3.7 Basin Planning

The City may, from time to time, develop and adopt watershed master plans that contain requirements specific to certain areas within the City's jurisdiction. Where such a watershed master plan has been adopted, all new development within the City shall conform to the approved plan. If no plan exists for that area of the proposed development, the City may waive this requirement, require the development to provide the necessary data, or declare a "special study area" until the watershed master plan for that area is completed. Interim plans may be accepted, providing they conform to the overall goals and objectives of the Stormwater Management Program and are otherwise consistent with the City's Ordinances and Regulations.

Stormwater Transfer

Planning and design of stormwater drainage systems should not result in the interwatershed transfer of stormwater. Channel modifications that create or increase flooding downstream shall be avoided; both for the benefit of the public and to prevent damage to private parties. Erosion and downstream sediment deposition increase of runoff peaks, and debris transportation must be avoided. It is the responsibility of the owner/developer to document, to the satisfaction of the Stormwater Administrator, that the proposed activity will not create such adverse conditions during the site plan review process.

The development process can significantly alter historical or natural paths. Development outfall systems shall discharge back into the natural drainageway at or near the historical location, unless otherwise approved by the City.

The policy of the City is to avoid transfer of storm drainage runoff from one watershed to another and to maintain historical drainage paths. However, the transfer of drainage from watershed to watershed is a viable alternative in certain instances and will be reviewed on a case-by-case basis by the City.

Floodplains

Certain areas within the City have been designated by the Federal Emergency Management Agency (FEMA) as flood hazard areas. All work impacting or adjacent to these areas shall be in compliance with all current FEMA regulations and permit requirements. The City has designated Floodplain Protection Overlay Districts (FPOD) consistent with Article 4.14 of its Concord Development Ordinance. All developments within a FPOD that apply for a Stormwater Management Permit must document that the development is in full compliance with the FPOD regulations.

Multi-purpose Use

Consideration shall be given to make all stormwater management facilities multipurpose facilities. Small parks, greenways, and other similar facilities shall be incorporated with major stormwater management facilities whenever possible, such that the hydraulic capacity and water quality treatment function of the facility is not compromised by these additional uses. River/Stream Overlay Districts and Stream Buffers shall be used for appropriate recreational and aesthetic purposes whenever feasible.

Access to Drainage Facilities

Easements, rights-of-way, or other legal access shall be provided to all stormwater management facilities for inspection, periodic maintenance, and infrequent repairs. Property owners are responsible for the maintenance and upkeep of the easement area. No permanent structures or other impediments to access shall be constructed within the area of easement.

1.3.8 Stormwater Runoff Quality

The policy of the City of Concord is to include stormwater quality considerations in planning of all stormwater facilities. Sediment, debris and other pollutants must be collected and removed from stormwaters. All stormwater facilities shall be compatible with the City's NPDES Phase II Permit and the Stormwater Management Program approved under that permit. The management of overall water quality of storm drainage will require submittal of a Stormwater Management Plan for new regulated developments (and redevelopments). The City has the right to inspect all construction sites and to enforce provisions of the Sedimentation and Erosion Control Permit and the City's Stormwater Management Permit.

1.3.9 Source and Structural BMPs

It is the policy of the City to encourage non-structural Best Management Practice (BMPs) for stormwater runoff and quality control wherever possible. When structural BMPs are required, they should be located as close to the source of runoff generation as possible. This policy is not intended to prevent the development of stormwater facilities that serve multiple parcels where there is benefit to the City and improvements to the control and treatment of stormwater runoff resulting from those facilities.

1.3.10 Operation and Maintenance

Operation and maintenance of stormwater facilities shall be required to ensure these facilities perform as designed. Prior to the construction of any stormwater management facility, the responsibility for the maintenance and operation of that facility shall be determined and an inspection and maintenance agreement, acceptable to the Stormwater Administrator, shall be filed. Property owners are responsible for the maintenance and upkeep of the facility.

Channel bed and bank erosion, drop structures, trash racks, pipe inlets and outlets, pumping facilities, and overall condition of the facilities shall be inspected as shown in Table 1-5 and repaired as necessary to avoid reduced conveyance capacity and ultimate failure. Sediment and debris shall be removed from channels, storm drains, and detention basins. Trash racks and inlets shall also be routinely cleared of debris to maintain system capacity. BMPs as part of the development shall be periodically serviced and repaired as necessary to keep these facilities functioning properly.

TABLE 1-5
MAINTENANCE SCHEDULE

Facility or Activity	Inspection/Cleaning Frequency	Maintenance Activity
Catch Basins and Manholes	Annually/Annually	Trash, Sediment, and Oil Removal
Pump Stations	Quarterly/During Autumn	Mechanical Checklist Debris Removal
Ditches and Swales	Every Autumn/Monthly during Summer	Mowing and Debris Removal
Drainageways and Streams	Annually or after large storm events/Annually	Grading Repairs (Erosion) Debris Removal
Pipes	Annually and after large storm events/every two to four years	Debris Removal TV Monitoring
Culverts	Annually/Annually	Debris Removal
Detention Facilities	Monthly/Annually City stormwater staff will perform an annual inspection of facility and owner's inspection and maintenance records	Trash, oil, and debris removal Control structure and vegetation control

Vehicle and maintenance equipment access shall be provided to all stormwater for maintenance and inspection. Developers shall be responsible for providing features to facilitate maintenance of drainage systems, including inlets, culverts, channels, ditches, and detention basins.

1.4 STORMWATER MANAGEMENT PLANS

1.4.1 Stormwater Management Plan Requirements

A Stormwater Management Plan shall be prepared for all development except that specifically exempted in the City's Stormwater Ordinance. The purpose of the plan is to identify existing and proposed hydrology and hydraulics of the site and the proposed storm drainage system. The plan shall also propose specific solutions to stormwater problems that would occur as a result of development. Detailed analysis of drainage basin hydrology and hydraulics is required. Solutions to drainage problems shall be noted and the capacity of facilities on and off-site shall be evaluated. Specific improvements including open channels, storm drains, grading, erosion and sediment control, inlets, culverts, detention/retention basins, and other improvements shall be located and sized to meet the requirements of the drainage systems as described in this Article and in accordance with the appropriate watershed master plan, if available. The drainage plan must describe the general treatment of drainageways, including safety and maintenance, and outline the protection of public facilities and the protection of private property adjacent to the waterways.

It is acknowledged that certain circumstances may preclude the use of certain requirements stipulated in this Article. It shall be the responsibility of the user to provide an explanation of the circumstance, the specific exception(s) requested, the justification(s) for the request, and the mitigative measures to be taken. Whenever a requirement set forth in this Article, or otherwise established by the City, can not be practically achieved, it is the responsibility of the developer to present a mitigation plan that achieves through alternative means substantially the same result as would have been achieved under the City's requirement.

1.4.2 Qualifications

All stormwater management, drainage or related facilities designs that are submitted as part of an application for a City of Concord Stormwater Management Permit shall be reviewed and sealed by a Registered Professional Engineer with a valid license from the state of North Carolina. The design professional shall attest that the design was conducted in accordance with the laws of the State of North Carolina and with policies of the City and with this Article.

1.4.3 Computations

Computations shall be submitted for review to the City and shall be in accordance with the procedures, standards, and criteria of this Article.

1.4.4 Plan Submittal Standards

All designs shall be accompanied by supporting data, graphs, calculations, sketches, and applicable references appropriate to the complexity of the proposed facility.

Plans and profiles shall be drawn on sheets no smaller than 24" x 36" to a horizontal scale of no smaller than 1"=50', and to a vertical scale of 1"=10', collectively referred to as the Site Plan. Exceptions are permitted on specific projects such as culverts and channel cross sections. For the purpose of applying for a Stormwater Management Permit, the Site Plan shall include, at minimum, the following information:

- Address or vicinity map showing the location of the activity.
- Subdivision name and date of the approved subdivision plat, if applicable.
- The date of the subdivision's approved Stormwater Management Permit, if applicable.
- Site boundaries.

- Street and other right-of-ways.
- Existing roadway width and pavement type.
- Stationing shall match street stationing and proceed upstream.
- North arrow shall point to the top of page or to the left.
- Elevation datum shall be United States Geological Survey (USGS).
- Existing and proposed structures and finished floor elevations.
- Existing and proposed driveway locations and types.
- Existing and proposed stormwater facilities (swales, pipes, inlets, basins, etc.).
- General drainage patterns indicated on a topographic map showing 1-foot (or smaller) contour intervals.
- Natural drainageways and direction of flow.
- City designated Stream Buffer overlays.
- Flood boundaries and/or elevations.
- Location and extend, and label the name of, any waterbody that is shown on the most recent version of either the 7.5-minute USGS topographic map or the Natural Resource Conservation Service (NRCS) Soil Survey map.
- Extent and phasing of land disturbing activities. If needed, a separate drawing can be provided for each phase.
- Other information that may be necessary to develop an understanding of the project.

Profiles shall indicate the proposed system (size and type of material) with flow-line elevations, flow-lines, gradients, left and right bank profiles, station numbers, inlets, manholes, ground-line and curb-line elevations, typical cross sections, riprap construction, filling details, minimum permissible building floor elevations within 100-year floodplain and adjacent to open drainage features, pipe crossings, design flow capacities and velocities, and title block.

Detail plans and sections shall be provided for all special system features such as detention/retention facilities, inlets, manholes, culverts, pipe bedding and backfill, ditch sections, and all related structures.

A complete list of the drawing requirements is included on a reproducible sheet included in Appendix A. No text presented on the drawings and documents shall be in a font smaller than 10-point type. The Stormwater Administrator may waive any of the format specifications and required items that are deemed not to be necessary for the review, reproduction, and storage of the documents.

At the City’s option, any or all of the above materials may be, or may be required to be, submitted in electronic formats compatible with the Stormwater Administrator’s computer systems and software.

1.4.5 Final As-Constructed Documents

Upon completion and final approval by the City, the original mylars depicting as-constructed (built) conditions shall be submitted to the City and shall remain the permanent property of the City. Final documents shall be sealed by a registered professional engineer with a valid license from the State of North Carolina. All associated costs shall be borne by the owner.

At the City's option, the Final As-Constructed documents may be, or may be required to be, submitted in electronic formats compatible with the Stormwater Administrator's computer systems and software.

1.4.6 Maintenance and Access Agreements and Restrictive Covenants

The City's NPDES Phase 2 permit also requires that provisions be made for the long term ownership and maintenance of stormwater control structures and best management practices. To that end, both Code of Ordinances section 60-88 (c) and Concord Development Ordinance sections 4.4.6. B. and 4.4.6. C. establish certain requirements for long-term maintenance and City access to stormwater control facilities. Specifically, the three ordinance sections require that property owners and developers enter into contracts with the City for access and long-term maintenance of stormwater control facilities. These sections also require developers and property owners to provide for the long term maintenance of future owners by establishing restrictive covenants. Contact the Concord City Attorney for more information on the covenants and maintenance agreements acceptable to the City. The City Attorney may be reached at:

Albert M. Benshoff
City Attorney
Legal Dept
PO Box 308
30 Market St.
Concord, NC 28026-0308
704 920 5114
FAX 704 784 1791
Benshoff@ci.concord.nc.us

1.5 NEW DEVELOPMENT AND REDEVELOPMENT MINIMUM REQUIREMENTS

All new development, including redevelopment, that includes land disturbing activity of one acre or greater shall prepare a Cabarrus County Erosion and Sediment Control Plan. Copies of the approved plan must be delivered to the City of Concord Plan Review Service Center (PRSC) upon submission of an application for a City Stormwater Management Permit.

All new developments, including redevelopments, which are required to obtain a City Stormwater Management Permit, shall meet all the minimum requirements presented in this section.

1.5.1 Minimum Requirement No. 1: Erosion and Sediment Control

All new development and redevelopment that includes land disturbing activities of ≥1 acre shall comply with all Cabarrus County Erosion and Sediment Control Requirements. Compliance with the Erosion and Sediment Control Requirements shall be demonstrated through implementation of an approved Erosion and Sediment Control Plan.

In addition, the following erosion and sediment control requirements shall be and are the responsibility of the developer:

No. 1.5.1.1: Protection of Adjacent Properties

Properties adjacent to the project site shall be protected from sediment deposition and are the responsibility of the developer.

No. 1.5.1.2: Protection of Stream Buffers

Designated stream buffers shall be protected from disturbance and all other adverse impact during and after the construction phase. Sediment ponds and traps, sediment barriers, and other appropriate BMPs shall be used to ensure that Buffers are not eroded, receive sediment deposition, or that the vegetation is damaged from the beginning of disturbance through final site stabilization.

No. 1.5.1.3: Controlling Off-Site Erosion

Properties and waterways downstream from development sites shall be protected from erosion due to increases in the volume, velocity, and peak flow rate of stormwater runoff from the project site.

No. 1.5.1.4: Construction Access Routes

Wherever construction vehicle access routes intersect paved roads, provisions must be made to minimize the transport of sediment (mud) onto the paved road. If sediment is transported onto a road surface, the roads shall be cleaned thoroughly at the end of each day. Sediment shall be removed from roads by shoveling or sweeping and be transported to a controlled sediment disposal area. Street washing shall be allowed only after sediment is removed in this manner.

No. 1.5.1.5: Removal of Temporary BMPs

All temporary erosion and sediment control BMPs shall be removed within 30 days after final site stabilization is achieved or after the temporary BMPs are no longer needed. Trapped sediment shall be removed or stabilized onsite. Disturbed soil areas resulting from removal shall be permanently stabilized.

No. 1.5.1.6: Control of Pollutants Other Than Sediment on Construction Sites

All pollutants other than sediment that occur onsite during construction shall be handled and disposed of in a manner that does not cause contamination of stormwater or groundwater.

No. 1.5.1.7: Maintenance

All temporary and permanent erosion and sediment control BMPs shall be maintained and repaired as needed to assure continued performance of their intended function. All maintenance and repair shall be conducted in accordance with recommendations presented in Section 10.

1.5.2 Minimum Requirement No. 2: Preservation of Natural Drainage Systems

Natural drainage patterns shall be maintained, and discharges from the site shall occur at the natural location, to the maximum extent practicable. Use of curb and gutters and other methods of development that disrupt the maintenance of sheet flow and the natural drainage patterns shall be minimized to the extent practicable and permissible by the Concord Development Ordinance.

1.5.3 Minimum Requirement No. 3: Preservation of Stream Buffers

The City has designated River/Stream Overlay Districts as described in Article 4 of the Concord Development Ordinance. In addition, the City's Stormwater Management Program requires that the stream buffer for a Class l stream shall be an undisturbed area of 50 feet plus four times the average percent slope of the area adjacent to the stream up to a maximum of 125 feet. An additional 20 foot vegetated setback from the stream buffer is required on all Class 1 streams.

For Class 2 streams, the stream buffer shall be measured from the average annual stream bank perpendicularly for a distance of 30 feet. No slope adjustments are required. There is an additional 10-foot vegetated setback from the undisturbed stream buffer on Class 2 streams.

Stream buffer areas shall be designated on recorded plats as easements. The plat shall be included with the Application for a Stormwater Management Permit.

All stream buffers shall be maintained by the landowner to maintain sheet flow to the maximum extent practical. Except where no practicable alternative exists, no new stormwater conveyance channels or outfalls may traverse or be constructed in a stream buffer.

Developers must provide certification, acceptable to the Stormwater Administrator that only development or other land disturbing activities that will occur within a designated stream buffer are "exempt or allowed" activities as shown within Table 1-6.

TABLE 1-6
TABLE OF USES AND ACTIVITIES WITHIN CONCORD STREAM BUFFERS

Use/Activity	Exempt	Allowed	Allowable with Mitigation	Prohibited
Airport facilities:				
➢ Airport facilities that impact equal to or less than 150 linear feet (LF) or one-third of an acre of stream buffer.		X		
➢ Airport facilities that impact greater than 150 LF or one-third of an acre of stream buffer.			X	
Archaeological activities	X			
Bridges		X		
Dam maintenance activities	X			

Use/Activity	Exempt	Allowed	Allowable with Mitigation	Prohibited
Drainage ditches, roadside ditches, and storm water outfalls through stream buffers:				
➢ Existing drainage ditches, roadside ditches, and stormwater outfalls provided that they are managed to minimize the sediment, nutrients and other pollution that convey to water bodies.	X			
➢ New drainage and roadside ditches and stormwater outfalls provided that a stormwater management facility is installed to control pollutant discharge and attenuate flow before the conveyance discharges through the stream buffer.		X		
➢ New drainage and roadside ditches and stormwater outfalls that do not provide control for pollutant discharge before discharging through the stream buffer.				X
➢ Excavation of the streambed in order to bring it to the same elevation as the invert of a ditch.				X
Drainage of a pond in a natural drainageway provided that a new stream buffer meeting the requirements of the City Stormwater Ordinance is established adjacent to the new channel.	X			
Driveway crossings of streams and other surface waters subject to this Rule:				
➢ Driveway crossings on single-family residential lots that disturb equal to or less than 25 LF or 2,500 square feet (SF) of stream buffer.	X			
➢ Driveway crossings on single-family residential lots that disturb greater than 25 LF or 2,500 SF of stream buffer.		X		
➢ In a subdivision that cumulatively disturb equal to or less than 150 LF or one-third of an acre of stream buffer.		X		
➢ In a subdivision that cumulatively disturb greater than 150 LF or one-third of an acre of stream buffer.			X	

Use/Activity	Exempt	Allowed	Allowable with Mitigation	Prohibited
Fences, provided that disturbance is minimized and installation does not result in removal of forest vegetation.	X			
Forest harvesting		X		
Fertilizer application: ➢ One-time fertilizer application to establish replanted vegetation. ➢ Ongoing fertilizer application.	X			X
Grading and revegetation provided that diffuse flow and the health of existing vegetation are not compromised and disturbed areas are stabilized.			X	
Greenway/hiking trails		X		
Historic preservation	X			
Landfills as defined by G.S. 130A-290				X
Mining activities: ➢ Mining activities that are covered by the Mining Act provided that new streams are established adjacent to the relocated channels. ➢ Mining activities that are not covered by the Mining Act or where new stream buffers are not established adjacent to the relocated channels. ➢ Wastewater or mining dewatering wells with approved NPDES permit.	X	X		X
Non-electric utility lines: ➢ Impacts other than perpendicular crossings.			X	

Use/Activity	Exempt	Allowed	Allowable with Mitigation	Prohibited
Non-electric utility line perpendicular crossing of streams and other surface waters. ➢ Perpendicular crossings that disturb equal to or less than 40 LF of stream buffer with a maintenance corridor equal to or less than 10 feet in width. ➢ Perpendicular crossings that disturb greater than 40 LF of stream buffer with a maintenance corridor greater than 10 feet in width. ➢ Perpendicular crossings that disturb greater than 40 LF but equal to or less than 150 LF of stream buffer with a maintenance corridor equal to or less than 10 feet in width. ➢ Perpendicular crossings that disturb greater than 40 LF but equal to or less than 150 LF of stream buffer with a maintenance corridor greater than 10 feet in width. ➢ Perpendicular crossings that disturb greater than 150 LF of stream buffer.	X	X X	X X	
On-site sanitary sewage systems - new ones that utilize ground absorption.				X
Overhead electric utility lines: ➢ Impacts other than perpendicular crossings.			X	
Overhead electric utility line perpendicular crossings of streams and other surface waters. ➢ Perpendicular crossings that disturb equal to or less than 150 LF of stream buffer. ➢ Perpendicular crossings that disturb greater than 150 LF of stream buffer.	X		X	
Periodic maintenance of modified natural streams such as canals and a grassed travelway on one side of the surface water when alternative forms of maintenance access are not practical.		X		

Use/Activity	Exempt	Allowed	Allowable with Mitigation	Prohibited
Playground equipment: ➢ Playground equipment on single-family lots provided that installation and use does not result in vegetation removal. ➢ Playground equipment installed on lands other than single-family lots or that requires vegetation removal.	X		X	
Ponds in natural drainageways, excluding dry ponds: ➢ New ponds provided that a stream buffer that meets the City's requirements is established adjacent to the pond. ➢ New ponds where a stream buffer is NOT established adjacent to the pond.		X	X	
Protection of existing structures, facilities, and streambanks when this requires additional disturbance of the stream buffer or the stream channel.			X	
Railroad impacts other than crossings of streams and other surface waters.			X	
Railroad crossings of streams and other surface waters subject to this Rule: ➢ Railroad crossings that impact equal to or less than 40 LF of stream buffer. ➢ Railroad crossings that impact greater than 40 LF but equal to or less than 150 LF or one-third of an acre of stream buffer. ➢ Railroad crossings that impact greater than 150 LF or one-third of an acre of stream buffer.	X	X	X	
Removal of previous fill or debris provided that diffuse flow is maintained and any vegetation removed is restored.		X		

Use/Activity	Exempt	Allowed	Allowable with Mitigation	Prohibited
Road crossings of streams and other surface waters:				
➢ Road crossings that impact equal to or less than 40 LF of stream buffer.	X			
➢ Road crossings that impact greater than 40 LF but equal to or less than 150 LF or one-third of an acre of stream buffer.		X		
➢ Road crossings that impact greater than 150 LF or one-third of an acre of stream buffer.			X	
Scientific studies and stream gauging.	X			
Stormwater management ponds excluding dry ponds:				
➢ New stormwater management ponds provided that a stream buffer is established adjacent to the pond.		X		
➢ New Stormwater management ponds where a stream buffer is NOT established adjacent to the pond.			X	
Stream restoration	X			
Streambank stabilization		X		
Temporary roads:				
➢ Temporary roads that disturb less than or equal to 2,500 SF provided that vegetation is restored within six months of initial disturbance.	X			
➢ Temporary roads that disturb greater than 2,500 SF provided that vegetation is restored within six months of initial disturbance.		X		
➢ Temporary roads used for bridge construction or replacement provided that restoration activities, such as soil stabilization and revegetation, are conducted immediately after construction.		X		

Use/Activity	Exempt	Allowed	Allowable with Mitigation	Prohibited
Temporary sediment and erosion control devices: ➢ To control impacts associated with uses approved by the Division of Water Quality (DWQ) or that have received a variance provided that sediment and erosion control for upland areas is addressed to the maximum extent practical outside the buffer. ➢ In-stream temporary erosion and sediment control measures for work within a stream channel.		 X X		
Underground electric utility lines: ➢ Impacts other than perpendicular crossings.			X	
Underground electric utility line perpendicular crossings of streams and other surface waters. ➢ Perpendicular crossings that disturb less than or equal to 40 LF of stream buffer. ➢ Perpendicular crossings that disturb greater than 40 LF of stream buffer.	 X	 X		
Vegetation management: ➢ Emergency fire control measures provided that topography is restored. ➢ Planting vegetation to enhance the stream buffer. ➢ Pruning forest vegetation, including understory vegetation, provided that the health and function of the vegetation is not compromised. ➢ Removal of individual trees that are in danger of causing damage to dwellings, other structures or human life. ➢ Removal of poison ivy. ➢ Removal of understory nuisance vegetation as defined in: Smith, Cherri L. 1998, Exotic Plant Guidelines; Department of Environment and Natural Resources, Division of Parks and Recreation, Raleigh, NC, Guideline #30.	 X X X X X X			

Use/Activity	Exempt	Allowed	Allowable with Mitigation	Prohibited
Water dependent structures as defined in 15A NCAC 2B .0202.		X		
Water supply reservoirs: ➢ New reservoirs provided that a stream buffer is established adjacent to the reservoir. ➢ New reservoirs where a Stream Buffer is NOT established adjacent to the reservoir.		X	X	
Water wells	X			
Wetland restoration	X			

1.5.4 Minimum Requirement No. 4: Source Control of Pollution

Source control BMPs shall be applied to all projects to the maximum extent practicable. Source control BMPs shall be selected, designed, and maintained according to all known and reasonable technology.

1.5.5 Minimum Requirement No. 5: Runoff Treatment BMPs

All projects shall provide treatment of stormwater. Treatment BMPs shall be sized to capture and treat the water quality design storm. The first priority for treatment shall be to filter and infiltrate as much as possible of the water quality design storm, defined as the runoff produced by the 1-year, 24-hour storm or the first inch of runoff, whichever is more, but only if site conditions are appropriate and groundwater quality will not be impaired.

Stormwater treatment BMPs shall not be built within a designated stream buffer, except for necessary conveyance systems as approved by the City or when an appropriate stream buffer is established and maintained around the BMP.

1.5.6 Minimum Requirement No. 6: Streambank Erosion Control

The requirement below applies only to situations where stormwater runoff is discharged to a stream, and must be met in addition to meeting all other minimum requirements.

Stormwater discharges to streams shall control streambank erosion by limiting the peak rate of runoff from individual development sites to no more than the existing condition peak runoff rate for the 1-year, 24-hour and the 10-year, 24-hour design storms. As the first priority, streambank erosion control BMPs shall utilize infiltration to the fullest extent practicable, but only if site conditions are appropriate and groundwater quality is protected.

Stormwater treatment BMPs shall not be built within a designed stream buffer, except where a recorded conservation easement of an area equivalent to the stream buffer is established around the BMP, where appropriate vegetation is established and maintained within that conservation easement, and where approved by the City Stormwater Administrator.

1.5.7 Minimum Requirement No. 7: Wetlands

The requirements below apply only to situations where stormwater discharges through a conveyance system into a wetland.

1. Stormwater discharges to wetlands must be controlled and treated to the extent necessary to meet the State Water Quality Standards.

2. Discharges to wetlands shall maintain the hydroperiod and flows of existing site conditions to the extent necessary to protect the characteristic uses of the wetland. Prior to discharging to a wetland, alternative discharge locations shall be evaluated, and natural water storage and infiltration opportunities outside the wetland shall be maximized.

3. Created wetlands that are intended to mitigate the loss of wetland acreage, function, and value shall also not be designed to treat stormwater.

4. In order for constructed wetlands to be considered treatment systems, they must be constructed on sites that are not wetlands and they must be managed for stormwater treatment. If these systems are not managed and maintained for a period exceeding three years, these systems may no longer be considered constructed wetlands. Discharges from constructed wetlands to waters of the state (including discharges to natural wetlands) are regulated under Section 404 of the Clean Water Act.

1.5.8 Minimum Requirement No. 8: Water Quality Sensitive Areas

Where the City determines that the minimum requirements do not provide adequate protection of water quality sensitive areas, either onsite or within the basin, more stringent controls shall be required to protect water quality.

1.5.9 Minimum Requirement No. 9: Off-Site Analysis and Mitigation

All major development projects, as designated by the Stormwater Administrator, shall conduct an analysis of off-site water quality impacts resulting from the project and shall mitigate these impacts. The analysis shall extend a minimum of one-fourth of a mile downstream from the project. The existing or potential impacts to be evaluated and mitigated shall include, at a minimum, but not be limited to:

- Excessive sedimentation

- Streambank erosion
- Discharges to groundwater contributing or recharge zones
- Violations of water quality standards
- Spills and discharges of priority pollutants

1.5.10 Minimum Requirement No. 10: Watershed Master Planning

Adopted and implemented watershed master plans may be used to modify any or all of the minimum requirements, provided that the level of protection for surface or groundwater achieved by the master plan will equal or exceed that which would be achieved by the minimum requirements in the absence of a basin plan. Watershed master plans shall evaluate and include, as necessary, retrofitting of BMPs for existing development and/or redevelopment in order to achieve watershed-wide pollutant reduction goals. Standards developed from master plans shall not modify any of the above requirements until the basin plan is formally adopted and fully implemented by the City.

1.5.11 Minimum Requirement No. 11: Operation and Maintenance

An operation and maintenance schedules and agreements shall be provided for all proposed stormwater facilities and BMPs, and the party (or parties) responsible for maintenance and operation shall be identified.

1.5.12 Minimum Requirement No. 12: Financial Liability

Performance bonding or other appropriate financial instruments may be required for any project in order to ensure compliance with these standards.

1.5.13 Exceptions

Exceptions to minimum requirements Nos. 1 through 11 may be granted prior to permit approval and construction. An exception may be granted, provided that a written finding of fact is prepared, that addresses the following:

1. The exception provides equivalent environmental protection and is in the overriding public interest; and that the objectives of safety, function, environmental protection and facility maintenance, based upon sound engineering, are fully met.

2. That there are special physical circumstances or conditions affecting the property such that the strict application of these provisions would deprive the applicant of all reasonable use of the parcel of land in question, and every effort to find creative ways to meet the intent of the minimum requirements has been made.

3. That the granting of the exception will not be detrimental to the public health and welfare, nor injurious to other properties in the vicinity and/or downstream, and to the quality of waters of the state.

4. The exception is the least possible exception that could be granted to comply with the intent of the minimum requirements.

1.6 SMALL PARCEL MINIMUM REQUIREMENTS

Many developments will not require a City Stormwater Management Permit. These developments meet the following criteria:

- Less than 1 acre will be ultimately disturbed by the development.
- Less than 20,000 SF of impervious area will be ultimately created by the development.
- Less than 1,000 CF of fill or excavation will occur during the development.
- The ultimate impervious percentage of the parcel will be less than 24 percent.

Developments meeting the above criteria still must control erosion and sedimentation and control and treat stormwater runoff. In general these developments must ensure that stormwater is conveyed in an adequately designed drainage system of natural drainage ways, grass swales, and other conveyances that are constructed and maintained to provide natural infiltration, control flow velocities, control flooding, extend the time of concentration of stormwater runoff, and to control to the maximum extent practicable the export of pollutants and other impacts of development.

The following minimum requirements shall be met for all such new developments:

1.6.1 Small Parcel Requirement No. 1: Construction Access Route

Construction vehicle access shall be, whenever possible, limited to one route. Access points shall be stabilized with quarry spall or crushed rock to minimize the tracking of sediment onto public roads and shall be the responsibility of the developer.

1.6.2 Small Parcel Requirement No. 2: Stabilization of Denuded Areas

All exposed and unworked soils shall be stabilized by suitable application of BMPs, including, but not limited to, sod or other vegetation, plastic covering, mulching, or application of ground base on areas to be paved. From October 1 through April 30, no soils shall remain exposed for more than two days. From May 1 through September 30, no soils shall remain exposed for more than seven days.

1.6.3 Small Parcel Requirement No. 3: Protection of Adjacent Properties

Adjacent properties shall be protected from sediment deposition by appropriate use of vegetative buffer strips, sediment barriers or filters, dikes or mulching, or by a combination of these measures and other appropriate BMPs.

1.6.4 Small Parcel Requirement No. 4: Maintenance

All erosion and sediment control BMPs shall be regularly inspected and maintained to ensure continued performance of their intended function.

1.6.5 Small Parcel Requirement No. 5: Other BMPs

As required by the City, other appropriate BMPs to mitigate the effects of increased runoff shall be applied.

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SECTION 2
STORMWATER RUNOFF

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2.1 INTRODUCTION

Hydrology is the scientific study of water and its properties, distribution, and effects on the earth's surface, soil, and atmosphere. Hydrologic analyses include estimation of peak runoff rates, volumes, and time distribution of stormwater runoff flows and are fundamental in the design of stormwater management facilities. This section addresses the movement of water over land resulting directly from precipitation in the form of stormwater runoff.

Land development changes how a watershed responds to precipitation. The most common effects are reduced infiltration and decreased travel time. Increased impervious surfaces and runoff velocities increase peak flow discharge volumes and rates. Total stormwater runoff volume is determined by the total drainage area of the receiving watershed, its infiltration characteristics, and the amount of precipitation.

2.2 DESIGN CRITERIA

This section presents rainfall data for storm events in the City of Concord (City) with recurrence intervals of 1-, 2-, 5-, 10-, 25-, 50-, and 100-years. A storm event with a 100-year recurrence interval is a storm with a magnitude that, on the average, has a one percent chance of being equaled or exceeded in any given year. A 2-year storm has a 50 percent chance of being equaled or exceeded in any given year. The type of storm distribution to be used, as well as criteria for selecting which recurrence interval to use as a design storm, is presented in Section 1.3, Design Policy

2.2.1 Design Storm Distribution and Hytegraph

All storm event hydrograph methods require the input of a rainfall distribution or design storm hyetograph. The design storm hyetograph is essentially a plot of rainfall depth versus time for a given design storm frequency and duration. It is usually presented as a dimensionless plot of unit rainfall depth (increment of rainfall depth for each time interval divided by the total rainfall depth) versus time (presented in Figure 2-1, on page 2-3). The total 24-hour rainfall volumes for the 1-, 2-, 5-, 10-, 25-, 50-, and 100-year storm events are provided in Table 2-1. The hyetograph provided in this section is to be used for all hydrograph analysis. The hyetograph (presented in Table 2-2, on page 2-4) is required for all design storms of 24-hour duration. It is the standard Soil Conservation Service (SCS) Type II rainfall distribution in 10-minute time intervals.

TABLE 2-1 APPROXIMATE 24-HOUR RAINFALL VOLUMES FOR CONCORD, NC	
Return Interval	**24-hour Volume (inches)**
1-year	2.9
2-year	3.5
5-year	4.4
10-year	5.1
25-year	6.0
50-year	6.8
100-year	7.6

2.2.2 Intensity-Duration-Frequency Curves

Intensity-duration-frequency (IDF) curves have been developed for the area and are shown in Figure 2-2, on page 2-5. The curves for 1-, 2-, 5-, 10-, 25-, 50-, and 100-year storm events with durations from 5 minutes to 24 hours. Ordinates from the Figure 2-2 have been provided in Table 2-3, on page 2-5.

2.2.3 Design Storm Recurrence Intervals

The selection of the recurrence interval to be used for the design storm shall be in accordance with Section 1.3, Design Policy.

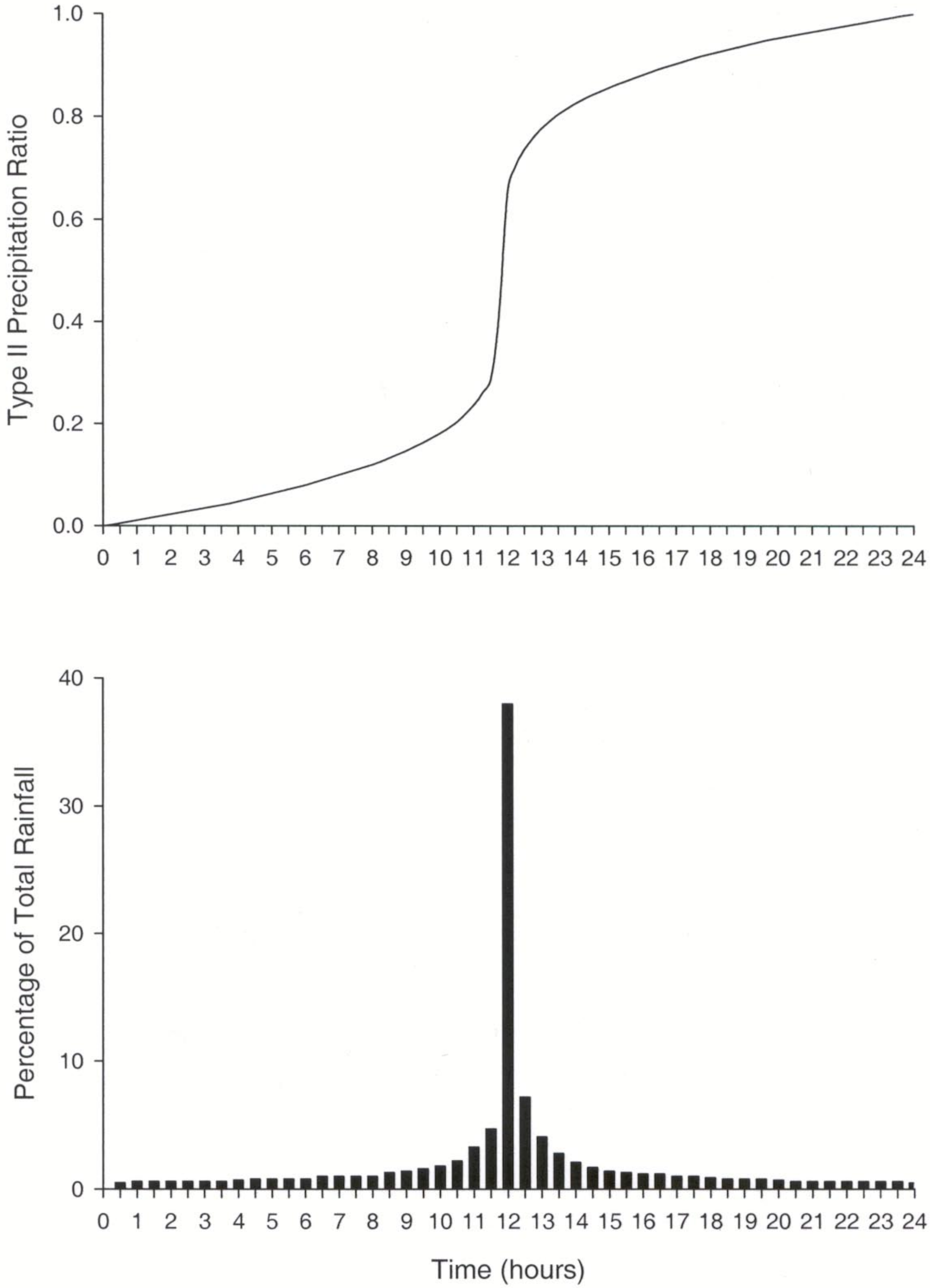


Figure 2-1 24-Hour Design Storm Hyetograph SCS Type II Precipitation Distribution

TABLE 2-2
ORDINATES OF THE SCS TYPE II RAINFALL DISTRIBUTION HYETOGRAPH

Time from Beginning of Storm (Minutes)	Percent Rainfall (%)	Cumulative Percent Rainfall (%)	Time from Beginning of Storm (Minutes)	Percent Rainfall (%)	Cumulative Percent Rainfall (%)	Time from Beginning of Storm (Minutes)	Percent Rainfall (%)	Cumulative Percent Rainfall (%)
0	0.00	0.00	490	0.40	12.40	970	0.40	88.50
10	0.13	0.13	500	0.43	12.83	980	0.40	88.90
20	0.17	0.30	510	0.47	13.30	990	0.40	89.30
30	0.20	0.50	520	0.47	13.77	1000	0.33	89.63
40	0.20	0.70	530	0.46	14.23	1010	0.34	89.97
50	0.20	0.90	540	0.47	14.70	1020	0.33	90.30
60	0.20	1.10	550	0.53	15.23	1030	0.33	90.63
70	0.20	1.30	560	0.54	15.77	1040	0.34	90.97
80	0.20	1.50	570	0.53	16.30	1050	0.33	91.30
90	0.20	1.70	580	0.6	16.90	1060	0.33	91.63
100	0.20	1.90	590	0.60	17.50	1070	0.30	91.93
110	0.20	2.10	600	0.60	18.10	1080	0.27	92.20
120	0.20	2.30	610	0.67	19.33	1090	0.27	92.47
130	0.20	2.50	620	0.73	19.50	1100	0.26	92.73
140	0.20	2.70	630	0.80	20.30	1110	0.27	93.00
150	0.20	2.90	640	1.00	21.30	1120	0.27	93.27
160	0.20	3.10	650	1.10	22.40	1130	0.26	93.53
180	0.20	3.30	660	1.20	23.60	1140	0.27	93.80
170	0.20	3.50	670	1.40	25.00	1150	0.27	94.07
190	0.20	3.70	680	1.57	26.57	1160	0.26	94.33
200	0.20	3.90	690	1.73	28.30	1170	0.27	94.60
210	0.20	4.10	700	6.93	35.23	1180	0.27	94.87
220	0.20	4.30	710	12.67	47.90	1190	0.23	95.10
230	0.23	4.53	720	18.40	66.30	1200	0.20	95.30
240	0.27	4.80	730	2.93	69.23	1210	0.20	95.50
250	0.27	5.07	740	2.40	71.63	1220	0.20	95.70
260	0.26	5.33	750	1.87	73.50	1230	0.20	95.90
270	0.27	5.60	760	1.53	75.03	1240	0.20	96.10
280	0.27	5.87	770	1.37	76.40	1250	0.20	96.30
290	0.26	6.13	780	1.20	77.60	1260	0.20	96.50
300	0.27	6.40	790	1.00	78.60	1270	0.20	96.70
310	0.27	6.67	800	0.93	79.53	1280	0.20	96.90
320	0.26	6.93	810	0.87	80.40	1290	0.20	97.10
330	0.27	7.20	820	0.73	81.13	1300	0.20	97.30
340	0.27	7.47	830	0.70	81.83	1310	0.20	97.50
350	0.26	7.73	840	0.67	82.50	1320	0.20	97.70
360	0.27	8.00	850	0.60	83.10	1330	0.20	97.97
370	0.33	8.33	860	0.57	83.67	1340	0.20	98.10
380	0.34	8.67	870	0.53	84.20	1350	0.20	98.30
390	0.33	9.00	880	0.47	84.67	1360	0.20	98.50
400	0.33	9.33	890	0.46	85.13	1370	0.20	98.70
410	0.34	9.67	900	0.47	85.60	1380	0.20	98.90
420	0.33	10.00	910	0.47	86.07	1390	0.20	99.10
430	0.33	10.33	920	0.43	86.50	1400	0.20	99.30
440	0.34	10.67	930	0.40	86.90	1410	0.20	99.50
450	0.33	11.00	940	0.40	87.30	1420	0.20	99.70
460	0.33	11.33	950	0.40	87.70	1430	0.17	99.87
470	0.34	11.67	960	0.40	88.10	1440	0.13	100.00
480	0.33	12.00						

Source: SCS TR-20 Manual

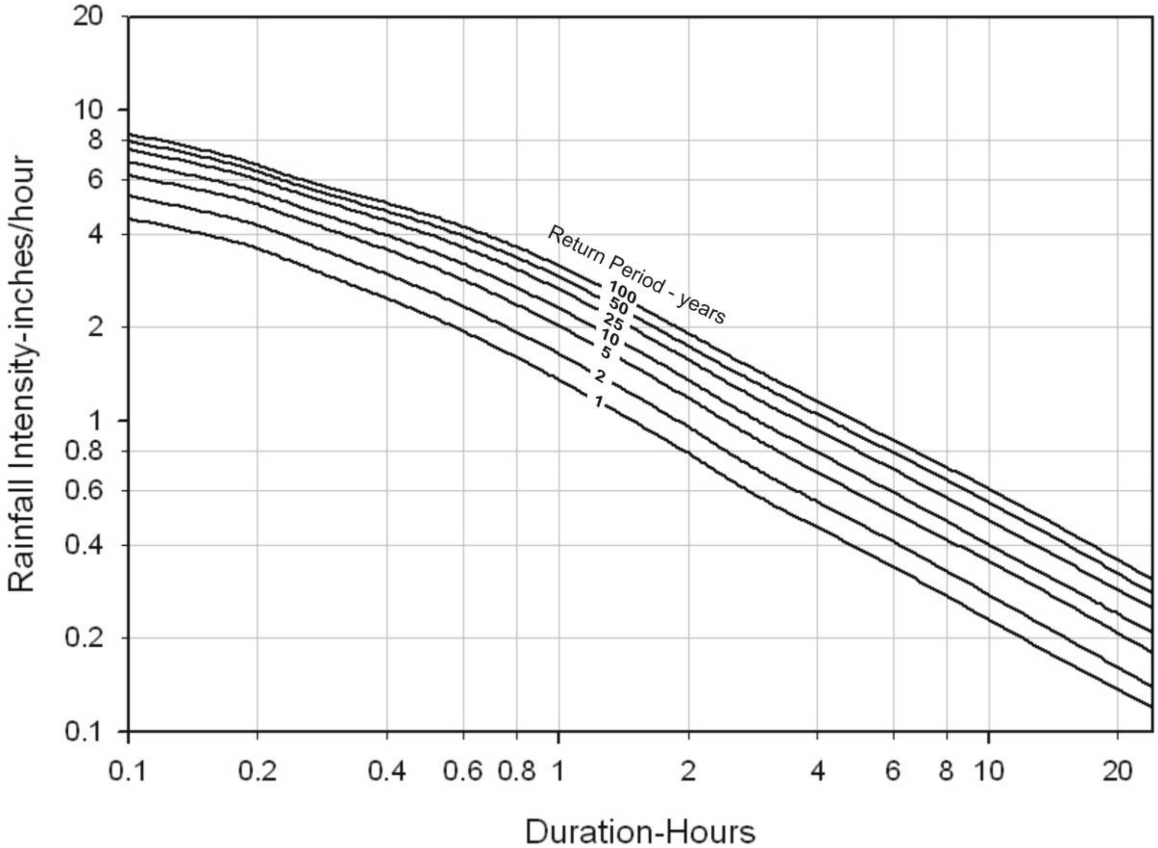


Figure 2-2 Intensity-Duration-Frequency Curves for Concord, NC

TABLE 2-3 ORDINATES OF THE IDF CURVES FOR CONCORD, NC								
Duration		**Return Period (years)**						
Hours	**Minutes**	**1**	**2**	**5**	**10**	**25**	**50**	**100**
0	5	4.80	5.66	6.60	7.26	8.00	8.52	8.98
	10	3.83	4.54	5.29	5.81	6.38	6.78	7.13
	15	3.19	3.80	4.46	4.90	5.39	5.72	6.01
	30	2.19	2.62	3.17	3.55	3.99	4.31	4.60
1		1.36	1.65	2.03	2.31	2.66	2.92	3.17
2		0.79	0.96	1.19	1.36	1.58	1.75	1.92
3		0.56	0.68	0.85	0.98	1.15	1.29	1.42
6		0.34	0.41	0.51	0.59	0.70	0.79	0.87
12		0.20	0.24	0.31	0.35	0.42	0.48	0.53
24		0.12	0.14	0.18	0.21	0.25	0.28	0.31

2.3 HYDROLOGIC METHODS

The design of properly sized storm drainage facilities requires some knowledge of the hydrologic behavior of the drainage basin under study. For most designs it is adequate to estimate only the peak discharge of the drainage area for the required frequency. While larger, more complex drainage basins may require the use of a method in order to estimate the discharge hydrograph. Two methods are presented in this section to satisfy their needs.

The Rational Method is recommended for street drainage and small drainage basins. It is an effective method for estimating peak discharges in small areas where rainfall intensity tends to be uniform for the area. Although the basic principles of the Rational Method apply to drainage areas greater than 20 acres, practice generally limits its use to some maximum area (50 acres).

For larger areas, storage and subsurface drainage flow cause an attenuation of the runoff hydrograph so that the rates of flow tend to be overestimated by the Rational Method. Because of the trend for overestimation of flows and the additional cost in drainage facilities associated with this overestimating, the application of a more sophisticated runoff computation technique is usually warranted for larger drainage areas. These larger drainage basins (0 to 2,000 acres) are modeled effectively by the SCS UH Method. This procedure is suitable for applications where the shape of the hydrograph and the volume of runoff are necessary, such as the design of detention facilities or water quality facilities.

2.3.1 <u>Rational Method</u>

The Rational Method is an empirical runoff formula that has gained wide acceptance because of its simple, intuitive treatment of peak storm runoff rates for small drainage basins. This method relates runoff to rainfall intensity, surface area, and surface characteristics by the formula:

$$Q = C_f \, CiA \tag{2-1}$$

where:

Q	=	peak runoff rate, in cubic feet per second
C_f	=	frequency factor to adjust the runoff coefficient for less frequent, high intensity storms
C	=	runoff coefficient
I	=	average rainfall intensity, for a duration equal to the time of concentration, in inches per hour
A	=	drainage area of the tributary to the point under consideration, in acres

The Rational Method is based on the following assumptions:

A. The rainfall occurs uniformly over the drainage area.

B. The frequency of the peak discharge is the same as the frequency of the average rainfall intensity (i.e., a 2-year rainfall intensity generates a 2-year discharge).

C. The time of concentration is the time required for runoff to become established and flow from the hydraulically most remote part of the drainage area to the basin outlet. This assumption applies to the most remote in time, not necessarily in distance.

Runoff Coefficient, C

The runoff coefficient, C, is the variable of the Rational Method for which it is most difficult to develop a precise determination. This provides the design professional with a degree of latitude to exercise his or her professional judgment. The following discussion is intended to provide a guide to promote the uniform application of runoff coefficients.

The runoff coefficient, C, accounts for abstractions for losses between rainfall and runoff which may vary with time for a given drainage area. These losses are caused by interception by vegetation, infiltration into permeable soils, retention in surface depressions, and evaporation and transpiration. In determining this coefficient, differing climatological and seasonal conditions, antecedent moisture conditions, and the intensity and frequency of the design storm should be considered.

Table 2-4, on page 2-8, represents recommended C values for various land uses. Where ranges are shown, adjustments should be made for level of development, surface type, soil type, and slope. It is often desirable to develop a composite runoff coefficient based in part on the percentage of different types of surfaces in the drainage area. This procedure can be applied to typical "sample" areas as a guide to the selection of usual values of the coefficient for the entire area. Suggested coefficients with respect to surface types are given in Table 2-5, on page 2-8.

The design professional shall use values in Tables 2-4 and 2-5 as guidance and derivation of these values shall be carefully documented and justified in the design drawings and specifications. Areas not conforming to these descriptions will be evaluated by calculating a composite runoff coefficient. Areas shall be evaluated based upon the ultimate development.

The coefficients in these two tables are applicable for storms of 1- to 10-year frequencies. These coefficients are based on the assumption that the design storm does not occur when the ground surface is frozen. Table 2-6, on page 2-9, represents correction factors to adjust the runoff coefficient for less frequent high intensity storms.

TABLE 2-4
RECOMMENDED RATIONAL METHOD RUNOFF COEFFICIENTS FOR LAND USE

Description of Area		Runoff Coefficient (Up to 10-Year Design Storm)
Business:	Downtown	0.70 to 0.95
	Neighborhood	0.50 to 0.70
Residential	RE – maximum of 1 du/ac	0.35
	RL – maximum of 2 du/ac	0.45
	RM-1 and RM-2 – maximum of 4 du/ac	0.50
	RV – maximum of 8 du/ac	0.57
	RC – maximum of 15 du/ac	0.65
Industrial:	Light	0.50 to 0.80
	Heavy	0.60 to 0.90
Parks and Cemeteries		0.10 to 0.25
Playgrounds		0.20 to 0.40
Railroad Yard		0.20 to 0.40
Unimproved		0.10 to 0.30

TABLE 2-5
SUGGESTED RATIONAL METHOD RUNOFF COEFFICIENTS FOR SURFACE TYPES

Character of Surface		Runoff Coefficient (Up to 10-Year Design Storm)
Pavement :	Asphaltic and Concrete	0.95
	Brick	0.85
Wooded		0.25
Packed gravel areas		0.55
Unpacked gravel areas		0.85
Driveways and Walkways		0.95
Roofs		0.95
Turf Slopes:	Flat, 0 to 1 percent	0.25
	Average, 1 to 3 percent	0.35
	Hilly, 3 to 10 percent	0.40
	Steep, 10 percent +	0.45
Cultivated Ground:	Flat, 0 to 1 percent	0.10
	Average, 1 to 3 percent	0.20
	Hilly, 3 to 10 percent	0.25
	Steep, 10 percent	0.30

TABLE 2-6
FREQUENCY FACTORS FOR THE RATIONAL METHOD

Recurrence Interval (years)	Adjustment Factor, C_f
1 to 10	1.00
25	1.10
50	1.20
100	1.25

Rainfall Intensity, i

Rainfall intensity, i, is the average rate of rainfall in inches per hour. Intensity is selected on the basis of design frequency of occurrence, a statistical parameter established by design criteria, and time of concentration. Rainfall intensity can be determined for the 1-, 2-, 5-, 10-, 25-, 50-, and 100-year return periods from Figure 2-2 and Table 2-3. Note that the total design storm depth (or volume) is not used in the Rational Method. This method determines only the peak discharge rate not the total runoff volume.

Travel Time, T_t

Travel time, T_t, is the time required for 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 the point of design. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system. T_c influences the shape and peak of the runoff hydrograph. Urbanization usually decreases T_c, thereby increasing the peak discharge; but T_c can be increased as a result of (a) ponding behind small or inadequate drainage systems, including storm drain inlets and road culverts; or (b) reduction of land slope through grading.

T_c is the sum of T_t values for the various consecutive flow segments.

$$T_c = T_{t1} + T_{t2} + \ldots T_{tm} \qquad (2\text{-}2)$$

where:

T_c	=	time of concentration, in minutes
m	=	number of flow segments
T_t	=	travel time, in minutes

Note: For application within the City, a minimum time of concentration of 5 minutes shall be used for each area or sub area to which the Rational Method is applied.

Travel time (T_t) is the ratio of flow length to flow velocity:

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

(2-3)

where:

T_t	=	travel time, in minutes
L	=	flow length, in feet
V	=	average velocity, in feet per second
60	=	conversion factor from seconds to minutes

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

Sheet Flow

Sheet flow is flow over plane surfaces. With sheet flow, the friction value (n_s, a modified Manning's 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) is used. These n_s values are for very shallow flow depths (approximately 0.1 foot or less) and are only used for travel lengths up to 300 feet in urban areas and 1,000 feet in non-urban areas. Table 2-7, on the next page, gives Manning's n_s values for sheet flow for various surface conditions.
For sheet flow determination, use Manning's kinematic solution to directly compute T_t:

$$T_t = \frac{0.42(n_s L)^{0.8}}{(P_2)^{0.5}(s_o)^{0.4}}$$

(2-4)

where:

T_t	=	travel time, in minutes
n_s	=	sheet flow Manning's effective roughness coefficient (from Table 2-7)
L	=	flow length, in feet
P_2	=	2-year, 24-hour rainfall depth, in inches, (see Table 2-1)
s_o	=	slope of hydraulic grade line, in land slope, feet per foot

TABLE 2-7
ROUGHNESS COEFFICIENTS (MANNING'S n_S)
FOR OVERLAND FLOW

Surface Description		n_s[1]
Smooth Surfaces (concrete, asphalt, gravel, or bare soil)		0.011
Fallow (no residue[2])		0.05
Cultivated Soils:	Residue Cover <20 percent	0.06
	Residue Cover >20 percent	0.17
Grass:	Short Grass Prairie	0.15
	Dense Grasses[3]	0.24
	Bermuda Grass	0.41
Range (natural)		0.13
Woods[4]:	Light Underbrush	0.40
	Dense Underbrush	0.80

1 The values are a composite of information compiled by Engman (1986).
2 Residue cover is cultivated or chopped plant material left on the field to prevent erosion and hold moisture.
3 Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
4 When selecting n_s, consider cover to a height of about 0.1 feet. This is the only part of the plant cover that will obstruct sheet flow.

Velocity Equation

A commonly used method of computing average velocity of flow, once it has measurable depth, is the following equation:

$$V = k\sqrt{s_o} \tag{2-5}$$

where:

V = velocity, in feet per second
k = time of concentration velocity factor, in feet per second
s_0 = slope of flow path, in feet per foot

"k" is computed for various land covers and channel characteristics with assumptions made for hydraulic radius using the following rearrangement of Manning's equation:

$$k = \frac{1.49R^{0.67}}{n} \tag{2-6}$$

where:

R	=	an assumed hydraulic radius
n	=	Manning's roughness coefficient for open channel flow

Typical "k" values are presented in Table 2-8, on the next page.

Shallow Concentrated Flow

Velocities for this type of flow can be calculated using the k_s values from Table 2-8 in which average velocity is a function of watercourse slope and type of channel. After computing the average velocity using the Velocity Equation above, the T_t for the shallow concentrated flow segment can be computed using the Travel Time Equation described above.

Open Channel Flow

Open channels are assumed to begin where the flow enters a definable system. This may include piped systems, ditches, channels visible on aerial photographs, or where lines indicating streams appear (in blue) on United State Geological Survey quadrangle sheets. The k_c values from Table 2-8 used in the Velocity Equation above or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full conditions. After average velocity is computed, the T_t for the channel segment can be computed using the Travel Time Equation described above.

Limitations

The following limitations apply in estimating T_t:

- Manning's kinematic solution should not be used for sheet flow longer than 300 feet in urban areas or 1,000 feet in non-urban areas.
- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate T_c. Storm sewers 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 a standard hydraulics textbook to determine average velocity in pipes for either pressure or non-pressure flow.
- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. A hydrograph should be developed to this point and a stage-storage-discharge technique should be used to determine an outflow rating curve through the culvert or bridge.

TABLE 2-8
"k" VALUES USED IN TRAVEL TIME/TIME OF CONCENTRATION CALCULATIONS

Shallow Concentrated Flow [after the initial 300 feet (1,000 feet in non-urban areas) of sheet flow, R = 0.1]	**k_s**
1. Forest with heavy ground litter and meadows (n = 0.10)	3
2. Brushy on ground with some trees (n = 0.060)	5
3. Fallow or minimum tillage cultivation (n = 0.040)	8
4. High grass (n = 0.035)	9
5. Short grass, pasture and lawns (n = 0.030)	11
6. Nearly bare ground (n = 0.025)	13
7. Paved and gravel areas (n = 0.012)	27
Channel Flow (intermittent - at the beginning of visible channels R = 0.2)	**k_c**
1. Forested swale with heavy ground litter (n = 0.10)	5
2. Forested drainage course/ravine with defined channel bed (n = 0.050)	10
3. Rock-lined waterway (n = 0.035)	15
4. Grassed waterway (n =0.030)	17
5. Earth-lined waterway (n = 0.025)	20
6. CMP pipe (n = 0.024)	21
7. Concrete pipe (0.012)	42
8. Other waterways and pipes	0.508/n[1]
Channel Flow (continuous stream, R = 0.4)	**k_c**
Meandering stream with some pools (n = 0.040)	20
Rock-lined stream (n = 0.035)	23
Grass-lined stream (n = 0.030)	27
Other streams, man-made channels and pipe	0.807/n[1]
[1] Manning's "n" values are based on facility characteristics.	

[PB1]

Example 1: The following is an example of travel time and time of concentration calculations:

Given: An existing drainage basin having a selected flow route composed of the following five segments:

Segment 1: L = 200 feet. Dense grasses (sheet flow)
s_o = 0.03 feet per foot, n_s = 0.24

Segment 2: L = 300 feet. Pasture (shallow concentrated flow)
s_o = 0.04 feet per foot, k_s = 11

Segment 3: L = 50 feet. Small pond (year around)
s_o = 0.00 feet per foot, k_c = 0

Segment 4: L = 300 feet. Grassed waterway (intermittent channel)
s_o = 0.05, k_c = 17

Segment 5: L = 500 feet Grass-lined stream (continuous)
s_o = 0.02, k_c = 27

Calculate T_t's for each reach and then sum them to calculate the drainage basin T_c.

Segment 1: Sheet flow (L<300 *feet*), $T_t = \dfrac{0.42\,(n_s L)^{0.8}}{(P_2)^{0.5}(s_o)^{0.4}}$

$$T_1 = \frac{(0.42)[(0.24)(200\ \textit{feet})]^{0.8}}{(3.5\ \textit{inches})^{0.5}(0.03\ \textit{feet per foot})^{0.4}} = 20.2\ \textit{minutes}$$

Segment 2: Shallow concentrated flow $V = k\sqrt{s_o}$

$$V_2 = (11)(0.04\ \textit{feet per foot})^{0.5} = 2.2\ \textit{feet per second}$$

$$T_2 = \frac{L}{60V} = \frac{(300\ \textit{feet})}{(60\ \textit{seconds per feet})(2.2\ \textit{feet per second})} = 2.3\ \textit{minutes}$$

Segment 3: Flat water surface

T_3 = 0 minutes

Segment 4: Intermittent channel flow $V = k\sqrt{s_o}$

$$V_4 = (17)(0.05\,\textit{feet per foot})^{0.5} = 3.8\,\textit{feet per second}$$

$$T_4 = \frac{(300\ \textit{feet})}{(60\ \textit{seconds per minute})(3.8\ \textit{feet per second})} = 1.3\ \textit{minutes}$$

Segment 5: Continuous stream $V = k\sqrt{s_o}$

$$V_5 = (27)(0.02\,\textit{feet per foot})^{0.5} = 3.8\,\textit{feet per second}$$

$$T_5 = \frac{(500\ \textit{feet})}{(60\ \textit{seconds per minute})(3.8\ \textit{feet per second})} = 2.2\ \textit{minutes}$$

$$T_c = T_1 + T_2 + T_3 + T_4 + T_5$$

$$T_c = 20.2 + 2.3 + 0 + 1.3 + 2.2 = 26\ \textit{minutes}$$

2.3.2 SCS Unit Hydrograph Method

A hydrograph is a graph of the discharge rate versus time, at a particular point. A hydrograph reflects precipitation and watershed characteristics, as well as geologic factors. A hydrograph can be separated into three segments: the rising limb, the crest segment, and the recession limb. The shape of the rising limb is especially sensitive to rainfall characteristics, while the shape of the recession limb is more sensitive to geologic characteristics and watershed slope. The crest segment is sensitive to both watershed and rainfall characteristics. The SCS UH Method may be used for determining peak runoff rates for rural basins with a drainage area of up to 2,000 acres. Other hydrograph methods may only be used with prior written approval by the City.

Additional background on this methodology can be found in "Urban Hydrology for Small Watersheds", Technical Release TR-55 (1986) and SCS National Engineering Handbook, Section 5 (1986). After October 1994, the United States Department of Agriculture Soil Conservation Service (USDA SCS) became known as the Natural Resource Conservation Service (NRCS). Materials and other references are denoted as SCS or NRCS based upon referenced edition or issue date.

Unit Hydrograph Definition

A UH is a special case of the flood hydrograph. Specifically, a UH is the hydrograph that results from one inch of precipitation excess generated uniformly over the watershed at a uniform rate during a specified period of time. There are five important concepts in this definition that warrant emphasis. First, the runoff occurs from precipitation excess, which can be defined as the difference between precipitation and losses. Losses include interception, depression storage, and infiltrated water that does not appear as direct runoff. Second, the volume of runoff is one inch, which is the same as the volume of precipitation excess. Third, the excess is applied at a constant rate (i.e., uniform rate). Fourth, the excess is applied with a uniform spatial distribution. Fifth, the intensity of the rainfall excess is constant over a specified period of time, which is called the duration.

The SCS method uses a dimensionless UH that is based on an extensive analysis of measured data. Unit hydrographs were evaluated for a large number of actual watersheds and then made dimensionless. An average of these dimensionless UH was developed as shown in Figure 2-3, on page 2-18. The time base of the dimensionless UH was approximately 5 times the time-to-peak and approximately 3/8 of the total volume occurred before the time-to-peak; the inflection point on the recession limb occurs at approximately 1.7 times the time-to-peak, and the UH had a curvilinear shape. The Dimensionless Curvilinear Unit Hydrograph and Equivalent Triangular Hydrograph is shown in Figure 2-4, on page 2-20, and the discharge ratios for selected values of the time ratio are given in Table 2-9.

Lag Time and Concentration Time

Two basic equations are used in defining the shape of the UH. The first equation defines the lag time of the basin, L, or the time from the midpoint of unit excess rainfall to the UH peak, t_p. A number of relationships are given for the dimensions of the SCS synthetic UH (see Figure 2-4, on page 2-20).

Lag time is defined as the time in hours from the center of mass of rainfall excess to peak discharge:

$$L = \frac{\ell^{0.8}(S+1)^{0.7}}{1{,}900y^{0.5}} \tag{2-7}$$

where:

L	=	lag time, in hours
ℓ	=	hydraulic length, in feet
y	=	slope, in percentage
S	=	maximum retention

$$S = \frac{1000}{CN} - 10$$

where:

CN	=	curve number

The time of concentration is related to the lag time by the equation:

$$t_c = 5/3L$$

(2-8)

where:

t_c = time of concentration, in hours

Runoff Parameters

All storm event hydrograph methods require the input of parameters which describe the physical drainage basin characteristics. These parameters provide the basis from which the runoff hydrograph is developed. This section describes the three key parameters (area, curve number, and time of concentration) used to develop the runoff hydrograph using the SCS UH Method.

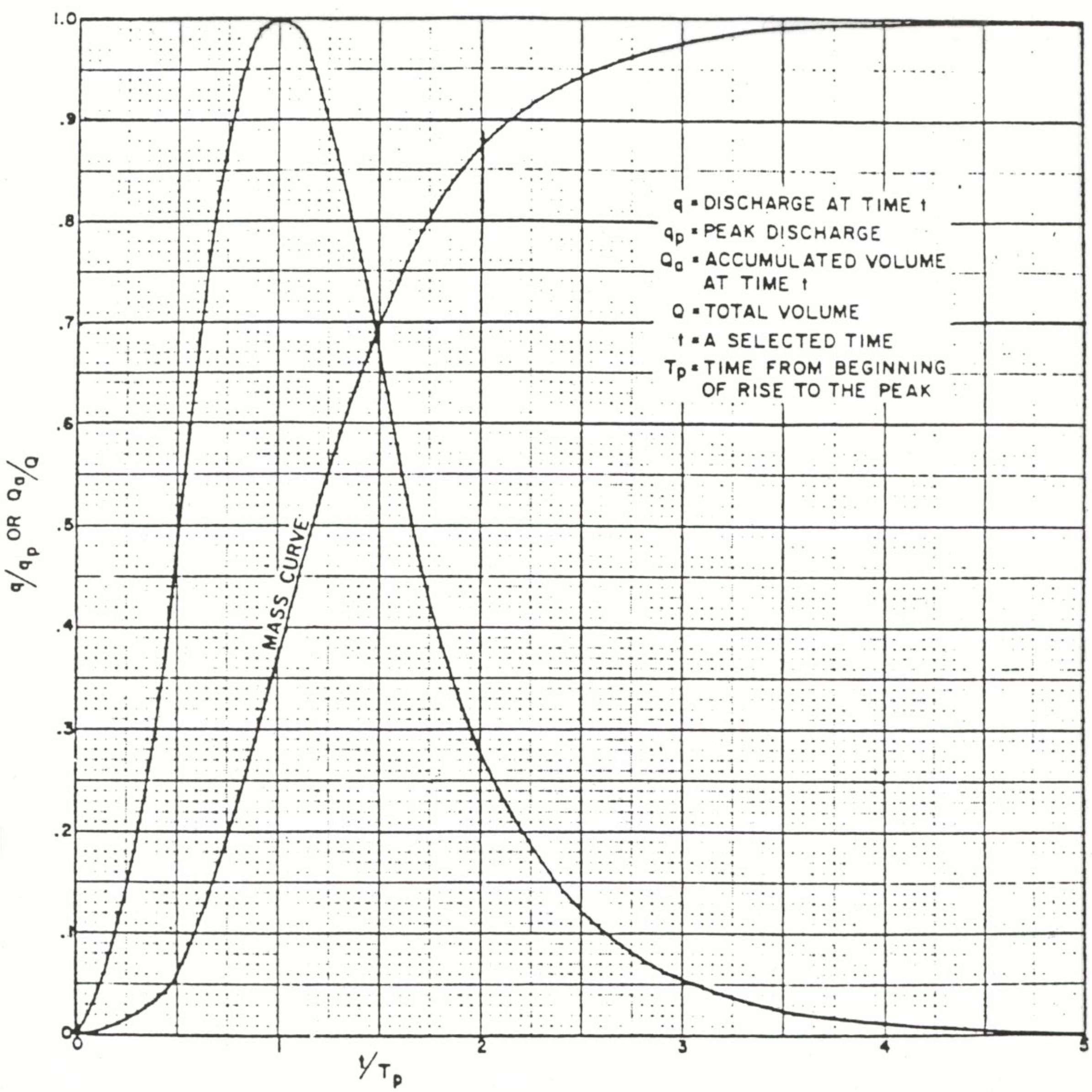


Figure 2-3 Dimensionless Unit Hydrograph and Mass Curve

TABLE 2-9
RATIOS FOR DIMENSIONLESS UNIT HYDROGRAPH AND MASS CURVE

Time Ratios t/t_p	Discharge Ratios q_P	Mass Curve Ratios Q_a/Q
0	0.000	0.000
0.1	0.030	0.001
0.2	0.100	0.006
0.3	0.190	0.012
0.4	0.310	0.035
0.5	0.470	0.065
0.6	0.660	0.107
0.7	0.820	0.163
0.8	0.930	0.228
0.9	0.990	0.300
1.0	1.000	0.375
1.1	0.990	0.450
1.2	0.930	0.522
1.3	0.860	0.589
1.4	0.780	0.650
1.5	0.680	0.700
1.6	0.560	0.751
1.7	0.460	0.790
1.8	0.390	0.822
1.9	0.330	0.849
2.0	0.280	0.871
2.2	0.207	0.908
2.4	0.147	0.934
2.6	0.107	0.953
2.8	0.077	0.967
3.0	0.055	0.977
3.2	0.040	0.984
3.4	0.029	0.989
3.6	0.021	0.993
3.8	0.015	0.995
4.0	0.011	0.997
4.5	0.005	0.999
5.0	0.000	1.000

Source: SCS National Engineering Handbook, Section 5 (1986).

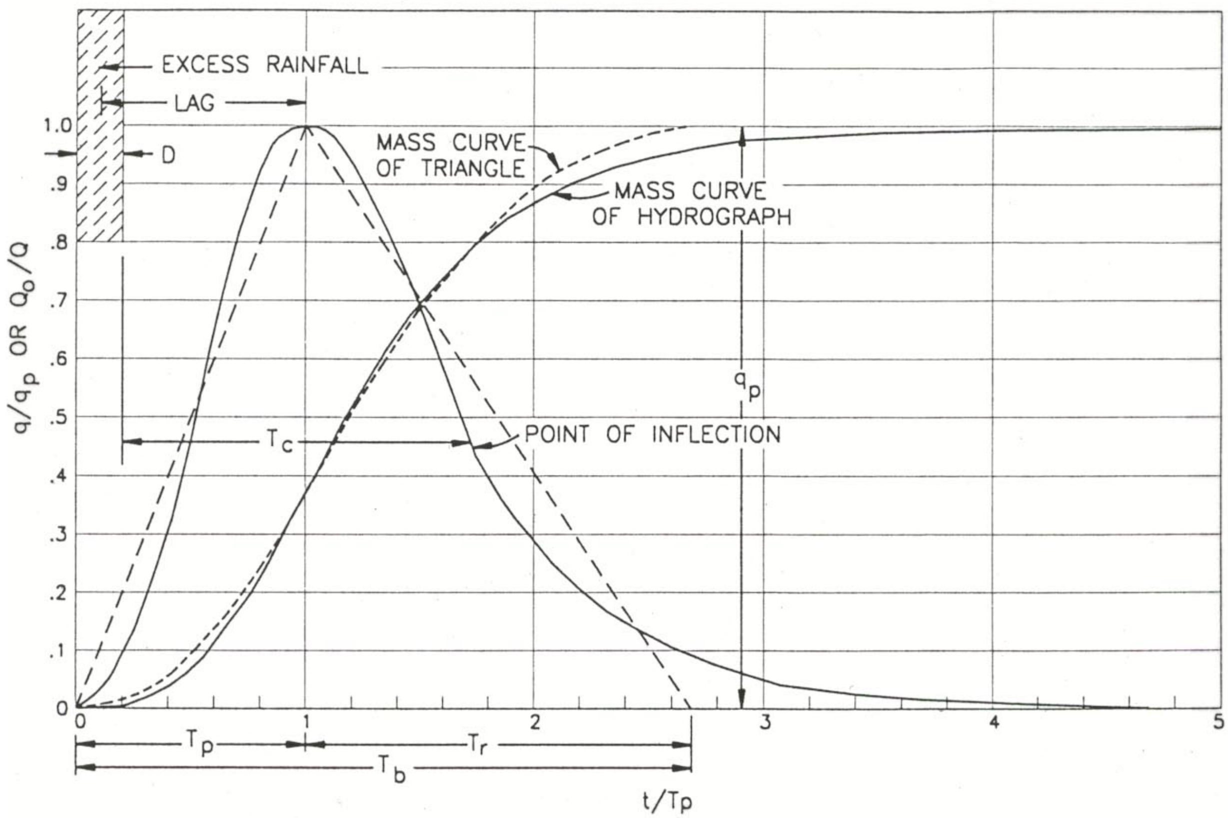


Figure 2-4 Dimensionless Curvilinear Unit Hydrograph and Equivalent Triangular Hydrograph

Area

Obtaining the highest degree of accuracy in hydrograph analysis requires the proper selection of homogeneous basin areas. Significant differences in land use within a given drainage basin must be addressed by dividing the basin area into subbasin areas of similar land use and/or runoff characteristics. For example, a drainage basin consisting of a concentrated residential area and a large forested area should be divided into two subbasin areas accordingly. Hydrographs should then be computed for each subbasin area and summed to form the total runoff hydrograph for the basin. Figure 2-5, on the next page, depicts the summation of UH flows for two excess rainfalls for the 4 hour time mark. This calculation must be made for the whole duration of all UH used.

To further enhance the accuracy of hydrograph analysis, all pervious and impervious areas within a given basin or subbasin should be analyzed separately, unless the impervious area discharges across pervious areas prior to entering the conveyance system. Separate analysis provides better results because there are typically significant differences between precipitation losses attributable to impervious and pervious soil covers; and there is typically a significant difference in travel times across paved areas and defined systems compared to unpaved areas and undefined systems.

This analysis may be done by computing separate hydrographs for each area and combining them to form the total runoff hydrograph. By analyzing pervious and impervious areas separately, the errors associated with averaging these areas are avoided and the true shape of the runoff hydrograph is better approximated.

Impervious areas not connected to a definable drainage system should be combined with the pervious areas. Situations where this may occur include: Roof down spouts draining to infiltration pockets, roof down spouts dispersed over lawn or landscaped areas, parking lots draining to infiltration facilities, and streets without curb and gutter or a definable ditch system.

Curve Number

The SCS method uses an index called the runoff curve number (CN) to represent the combined hydrologic effect of the soil type, land use, hydrologic condition of the soil cover, and the antecedent soil moisture. The CN indicates the runoff potential of soil which is not frozen. Higher CNs reflect a higher runoff potential.

The following conditions and limitations apply when using the SCS CN to estimate runoff:

- Understand that initial abstractions (I_a) include interception, initial infiltration, surface depression storage, and evapotransporation.
- Runoff from frozen ground cannot be estimated using this procedure.
- The CN method becomes less accurate when runoff is less than 0.5 inches. When this situation occurs, use of another procedure is recommended.
- This procedure applies only to direct runoff.

If the weighted CN is less than 30, use of another procedure is recommended.

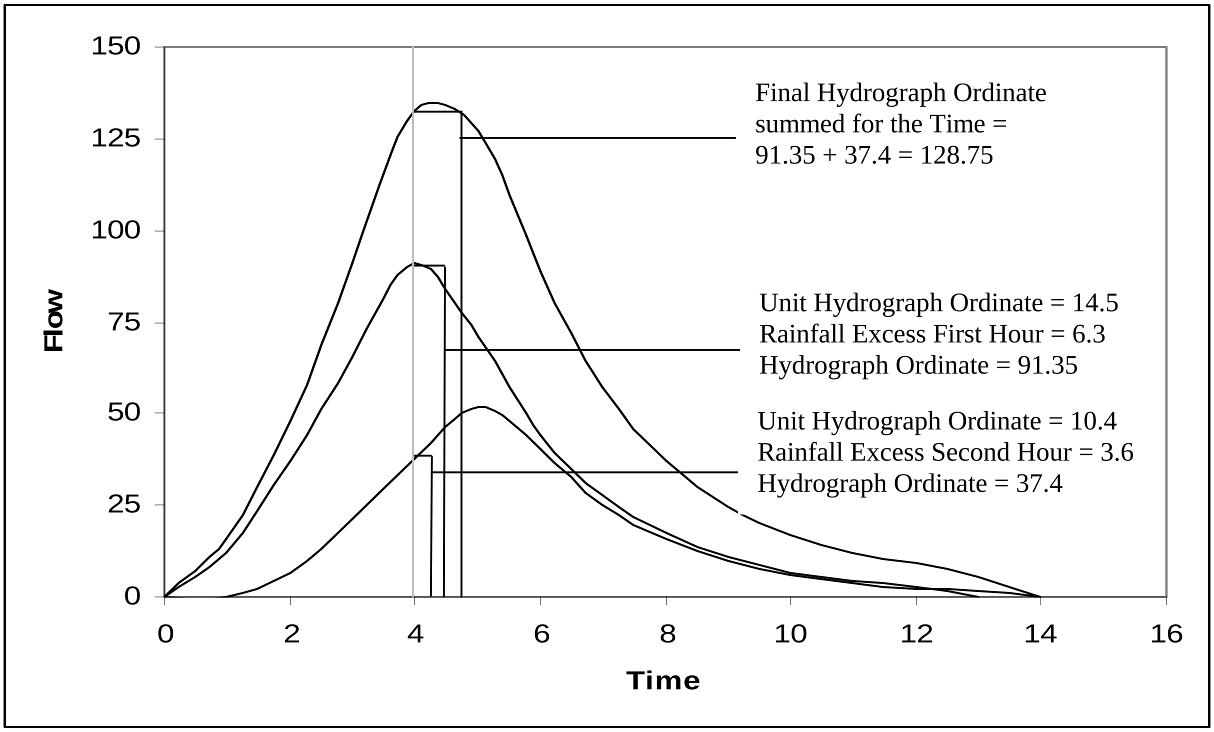


Figure 2-5 Combination of Unit Hydrographs of Varying Rainfall Excess Depths

The SCS soil classification system consists of four Hydrologic Soil Groups (HSG) which are characterized as follows:

HSG A:	Low runoff potential. Deep sand, deep loess, and aggregated silts.
HSG B:	Moderate runoff potential. Shallow loess and sandy loam.
HSG C:	Moderate to high runoff potential. Clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay.
HSG D:	High runoff potential. Soils that swell significantly when wet, heavy plastic clays and certain saline soils.

The "Soil Survey of Cabarrus County, North Carolina", prepared by the USDA SCS provides a detailed description of the soils in Cabarrus County. This is generally the best means of identifying soil groups.

The SCS cover classification includes three factors: (1) land use, (2) treatment, and (3) hydrologic condition. Identified land uses include: fully developed urban areas, developing areas, and agricultural land. The land uses are subdivided by treatment practices that describe the type of cover (Table 2-10). The hydrologic condition reflects the level of land management which are given as poor, fair, and good. Table 2-11 contains a list of soil types and HSG in the City.

Antecedent moisture conditions have a significant effect on runoff potential (CN). The SCS has developed three antecedent moisture conditions, which are described below:

Condition I: Soils are dry but not to wilting point.

Condition II: Average conditions.

Condition III: Heavy or light rainfall and low temperatures have occurred within the 5 days; saturated soils.

Antecedent moisture condition (AMC) Condition II, represented in Tables 2-10 and 2-11, shall be used in all design calculations unless a different condition is specified by the City.

Curve Number Runoff Equation

The volume of runoff is dependent on the volume of precipitation and the volume of storage available for retention. Table 2.1 from Section 2.1.2 has been reproduced again in this section.

Approximate 24-hour Rainfall Volumes for Concord, NC	
Return Interval	**24-hour Volume (inches)**
1-year	2.9
2-year	3.5
5-year	4.4
10-year	5.1
25-year	6.0
50-year	6.8
100-year	7.6

The runoff equation based on this concept is:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \tag{2-9}$$

where:

Q	=	runoff, in inches
P	=	rainfall, in inches
S	=	potential maximum retention after runoff begins, in inches
I_a	=	initial abstraction, in inches

Initial abstraction (I_a) includes all losses before runoff begins. These losses include interception, initial infiltration, surface depression storage, and evapotransporation,. Initial abstraction is based on the following empirical relationship that shall be used for all design calculations unless otherwise directed by the City.

$$I_a = 0.2S \tag{2-10}$$

Substituting $I_a = 0.2S$ into Equation 2-9:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \tag{2-11}$$

Potential maximum retention (S) is related to the soil and cover conditions of the watershed through the CN:

$$S = \frac{1000}{CN} - 10 \tag{2-12}$$

The precipitation excess for a storm can also be obtained using the graphical solution shown on Figure 2-6. For example, a precipitation of 5 inches with a curve number of 80 generates 2.9 inches of runoff.

TABLE 2-10
RUNOFF CURVE NUMBER FOR URBAN AREAS AND OTHER AGRICULTURAL LANDS[1]

Cover Description Cover Type and Hydrologic Condition	Average Percent Impervious Area[2]	Curve Numbers for Hydrologic Soil Group			
		A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]					
Poor Condition (grass cover <50 percent)		68	79	86	89
Fair Condition (grass cover 50 to 75 percent)		49	69	79	84
Good Condition (grass cover >75 percent)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc., (excluding right-of-ways)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-ways)		98	98	98	98
Paved; open ditches (including right-of-ways)		83	89	92	93
Gravel (including right-of-ways)		76	85	89	91
Dirt (including right-of-ways)		72	82	87	89
Urban districts:					
Commercial and business	85	89	92	94	95
Industrialf	72	81	88	91	93
Residential districts by average lot size:					
RE - maximum of 1 dwelling per acre	20	51	68	79	84
RL - maximum of 2 dwellings per acre	25	54	70	80	85
RM-1 - maximum of 3 dwellings per acre	30	57	72	81	86
RM-2 - maximum of 4 dwellings per acre	38	61	75	83	87
RV - maximum of 8 dwelling/acre	65	77	85	90	92
RC - maximum of 15 dwelling/acre	80	86	90	93	96
Developing urban areas and agricultural land					
Newly graded areas (pervious areas only, no vegetation)[4]		77	86	91	94
Pasture, grassland, or range – continuous forage for grazing[5]	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 elements[6]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	[7]30	48	65	73
Woods-grass combination (orchard or tree farm)[8]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods[9]	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
Source: SCS, Urban Hydrology for Small Watersheds, Technical Release No. 55.					

Notes for Table 2-10

[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. CNs for other combinations of conditions may be computed by using Figures 2-7 or 2-8 located in this Article.

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

[4] Composite CNs to use for the design of temporary measures during grading and construction should be computed using Figures 2-7 or 2-8, located in this Article based on the degrees of development (impervious area percentage) and the CNs for the newly graded pervious area.

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

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

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

[8] CNs shown was computed for areas with 50 percent woods and 50 percent grass (pasture) cover. Other combinations of conditions may be computed from the CNs for woods and pastures.

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

TABLE 2-11
SOIL TYPES IN CONCORD, NC

Soil Name	Symbol	HSG*
Altavista	AaB	C
Appling	ApB	B
Armenia	Ar	D
Badin	Ba	C
Cecil	Cc	B
Cecil-Urban complex	Ce	B
Chewacla	Ch	C
Cullen	Cu	C
Coronaca	Co	B
Enon	En	C
Enon-Urban complex	Eo	C
Georgeville	Ge	B
Goldston	Go	C
Hiwassee	Hw	B
Iredell	Id	C/D
Mecklenburg	Me	C
Mecklenburg-Urban complex	Mk	C
Misenheimer	Ms	C
Pacolet	Pa	B
Pacolet-Udorthents complex	Pc	B
Poindexter	Po	B
Sedgefield	Sf	C
Udorthents	Ud	-
Urban land	Ur	-
Vance	Va	C
Wehadkee	We	D

*SCS Hydrologic Soil Group

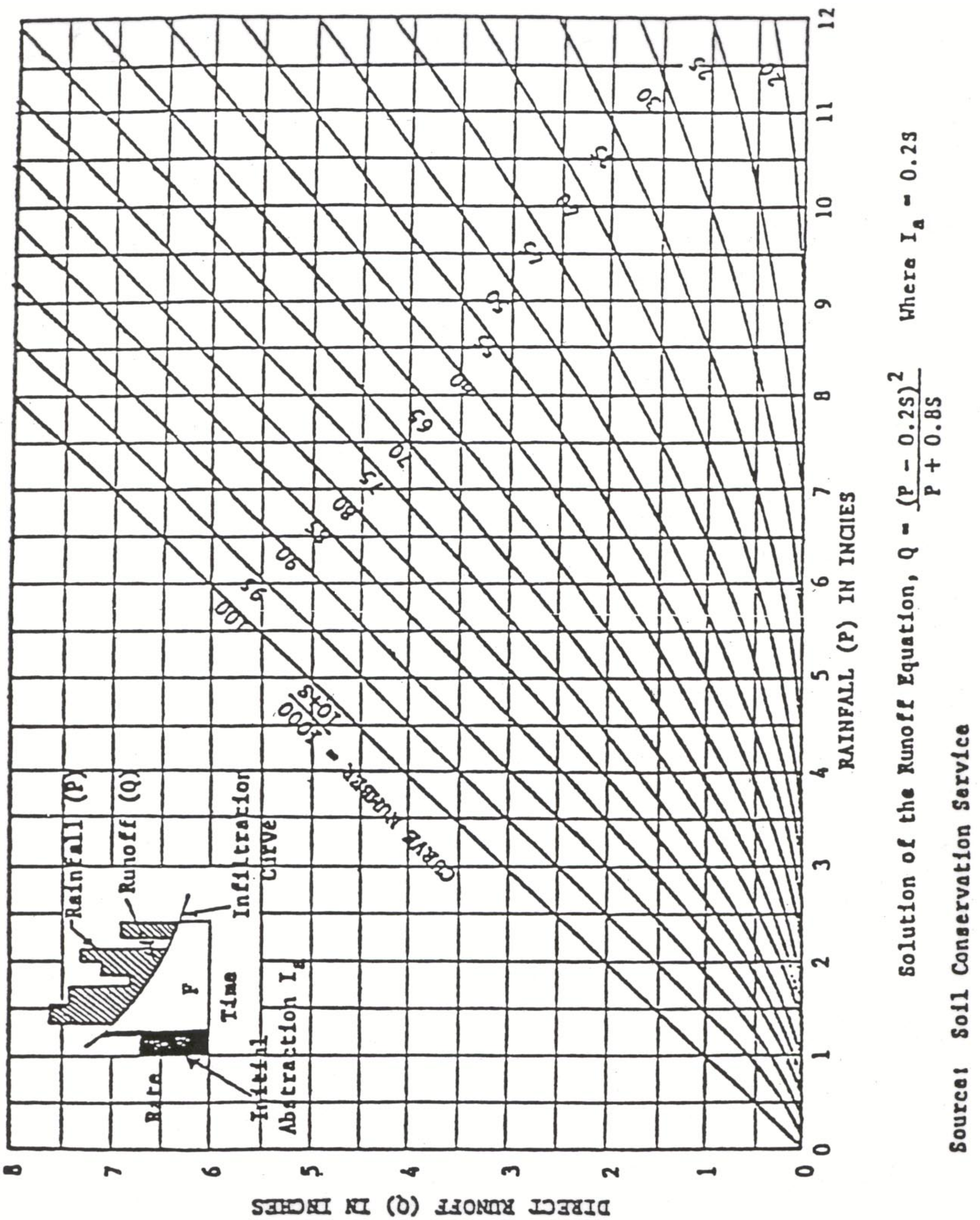


Figure 2-6 SCS Solution of the Runoff Equation

Example 2: **This example illustrates the procedure to use for computing a composite curve number and determining runoff for a basin with mixed land uses.**

Given: 300-acre basin.
70 percent of the basin is an Enon soil, Hydrologic Soil Group C.
30 percent of the watershed is a Cecil soil, Hydrologic Soil Group B.

200 acres of the watershed, including all HSG B soils and 100 acres of the HSG C soil is 1/2-acre residential lots. The residential lots have directly connected impervious areas of 25 percent and the pervious area is considered in good hydrologic soil condition.

Remaining area is open space and in good hydrologic condition.

Find: Determine the weighted CN for this basin and the volume of runoff from the 25-year 24-hour storm event.

Solution:

1. The 25-year 24-hour precipitation for the Concord area is 6.0 inches (Table 2-1).

2. Following the procedure outlined in the worksheet on the following page, the weighted CN for the basin can be determined and the volume of runoff can be calculated.

Example 2:

1. Runoff CN

Soil Name	Hydrologic Group	Cover Description	Curve Number[1] Table 2-10	Curve Number[1] Figure 2-7	Curve Number[1] Figure 2-8	Area (acres)	Product of CN*Area
Cecil	B	1/2 acre residential, 25 percent impervious, Good condition	70			90	6300
Enon	C	1/2 acre residential, 25 percent impervious, Good condition	80			100	8000
Enon	C	Open space, Good condition	74			110	8140
					Totals	300	22440

[1] Use only one CN source per line

$$CN_{weighted} = \frac{\sum (CN \times Area)}{\sum Area}$$

$$CN_{weighted} = \frac{22{,}440_{acres}}{300_{acres}} = 74.8 \qquad \text{Use CN} = 75$$

2. Runoff

Storm #1		
Frequency	25	year
Rainfall, P (24-hour)	6.0	inches
Runoff, Q	3.3	inches

(Use P and CN with Figure 2-6 and Eqns 2-11 and 2-12)

Example 3: **Example 3 is the same as Example 2, except the directly connected impervious area for the 1/2-acre residential lots differs from that presented in Table 2-10.**

Given: The 1/2-acre residential lots directly connected impervious areas are 35 percent, which differs from what is presented in Table 2-10.

Solution: Use Figure 2-7 to obtain a representative CN for the 1/2-acre residential lots. Using footnote 2 in Table 2-10, enter Figure 2-7 using CN = 61 for HSG B and CN - 74 for HSC C and obtain CN of 74 and 82, respectively.

Example 3:

1. Runoff CN

Soil Name	Hydrologic Group	Cover Description	Curve Number[1] Table 2-10	Curve Number[1] Figure 2-7	Curve Number[1] Figure 2-8	Area (acres)	Product of CN*Area
Cecil	B	1/2 acre residential, 35 percent impervious, Good condition		74		90	6660
Enon	C	1/2 acre residential, 35 percent impervious, Good condition		82		100	8200
Enon	C	Open space, Good condition	74			110	8140
					Totals	300	23000

[1] Use only one CN source per line

$$CN_{weighted} = \frac{23{,}000_{acres}}{300_{acres}} = 76.7 \qquad \text{Use CN} = 77$$

2. Runoff

Storm #1		
Frequency	25	year
Rainfall, P (24-hour)	6.0	inches
Runoff, Q	3.5	inches

(Use P and CN with Figure 2-6 and Eqns 2-11 and 2-12)

Example 4: Example 4 is the same as Example 2, except 50 percent of the impervious area is not directly connected for the 1/2-acre residential lots.

Solution: Use Figure 2-8, on page 2-33, to determine the representative CN for the 1/2-acre residential lots. This adjustment to impervious area generates a representative CN of 78.

Example 4:

1. Runoff CN

Soil Name	Hydrologic Group	Cover Description	Curve Number[1]			Area (acres)	Product of CN*Area
			Table 2-10	Figure 2-7	Figure 2-8		
Cecil	B	1/2 acre residential, 25 percent impervious, 50 percent connected impervious, Good condition	70			90	6300
Enon	C	1/2 acre residential, 25 percent impervious, 50 percent unconnected impervious, Good condition			78	100	7800
Enon	C	Open space, Good condition	74			110	8140
					Totals	300	22240

[1] Use only one CN source per line

$$CN_{weighted} = \frac{22{,}400\,acres}{300\,acres} = 74.7 \qquad \text{Use CN} = 75$$

2. Runoff

Storm #1		
Frequency	25	year
Rainfall, P (24-hour)	6.0	inches
Runoff, Q	3.3	inches

(Use P and CN with Figure 2-6 and Eqns 2-11 and 2-12)

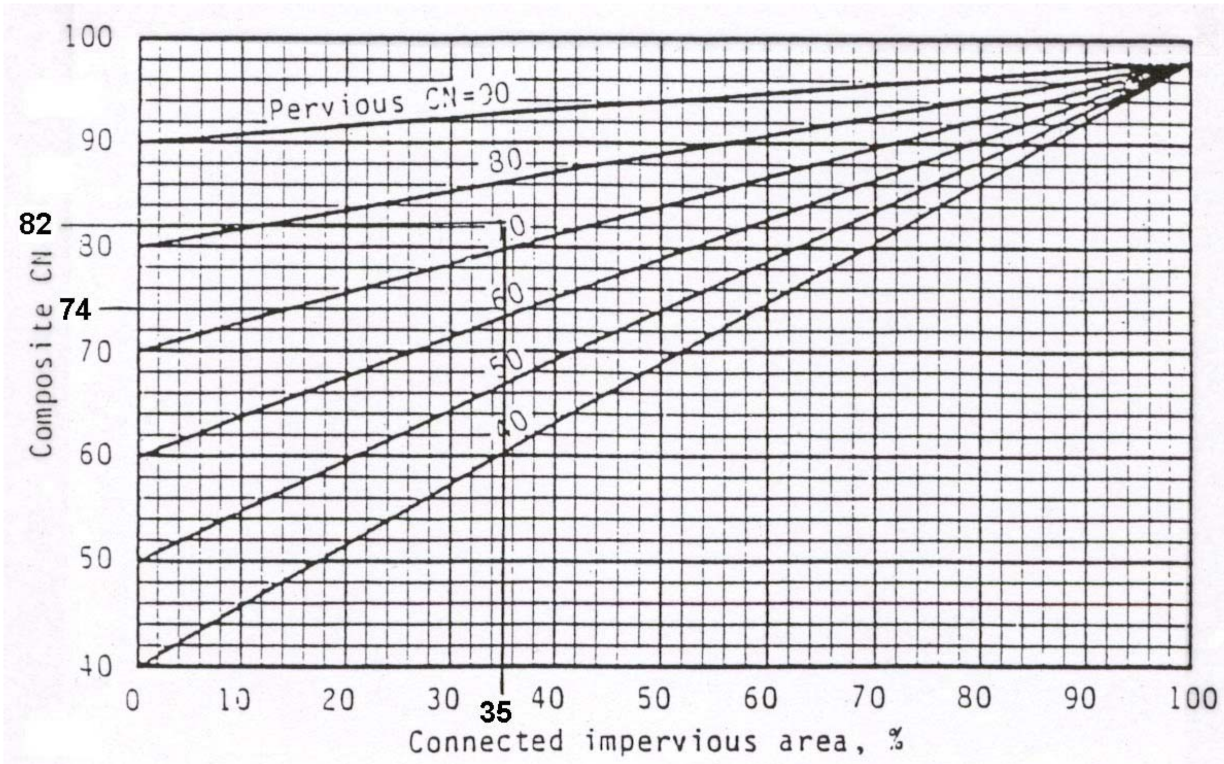


Figure 2-7 Composite Curve Number with Connected Impervious Areas

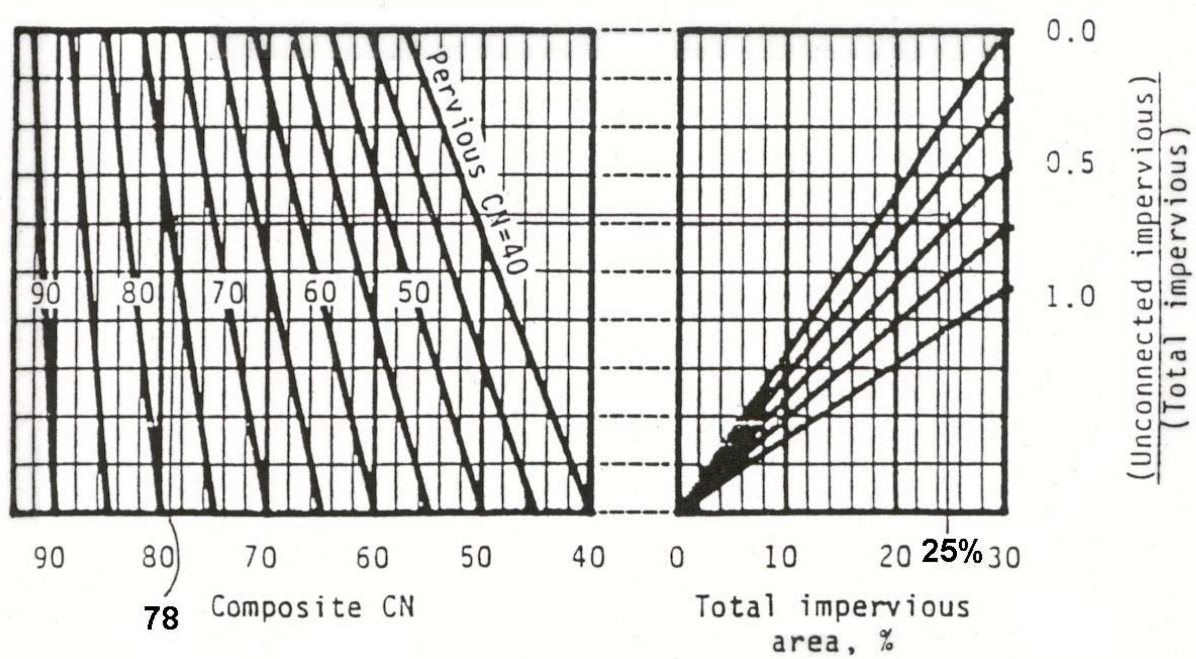


Figure 2-8 Composite Curve Number with Unconnected Impervious Areas and Total Impervious Area less than 30 percent

Travel Time and Time of Concentration

Travel time (T_t) is defined as the time it takes for water to travel from one location to another location within the watershed. Time of concentration (T_c) is defined as the time required for water to travel from the hydraulically most distant point in the watershed to the point of design. This method assumes that time of concentration can include travel components of overland flow (sheet flow), flow through shallow swales and gutters, and channel flow.

Limitations for time of concentration include overland flow not exceeding 300 feet for urban areas and 1,000 feet for rural areas and a minimum T_c of 0.1 hour.

Computation of Travel Time and Time of Concentration

Travel time and time of concentration are computed using the procedure presented in SCS TR55. This procedure determines the average velocity for each travel component which is converted to a travel time using the following equation:

$$T_t = \frac{L}{3600V} \tag{2-13}$$

where:

T_t = travel time, in hours
L = flow length, in feet
V = average velocity, in feet per second
3600 = conversion factor from seconds to hours

Time of Concentration (T_c):

$$T_c = T_{t\,\text{sheet flow}} + T_{t\,\text{shallow concentrated flow}} + T_{t\,\text{open channel flow}} + \ldots T_{tm} \tag{2-14}$$

where:

T_c = time of concentration, in hours
T_{tm} = number of travel time components

Sheet flow

Sheet flow represents flow of runoff over plane surfaces. The procedure used is Manning's Kinematic solution to determine travel time. This equation is identical to Equation 2-4 for the Rational Equation except the time is set in hours rather than minutes. Sheet flow continues to be limited to less than 300 feet for urban areas and 1,000 feet for rural areas.

The following assumptions apply when using this equation to calculate overland flow: 1) shallow steady uniform flow, 2) constant intensity of rainfall excess, 3) a rainfall duration of 24 hours and a storm frequency of 2 years, and 4) a minor effect of infiltration on travel time.

$$T_t = \frac{0.007(nL)^{0.8}}{P_2^{\,0.5} s^{0.4}} \tag{2-15}$$

where:

T_t	=	travel time, in hours
n	=	Manning's roughness coefficient, refer to Table 2-7
L	=	flow length, in feet
P_2	=	2-year, 24-hour rainfall, in inches, refer to Table 2-1
s	=	slope of hydraulic grade line (land slope), in feet per foot

Shallow Concentrated Flow

This method assumes that, after a maximum travel distance, sheet flow becomes shallow concentrated flow. Average velocity of shallow concentrated flow for both paved and unpaved surfaces is determined from Figure 2-9, on page 2-36. Shallow concentrated flow over paved surfaces is representative of gutter flow.

Channel Flow

This procedure assumes that open channel flow begins where channels are visible either from aerial photos that identify a channel or through field observations. Manning's equation is used to calculate velocity in the channels. This approach assumes that the channel is flowing bank full. Manning's equation is:

$$V = \frac{1.49 r^{2/3} s^{1/2}}{n} \tag{2-16}$$

where:

V	=	average velocity, in feet per second
r	=	hydraulic radius feet = area divided by wetted perimeter
s	=	slope of the hydraulic grade line (channel slope), in feet per foot
n	=	Manning's roughness coefficient for open channel flow
A	=	area of flow, in square feet
WP	=	perimeter of channel that is wet by water, in feet

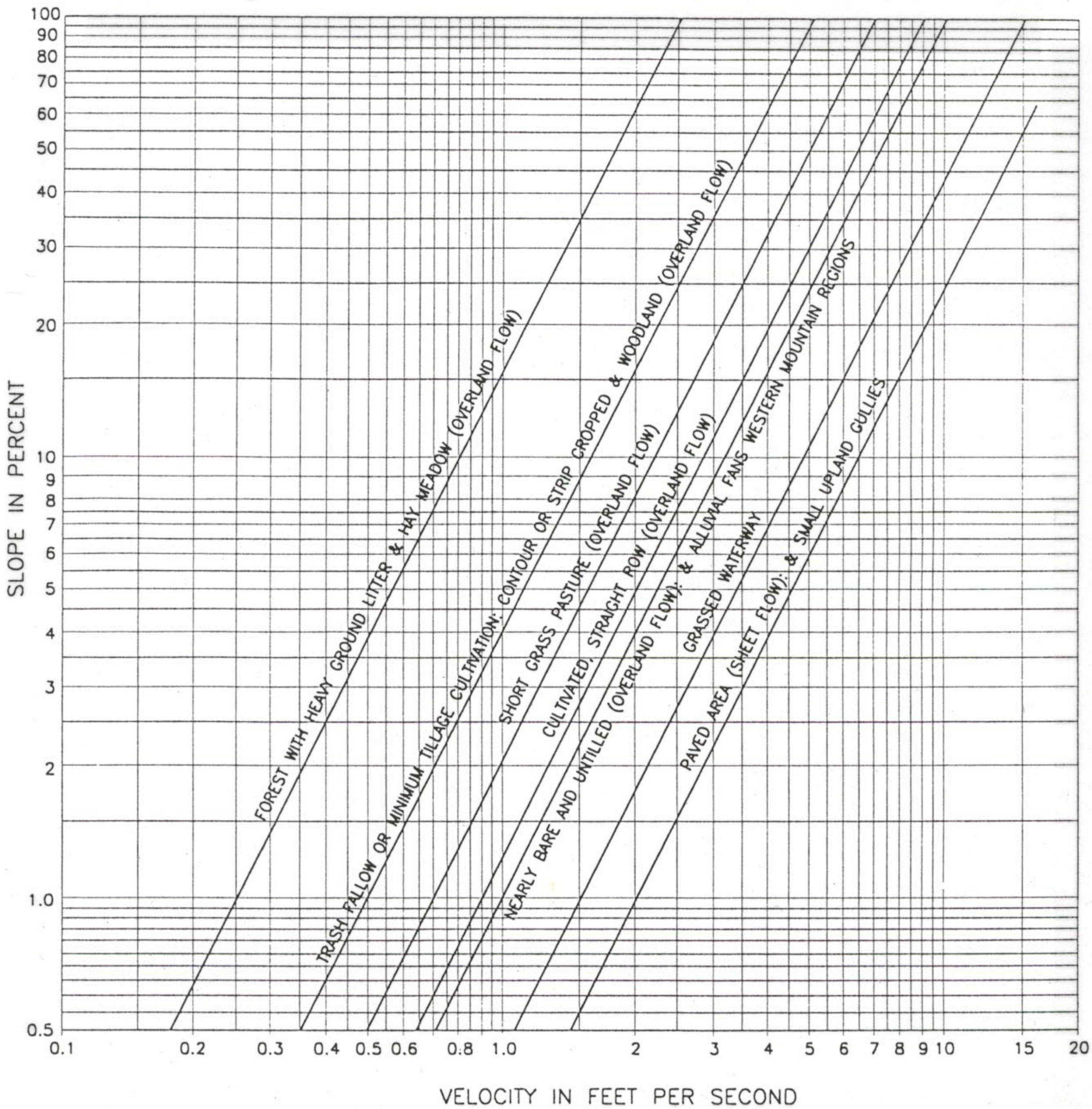
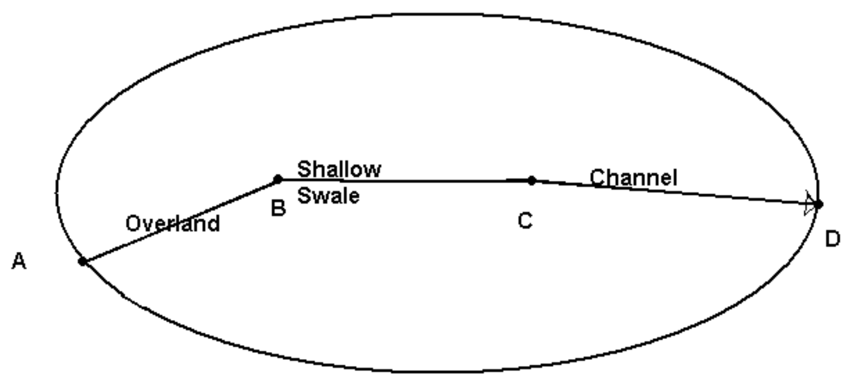


Figure 2-9 Average Velocities for Estimating Travel Time for Shallow Concentrated Flow

Example 5: Compute travel time and time of concentration using the SCS TR55 Methodology. Includes an overland flow component, shallow concentrated flow component, and an open channel flow component.

Find: Determine the time of concentration at Point D.

Schematic:



Given:

Segment	
A - B	sheet flow dense grass length (L) = 250 feet slope (s) = 0.005 feet per foot
B - C	shallow concentrated flow unpaved (local swale) length (L) = 800 feet slope (s) = 0.015 feet per foot
C - D	channel flow 4 foot base, 2 feet in depth, and 2:1 sideslopes length (L) = 2,000 feet slope (s) = 0.004 feet per foot Manning's roughness (n) = 0.05

Solution: The travel time for each flow component is computed and their summation represents the time of concentration to Point D.

Example 5:

Present Area

Sheet flow	Segment	A-B	
Surface description		Dense grass	
Manning's roughness coefficient, n		0.24	(Table 2-7)
Flow length, L (<300 feet)		250	feet
Two-year, 24-hour rainfall, P_2		3.5	in (Table 2-1)
Land slope, s		0.005	feet per foot
$T_t = \frac{0.007(nL)^{0.8}}{P_2^{0.5}s^{0.4}}$	$T_t =$	0.82	hour

Shallow Concentrated flow	Segment	B-C	
Surface description		unpaved	
Flow length, L		800	feet
Watercourse slope, s		0.015	feet per foot
Average velocity, V		2.0	feet per second (Figure 2-9)
$T_t = \frac{L}{3600V}$	$T_t =$	0.11	hour

Channel flow	Segment	C-D	(assumed full bank flow)
Cross-sectional flow area, a		16	$feet^2$
Wetted perimeter, p_w		12.94	feet
Hydraulic radius, r		1.24	feet
Channel slope, s		0.004	feet per foot
Manning's roughness coefficient, n		0.05	
$V = \frac{1.49r^{2/3}s^{1/2}}{n}$		2.17	feet per second
Flow length, L		2000	feet
$T_t = \frac{L}{3600V}$	$T_t =$	0.26	hour
Watershed $T_c = \sum T_t$	$T_c =$	1.19	hour

Use T_c of 1.2 hours

SECTION 3
STREET DRAINAGE

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3.1 INTRODUCTION

Roadways or streets in the City of Concord (City) serve an important and necessary drainage service even though their primary function is the movement of traffic. However, good planning of streets can substantially help in reducing the size, and sometimes reducing the extent, of a storm drainage system in newly urbanized areas. Traffic and drainage uses are compatible up to a point, beyond which drainage must be secondary to traffic needs.

3.2 DESIGN CRITERIA

Design criteria for the collection and transport of runoff on public streets are based on a reasonable frequency of traffic interference. That is, depending on the street classification, certain traffic lanes can be fully or partially inundated during the design storm return period. Higher frequency storms which occur more often will produce less runoff and will not inundate the entire street.

Planning and design for urban storm runoff must be considered from the viewpoint of both the minor and major storm occurrences. The objective of major storm runoff planning and design is to eliminate major damage and loss of life. The minor storm drainage system is necessary to eliminate inconvenience and high street maintenance costs, and reduce frequently recurring minor damage. Table 3-1 outlines the design criteria for the minor and major storm.

TABLE 3-1
DESIGN CRITERIA FOR STREET DRAINAGE

Minor Storm	Major Storm
4 inches/hour rainfall intensity	100-year storm

3.2.1 <u>Street Capacity for Minor Storm</u>

Determination of street capacity for the minor storm shall be based upon pavement encroachment or spread, the commonly referred terminology. The pavement encroachment for the design storm shall be limited as set forth in Section 1.3 Design Policy and summarized on the next page in Tables 3-2 and 3-3. When the maximum encroachment or cross street flow depth is reached, a separate storm drainage system, additional inlets, or additional storm drainage capacity shall be provided and designed on the basis of the minor storm. Table 3-2 outlines the minor storm criteria for allowable street use for encroachment.

TABLE 3-2 MINOR STORM RUNOFF ALLOWABLE STREET USE	
Street Classification	**Maximum Pavement Encroachment**
Alley and Local Street	Flow may spread to crown of street. Velocities may not exceed 8 feet per second. No curb overtopping.
Collector	Flow spread must leave at least one half of a travel lane in each direction free of water. Velocities may not exceed 8 feet per second. No curb overtopping.
Thoroughfare	Flow spread must leave at least one lane in each direction free of water. Velocities may not exceed 8 feet per second. No curb overtopping.
Freeway	No encroachment is allowed on any travel lanes.

Cross flows, those flows that pass over the crown of a street, also need to be regulated for traffic interference and public safety concern. Table 3-3 outlines the minor storm criteria for cross flows.

TABLE 3-3 MINOR STORM RUNOFF ALLOWABLE CROSS STREET FLOW	
Street Classification	**Maximum Cross Flow Depth**
Alley and Local Street	None
Collector	None
Thoroughfare	None
Freeway	None

Calculating Gutter Capacity

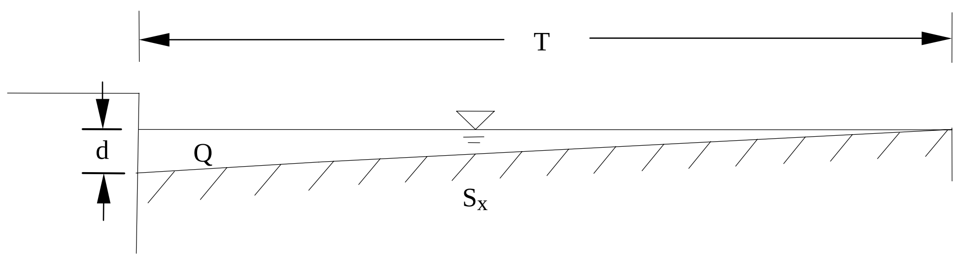
There are a variety of gutter sections available for use in street drainage today. The geometry of these gutters may be determined by the need for additional carrying capacity or the ability for safe passage of pedestrian traffic. The gutter may have a straight transverse slope, a composite transverse slope, or transverse slopes composed of two straight lines (V-shape). When the allowable encroachment has been determined, the gutter (that portion of the street used to convey runoff) capacity can be computed using a modified version of Manning's formula.

A straight transverse slope section or standard gutter usually resembles a triangular shape with the curb forming the vertical leg of the triangle. For standard gutters with identical cross slopes for the street and gutter, a modified version of the Manning's equation has been developed. Manning's equation is modified to better describe the hydraulic radius of a gutter section. The equation in terms of cross slope and width of flow at the curb is:

$$Q = \left[\frac{0.56}{n}\right] S_x^{1.67} s^{0.5} T^{2.67} \qquad (3\text{-}1)$$

where:

Q	=	discharge, in cubic feet per second
n	=	Manning's roughness coefficient
S_x	=	cross slope of the street, in feet per foot
s	=	longitudinal slope, in feet per foot
T	=	width of flow (spread), in feet



The resistance of the curb face is neglected in the equation since the resistance is negligible when the cross slope is 10 percent or less.

The depth of flow in a standard gutter can be calculated by the following equation.

$$d = TS_x \qquad (3\text{-}2)$$

where:

d	=	depth of flow at curb or deepest point, in feet
T	=	width of flow (spread), in feet
S_x	=	cross slope of the street, in feet per foot

Manning's "n" values for different street and gutter roughness conditions are presented in Table 3-4, on the next page.

To increase the capacity of a gutter, the gutter cross slope may be steepened with respect to the cross slope of the street. These gutters are termed composite gutters. For composite gutters, the capacity is determined for the depressed section and the area above the depressed section separately. The following series of equations demonstrates the calculations required for determining the capacity for these types of gutters.

$$Q = Q_s + Q_w \tag{3-3}$$

where:

Q	=	discharge, in cubic feet per second
Q_s	=	discharge in section above discharge, in cubic feet per second
Q_w	=	discharge in depressed section, in cubic feet per second

TABLE 3-4
MANNING'S ROUGHNESS COEFFICIENTS FOR STREETS AND GUTTERS

Surface Type	"n" Value
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
Brick	0.016

Equation 3-3, above, has been simplified to the following equation in terms of cross slopes and widths based on the relationship of frontal flow (gutter section) to side flow (street section).

$$Q = \frac{Q_s}{1 - E_0} \tag{3-4}$$

where:

$$E_0 = \frac{1}{1 + \dfrac{S_w/S_x}{\left[1 + \dfrac{S_w/S_x}{(T/W) - 1}\right]^{2.67} - 1}} \tag{3-5}$$

S_w	=	cross slope of depressed section, in feet per foot
S_x	=	cross slope of street, in feet per foot
W	=	width of depressed section, feet
T	=	width of flow (spread), in feet

If the cross slope of the depressed section is not known or provided it may be calculated from the following equation.

$$S_w = S_x + \frac{a}{W} \tag{3-6}$$

Flow depth at the curb can be calculated from the following equation using the spread, cross slope of the street, and depth of the depressed section.

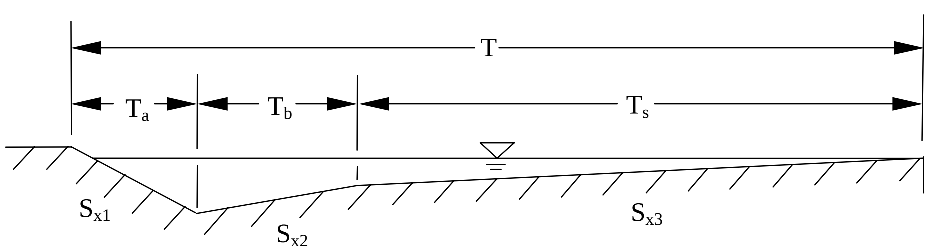
$$d = a + TS_x \tag{3-7}$$

Gutters with transverse slopes composed of two straight lines (V-shape) are commonly referred to as valley gutters. The valley may also be composed of a smooth parabolic cross section. The smooth parabolic gutters are commonly referred to as roll back or mountable types. The capacity for all valley type gutters is approximated by the same methodology. For valley gutters, an adjusted cross slope of the gutter section is calculated by the following equation.

$$S_a = \frac{S_{x1} S_{x2}}{S_{x1} + S_{x2}} \tag{3-8}$$

where:

S_a	=	adjusted cross slope, in feet per foot
S_{x1}	=	cross slope of interior gutter side, in feet per foot
S_{x2}	=	cross slope of exterior gutter side, in feet per foot



Use Equation 3-1 and the adjusted cross slope from Equation 3-8 as S_w to determine the capacity of the valley gutter. If it is necessary to determine the spread associated with a given flow capacity, use Equations 3-1 and 3-8 to calculate a preliminary spread, T', first. If T' is less than T_a+T_b, then T is equal to T' and the spread is being conveyed only in the valley gutter section. If T' is greater than T_a+T_b, then part of the spread is also being conveyed in the street section. To determine the final spread, T, that is also being partially conveyed in the street, requires an assumption of T' and iteration between Equations 3-1 and 3-8 until T' converges. This process is documented in Example 2.

Calculating Velocity In Gutter

The average velocity of flow in the gutter can be calculated by modifying Manning's Equation. Equation 3-9 can be used to approximate velocity in standard gutter sections. The equation in terms of cross slope and width of flow in the pavement assumes that the discharge in the gutter varies uniformly between the sections and is:

$$V = \frac{1.12}{n} s^{0.5} S_x^{0.67} T^{0.67} \tag{3-9}$$

where:

V	=	velocity, in feet per second
s	=	longitudinal slope, in feet per foot
S_x	=	cross slope, in feet per foot
T	=	width of flow (spread), in feet
n	=	Manning's roughness coefficient

If a channel has zero flow at the upstream end, the average velocity occurs at the point where spread is equal to 65 percent of the maximum spread. For channel sections with discharges greater than zero at the upstream section, the spread at average velocity, T_a, is given by Table 3-5, on the next page. In Table 3-5, T_1 is the spread at the upstream section and T_2 is the spread at the downstream section. The average spread is then used in Equation 3-8.

TABLE 3-5
SPREAD AT AVERAGE VELOCITY IN A TRIANGULAR GUTTER SECTION

$\frac{T_1}{T_2}$	$\frac{T_a}{T_2}$
0.0	0.65
0.1	0.66
0.2	0.68
0.3	0.70
0.4	0.74
0.5	0.77
0.6	0.82
0.7	0.86
0.8	0.90

Average velocity calculations for other cross sections are best determined by software packages. The Federal Highway Administration's HY-22 and Visual Urban (HY-22 for Windows) are free, downloadable software packages that perform this and other calculations from this Section.

Example 1: **Gutter Carrying Capacity**

Given: 6-inch vertical curb
2-feet-wide by 0.1-feet-deep gutter
2 percent pavement crown slope
Residential collector without on-street parking (24-foot pavement width)
1 percent longitudinal street grade
Assume n = 0.016

Find: Capacity in gutter for minor storm

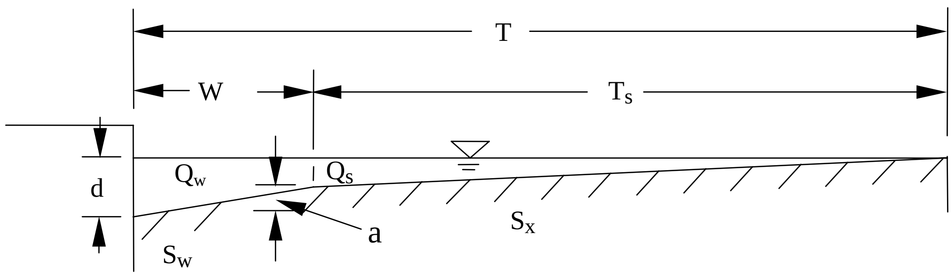
Solution:

1) Determine allowable pavement encroachment.

From Table 3-2, a collector street has an allowable encroachment of half a travel lane in a single direction. Therefore, encroachment may spread to a width of 8 feet. Half of one travel lane in a single direction is 6 feet wide plus the 2 feet provided by the gutter section.

2) Calculate capacity for the gutter.

The gutter is a composite gutter where the capacity for the depressed section and street section will be determined separately.



$T = 8$ feet	$d = 0.5$ feet	$s = 0.01$ feet per foot
$T_s = 6$ feet	$a = 0.1$ feet	$S_X = 0.02$ feet per foot
$W = 2$ feet		

Calculate the total capacity, Q, with Equation 3-4 which sums the capacity of both the street and gutter. The capacity of the street section is determined by Equation 3-1. The capacity of the depressed gutter section is determined by Equation 3-5 but the gutter slope, S_w, must be calculated from Equation 3-6 since it is not known.

a) Find Q_s: Substitute T_s for T in Equation 3-1 and solve for Q.

$$Q = \left[\frac{0.56}{n}\right] S_x^{1.67} s^{0.5} T^{2.67}$$

$$Q_s = \left[\frac{0.56}{0.016}\right] (0.02 \textit{ feet per foot})^{1.67} (0.01 \textit{ feet per foot})^{0.5} (6 \textit{ feet})^{2.67}$$

$$Q_s = 0.61 \textit{ cubic feet per second}$$

b) Find S_w: Solve Equation 3-6 for S_w.

$$S_w = S_x + \frac{a}{W}$$

$$S_w = 0.02 \textit{ feet per foot} + \frac{0.1 \textit{ feet}}{2 \textit{ feet}}$$

$$S_w = 0.07 \ \textit{feet per foot}$$

c) Find E_0: Solve Equation 3-5 for E_0.

$$E_0 = \frac{1}{1 + \dfrac{S_w / S_x}{\left[1 + \dfrac{S_w / S_x}{(T/W) - 1}\right]^{2.67} - 1}}$$

$$E_0 = \frac{1}{1 + \dfrac{(0.07 \ \textit{feet per foot}) / 0.02 \ (\textit{feet per foot})}{\left[1 + \dfrac{(0.07 \ \textit{feet per foot}) / 0.02 \ (\textit{feet per foot})}{(8 \ \textit{feet} \text{ by } 2 \ \textit{feet}) - 1}\right]^{2.67} - 1}}$$

$$E_0 = 0.66$$

d) Find Q: Solve Equation 3-4 for Q.

$$Q = \frac{Q_s}{1 - E_0}$$

$$Q = \frac{0.61 \ \textit{cubic feet per second}}{1 - 0.66}$$

$$Q_s = 1.8 \ \textit{cubic feet per second}$$

Example 2: Spread in a Valley Gutter Section

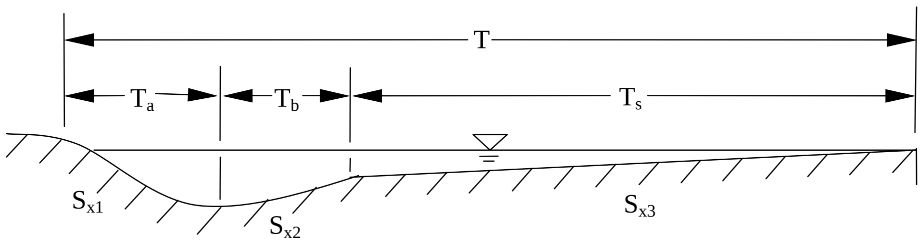
Given:
6-inch vertical curb
2.5 feet-wide gutter
2 percent pavement crown slope
Mountable curb with 3 percent gutter slope
Capacity of 10 cubic feet per second
1 percent street grade
Assume n = 0.016

Find: Spread for the given capacity.

Solution:

1) Calculate spread for the given capacity.

The gutter is a valley gutter where the capacity is distributed among the valley and street section.



s	=	0.01 feet per foot	S_{x1}	=	1 feet per foot	T_a	=	0.5 feet
S_x	=	0.02 feet per foot	S_{x2}	=	0.03 feet per foot	T_b	=	2 feet
d	=	0.5 feet						

Calculate the spread with Equation 3-1 by first calculating intermediate slope from Equation 3-8. Compare spread from intermediate slope calculation with valley section width, W. If spread is less than valley section width, then the spread is entirely contained within the gutter. If spread is greater than valley section width, then the spread is partially being conveyed along the street.

a) Find S_a: Solve Equation 3-8 for S_a.

$$S_a = \frac{(1 \textit{ feet per foot})(0.03 \textit{ feet per foot})}{1 \textit{ feet per foot} + 0.03 \textit{ feet per foot}}$$

$S_a = 0.029$ *feet per foot*

b) Find T': Rearrange Equation 3-1 and solve for T'.

$$T' = \left[\frac{Qn}{0.56 S_a^{1.67} s^{0.5}}\right]^{0.375}$$

$$T' = \left[\frac{(1 \text{cubic } \textit{feet per second})(0.016)}{(0.56)(0.029 \textit{ feet per foot})^{1.67}(0.01 \textit{ feet per foot})^{0.5}}\right]^{0.375}$$

$T' = 5.71 \ \textit{feet}$

$T' > T_a + T_b = 2.5 \ \textit{feet}$

c) Find S_a: Calculate a new adjusted slope by assuming a spread and weighting the slopes with S_{x2} and S_x. Assume a spread of 7 feet, weight slopes, and solve Equation 3-8 for S_a.

$$S = \frac{(2 \ \textit{feet})(0.03 \ \textit{feet per foot}) + (5 \ \textit{feet})(0.02 \ \textit{feet per foot})}{7 \ \textit{feet}}$$

$S = 0.0229 \ \textit{feet per foot}$

$$S_a = \frac{(1 \ \textit{feet per foot})(0.0229 \ \textit{feet per foot})}{1 \ \textit{feet per foot} + 0.0229 \ \textit{feet per foot}}$$

$S_a = 0.0224 \ \textit{feet per foot}$

d) Find T': Substitute S_a and into Equation 3.1 for S_x and solve for T'.

$$T' = \left[\frac{(1 \textit{cubic foot per second})(0.016)}{(0.56)(0.0224 \ \textit{feet per foot})^{1.67} (0.01 \ \textit{feet per foot})^{0.5}} \right]^{0.375}$$

$T' = 6.72 \ \textit{feet}$

e) Find S_a: Reiterate between Steps c) and d) until the spread, T' converges.

$$S = \frac{(2 \ \textit{feet per foot})(0.03 \ \textit{feet per foot}) + (4.72 \ \textit{feet per foot})(0.02 \ \textit{feet per foot})}{6.72 \ \textit{feet}}$$

$S = 0.0230 \ \textit{feet per foot}$

$$S_a = \frac{(1 \ \textit{feet per foot})(0.0230 \ \textit{feet per foot})}{1 \ \textit{feet per foot} + 0.0230 \ \textit{feet per foot}}$$

$S_a = 0.0225 \ \textit{feet per foot}$

f) Find T': Substitute S_a and into Equation 3.1 for S_x and solve for T'.

$$T' = \left[\frac{(1\,\textit{cubic feet per second})(0.016)}{(0.56)(0.0225\ \textit{feet per foot})^{1.67}(0.01\,\textit{feet per foot})^{0.5}}\right]^{0.375}$$

T' = 6.70 *feet* ≈ 6.72 *feet*

3.2.2 Street Capacity for Major Storm

Determination of the allowable capacity for the major storm shall be based upon allowable depth and inundated area. Excessive depths at the crown and gutter areas of the street may present hazardous conditions for emergency vehicles. When the maximum inundation depth or cross street flow depth is reached, a separate storm drainage system, additional inlets, or additional storm drainage capacity shall be provided and designed on the basis of the major storm. The allowable depth and inundated area for the major storm shall be limited as set forth in Section 1.3 Design Policy and summarized here in Tables 3-6 and 3-7. Table 3-6 outlines the major storm criteria for allowable street inundation.

TABLE 3-6
MAJOR STORM RUNOFF ALLOWABLE STREET INUNDATION

Street Classification	Allowable Depth and Inundated Areas
Alley and Local Street	The depth of water over the gutter flow line shall not exceed 12 inches. Velocities shall not exceed 8 feet per second.
Major Collector and Both Thoroughfares	The depth of water at the street crown shall not exceed 6 inches in order to allow operation of emergency vehicles. The depth of water over the gutter flow line shall not exceed 12 inches. Velocities shall not exceed 8 feet per second.
Freeway	No inundation is allowed.

Cross flows, flows that pass over the crown of a street, also need to be regulated for traffic interference and public safety concern. Table 3-7 outlines the major storm criteria for cross flows.

TABLE 3-7
MAJOR STORM RUNOFF ALLOWABLE CROSS STREET FLOW

Street Classification	Maximum Cross Flow Depth
Alley and Local Street	12 inches
Collector and Thoroughfare	6 inches or less over crown
Freeway	None

Calculating Gutter Capacity

When the allowable depth and inundated area is determined from Tables 3-6 and 3-7, the flow capacity shall be calculated using Equation 3-1 with an "n" value applicable to the actual boundary conditions encountered.

3.2.3 Ponding

The term "ponding" shall refer to areas where runoff is restricted to the street surface by sump inlets, street intersections, low points, intersections with drainage channels, or other reasons.

Minor Storm

Limitations for pavement encroachment by ponding for the minor storm shall be those presented in Tables 3-2 and 3-3. These limitations shall determine the allowable depth at inlets, gutter turnouts, culvert headwaters, and other hydraulic structures.

Major Storm

Limitations for depth and inundated area for major storms shall be those presented in Tables 3-6 and 3-7. These limitations shall determine the allowable depth at inlets, gutter turnouts, culvert headwaters, and other hydraulic structures.

3.3 INTERSECTION LAYOUT CRITERIA

The following design criteria are applicable at intersections of urban streets. Gutter capacity limitations covered in Section 3.2 Design Criteria shall apply along the street, while this section shall govern at the intersection.

3.3.1 Gutter Capacity, Minor Storm

Pavement Encroachment

Limitations at intersections for pavement encroachment shall be as given in Tables 3-2 and 3-3.

Intersections typically comprise streets of varying cross sections. The capacity of gutters on the intersecting streets or even in the transitioning area may create encroachment problems elsewhere. Therefore, the capacity of each gutter approaching an intersection shall be calculated based upon the most critical cross section as determined. Sections with substantial variation in longitudinal slopes shall be calculated on either of the following two criteria:

A. Flow Direction Change at Intersection

When the gutter flow must undergo a direction change at the intersection greater than 45 degrees, the slope used for calculating capacity shall be the effective gutter slope, defined as the average of the gutter slopes at 0 feet, 25 feet, and 50 feet from the geometric point of intersection of the two gutters (see Figure 3-1).

B. Flow Interception by Inlet

When gutter flow will be intercepted by an inlet on continuous grade at the intersection, gutter slope shall be utilized for calculations. Under this condition, the points for averaging shall be 0 feet, 25 feet, and 50 feet upstream from the inlet (see Figure 3-1).

3.3.2 Gutter Capacity, Major Storm

Allowable Depth and Inundated Area

The allowable depth and inundated area for the major storm shall be limited as set forth in Tables 3-6 and 3-7.

Capacity

The carrying capacity of each gutter approaching an intersection shall be calculated, based upon the most critical cross section.

The grade used for calculating capacity shall be as covered in Gutter Capacity, Design Storm.

3.3.3 Ponding

Minor Storm

The allowable pavement encroachment for the minor storm shall be as presented in Tables 3-2 and 3-3.

Major Storm

The allowable depth and inundated area for the major storm shall be as presented in Tables 3-6 and 3-7.

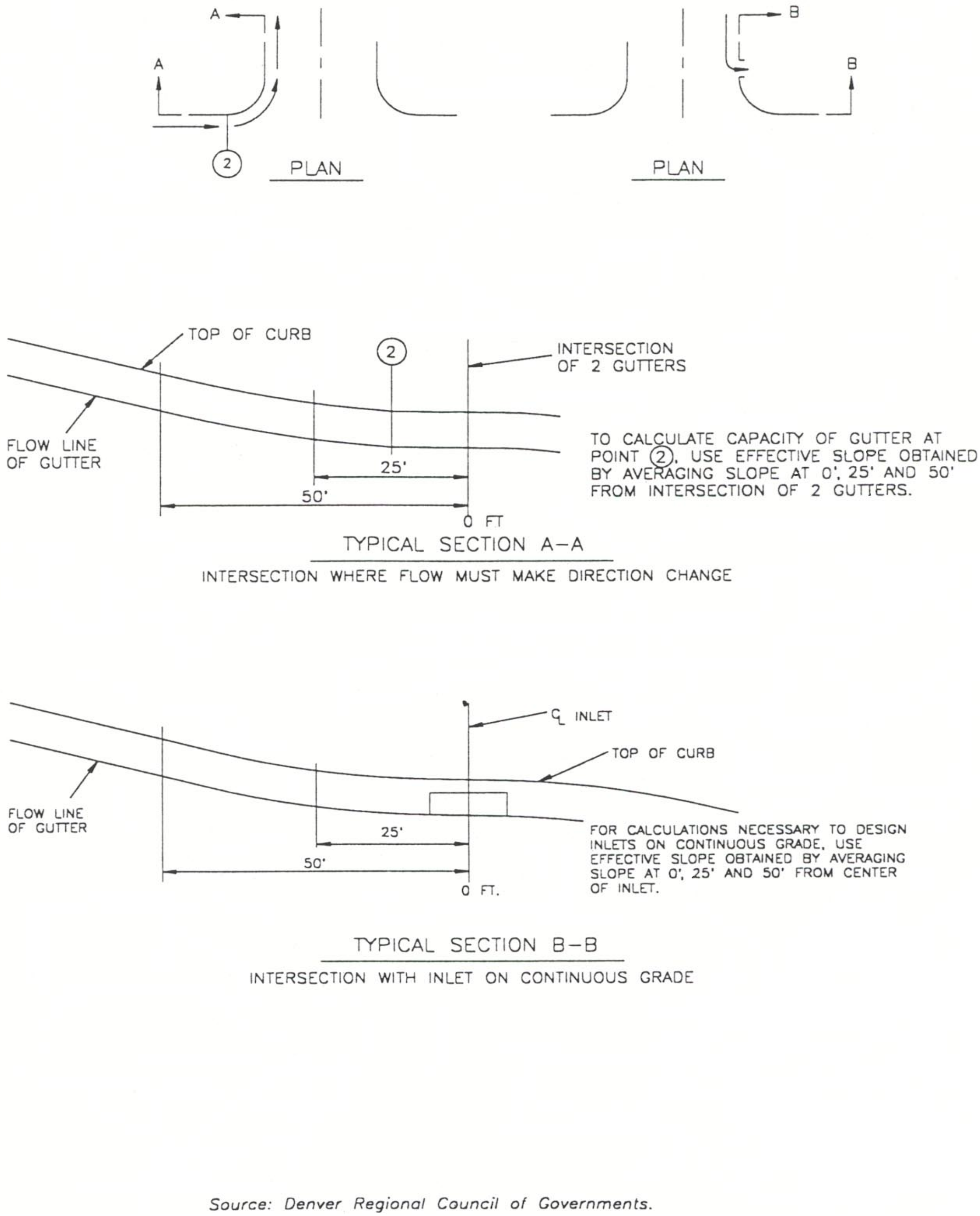


Figure 3-1 Intersection Drainage

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SECTION 4
STORM INLETS

Page

TABLE

FIGURE

4.1 INTRODUCTION

The purpose of this section is to present the significance of the hydraulic elements of storm inlets and their appurtenances to a storm drainage system. The hydraulic capacity of a gutter inlet depends upon its geometry and upon the characteristics of the gutter flow. The inlet capacity governs both the rate of water removed from the gutter and the amount of water that can enter the storm sewer system. Many costly storm sewers flow at less than design capacity because the storm runoff cannot get into the sewers. Inadequate inlet capacity or poor inlet location may cause flooding on the traveled way and create a safety hazard or interrupt traffic. Limitations on depth of street inundation are presented in Section 1.3 Design Policy and Section 3 Street Drainage.

4.2 INLET TYPES

Gutter inlets can be divided into the following three major classes, each with many variations: (1) curb-opening inlets; (2) grate inlets; and, (3) combination inlets. The City of Concord (City) has approved the use of specific inlets, as identified in Figure 4-1. The placement of two inlets side by side is referred to as double inlets and is acceptable.

Brief descriptions of the allowable inlet types follows and is highlighted in Figure 4-1:

A. Grate inlets. These inlets consist of an opening in the storm sewer system covered by one or more grate. Their use within the City is limited to depressed areas outside the curb and gutter section, such as within roadway medians and ditches and along storm sewers connecting other curb and gutter systems. Grate inlets situated in these conditions are typically referred to as drop inlets.

B. Combination inlets. These units consist of both a curb-opening and a grate inlet acting as a single unit.

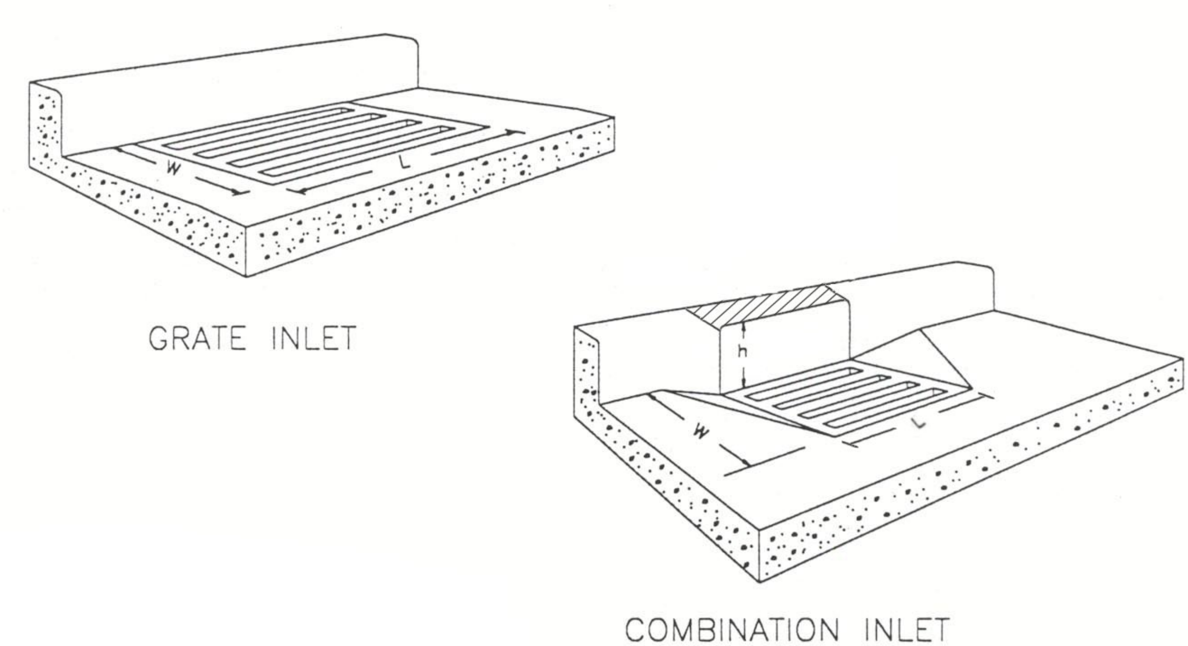
In mathematical form, inlet efficiency, E, is defined by Equation 4-1:

$$E = \frac{Q_i}{Q} \tag{4-1}$$

where:

E	=	efficiency of inlet
Q_i	=	intercepted flow by inlet, in cubic feet per second
Q	=	total gutter flow, in cubic feet per second

The discharge that bypasses the inlet, Q_c, is termed carry-over or bypass flow. The intercepted flow of all inlet configurations increases with increasing gutter flow; however, inlet efficiency generally decreases with increasing gutter flow.



Administration, HEC No. 12, Drainage of Highway Pavements.

Figure 4-1 Allowable Inlet Types

Factors affecting gutter flow also affect inlet interception capacity. The interception capacity of a grate inlet depends on the amount of water flowing over the grate, the size and configuration of the grate, and the velocity of flow in the gutter. The efficiency of a grate is dependent on the same factors and total flow in the gutter.

The interception capacity of a combination inlet consisting of a grate and curb-opening does not differ materially from that of a grate. Interception capacity and efficiency are dependent on the same factors which affect grate capacity and efficiency. However, as the depth of water in the gutter increases, the curb-opening greatly increases the interception capacity. Curb-opening inlet interception capacity and efficiency are increased even further by the use of a gutter depression at the curb-opening or a depressed gutter which concentrates runoff closer to the curb line.

4.3 INLET CLOGGING

All types of inlets are subject to clogging. Attempts to simulate clogging tendencies in the laboratory have not been successful, except to demonstrate the importance of parallel bar spacing in debris handling efficiency. Grates with wider spacings of longitudinal bars pass debris more efficiently. Problems with clogging are largely local since the amount of debris varies significantly from one neighborhood to another. Some neighborhoods may contend with only a small amount of debris while others experience extensive clogging of drainage inlets. Although, the City has an ongoing maintenance program for inlet clearing and cleaning, clogging should always be considered in the design of storm inlets. The total length or area of the inlet should be adjusted to account for clogging.

Common practice has been to apply a standard reduction percentage, such as 50 percent for grates and 25 percent curb openings, to the length or area of an inlet. This; however, presents a problem of excessive placement of inlets due to the reduction in capacity of all inlets, when typically only the initial inlet in a series becomes clogged. Equation 4-2 provides a relationship for distributing the effect of clogging over a series of inlets.

$$C = \frac{C_o}{N(1-e)} \tag{4-2}$$

where:

C = multiple unit clogging factor
C_o = single unit clogging factor (50 percent for grates and 25 percent for curb-openings)
N = number of inlets in series
e = decay ratio less than unity (0.5 for grates and 0.25 curb-openings)

The interception of an inlet on a grade is proportional to the inlet length and a sag is proportional to the inlet opening area. Therefore, the clogging factor shall be applied to the length for inlets on grade:

$$L_e = L\,(1-C) \tag{4-3}$$

where:

L_e	=	effective unclogged length, in feet
L	=	length of inlet, in feet
C	=	multiple unit clogging factor

For inlets in sag the clogging factor shall be applied to the opening area of the inlet:

$$A_e = A\,(1-C) \tag{4-4}$$

where:

A_e	=	effective unclogged area, in square feet
A	=	area of inlet, in square feet
C	=	multiple unit clogging factor

4.4 INLET LOCATION

In general, inlets should be placed at all low points in the gutter grade and at intersections to prevent the gutter flow from crossing traffic lanes of the intersecting road. In urban locations, inlets are normally placed upgrade from pedestrian crossings to intercept the gutter flow before it reaches the crosswalk. Location should also be coordinated to avoid inlet location and the effect of ponding near pedestrian facilities (e.g., ramps) governed by the regulations of the American Disability Act. Gutter flow should be intercepted where pavement surfaces transition between superelevated and normal sections. Placement of inlets before the change in the pavement lessen water flowing across the roadway and thus preventing potential driving hazards, such as hydroplaning and icing. Where a curbed roadway crosses a bridge, the gutter flow should be intercepted and not be permitted to flow onto the bridge.

Inlet spacing shall be limited to a maximum distance of 300 feet where other design criteria, such as pavement encroachment and depth do not control within that distance.

4.4.1 Spacing of Inlets on Grade

Inlets should be spaced so as to limit the spread of the water on the pavement in accordance with criteria in Section 3.2 Design Criteria for street drainage.

With the maximum spread fixed and with a given pavement cross and longitudinal slope, the flow in the gutter is also fixed and can be calculated as shown in Section 3.2 Design Criteria. The spacing of inlets must be equal to or less than the length of pavement needed to generate the discharge corresponding to the allowable spread on the pavement. The flow bypassing each inlet must be included in the flow arriving at the next inlet.

An example of the computations for inlet spacing for a combination inlet follows:

Example 1: Initial Inlet Spacing

Given: Non-residential minor collector (36-foot pavement width)
2-foot curb-and-gutter section
2 percent pavement crown slope
1 percent longitudinal street slope
Composite C for pavement and shoulder = 0.95, as determined from contributing area, see Table 2-4
Contributing area is 200 feet to each side of the centerline
3-foot long by 2-foot wide combination inlet with 2 inch depression
Assuming n = 0.014 for rough asphalt Table 3-4

Find: Maximum design inlet spacing during the minor storm

Solution:

1) Determine allowable pavement encroachment.

From Table 3-2, a collector street has an encroachment that requires half a single lane in each direction to be free of encroachment. Therefore, encroachment may spread to a width of 11 feet. Half a full travel lane in a single direction is 9 feet wide plus the 2 feet provided by the gutter section.

2) Compute the runoff from the contributing area, given the rainfall intensity for a minor storm, I, and the coefficient of runoff, C. The rainfall intensity for a minor storm is 4 inches/hour. Also, remember to convert area to acres before computing and that the Rational Method equation provided below is not dimensionally correct (i.e., the units will not cancel out).

$$Q = CIA$$

$$Q = (0.95)\,(4\ _{\textit{inches per hour}})\left[\frac{200\ _{\textit{feet}} \times L\ _{\textit{feet}}}{43{,}560\ _{\textit{feet}^2\ \textit{per hour}}}\right]$$

$$Q = 0.017L$$

3) Compute the discharge for the standard gutter section that is limited by the allowable spread, T = 11 feet. See Section 3.2 Design Criteria for calculation methodology.

Q = 3.09 cubic feet per second

4) Compute the location of the first inlet by substituting the allowable capacity, Q, into the relationship derived in Step 2.

$$Q = 0.017L$$

$$L = \frac{3.09 \ \textit{cubic feet per second}}{0.017}$$

$$L = 181 \ \textit{feet}$$

This distance represents the minimum length at which the initial inlet must be located to meet the spread criterion. Subsequent inlet spacing intervals are further determined by spread criterion; however, the capacity, Q, will include not only the contributing drainage but that of the bypassed flow from the previous inlet as well.

4.4.2 Spacing of Inlets in Sag

Sag vertical curves differ one from another in the potential for ponding, and criteria adopted for inlet spacing in sags should be applied only where traffic could be unduly disrupted if an inlet became clogged or runoff from the design storm was exceeded. Therefore, criteria adopted for inlet spacing in sag vertical curves are not applicable to the sag curve between two positive or two negative longitudinal slopes. Also, they should not be applied to locations where ponding depths could not exceed curb height and ponding widths would not be unduly disruptive, as in sag locations on embankment.

For minor streets, those including minor collectors, local streets, and alleys, a double catchbasin shall be placed in the sag to facilitate drainage and prevent excessive ponding. This approach is sufficient to address the hazard and inconvenience caused by ponding on a minor street without requiring an excessive inlet system.

For major streets, including major collectors, thoroughfares, and expressways, it is engineering practice to place flanking inlets on each side of the inlet at the low point in the sag. The flanking inlets should be placed so that they will limit spread on low gradient approaches to the level point and act in relief of the inlet at the low point if it should become clogged or if the design spread or depth is exceeded.

The flanking approach requires three inlets, one at the low point and one on each side of this point, where the grade elevation is approximately 0.2 feet higher than at the low point. The additional inlets furnish added capacity to allow for flow bypassing the upgrade inlets and provide a safety factor if the sag inlet becomes clogged. These inlets limit the deposition of sediment on the road in the sag, and they also reduce flow arriving at the low point and thereby prevent ponding on the road. The inlets should be spaced so as to limit the spread and depth of water on the pavement to the criterion outlined in Section 3.2 Design Criteria.

Table 4-1 shows the spacing required for various depths at curb criteria and vertical curve lengths defined by the following dimensionless coefficient:

$$X = (200dK)^{0.5} \quad (4\text{-}5)$$

where:

X = distance from the low point, feet
d = depth at curb, feet
K = dimensionless Coefficient, Table 4-1

$$K = \frac{L}{A} \quad (4\text{-}6)$$

where:

K = dimensionless coefficient
L = length of vertical curve, in feet
A = algebraic difference in approach grades (G_2 - G_1)

TABLE 4-1
DISTANCE TO FLANKING INLETS IN SAG VERTICAL CURVE LOCATIONS

Depth at Curb (feet)									
Speed (mph)	**"K" L/A**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**
20	20	20	28	35	40	45	49	53	57
25	30	24	35	42	49	55	60	65	69
30	40	28	40	49	57	63	69	75	80
35	50	32	45	55	63	71	77	84	89
40	70	37	53	65	75	84	92	99	106
45	90	42	60	73	85	95	104	112	120
50	110	47	66	81	94	105	115	124	133
55	130	51	72	88	102	114	125	135	144
60	160	57	80	98	113	126	139	150	160
	*167	58	82	100	116	129	142	153	163
65	180	60	85	104	120	134	147	159	170
70	220	66	94	115	133	148	162	176	188
Note: * Maximum drainage K = 167									

The AASHTO policy on geometrics specifies maximum K values for various design speeds, as shown in Table 4-1.

Example 2: Spacing of Inlet in a Sag

Given: 2 percent pavement crown slope
Allowable spread not to exceed 10 feet
Speed limit for highway is 55 MPH
Dimensionless coefficient, K = 130

Find: Location of flanking inlets if determined: (1) in relief of the inlet at the low point when depth at the curb exceeds design depth, and (2) when depth at the curb is 0.1 foot less than depth at design spread.

Solution:

1) Find depth at curb for allowable spread using Equation 3-2

$$d = TS_x$$

$$d = (10\ \textit{feet})(0.02\ \textit{feet per foot})$$

$$d = 0.2\ \textit{feet}$$

a) From Table 4-1, using K = 130 and a depth of 0.2 feet, spacing to flanking inlet = 72 feet

b) From Table 4-1, using a depth 0.1 feet less than the design depth of 0.2 feet, yields a depth of 0.1 feet. For the same K and depth at curb of 0.1 feet, spacing to flanking inlets = 51 feet

The purpose in providing Table 4-1 is to facilitate the selection of criteria for the location of flanking inlets based on the ponding potential at the site, the potential for clogging of the inlet at the low point, design spread, design speeds, traffic volumes, and other considerations which may be peculiar to the site under consideration. The depth at curb criterion which does not vary with these considerations neglects consideration of cross slope and design spread; it may be unduly conservative at some locations. Location of flanking inlets at a fixed slope rate on the vertical curve also neglects consideration of speed facilities and is not at all conservative for high speed facilities.

Except where inlets become clogged, spread on low gradient approaches to the low point is a more stringent criterion for design than the interception capacity of the sag inlet. A gradient of 0.5 percent is to be maintained within 50 feet of the level point in order to provide for adequate drainage.

4.5 GRATE INLETS

Grate inlets will intercept all of the gutter flow passing over the grate (frontal flow) if the grate is sufficiently long and the gutter flow velocity is low. Only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs. A part of the flow along the side of the grate will be intercepted, dependent on the cross slope of the pavement, the length of the grate, and flow velocity. The City of Concord Technical Standards for Streets has standards and details for acceptable grate inlets for use in the City. Use North Carolina Department of Transportation standards and details for grate inlet types, construction, and performance where local City standards and details are not available (e.g. grates for use in bike lanes where required).

4.5.1 Capacity of Grate Inlets on Grade

The ratio of frontal flow to total gutter flow, E_o, for a straight cross slope is expressed by Equation 4-7 for either straight cross slopes or depressed gutter sections.

$$E_o = \frac{Q_w}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{2.67} \tag{4-7}$$

where:

Q	=	total gutter flow, in cubic feet per second
Q_w	=	flow in width W, in cubic feet per second
W	=	width of grate, in feet
T	=	total spread of water, in feet

The ratio of side flow, Q_s, to total gutter flow is:

$$\frac{Q_s}{Q} = 1 - \frac{Q_w}{Q} = 1 - E_o \tag{4-8}$$

where:

Q_s	=	ratio of side flow, in cubic feet per second
Q	=	total gutter flow, in cubic feet per second
Q_w	=	flow in width W, in cubic feet per second
E_o	=	ratio of frontal to total gutter flow

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by Equation 4-9 which takes into account grate length, bar configuration, and gutter velocity at which splash-over occurs. This ratio is equivalent to frontal flow interception efficiency.

$$R_f = 1 - 0.09(V - V_o) \tag{4-9}$$

where:

R_f	=	frontal flow interception efficiency
V	=	velocity of flow in the gutter, in feet per second
V_o	=	gutter velocity where splash-over first occurs, in feet per second

The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, is expressed by the following equation.

$$R_s = \frac{1}{1 + \frac{0.15V^{1.8}}{S_x L^{2.3}}} \tag{4-10}$$

where:

R_s	=	side flow interception efficiency
V	=	velocity of flow in the gutter, in feet per second
S_x	=	cross slope, in feet per foot
L	=	length of the grate, in feet

The efficiency, E, of a grate is expressed as:

$$E = R_f E_o + R_s (1 - E_o) \tag{4-11}$$

where:

E	=	grate efficiency
R_f	=	frontal flow interception efficiency
E_o	=	ratio of frontal flow to total gutter flow
R_s	=	side flow interception efficiency

The first term on the right side of Equation 4-11 is the ratio of intercepted frontal flow to total gutter flow, and the second term is the ratio of intercepted side flow to total side flow. The second term is insignificant with high velocities and short grates.

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q\,[R_f E_o + R_s(1 – E_o)] \tag{4-12}$$

where:

Q_i	=	flow intercepted, in cubic feet per second
E	=	grate efficiency
Q	=	total gutter flow, in cubic feet per second
R_f	=	frontal flow interception efficiency
E_o	=	ratio of frontal flow to total gutter flow
R_s	=	side flow interception efficiency

Example 3: Interception Capacity of Grate Inlets on Grade

Given: Non-residential major collector (36-foot pavement width)
2-foot curb-and-gutter section
2 percent pavement crown slope
1 percent longitudinal street slope
3-foot long by 2-foot wide grate inlet with 2 inch depression
Assume n = 0.016 for rough asphalt form Table 3-4

Find: Interception capacity of a 3-foot long and 2-foot wide standard grate

Solution:

1) Determine allowable pavement encroachment.

From Table 3-2, a major collector street has an allowable encroachment of half a travel lane in a single direction. Therefore, encroachment may spread to a width of 11 feet. Half of one travel lane in a single direction is 9 feet wide plus the 2 feet provided by the gutter section.

2) Calculate the ratio of frontal flow to total gutter flow, E_0, using Equation 4-7.

$$E_0 = 1 - \left(1 - \frac{W}{T}\right)^{2.67}$$

$$E_0 = 1 - \left(1 - \frac{2\ \mathit{feet}}{11\ \mathit{feet}}\right)^{2.67}$$

$$E_0 = 0.41$$

4.5.2 Capacity of Grate Inlets in Sag

A grate inlet in sag operates first as a weir having a crest length roughly equal to the outside perimeter (P) along which the flow enters. Bars are disregarded and the side against the curb is not included in computing P. Weir operation continues to a depth, d, of about 0.4 feet above the top of grate and the discharge intercepted by the grate is:

$$Q_i = 3.0Pd^{1.5} \qquad (4\text{-}13)$$

where:

Q_i	=	rate of discharge into the grate opening, in cubic feet per second
P	=	perimeter of grate opening, in feet, disregarding bars and neglecting the side against the curb
d	=	depth of water at grate, in feet

When the depth at the grate exceeds about 1.4 feet, the grate begins to operate as an orifice and the discharge intercepted by the grate is:

$$Q_i = 0.67A_8(2gd)^{0.5} = 5.37A_8d^{0.5} \qquad (4\text{-}14)$$

where:

Q_i	=	rate of discharge into the grate opening, in cubic feet per second
A_g	=	clear opening of the grate, in square feet
g	=	acceleration of gravity, 32.2 feet per second squared
d	=	depth of ponded water above top of grate, in feet

Between depths of approximately 0.4 feet and approximately 1.4 feet (over the grate), the operation of the grate inlet is indefinite due to vortices and other disturbances. The capacity of the grate is somewhere between that given by the weir and orifice equations.

Except where inlets become clogged, spread on low gradient approaches to the low point is a more stringent criterion for design than the interception capacity of the sag inlet. A minimum gradient of 0.5 percent should be maintained within 50 feet of the level point in order to provide for adequate drainage.

Drop inlets being used in depressed areas in roadway medians or ditches or along storm sewers that connect major curb and gutter systems are governed by the same hydraulic principles. A drop inlet operates as a weir to certain depth at which it transitions into orifice flow. Equations 4-13 and 4-14 govern the performance of drop inlets. There are no specific criteria for drop inlets. However, design should be taken into consideration so that the flooding of adjacent property, buildings, utility infrastructure, private roadway, and pedestrian access is minimized.

Example 4: Interception Capacity of Grate Inlet in Sag

Given: Local street (24-foot pavement width)
Flow from right and left of grate, Q_1 = 3.6 cfs and Q_2 = 4.4 cfs, respectively
3 percent pavement crown slope
1 percent longitudinal street slope
6-inch vertical curb

Find: Grate inlet size for specified flow and depth at curb under minor storm conditions.

Solution:

1) Determine allowable pavement encroachment.

From Table 3-2, a local street has an allowable encroachment to the crown of the road. Therefore, encroachment may spread to a width of 14 feet. Each travel lane is 12 feet wide plus the 2 feet provided by the gutter section. Also, the curb is not allowed to overtop.

2) Calculate design discharge, Q_i, and use Equation 4-13 to determine the needed perimeter, P, to handle the capacity.

$$Q_i = 3.6 \textit{ cubic feet per second} + 4.4 \textit{ cubic feet per second} = 8.0 \textit{ cubic feet per second}$$

$$P = \frac{Q_i}{3d^{1.5}}$$

$$P = \frac{8 \textit{ cubic feet per second}}{3(0.5 \textit{ feet})^{1.5}}$$

$$P = 7.54 \textit{ feet}$$

3) From Step 2, a grate must have a perimeter of 7.54 feet to intercept 8 $feet^3$ per second at a depth of 0.5 feet. Therefore, a single 3-foot long by 2-foot wide grate with a perimeter of 7 feet (the specified grate per City of Technical Standards for Streets) would not meet the requirement for capacity. Either double inlets would need to be constructed next to each other or additional inlets will need to be placed upgrade of the sag.

4) Verify that the minimum grade requirements for a vertical curve are being met by using Equation 3-1 and solving for the spread, T.

$$T = \left[\frac{Qn}{0.56 S_a^{1.67} s^{0.5}} \right]^{0.375}$$

$$T_1 = \left[\frac{(3.6 \textit{ cubic feet per second})(0.016)}{0.56\,(0.03 \textit{ feet per foot})^{1.67}(0.003 \textit{ feet per foot})^{0.5}} \right]^{0.375}$$

$$T_2 = \left[\frac{(4.4 \textit{ cubic feet per second})(0.016)}{0.56(0.03 \textit{ feet per foot})^{1.67}(0.003 \textit{ feet per foot})^{0.5}} \right]^{0.375}$$

$T_1 = 11.34$ *feet*

$T_2 = 12.22$ *feet*

T_1 and T_2 are less than the allowable encroachment of 14 feet

A double 3 by 2 foot grate is adequate to intercept the design storm at a spread width which does not exceed design spread and American Association of Highway and Transportation Officials requirements are met. The tendency of grate inlets in sag to clog may warrant a combination inlet on the low-gradient approaches.

4.6 COMBINATION INLETS

Combination inlets incorporate a vertical opening in the curb along side the grate placed on grade. The grate portion operates identical to that of a grate only inlet however, the curb opening will intercept a portion of the flow where the flow depth is sufficient (i.e. sag conditions) or clogging of the grate has occurred. The Concord Technical Standards for Streets has standards and details for acceptable combination inlets for use in the City. Use North Carolina Department of Transportation standards and details for grate inlet types, construction, and performance where local Concord standards and details are not available (e.g. grates for use in bike lanes where required). Standards for a double catchbasin and open throat inlet are provided in Figures 4-2 through 4-5 at the end of this section. These standards were adapted from Charlotte-Mecklenburg Land Development Standards and other sources.

4.6.1 Capacity of Combination Inlets on Grade

The interception capacity of combination inlets on grade where the curb opening and the grate are of equal length does not increase interception appreciably greater than a grate alone. Capacity is computed by neglecting the capacity of the curb opening in this situation. Capacity for combination inlets should be calculated using the methodology outlined in Section 4.5 Grate Inlets

4.6.2 Capacity of Combination Inlets in Sag

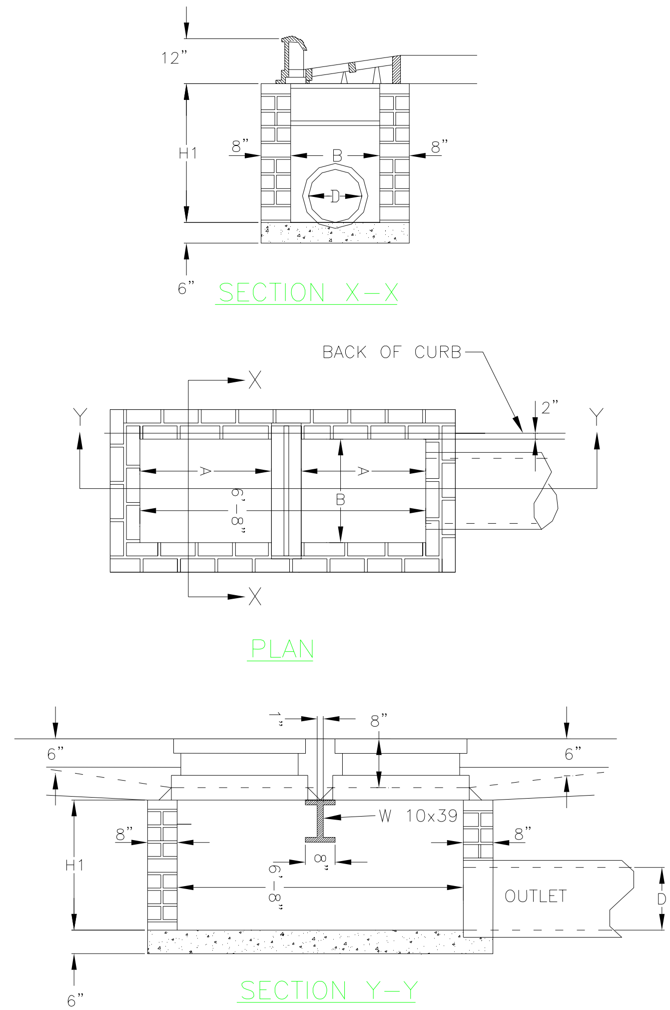
Combination inlets shall be required for use in sags. The interception capacity of the combination inlet is essentially equal to that of a grate alone in weir flow. In orifice flow, the capacity is equal to the capacity of the grate plus the capacity of the curb opening.

Equation 4-13 can be used for weir flow in combination inlets in sag locations. Where depth at the curb is such that orifice flow occurs, the interception capacity of the inlet is computed by adding Equation 4-13 to that of the curb opening capacity term:

$$Q_i = 0.67A_g(2gd)^{0.5} + 0.67hL\,(2gd_i)^{0.5} \tag{4-20}$$

where:

Q_i = intercepted flow by inlet, in cubic feet per second
A_g = clear area of the grate, in square feet
g = acceleration due to gravity, 32.2 feet per second squared
d = depth at the curb, in feet
h = height of curb-opening orifice, in feet
L = length of curb-opening, in feet
d_i = depth at lip of curb-opening, in feet



GENERAL NOTES:

1. ALL CONCRETE TO BE 3600 P.S.I COMPRESSIVE STRENGTH.
2. MORTAR JOINTS SHOULD BE BETWEEN 3/8" AND 5/8" THICK.
3. ALL CATCH BASINS OVER 3'–6" IN DEPTH TO BE PROVIDED WITH METAL STEPS ON 1'–2" +/– CENTERS. STEPS SHALL BE IN ACCORDANCE WITH STD. 20.12.
4. CONCRETE BRICK MAY BE USED IN LIEU OF HARD COMMON CLAY BRICK. JUMBO BRICK WILL BE PERMITTED.
5. FOR 8'–0" IN HEIGHT OR LESS USE 8" WALL, OVER 8'–0" IN HEIGHT USE 12",USE 12" WALL TO 6'–0" FROM TOP OF WALL, AND 8" WALL FOR THE REMAINING 6'–0".
6. ALL PIPE IN STORM DRAIN STRUCTURE SHALL BE STRUCK EVEN WITH THE INSIDE WALL, GROUTED AND BRUSHED SMOOTH.
7. WEEP HOLE(S) SHALL BE PLACED IN BACK WALL. A STONE DRAIN CONSISTING OF 1 (ONE) CUBIC FOOT OF NUMBER 78M STONE CONTAINED IN A BAG OF POROUS FABRIC SHALL BE PLACED AT EACH WEEP HOLE.

DIMENSIONS OF BOX AND PIPE

PIPE D	SPAN A	WIDTH B	HEIGHT MIN. H1	W 10 LENGTH
15"	3'–0"	2'–4"	2'–6"	2'–10"
18"	3'–0"	2'–4"	2'–10"	2'–10"
24"	3'–0"	2'–4"	3'–4"	2'–10"

Figure 4-2 Standard for Double Catch Basin for 15" through 24" Pipe

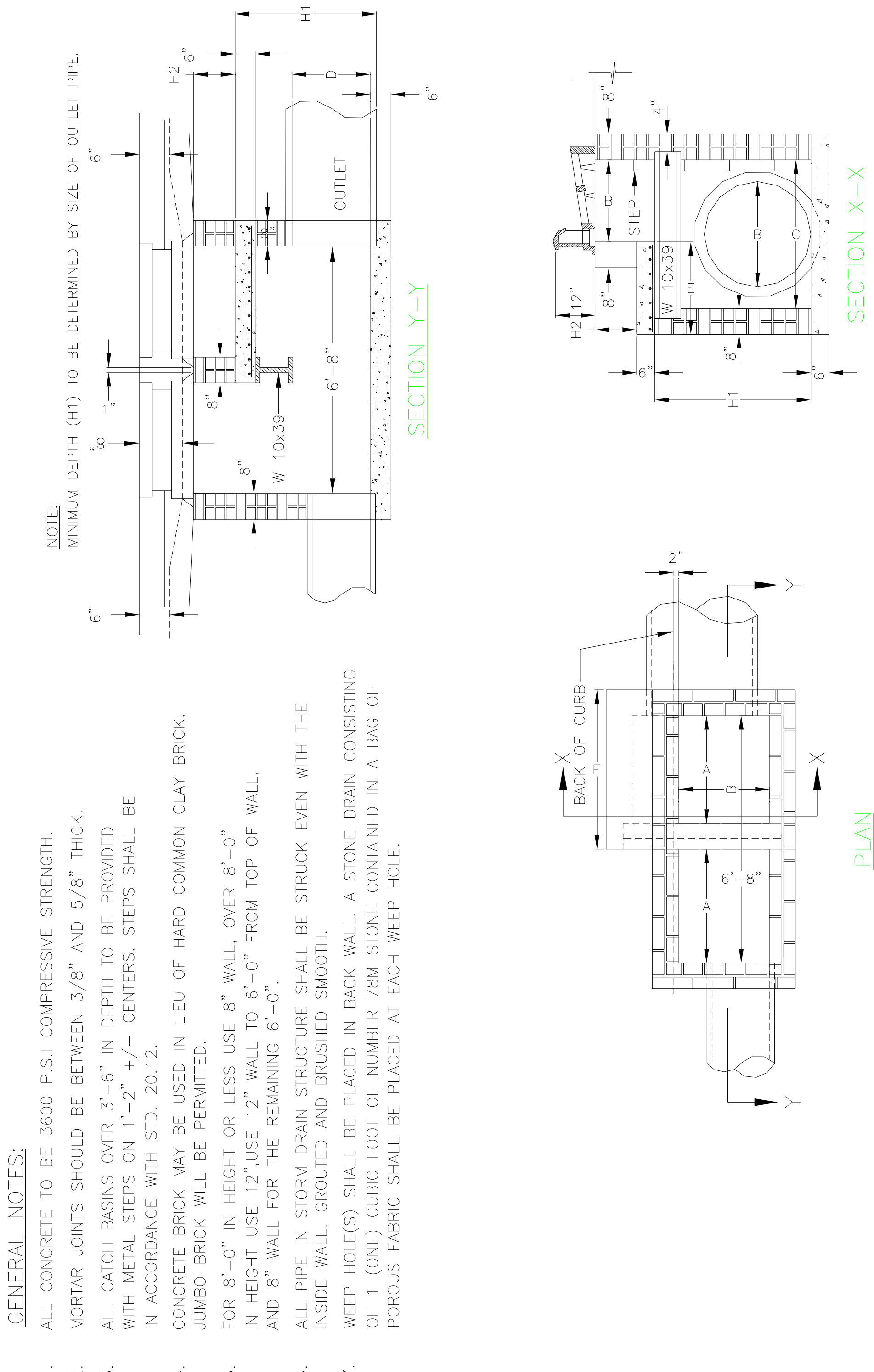


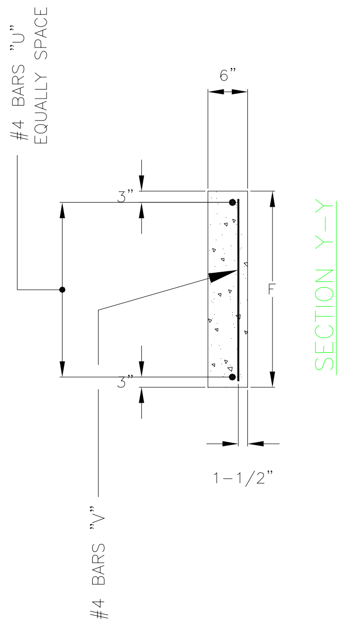
Figure 4-3 Standard for Double Catch Basin for 30" through 36" Pipe (Sheet A)

15" THRU 24" PIPE

DIMENSIONS OF BOX AND PIPE						
PIPE	SPAN	WIDTH	WIDTH	HEIGHT	HEIGHT	W 10
D	A	B	C	MIN. H1	H2	LENGTH
15"	3'-0"	2'-4"	-	2'-6"	-	2'-10"
18"	3'-0"	2'-4"	-	2'-10"	-	2'-10"
24"	3'-0"	2'-4"	-	3'-4"	-	2'-10"

30" THRU 36" PIPE

DIMENSIONS OF BOX AND PIPE							COVER		TOP SLAB REINFORCMENT						
							DIMENSION		BARS - U		BARS - V		BARS - W		TOT.
PIPE	SPAN	WIDTH	WIDTH	HEIGHT	HEIGHT	W 10									
D	A	B	C	MIN. H1	H2	LENGTH	E	F	NO.	LENGTH	NO.	LENGTH	NO.	LENGTH	LBS.
30"	3'-0"	2'-4"	3'-4"	3'-2"	VAR.	4'-0"	1'-10"	4'-4"	4	1'-6"	3	4'-1"	3	4'-1"	45
36"	3'-0"	2'-4"	3'-10"	3'-8"	VAR.	4'-6"	2'-4"	4'-4"	4	2'-0"	4	4'-1"	3	4'-1"	49



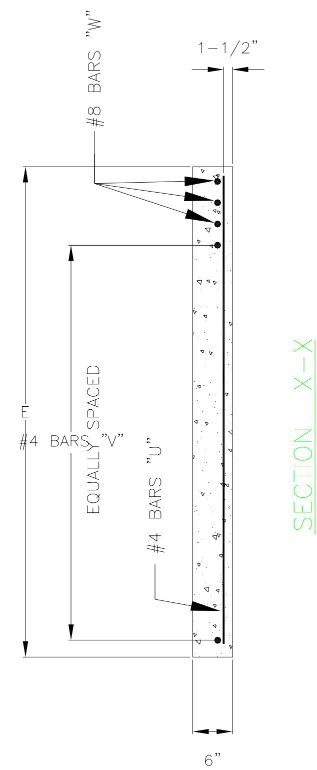


Figure 4-4 Standard for Double Catch Basin for 30" through 36" Pipe (Sheet B)

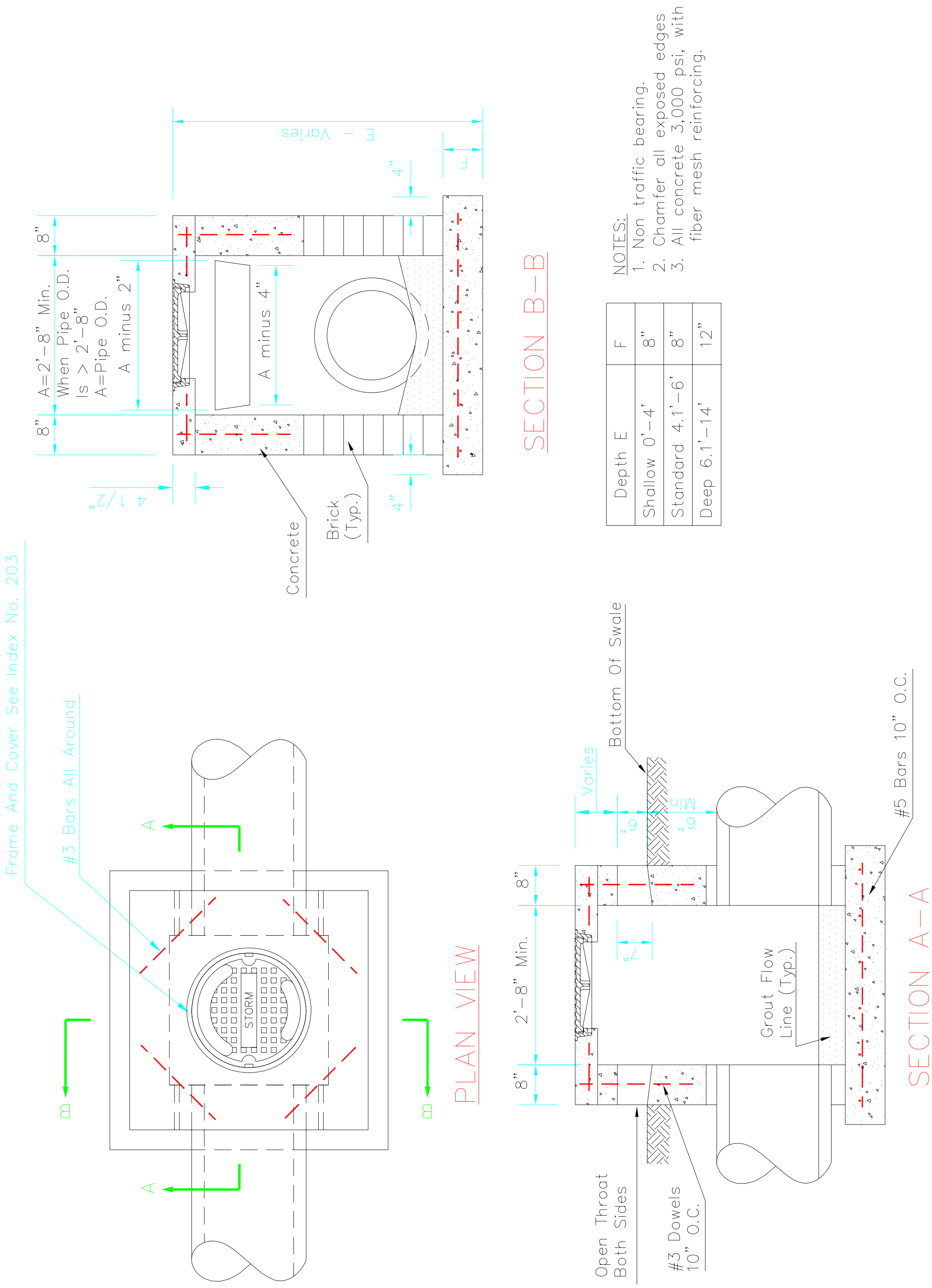


Figure 4-5 Standard for Open Throat Inlet

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SECTION 5
STORM SEWERS

Page

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5.1 INTRODUCTION

This section addresses storm sewer design and function in complex drainage systems. Hydraulically, storm drainage systems consist of conduits (open or enclosed) that transport unsteady and non-uniform free flow. Storm sewers are designed for open-channel flow to satisfy, as well as make possible, the requirements for unsteady and non-uniform flow.

The hydraulics of storm sewers with open channel flow is described by Manning's Equation. Designers must verify that the open channel flow assumption is valid by calculating the hydraulic grade line. All structures and their effect on the hydraulics of the system must be considered.

The design of a storm drainage system is governed by the following seven conditions:

A. Compliance with the City of Concord (City) standards.

B. The system must accommodate the surface runoff resulting from the selected design storm with no damage to physical facilities and minimum interruption of normal traffic.

C. Runoff resulting from major storms must be anticipated and discharged free from impedance without serious damage to physical facilities (such as conveyance past finished floor elevations of buildings and under roadways without washing out embankments and subgrades).

D. The storm drainage system must have a maximum reliability of operation with respect to being structurally sound to its environment where it is placed and performing hydraulically to its intended function for the entire life of design.

E. The storm drainage system must require minimum maintenance (cleaning and clearing obstructions) and must be accessible for maintenance operations.

F. The storm drainage system must be adaptable to future expansion with minimum additional cost and designed to accommodate build-out conditions in the upstream reaches of the drainage area.

G. Site design, swales and natural flow features should be utilized to reduce the need for extensive storm sewer systems whenever possible.

5.2 GENERAL CRITERIA

5.2.1 Frequency of Design Runoff

The runoff design frequency is a function of operational and economic criteria with a special emphasis on public safety. As discussed in other sections of this Article, some types of facilities do not require high levels of protection and periodic flooding is not objectionable. However, for all facilities, the designer must consider the impact of a 100-year design storm and provide for its passage without the loss of life or major property damage.

Table 5-1 indicates the minimum acceptable design storm frequencies for storm sewer system facilities.

TABLE 5-1
STORM SEWER SYSTEM FACILITYDESIGN STORM FREQUENCY

Facility	Storm Return Period (Frequency)
Streets and Gutters	10 years
Inlets	10 years
Storm Sewers	10 years
Cross culverts	25 years
Major Drainage System (culverts over FEMA regulated floodways)	100 years

In addition to meeting the minimum design storm criteria presented in Table 5-1, storm sewer facilities must also be designed to allow the storm drainage system to meet the hydraulic and detention requirements outlined in Section 8 Hydraulics of Detention.

5.2.2 Velocities and Grades

Minimum Grades

Storm sewers should operate with flow velocities sufficient to prevent excessive deposition of solid material, which would result in clogging. The controlling velocity occurs near the bottom of the conduit and is considerably less than the mean velocity. Storm sewers shall be designed to have a minimum allowable slope of 0.5 percent or the slope that will produce a mean velocity of 2.5 feet per second during the design flow, whichever is greater. Any variance of minimum slope must be approved by the City. Outlets of pipes on minimum grade should be designed to avoid sedimentation at the outfall.

Maximum Velocities

Maximum velocities are important for the long-term protection of the interior material of the conduit. The maximum velocity shall not exceed 15 feet per second for reinforced concrete pipe (RCP) or 10 feet per second in corrugated metal pipe or plastic pipe.

Energy dissipaters shall be required at outfalls where pipe flow velocities may exceed erosive velocities defined in the North Carolina Erosion and Sediment Control Planning and Design Manual. These dissipaters must meet the requirements of the North Carolina Sedimentation Pollution Control Act and the Cabarrus County Erosion Control Ordinance.

5.2.3 Pipe Sizes and Material Types

Pipes which are to become an integral part of the public storm sewer system shall be circular and have a minimum diameter of 15 inches for gravity flow. If alternate shapes are required for utility clearance or special conditions, the designer must contact the City for approval. All pipe design and installation must meet the manufacturer's recommendation for minimum depth of cover.

A key consideration in selection of pipe material type involves the design life of the pipe. Required pipe design life shall be a minimum 50 years as certified by the manufacturer. The design professional must meet all manufacturer requirements on which the design life is based. For example, bedding requirements are critical to meeting the pipe design life. New construction or replacement of existing storm sewers within the right-of-way or connecting systems shall require reinforced concrete pipe unless approval from the City for other pipe material has been granted. All reinforced concrete storm sewers shall meet, at a minimum, the requirement of American Association of State Highway Transportation Officials (AASHTO) M170 Classes III-V.

In selecting a roughness coefficient, consideration shall be given to the average conditions during the useful life of the structure. An increased "n" value shall be used primarily in analyzing old conduits where alignment is poor and joints have become rough. If, for example, concrete pipe is being designed at a location and there is reason to believe that the roughness would increase through erosion or corrosion of the interior surface, slight displacement of joints, or entrance of foreign materials, the designer should select a roughness coefficient representing the average condition using engineering judgment. Any selection of "n" values below the minimum or above the maximum, either for monolithic concrete structures, concrete pipe, or corrugated metal pipe, requires the written approval of the City.

The coefficients of roughness listed in Table 5-2, on page 5-4, are used in direct solution of Manning's Equation.

TABLE 5-2
ROUGHNESS COEFFICIENTS FOR STORM SEWERS

Materials of Construction	Design Coefficient[1]
Concrete Pipe	0.012-0.015
Corrugated Metal	0.020-0.027
Cast Iron	0.011-0.015
Vertified Clay Pipe	0.011-0.017
Brick with cement mortar	0.012-0.017
Plastic	0.010-0.015
PVC	0.010

[1] Designer may select a single representative "n"

5.2.4 Manhole Location

Manholes shall be located at pipe junctions and changes in alignment and slope sections. Manholes shall be located at intervals not to exceed 500 feet for any pipe.

5.2.5 Alignment

In general, storm sewer alignment between structures shall be straight. Any pipe deflection must be approved by the City and in no case can it exceed manufacturer's recommendation. Storm sewers must be outside the curb line and street crossings shall be at 90 degrees. The City must approve all curved alignments prior to installation.

5.3 FLOW IN STORM SEWERS

5.3.1 Pipe Flow

All storm sewers shall be designed by the application of the Continuity Equation and Manning's Equation, as follows:

$$Q = AV \tag{5-1}$$

$$Q = \frac{1.49}{n} AR^{0.67} S_f^{\ 0.5} \tag{5-2}$$

where:

Q = pipe flow, in cubic feet per second
A = cross-sectional area of pipe, in square feet
V = velocity of flow, in feet per second

n	=	Manning's coefficient of roughness of pipe
R	=	hydraulic radius equal to area divided by wetted perimeter, in feet
WP	=	wetted perimeter, in feet
S_f	=	friction slope of pipe, in feet per foot slope of energy grade line

Several general rules must be observed when designing storm sewer sections. When followed, they tend to alleviate or eliminate the common mistakes made in storm sewer design. These rules are as follows:

A. Select pipe size and slope so that the velocity of flow will increase progressively, or at least will not appreciably decrease, at inlets, bends, or other changes in geometry or configuration.

B. Do not discharge the contents of a larger pipe into a smaller one, even though the capacity of the smaller pipe may be greater due to steeper slope.

C. At changes in pipe size from a smaller to a larger pipe, match the soffits (inside top surface) of the two pipes at the same level rather than matching the flow lines (in situations with limiting slopes, match the 0.8 diameter point of each pipe).

D. During the design process, compare the pipe slope of a particular run to critical slope. If the slope of the pipe is greater than critical slope, the segment will likely operate under entrance control instead of the originally assumed normal flow. Conduit slope should be kept below critical slope if possible. This approach also reduces the possibility of hydraulic jumps.

5.3.2 Bernoulli Equation

The law of conservation of energy as expressed by the Bernoulli Equation is the basic principle most often used in hydraulics. Energy cannot be lost, thus in a hydraulic system the sum of all energies along a flow line is constant. The total energy in mathematical form is given in Equation 5-3.

$$E = y + \frac{V^2}{2g} + \frac{P}{\gamma} = \text{constant} \qquad (5\text{-}3)$$

where:

E	=	total energy head, in feet
y	=	depth of water, in feet
V	=	mean velocity, in feet per second
P	=	pressure at given location, in pounds per square foot
g	=	acceleration of gravity, 32.2 feet per second squared
γ	=	specific weight of water, in pounds per cubic foot

The principle states that the energy head at any cross section must equal that in any other downstream section plus the intervening losses. In open channels, the flow is primarily controlled by the gravitational action of the moving fluid, which overcomes the hydraulic energy losses. The following version of the Bernoulli Equation represents the hydraulic principles in open channel flow.

$$h = y + \frac{V^2}{2g} + Z \qquad (5\text{-}4)$$

where:

H	=	total energy head, in feet
y	=	depth of water, in feet
V	=	mean velocity, in feet per second
Z	=	height above datum, in feet
g	=	acceleration of gravity, 32.2 feet per second squared

The total energy at point one is equal to the total energy at point two. The terms are defined as above and h_L is the total headloss between sections 1 and 2.

$$y_1 + Z_1 + \frac{V_1^2}{2g} = y_2 + Z_2 + \frac{V_2^2}{2g} + h_L \qquad (5\text{-}5)$$

The Bernoulli Equation is rewritten for pressure or closed conduit flow. The terms are defined as above.

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma} + Z_1 = \frac{V_2^2}{2g} + \frac{P_2}{\gamma} + Z_2 + h_L \qquad (5\text{-}6)$$

Figure 5-1 is a graphical representation of the energy in open channel flow and closed conduit flow. The following variables are presented in Figure 5-1.

H	=	total energy head, in feet
y	=	depth of water, in feet
$V^2/2g$	=	velocity head, in feet
EGL	=	energy grade line, in feet
S_o	=	slope of bottom, feet per foot
h_L	=	head loss (all types), in feet
V	=	mean velocity, in feet per second
Z	=	height above datum, in feet
HGL	=	hydraulic grade line, in feet
S_f	=	slope of EGL, in feet per foot
S_w	=	slope of HGL, in feet per foot

P/γ = pressure head, in feet

The sum of the pressure head, P/γ and the elevation head, y, is called the piezometric head. This is the height to which water would rise in a pipe with one of its ends inserted into an arbitrary point in the flow field. The line connecting points of piezometric measurements along the path of flow is called the hydraulic grade line.

$$HGL = \frac{P}{\gamma} + y \tag{5-7}$$

where:

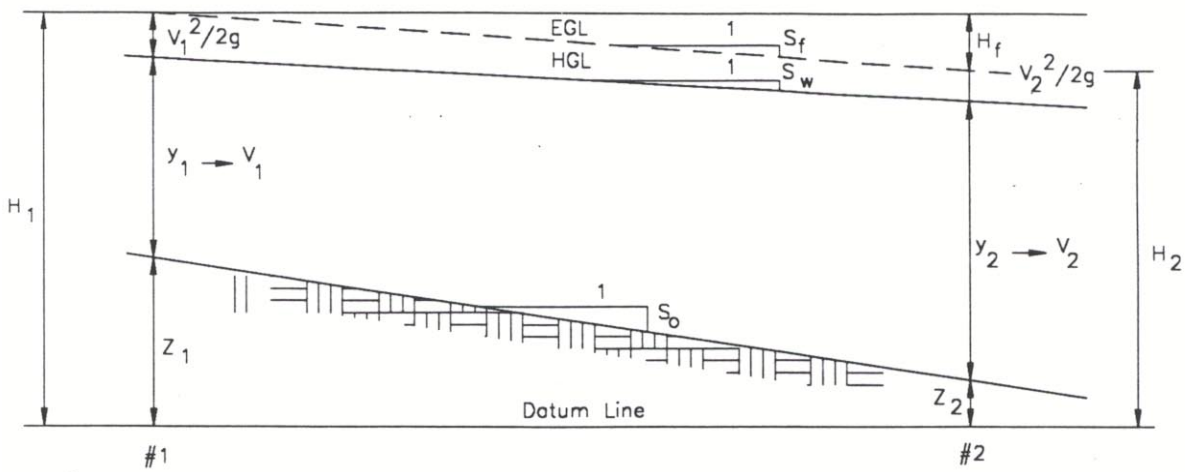
HGL = hydraulic grade line, in feet
P/γ = pressure head, in feet
y = elevation head, in feet

The energy grade line is equal to the hydraulic grade line plus the velocity head, $V^2/2g$.

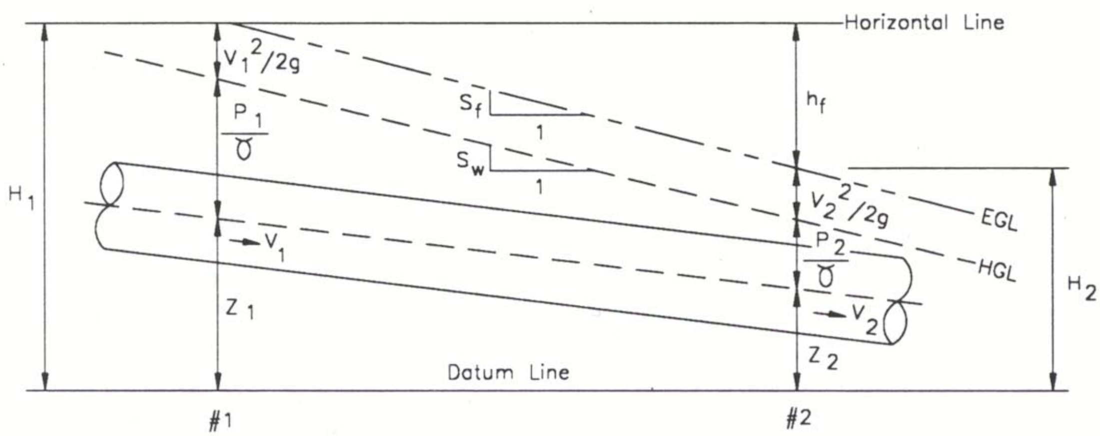
$$EGL = \frac{P}{\gamma} + y + \frac{V^2}{2g} \tag{5-8}$$

where:

EGL = energy grade line, in feet
P/γ = pressure head, in feet
y = elevation head, in feet
$V^2/2g$ = velocity head, in feet



Energy in open–channel flow



Energy in closed–conduit flow

Source: American Iron and Steel Institute. Modern Sewer Design.

Figure 5-1 Energy in Open-Channel Flow and Closed Conduit Flow

5.3.3 Critical Flow and Depth

Critical depth occurs in flow with a free surface and can be defined as (1) the depth at which, for a given energy content of the water in a conduit, maximum discharge occurs; or (2) the depth at which in a given conduit a given quantity of water flows with the minimum content of energy. Computation of the critical depth y_c can be accomplished in a trial-and-error manner to minimize energy E by using the nomograph in Figure 5-2 for determining critical depth in circular conduits. Uniform flow at critical depth will occur when the grade or slope of the conduit is nearly equal to the loss of head per foot resulting from flow at this depth.

5.4 ENERGY GRADIENT AND PROFILE OF STORM SEWERS

When using Bernoulli's Equation in the hydraulic design of storm sewers, designers must account for energy losses. These losses are commonly referred to as head losses, and are classified as either friction or minor losses. Friction losses are due to forces between the fluid and the boundary material, while minor losses are a result of the geometry of sewer appurtenances such as manholes, bends, and either an expanding or contracting transition. Minor losses can constitute a significant portion of the total head loss.

When storm sewer systems are designed for full flow, the design professional shall establish the head losses caused by flow resistance in the conduit, changes of momentum and interference at junctions and structures. This information is then used to establish the design water surface elevation at each structure.

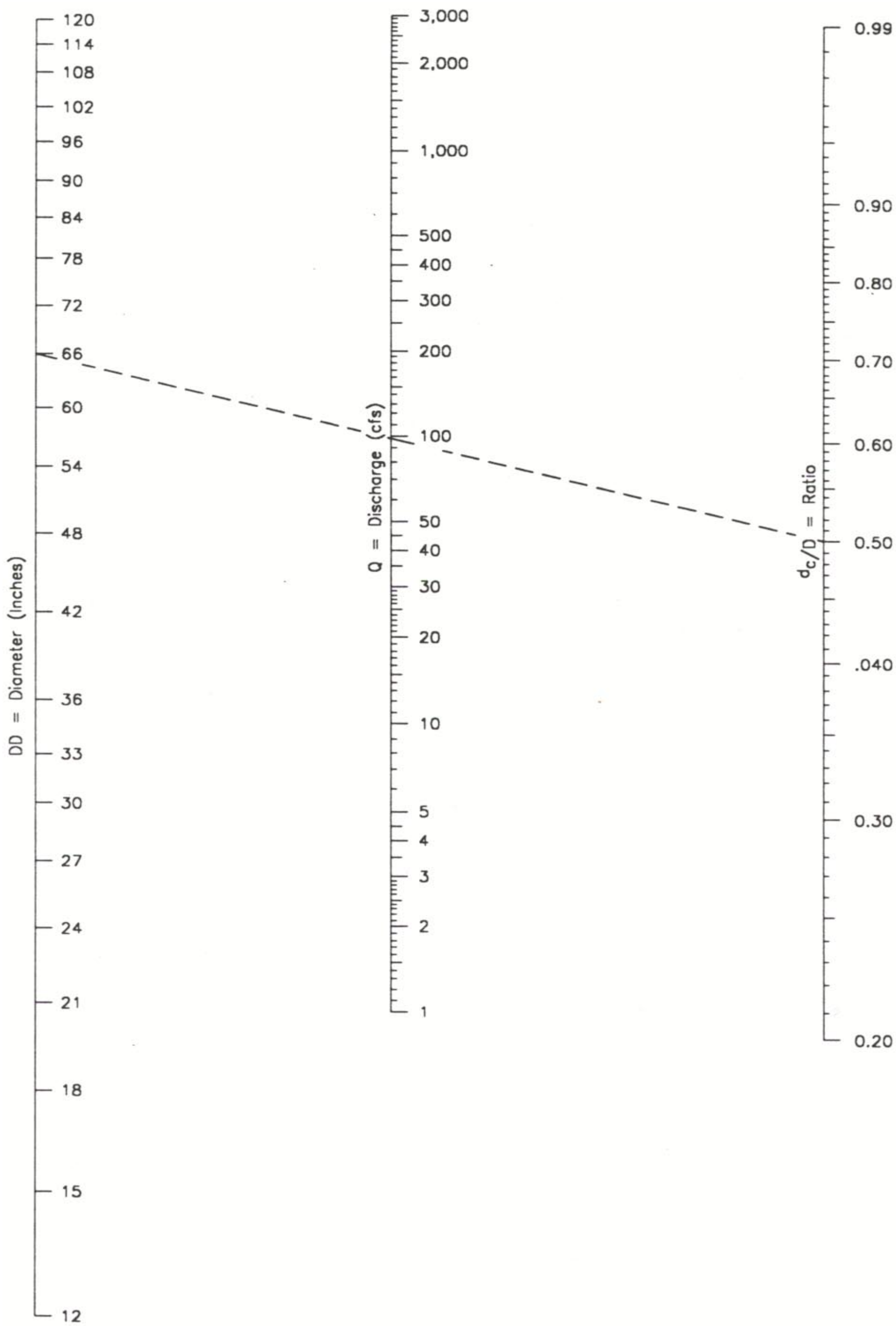
It is not necessary to compute the energy grade line of a conduit section if all three of the following conditions are satisfied;

A. The slope(s) and the pipe size(s) are chosen so that the slope is equal to or greater than the friction slope.

B. The inside top surfaces (soffits) of successive pipes are lined up when changing sizes.

C. The water surface at the point of discharge will not rise above the top of the outlet.

In such cases, the pipe does not operate under pressure and the slope of the water surface under capacity discharge will approximately parallel the slope of the invert of the pipe, assuming the minor losses are reasonable (less than 10 percent). Minor or local losses are energy losses resulting from rapid changes in the direction or magnitude of the velocity. The term minor loss is appropriate for pipelines that include long reaches of uniform straight pipe. However, for short pipes, it is a misnomer, because the minor losses may be greater than the friction losses.



Source: Texas Highway Department.

Figure 5-2 Critical Depth for Circular Conduits

In the absence of these conditions or when it is desired to check the system against a larger storm event than that used in sizing the pipes, the hydraulic and energy grade lines shall be computed and plotted. Minor losses due to turbulence at structures shall be determined by the procedure described below. If the storm sewer system could be extended at some future date, present and future operation of the system must be considered.

The final hydraulic design of a system should be based on the procedures set forth in this Article. The conduits are treated as either open channel flow or full pipe flow, as the case may be. For open channel flow, the energy grade line is used as a base for calculation, while the hydraulic grade line is used for full pipe flow. The following procedure is applicable to storm sewers flowing with a free water surface, or open channel flow. The basic approach to the design of open channel flow in storm sewers is to calculate the energy grade line along the system. It is assumed that the energy grade line is parallel to the pipe grade and that any losses other than pipe friction may be accounted for by assuming point losses at each manhole.

5.4.1 Friction Head Loss

The pipe friction can be evaluated by modifying the Manning's Equation.

$$S_f = \left[\frac{Qn}{1.49AR^{0.67}}\right]^2 \tag{5-9}$$

where:

S_f	=	friction slope of pipe, in feet per foot
Q	=	pipe flow, in cubic feet per second
n	=	Manning's roughness coefficient
A	=	cross-sectional area of pipe, in square feet
R	=	hydraulic radius equal to area divided by wetted perimeter, in feet
WP	=	wetted perimeter, in feet

The pipe friction head loss is equal to the friction slope of the pipe multiplied by the pipe length.

$$h_f = S_f L \tag{5-10}$$

where:

h_f	=	pipe friction head loss, in feet
S_f	=	friction slope of pipe, in feet per foot
L	=	length of pipe, in feet

5.5 DESIGN PROCEDURE FOR STORM SEWER SYSTEMS

5.5.1 Preliminary Design

A. Prepare a drainage map of the entire area that will be impacted by proposed improvements. Contour maps serve as excellent drainage area maps when supplemented by field reconnaissance.

B. Make a preliminary layout of the proposed storm drainage system, locating all inlets, manholes, mains, laterals, ditches, culverts, outlet, etc.

C. Outline the drainage area for each inlet in accordance with present and future street development conditions.

D. Label each drainage area with an identification number, the size of area, and the direction of surface runoff by small arrows.

E. Show all existing underground utilities.

F. Establish design rainfall frequency for each storm of interest.

G. Establish inlet time of concentration for each point of interest.

H. Calculate flows at each point of interest for each storm of interest.

I. Establish the typical cross section of each street.

J. Establish permissible spread of water on all streets within the drainage area.

K. Determine the outlet location for the system and assess capacity and impact on the receiving feature (stream, existing storm sewer system, etc.).

L. Include Steps A through K with plans submitted for review. The submitted drainage map shall be suitable for permanent filing with the appropriate agency and shall be of reproducible copy quality.

5.5.2 Storm Sewer System

The stormwater runoff calculated to enter each inlet will be further conveyed by the proposed storm sewer system. This system must have the capacity to safely carry runoff from the whole contributing drainage area for the selected design storm to its discharge point/outfall.

The flow rate in the pipe at certain points of the storm system isn't necessarily the summation of all inflows from the contributing partial drainage areas at the same point. Varying times of concentration cause the hydrographs for other contributing areas to peak at different times.

Therefore, the methods that calculate only peak discharge, such as Rational Method, have a tendency to overestimate the size of the system, because they do not consider the peak timing. This does not negatively impact systems with smaller drainage areas (such as 200 acres), but the trunk systems draining larger areas should be sized utilizing flows generated by a method that utilizes hydrographs.

Determining Type of Flow

The type of flow in the system must be determined before treating conduits as open channels. This determination must be carried out by calculating the hydraulic grade line at the downstream end of the system and progressively proceeding upstream.

A. Discharge Point

The discharge point of the storm sewer usually establishes a starting point for the hydraulic calculations. If the discharge is submerged, as when the water level of the receiving water body is above the crown of the pipe, the exit loss should be added to the water level and calculations for head loss in the sewer should start from this point. If the hydraulic grade line is above the pipe crown at the next upstream manhole, full flow calculations should be used. If the hydraulic grade line is below the pipe crown at the upstream manhole, then open channel flow calculations must be used at the manhole.

When the discharge is not submerged, a flow depth must be determined at some control section to allow calculations to proceed upstream. If the tailwater depth is less than $(D + d_c)/2$, set the tailwater elevation equal to $(D + d_c)/2$, where D equals the pipe diameter, and d_c equals the critical depth, both in feet, otherwise use the tailwater depth. The hydraulic grade line is then projected to the upstream manhole. Full flow calculations may be utilized at the manhole if the hydraulic grade is above the pipe crown.

The assumption of straight hydraulic grade lines is not entirely correct, since backwater and drawdown exists, but should be accurate enough for the size of pipes usually considered for storm sewers. If additional accuracy is justified for large conduits with diameters larger than 48 inches or where the result will have a very significant effect on design, backwater and drawdown curves should be calculated.

B. Piped System

As outlined for the discharge point, at each manhole the same type of procedure must be repeated.

The water depth in each manhole must be calculated to verify that the water level is above the crown of all pipes. Whenever the level is below the crown of a pipe, open channel methods are acceptable.

Storm Sewer Pipe

When the initial energy gradient is established and the design discharge is determined, the Continuity and Manning's Equations (5-1 and 5-2) may be used to determine the pipe size and velocity.

Junctions, Inlets, and Manholes

A. Determine the invert elevations at the upstream and downstream end of the pipe section in question. The elevation of the invert of the upstream end of the pipe is equal to the elevation of the downstream end of the pipe (invert) plus the product of the length of the pipe and pipe gradient, S_o.

B. Determine the velocity of flow for the incoming pipe (main line) at the junction, inlet, or manhole at the design point.

C. Determine the velocity of flow for the outgoing pipe (main line) at the junction, inlet, or manhole at the design point.

D. Compute the velocity head (Equation 5-8) for the outgoing velocity (main line) at the junction, inlet, or manhole at the design point.

E. Compute the velocity head for the incoming velocity (main line) at the junction, inlet, or manhole at the design point.

F. Determine the head loss coefficient, k_m, at the junction, inlet, or manhole at the design point.

G. Compute the head loss at the junction, inlet or manhole. See Table 5-3, on the next page, for coefficients and use the incoming velocity unless otherwise specified.

$$h_j = k_m \left[\frac{V_2^2 - V_1^2}{2g} \right]$$

TABLE 5-3

COEFFICIENTS FOR MINOR LOSSES (k_m) IN STORM SEWER SYSTEMS	
Minor Loss Type	**k_m**
Terminal Loss (beginning structure)	1.0
Entrance Loss	0.5[1]
Incoming Opposing Flows	1.0[2]
Angled incoming pipes losses –	
Angle, in degrees	
15	0.19
30	0.35
45	0.47
60	0.56
75	0.64
90	0.70

[1] Assuming a square-edged entrance.
[2] Identical flows entering perpendicular to the outflow. Use the combined outflow velocity.

H. Compute energy gradient at the upstream end of the junction as if the junction were not there.

I. Add the head loss to the calculated energy gradient elevation to obtain the energy gradient elevation at upstream end of junction.

All information shall be recorded on the plans or in tabular form convenient for review.

Major Storm System

A major storm system is one which is located above a flood way. Check the proposed system for the 100-year storm event. Modify the proposed system or provide additional flow capacity as required to accommodate the major design storm according to the requirements stated in Section 2 Stormwater Runoff, Section 3 Street Drainage, and Section 4 Storm Inlets.

5.5.3 Inlet System

Determining the size and suitable location of inlets is largely a trial-and-error procedure. The guide to the preferred procedure is contained/summarized in the following steps:

A. Beginning at the upstream end of the project outlines a trial drainage area and calculate the runoff from it.

B. Compare the calculated runoff to allowable street capacity. If the calculated runoff is greater than the allowable street capacity, reduce the size of the trial area. If the calculated runoff is less than street capacity, increase the size of the trial area.

Repeat this procedure until the calculated runoff equals the allowable street capacity. This is the first point at which a portion of the flow must be removed from the street. The percentage of flow to be removed will depend on street capacity versus runoff entering the street downstream.

C. Record the drainage area, time of concentration, and calculated runoff for the area. This information shall be recorded on the plans or in tabular form convenient for review.

D. If an inlet is to be used to remove water from the street, size the inlet(s) and record the inlet size, amount of intercepted flow, and amount of flow carried over (bypassing the inlet).

E. Continue the above procedure for other areas until a complete system of inlets has been developed. Compare the time of concentration for the area to the time of concentration for the upstream contributing areas. Use the longer time of concentration to calculate the discharge at the inlet. Remember to account for carry-over flow from one inlet to the next. Add the carry-over flow to the calculated discharge to obtain the design discharge at the inlet. The difference between the design discharge and the inlet discharge is carry-over flow and is bypassed to the next downstream inlet.

F. After a complete system of inlets has been established, modifications should be made to accommodate special situations such as large runoff point sources, variation in street alignments and grades or areas not to be inundated (pedestrian crossings and ramps).

G. Record information from Steps C and D for all inlets.

H. After the inlets have been assigned a location and sized the inlet pipes can be designed. Inlet pipes, or connector pipes, convey runoff intercepted by the inlets to the storm sewer main line.

I. Inlet pipes are sized to carry the flow intercepted by the inlet. Inlet pipe capacity may be controlled by the available gradient or by pipe conditions at the inlet. Inlet pipe sizes should be determined for both inlet and outlet conditions and the larger size used in the design.

5.5.4 <u>Storm Sewer Collection System Design by the Rational Method</u>

The Rational Method is a commonly used method for storm sewer collection system design. Table 5-4 provides computational guidance for designing a storm sewer system by the Rational Method. Table 5-4 may also serve as a reference for submitting data and results output from a software program that may be used for the design. The minimum time of concentration acceptable in the

Rational Method is 5 minutes. Also refer to Charlotte-Mecklenburg Storm Water Design Manual (dated 1993) pages 3-11 through 3-15.

TABLE 5-4
GUIDANCE TABLE FOR DESIGNING A STORM SEWER SYSTEM WITH THE RATIONAL METHOD

#	Variable	Explanation
1	Location of Design Point	Determine the design point location.
2	Basins	List delineated basins contributing runoff to this point. Plans must be clearly delineated in submittal.
3	Length	Enter length of flow path between previous design point of contributing subbasins and design point under consideration.
4	Inlet Time	Determine the inlet time for the particular design point. For the first point on a system, the inlet time will be equal to the time of concentration. For subsequent design points, inlet time should also be tabulated to determine if it may be of greater magnitude than the accumulated time of concentration from upstream basins. If the inlet time exceeds the time of concentration from the upstream basin, the inlet time should be substituted for time of concentration and used for this and subsequent basins. See the Section 2 Stormwater Runoff of this Article to determine inlet time.
5	Flow Time - Street Flow	Enter the appropriate flow time between the upstream design points and the design point under consideration. The flow time on the street should be used if a significant portion of the flow from the above basin is carried in the street.
6	Flow Time - Pipe Flow	Pipe flow time should generally be used unless there is significant carry-over from above basins in the street.
7	Time of Concentration	The time of concentration is the summation of the previous design point time of concentration and the intervening flow time.
8	Coefficient 'C'	Rational Method Runoff Coefficient, "C", for the basins listed in Column 2 should be determined and listed. The "C" value should be weighted if the basins contain areas with different "C" values. Build-out conditions for upstream areas should be taken into consideration.

TABLE 5-4 continued
COMPUTATIONAL GUIDANCE FOR DESIGNING A STORM SEWER DESIGN WITH THE RATIONAL METHOD

#	Variable	Explanation
9	Intensity - "I"	The intensity to be applied to the basins under consideration is obtained from the time-intensity-frequency curve developed for the specific area under consideration based upon the depth-duration-frequency curves presented in Section 2 Stormwater Runoff. The intensity is determined from the time of concentration and the frequency of design.
10	Area - "A"	The area (in acres) of the basins listed in Column 2 is tabulated in this column. Subtract ponding areas which do not contribute to direct runoff such as rooftop and parking lot ponding areas.
11	Direct Runoff	Direct runoff from the tributary basins listed in Column 2 is calculated and tabulated in this column by multiplying Columns 8, 9, and 10 together.
12	Other Runoff	Runoff from other sources, such as controlled releases from rooftops, parking lots, base flows from groundwater, and any other source, are identified in this column.
13	Total Runoff	The total of runoff from the previous design point summation plus the incremental runoff listed in Columns 11 and 12 is identified in this column.
14	Street Longitudinal Slope in Percent	The proposed street slope is listed in this column.
15	Allowable Capacity in the Street	The allowable capacity for the street is entered in this column. Allowable capacities should be calculated in accordance with procedures set forth in Section 3 Street Drainage.
16	Pipe Slope in Percent	Enter the proposed pipe grade.
17	Pipe Size in Inches	Enter the required pipe size to convey the quantity of flow necessary in the pipe.
18	Pipe Capacity	Enter the capacity of the pipe flowing full with the slope expressed in Column 16.
19	Street Design	Tabulate the quantity of flow to be carried in the street.
20	Street Velocity	Enter the actual velocity of flow for the volume of runoff to be carried in the street.

TABLE 5-4 continued
COMPUTATIONAL GUIDANCE FOR DESIGNING A STORM SEWER DESIGN WITH THE RATIONAL METHOD

#	Variable	Explanation
21	Pipe Design	Enter the quantity of flow determined to be carried in the pipe.
22	Pipe Velocity	Tabulate the actual velocity of flow in the pipe for the design Q.
23	Remarks	Include any remarks or comments, which may affect or explain the design. When routing the 100 year design storm through the system, required elevations for adjacent buildings must be listed in this column.

SECTION 6
OPEN CHANNELS

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6.1 INTRODUCTION

Open channels designed for use in drainage systems have significant cost and capacity advantages. Open channels may be used for recreational and aesthetic purposes, habitat improvement, in-stream storage, and groundwater recharge. Challenges for their use in a public drainage system include potential right-of-way constraints and maintenance costs. Careful planning and design are required to increase the benefits and minimize the challenges.

The ideal open channel is a stabilized watercourse developed by nature over time, characterized by a stable bed and banks. The benefits of such a channel are:

A. Adequate channel storage can decrease downstream peak flows.

B. The maintenance needs of properly stabilized channels are minimal.

C. Flow volumes in the public drainage system may be reduced by natural subsurface infiltration.

D. The disturbance of native vegetation and wildlife is minimized.

E. The channel can provide a desirable green belt that can be used as a recreational area adding significant social benefits.

A stabilized natural channel, or the artificial, man-made channel which most nearly conforms to the character of a stabilized natural channel, is the most preferable design option.

Channel stability, particularly in unprotected alluvial materials, is a problem in urban hydrology because of the significant difference between low flow and peak storm runoff rates. A natural channel must be studied in sufficient detail to determine the measures needed to mitigate potential bottom scour and bank degradation. Proactive erosion control measures should be applied at reasonable cost to preserve the natural appearance without sacrificing hydraulic efficiency. This section provides the criteria and methodology for open channel design.

6.2 DESIGN CRITERIA

6.2.1 <u>Design Frequency</u>

Open channels shall be designed in a manner such that the water surface elevation resulting from the 100-year storm event is at least 2 feet below the lowest finished floor elevation of any existing residential dwelling or public, commercial and industrial building unless the building is flood-proofed. Open channels constructed to convey only stormwater flow for a short period of time shall be designed to convey flow generated by the 10-year design storm event under bank full conditions. Bank full conditions exist when flow is conveyed within the main area of a channel without overtopping the channel bank. Intermittent and perennial stream designs, which convey base flow entirely or partly throughout the year such as blue line stream indicated on United States Geological

Survey topography mapping or Natural Resource Conservation Service (NRCS) soil mapping, shall refer to natural stream design criteria for proper design techniques and construction.

6.2.2 Maintenance Easement

A protected maintenance access corridor shall be provided and recorded along all newly constructed channels. This access corridor shall provide a minimum access width of 20 feet from the channel bank on each side unless otherwise approved by the City of Concord (City). For some small channels, a maintenance corridor may only be required on one side. These restricted maintenance corridors shall be sufficient to allow access by maintenance equipment.

6.3 TYPES OF CHANNELS

Channels are defined as natural or artificial. Natural channels include all watercourses that have developed over time by natural processes. Artificial channels are those constructed or significantly altered by human effort and include roadside ditches and vegetative-lined or improved channels.

6.3.1 Natural Channels

Many natural channels have mild slopes, are reasonably stable, and are not in a state of serious degradation or aggradation. However, if a natural channel is to be used as a drainage feature for storm runoff from an urbanizing area, the altered flow regime resulting from the increase in impervious area may cause scour and erosion. Hydraulic analyses must be performed to identify the erosion tendencies in natural channels. Some on-site modification of the natural channel may be required to assure a stabilized condition. State and Federal regulations may dictate the actions needed to be taken in natural channels where erosion and scouring either is or may occur after development.

The investigation necessary to assure adequate capacity of the natural channel is unique for every channel. The design professional must prepare cross sections of the channel, define the water surface profile for the minor and major design storm, investigate the bed and bank material to determine erosion tendencies, and study the bank slope stability of the channel under design flow conditions. Supercritical flow does not normally occur in natural channels, but calculations must be performed to ensure that this condition will not occur.

6.3.2 Vegetative-Lined Channels

Vegetative-lined channels are the most desirable of the artificial channel types. Vegetation will stabilize the body of the channel, consolidate the soil mass of the bed, reduce erosion on the channel surface, and control the movement of soil particles along the channel bottom. Channel storage, lower velocities, and the vegetative multiple-use benefits create significant advantages over other artificial channels.

The vegetated channel lining creates turbulence, which reduces hydraulic energy and flow velocity. Therefore, the design professional must give full consideration to sediment deposition and scour, as well as hydraulics.

6.3.3 Concrete-Lined Channels

Concrete channel design shall only be allowed upon approval by the City, when project constraints dictate its use.

Concrete linings must be designed to withstand the various forces and actions that tend to overtop the bank lining, deteriorate the lining, erode the soil beneath the lining, and erode unlined areas. Maintenance needs should be included in any drainage plan utilizing concrete-lined channels.

6.3.4 Rock-Lined Channels

Rock-lined channels are constructed from ordinary riprap or wire enclosed riprap (gabions). The rock lining increases turbulence in the flow resulting in a loss of hydraulic energy and reduced flow velocities. The rock lining also permits a higher design velocity, and therefore, a steeper design slope than vegetative-lined channels. Rock linings are also used for erosion control at culvert/storm drain outlets, at sharp channel bends, at channel confluences, and at locally steepened channel sections. Correct sizing and material placement are essential to good performance. Incorrectly designed rock-lined channels can result in excessive maintenance requirements. The use of undersized stone for riprap can lead to washouts of the placed stone, whereas the use of oversized placed stone may contribute to localized scouring and erosion. In either situation, corrective maintenance would be required to stabilize the channel surface. Maintenance needs should be included in any drainage plan utilizing rock-lined channels.

Channels designed with riprap or gabion linings shall only be allowed upon approval by the City, when project constraints dictate their use. The use of riprap for permanent erosion control is permitted only if the site is unsuitable for vegetative lined channels. Rock-lined channels consisting of grouted riprap will not be permitted for use in the City.

6.3.5 Bioengineered Channels

Bioengineering may be utilized for channel slope stabilization where feasible. Adequate water supply must be available during the growing season to keep the vegetation viable.

Bioengineered slopes typically include 6- to 12-inch soil lifts/layers wrapped by biodegradable matting material. Deeply rooted vegetation is placed between the soil layers to create stability. Appropriate methodology documentation by a qualified, experienced bioengineering design professional must be provide for these channels.

6.4 CHANNEL DISCHARGE

Designing a stable channel under dynamic channel conditions requires an understanding of sediment transport and stream channel response. For example, unlined channels must be designed to minimize excessive scour while lined channels must be designed to prevent deposition of sediments. Unlined channels are most successful when designed under the concept of dynamic equilibrium.

All variables used in fluid mechanics and hydraulics fall into one of three classes: those describing the boundary geometry, flow, and fluid. Various combinations of these variables define parameters that describe the state of flow in open channels.

6.4.1 Manning's Equation

Careful attention must be given to the design of drainage channels to provide adequate capacity and allow for minimum maintenance. The hydraulic characteristics of open channels shall be determined by using Manning's Equation, commonly expressed as:

$$V = \frac{1.49}{n} R^{0.67} S_f^{\ 0.5} \qquad (6\text{-}1)$$

where:

V	=	average velocity, in feet per second
R	=	hydraulic radius of channel, A/WP, in feet
S_f	=	slope of the energy gradient, in feet per foot
n	=	Manning's coefficient of channel roughness
WP	=	wetted perimeter of channel wetted by water, in feet
A	=	cross sectional area of channel flow, in square feet

6.4.2 Uniform Flow

Manning's Equation is an accurate representation of flow conditions only when the rate of flow and channel characteristics (roughness, cross section geometry and slope) remain relatively constant, hence, uniform flow. For a channel of given roughness, discharge and slope, there is only one possible depth for maintaining a uniform flow. This depth is commonly expressed as the normal depth. The corresponding discharge is expressed as the normal discharge. Under uniform flow conditions, the water surface profile is parallel to both the energy grade line and the bottom of the channel.

Uniform flow is most often considered a theoretical abstraction. A channel is commonly designed on the assumption it will convey uniform flow at normal depth, but this is difficult, if not impossible, to achieve under non-laboratory conditions. The actual flow depth can differ appreciably from the normal flow depth.

Normal depth computations are made so frequently that it is convenient to use nomographs for various types of open channel cross sections to eliminate the need for trial-and-error solutions. A nomograph for estimating uniform flow for trapezoidal channels is shown in Figure 6-1, on page 6-7.

6.4.3 Critical Flow

Flowing water contains potential and kinetic energy. The relative values of the potential and kinetic energy are important in the analysis of open channel flow. The potential energy is represented by the depth of water plus the elevation of the channel bottom above a datum. The kinetic energy is represented by the velocity head, $V^2/2g$. The specific energy or head is equal to the depth of water plus the velocity head. In this case, the datum is taken as the flow line of the channel.

$$H = d + \frac{V^2}{2g} \tag{6-2}$$

where:

H	=	specific energy head, in feet
d	=	depth of flow, in feet
V	=	average channel flow velocity, in feet per second
g	=	acceleration of gravity, 32.2 feet per second squared

When depth of flow is plotted against specific energy for a given channel discharge at a section, the resulting curve shows that, at a given specific energy, there are two possible flow depths (see Figure 6-2, on page 6-8). At minimum energy, only one depth of flow exists. This is known as the critical depth.

The effect of gravity upon the state of flow is represented by a ratio of the inertial forces to gravity forces. This ratio is known as the Froude Number, Fr, and is used to categorize the flow. Determination of the point of critical flow is very necessary to ensure that a channel design is stable. This is especially important in natural or vegetative lined channels where inappropriate velocity can lead to bank instability problems.

The critical state of flow through an open channel is characterized by several important conditions. Condition E is valid only for rectangular channel geometry.

A. The specific energy is a minimum for a given discharge.

B. The discharge is a maximum for a given specific energy.

C. The specific force is a minimum for a given discharge.

D. The Froude Number is equal to 1.0.

E. The velocity head is equal to half the hydraulic depth in a channel of small slope.

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 flow at or near the critical state is unstable, because minor changes in specific energy, such as from channel debris, can cause a major change in depth.

In the analysis of non-rectangular channels, the Froude Number equation is rewritten below. The hydraulic mean depth of flow is defined as the cross sectional area divided by the top width. Equation 6-3 can also be used in a trial and error approach to solve for critical depth of non-rectangular. Simply set the Froude Number to one, assume a depth, and solve till both sides equate.

$$Fr = \left[\frac{Q^2 T}{gA^3}\right]^{0.5} \tag{6-3}$$

where:

Fr	=	Froude Number
Q	=	discharge in channel, in cubic feet per second
T	=	top width of channel, in feet
g	=	acceleration of gravity, 32.2 per second squared
A	=	cross-sectional area, in square feet

Critical depth for rectangular sections is defined by the following relationship.

$$d_c = \frac{V^2}{g} \tag{6-4}$$

where:

d_c	=	critical depth, in feet
V	=	average channel flow velocity, in feet per second
g	=	acceleration of gravity, 32.2 feet per second squared

The Froude Number, among its many definitions, defines the relationship between inertial and gravitational forces of a discharge for a specific channel. The ratio of this relationship with respect to 1 also indicates its flow regime. A Froude Number of 1 represents critical flow. If the Froude Number is greater than 1, the flow is supercritical, but when the Froude Number is less than 1, the flow is subcritical. Supercritical is characterized by higher velocities and shallower depths. These characteristics present the potential for erosive conditions that would require more protective channel lining. Subcritical flow is characterized by lower velocities but higher depths; subcritical flow is more typical of natural and artificial watercourses.

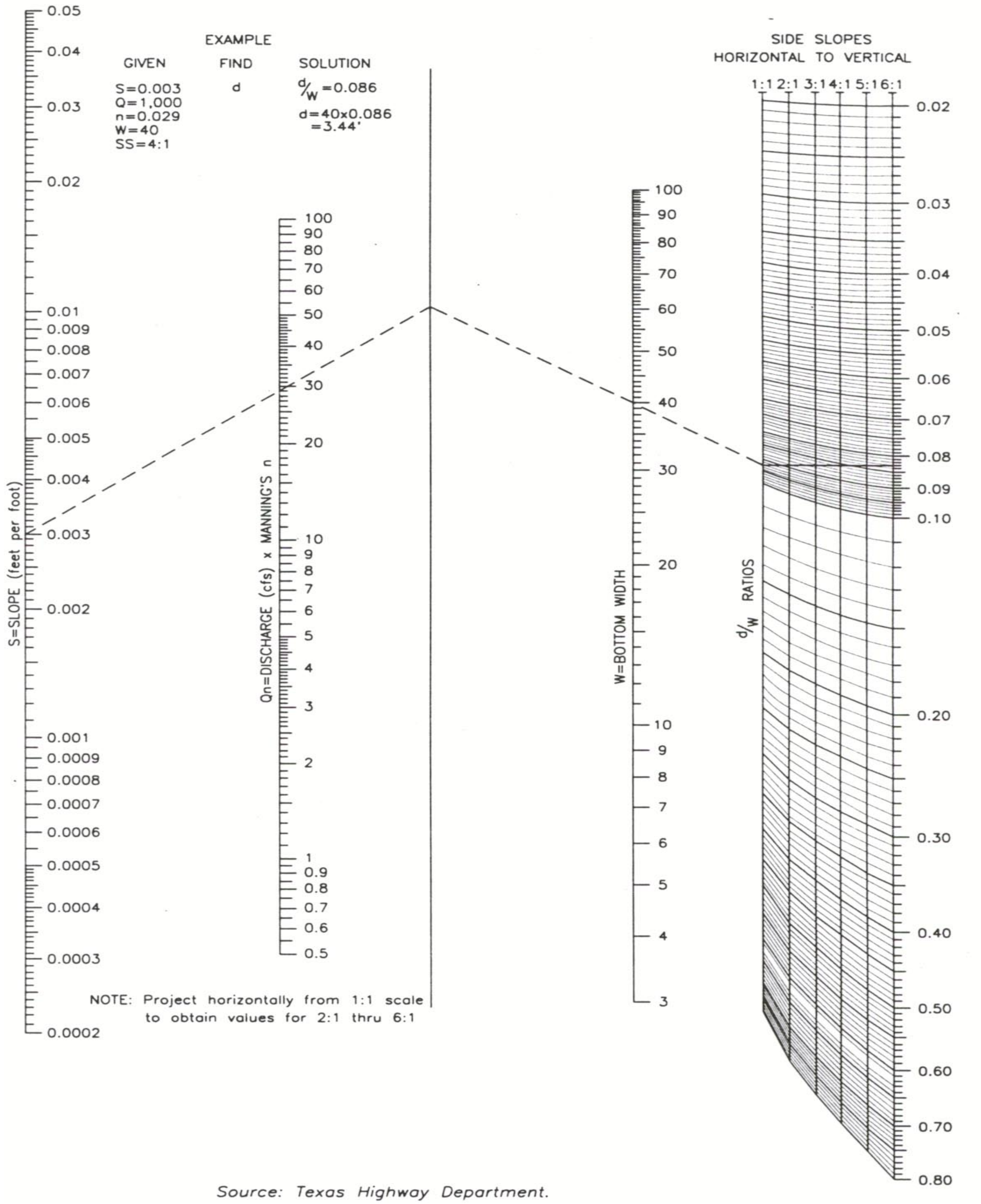
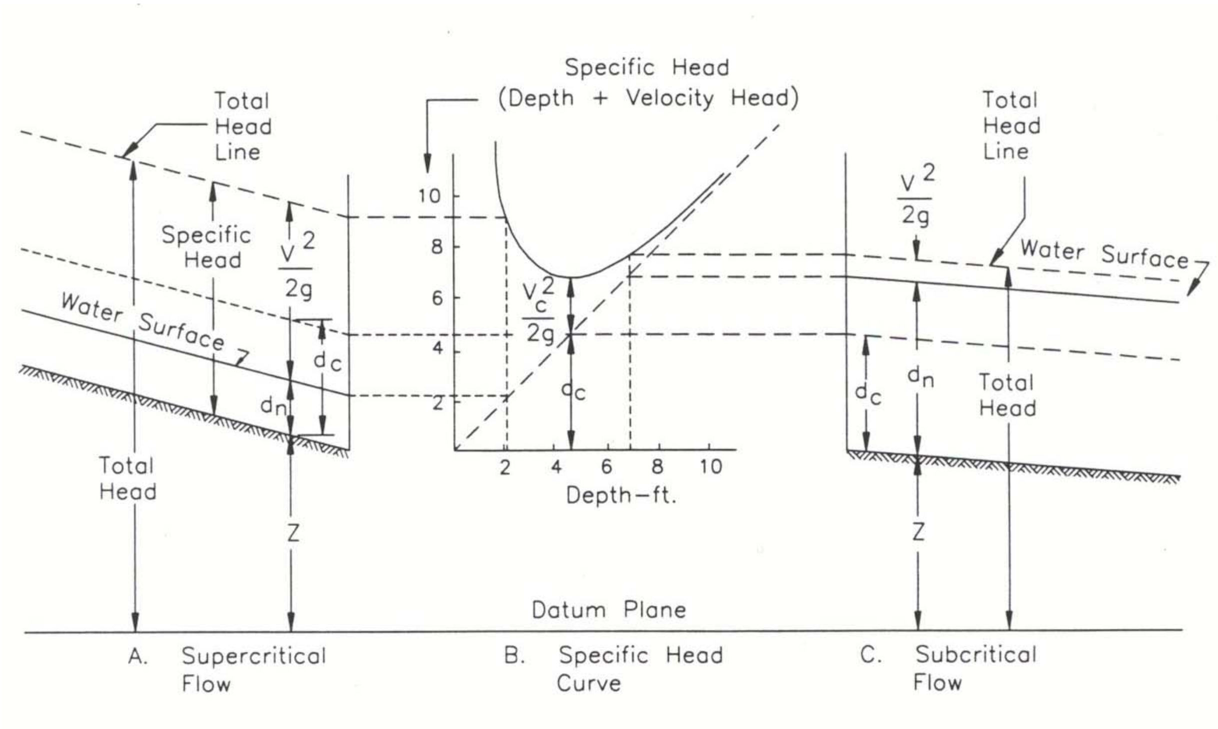


Figure 6-1 Uniform Flow for Trapezoidal Channels



Source: Federal Highway Administration, HDS No. 3 *Design Charts for Open Channel*

Figure 6-2 Definition Sketch of Specific Head

6.4.4 Gradually Varied Flow

Gradually varied flow is used to describe a type of steady non-uniform flow. The change in the depth and velocity occur gradually over a considerable length of channel and the non-uniformity of the flow is not pronounced. The most common occurrence of gradually varied flow in storm drainage is the backwater created by culverts, storm drain inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel and the water surface profile must be computed using backwater techniques. Backwater calculations can be tedious and iterative for some methodologies. Section 6.7 Water Surface Profile Analysis offers some guidance in selecting software for performing these calculations. Guidance for performing these calculations manually is available in Chapter 4 of the Charlotte-Mecklenburg County Storm Water Design Manual.

6.4.5 Rapidly Varied Flow

Rapidly varied flow is characterized by very pronounced curvature of the streamlines. The change in curvature may become so abrupt that the flow profile is virtually broken, resulting in a state of high turbulence. Several mathematical solutions exist for some cases of rapidly varied flow. Design professionals generally rely on empirical solutions of specific problems. The two cases of rapidly varied flow (weir flow and the hydraulic jump) that occur commonly in storm drainage will be discussed in this section.

Weir Flow

Weir calculations are commonly used for spillway outlets in detention ponds. Weirs can also be used for flow measurement, flow diversion, and energy dissipation. The general form of the equation for horizontal crested weirs is in Section 8 Hydraulics of Detention.

Hydraulic Jump

In urban hydraulics, a hydraulic jump may occur at grade changes, grade control structures (i.e., check drops), or at the outlet of an emergency spillway for detention ponds. The evaluation of hydraulic jumps is important since there are associated losses of energy and erosive forces. For hard-lined facilities such as concrete channels, the forces and the change in energy can affect the structural stability or the hydraulic capacity. For vegetative-lined channels, the erosive forces must be controlled to prevent serious damage. Control is usually obtained by check drops or grade control structures which confine the erosive forces to a protected area.

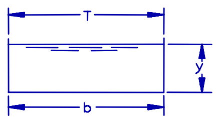
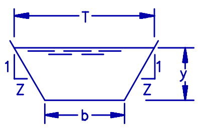
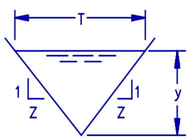
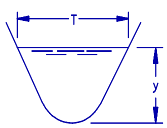
The jump can be adequately defined for box culverts/drains and for spillways using the jump characteristics of rectangular sections. The approximate jump location can be determined by intersecting the energy grade line of the supercritical and subcritical flow reaches. The primary concerns are: (1) if the channel is designed adequately to withstand the forces which may cause localized scouring or damage to the channel, and (2) if the jump will affect the hydraulic characteristics. The effect on capacity can be determined by evaluating the energy grade line taking into account the energy lost by the jump. In general, for a Froude Number less than 2.0, the loss of energy is less than 10 percent. These calculations must be included with the required submittals.

6.5 DESIGN CONSIDERATIONS

Typical channel cross sections are rectangular, trapezoidal, triangular, and parabolic in shape. A triangular channel is a special type of trapezoidal section with a bottom width of zero. Due to the difficulty of maintenance, their application is generally not feasible. Trapezoidal channels of varying bottom widths and side slopes are the most commonly constructed channels. Parabolic channels typically occur over time as corners fill in and bottoms stabilize. Formulas used in channel size design for typical cross section geometrics are presented in Table 6-1, on the next page.

Artificial open channels are commonly designed to have trapezoidal sections of adequate cross sections to incorporate ease of maintenance, uncertainties in runoff estimates, differences in channel roughness coefficients, channel obstructions, and sediment accumulations. These designs tend to incorporated trickle channels for periods of low flow as well. The channel configurations may be dictated by right-of-way constraints and where hard-lined channels are required. State and Federal regulations can also dictate the type of channel and lining.

TABLE 6-1
GEOMETRIC ELEMENTS OF CHANNEL SECTIONS

Section	Area A	Wetted Perimeter WP	Hydraulic Radius R	Top Width T
 Rectangular	by	$b+2y$	$\frac{by}{b+2y}$	b
 Trapezoidal	$by+Zy^2$	$b+2y\sqrt{Z^2+1}$	$\frac{by+Zy^2}{b+2y\sqrt{Z^2+1}}$	$b+2Zy$
 Triangular	Zy^2	$2y\sqrt{Z^2+1}$	$\frac{Zy}{2\sqrt{Z^2+1}}$	$2Zy$
 Parabolic	$\frac{2}{3}Ty$	$T+\frac{8y^2}{3T}$	$\frac{2T^2y}{3T^2+8y^2}$	$\frac{3A}{2y}$

Source: Chow, Ven Te, 1959; Open-Channel Hydraulics

Determination of a representative Manning's "n" value is critical in the analysis of the hydraulic characteristics of an open channel. The "n" value for each channel reach should be based on the individual channel characteristics. Typical minimum, normal, and maximum roughness coefficients for various types of open channels are presented in Table 6-2, on the next page. Table 6-3, on page 6-14, presents a variable "n" value dependent on the depth of flow in the channel. Experience and judgment should also be used in selecting the proper "n" value for a channel. When working with a detailed hydraulic analyses "n" values should be calibrated, whenever possible, to compare the calculated with known water surface conditions.

TABLE 6-2
TYPICAL ROUGHNESS COEFFICIENTS ("n") FOR OPEN CHANNELS

Type of Channel and Description	Minimum	Normal	Maximum
Excavated or Dredged			
a. Earth, straight and uniform:			
1. Clean, recently constructed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish:			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged:			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts:			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut:			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
Natural Streams - Minor streams (top width at flood stage < 100 feet)			
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 stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150

TABLE 6-2 (Continued)
TYPICAL ROUGHNESS COEFFICIENTS ("n") FOR OPEN CHANNELS

Type of Channel and Description	Minimum	Normal	Maximum
Lined or Built-Up Channels			
a. Corrugated Metal	0.021	0.025	0.030
b. Concrete:			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	
8. On irregular excavated rock	0.022	0.027	
c. Concrete bottom, float finished with sides of:			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
d. Gravel bottom with sides of:			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
e. Asphalt:			
1. Smooth	0.013	0.013	
2. Rough	0.016	0.016	
f. Rock-lined:			
1. Riprap	0.023	0.033	0.036
2. Grouted riprap	0.020	0.023	0.026
3. Gabions	0.025		0.033

Source: Chow, Ven Te, 1959; Open-Channel Hydraulics.

TABLE 6-3
MANNING'S ROUGHNESS COEFFICIENTS ("n") FOR STRAIGHT CHANNELS WITHOUT SHRUBBERY OR TREES

Grass Condition	Depth of Flow of 0.7 to 1.5 feet	Depth of Flow greater than 3.0 feet
Bermudagrass, Buffalograss, Kentucky Bluegrass:		
a. Mowed to 2 inches	0.035	0.030
b. Length 4 to 6 inches	0.040	0.030
Good stand, any grass:		
a. Length of 12 inches	0.070	0.035
b. Length of 24 inches	0.100	0.035
Fair stand, any grass:		
a. Length of 12 inches	0.060	0.035
b. Length of 24 inches	0.070	0.035

A Manning's roughness coefficient is used in the design of every channel, no matter what type of natural or man-made lining is used, even if no lining is used. This value and a description of how this value is to be maintained must be included in the maintenance requirements.

If the channel is grass-lined, the frequency and height of grass after mowing should be specified. If the channel and floodplain is composed of grass, brush, and small trees, then the frequency, type of maintenance, and height after maintenance should be specified.

Where applicable, unlined open channels of a given soil type should have sufficient gradient to provide self-cleaning flow velocities but not be so great as to create excessive erosion. Maximum permissible design flow velocities for earth channels are presented in Table 6-4, on page 6-15. Table 6-5, on page 6-16, presents maximum permissible velocities for earth channels with varied grass linings and sloping configurations. Lined channels, drop structures, check dams, or concrete spillways may be required to control erosion that results from high channel flow velocities. Overall, the design of open channels, including stable, alluvial channel systems, is tied closely to the criteria for erosion and sediment control.

TABLE 6-4
MAXIMUM PERMISSIBLE DESIGN OPEN CHANNEL FLOW VELOCITIES IN EARTH*

Soil Types	Permissible Mean Channel Velocity (feet per second)
Fine Sand (noncolloidal)	2.0
Coarse Sand (noncolloidal)	4.0
Sandy Loam (noncolloidal)	2.5
Silt Loam (noncolloidal)	3.0
Ordinary Firm Loam	3.5
Silty Clay	3.5
Fine Gravel	5.0
Stiff Clay (very colloidal)	5.0
Graded, Loam to Cobbles (noncolloidal)	5.0
Graded, Silt to Cobbles (colloidal)	5.5
Alluvial Silts (noncolloidal)	3.5
Alluvial Silts (colloidal)	5.0
Coarse Gravel (noncolloidal)	6.0
Cobbles and Shingles	5.5
Hard Shales and Hard Pans	6.0
Soft Shales	3.5
Soft Sandstone	8.0
Sound rock (igneous or hard metamorphic)	20.0

* These velocities shall be used in conjunction with scour calculations and as approved by the City Engineer.

Source: Chow, Ven Te, 1959: Open Channel Hydraulics.

TABLE 6-5
MAXIMUM PERMISSIBLE VELOCITIES[1] IN VEGETATIVE-LINED CHANNELS

Channel Slope (%)	Soil Characteristics	Vegetative Lining[2]	Permissible Velocity[3] (feet per second)
0-5	Easily Erodible Non-plastic (Sands and Silts)	Bermuda grass	5.0
		Tall fescue	4.5
		Bahia grass	4.5
		Kentucky bluegrass	4.5
		Grass-legume mixture	3.5
	Erosion Resistant Plastic (Clay mixes)	Bermuda grass	6.0
		Tall fescue	5.5
		Bahia grass	5.5
		Kentucky bluegrass	5.5
		Grass-legume mixture	4.5
5-10	Easily Erodible Non-plastic (Sands and Silts)	Bermuda grass	4.5
		Tall fescue	4.0
		Bahia grass	4.0
		Kentucky bluegrass	4.0
		Grass-legume mixture	3.0
	Erosion Resistant Plastic (Clay mixes)	Bermuda grass	5.5
		Tall fescue	5.0
		Bahia grass	5.0
		Kentucky bluegrass	5.0
		Grass-legume mixture	3.5
10-15	Easily Erodible Non-plastic (Sands and Silts)	Bermuda grass	3.5
		Tall fescue	2.5
		Bahia grass	2.5
		Kentucky bluegrass	2.5
	Erosion Resistant Plastic (Clay mixes)	Bermuda grass	4.5
		Tall fescue	3.5
		Bahia grass	3.5
		Kentucky bluegrass	3.5

Source: USDA-NRCS Modified
[1] Permissible velocity based on 10-year design storm peak runoff.
[2] Soil erodibility based on resistance to soil movement from concentrated flowing water.
[3] Before grass is established, permissible velocity is determined by the type of temporary liner used.

6.6 DESIGN STANDARDS

The design standards for open channels cannot be presented in a step-by-step fashion because of the wide range of options available to the design professional. Certain planning and conceptual criteria are particularly useful in the preliminary design of a channel. These criteria, which have the greatest effect on the performance and cost of the channel, are discussed below. Design submittals shall be in a clear and concise format convenient for review and shall include, but not be limited to, (1) storm runoff computations and mapping, (2) hydraulic design computations, assumptions, references, sketches and drawings, (3) floodplain mapping, (4) and all other pertinent data.

6.6.1 Natural Channels

The design criteria and evaluation techniques for natural channels are:

A. The channel and areas above the bankfull elevation (floodplain), shall have adequate capacity for major storm runoff.

B. Natural channel segments with a Froude Number greater than 0.95 for any flow shall be protected from erosion.

C. The water surface profiles shall be defined so that the major storm floodplain can be mapped.

D. Filling of the flood fringe reduces valuable floodplain storage capacity and tends to increase downstream runoff peaks. Filling of the flood fringe is subject to the restriction of floodplain regulations.

E. Roughness factors "n" that are representative of inadequately maintained or "in need of maintenance" conditions shall be used for the analysis of water surface profiles.

F. Roughness factors "n" that are representative of maintained channel conditions shall be used to determine velocity limitations.

G. Erosion control structures, such as riprap, check drops, or check dams, may be required to control flow velocities, including the initial storm runoff.

H. Plan and profile drawings of the major storm floodplain, including flooding limits, shall be prepared. Appropriate allowances for future bridges or culverts, which can raise the water surface profile and cause the floodplain to be extended, shall be included in the analysis.

With most natural waterways, grade control structures should be constructed at regular intervals to decrease slope and control erosion. However, these channels should be left in as near natural condition as possible. For that reason, extensive modifications should not be undertaken unless they are found to be necessary to avoid excessive erosion with subsequent deposition downstream.

The usual rules of freeboard depth (the difference between the water surface for the design storm flow and the top of the constructed bank), curvature, and other guidelines applicable to artificial channels do not necessarily apply to natural channels. There are significant advantages that may occur if the design professional incorporates into his/her planning the overtopping of the channel and localized flooding of adjacent areas that are laid out and developed for the purpose of being inundated during the major storm runoff. Freeboard can be used to gauge the adequacy of a natural channel for future changes in runoff.

6.6.2 Vegetative-Lined Channels

Key parameters in vegetative-lined channel design include velocity, slope, roughness, depth, freeboard, curvature, cross section shape, and lining material. Other factors such as water surface profile computation, erosion control, drop structures, and transitions also play an important role. A discussion of these parameters is presented below.

A. Flow Velocity and Capacity

The maximum normal depth velocity should not exceed 5 feet per second for grass-lined channels. The Froude Number (turbulence factor) shall be less than 0.8 for grass-lined channels. Grass-lined channels having a Froude Number greater than 0.8 are not permitted. The minimum velocity should be greater than 2 feet per second for self-cleansing.

B. Longitudinal Channel Slopes

Vegetative-lined channels normally have slopes of 0.2 to 0.5 percent. Where the natural topography is steeper than desirable, drop structures should be utilized to maintain design velocities.

C. Freeboard for Drainage Ways

Except where localized overflow is desirable for additional ponding benefits or other reasons, the freeboard can be calculated with the following equation:

$$H_{FB} = 1.0 + \frac{V^2}{2g} \tag{6-9}$$

where:

H_{FB}	=	freeboard height, in feet
V	=	average channel flow velocity, in feet per second
g	=	acceleration of gravity, 32.2 feet per second squared

The minimum freeboard should be 1-foot above the computed water surface elevation. Freeboard should not be obtained by the construction of levees.

An approximation of the superelevation of the water surface at a curve for a trapezoidal channel can be obtained from the following equation:

$$h = \frac{V^2 T}{g r_c} \tag{6-10}$$

where:

h	=	superelevation, in feet
V	=	average channel flow velocity, in feet per second
T	=	top width of channel, in feet
g	=	acceleration of gravity, 32.2 feet per second squared
r_c	=	centerline radius of curvature, in feet

The freeboard shall be measured above the superelevated water surface.

D. Curvature

The centerline curvature should have a minimum radius of twice the top width of the design flow water surface, i.e., the length of the water surface perpendicular to the flow line, but not less than 100 feet.

E. Cross Sections

The channel shape may be almost any type suitable to the location and to the environmental conditions. Often, the shape can be chosen to suit open space and recreational needs. However, limitations for the major storm design flow include:

1. Base Flow Channel

The base flow should be carried in a base flow channel. The minimum capacity should be 1 to 3 percent of the 100-year flow, but not less than 1 cubic foot per second. Base flow channels shall be constructed of materials to minimize erosion, to facilitate maintenance, and to aesthetically blend with the adjacent vegetation and soils.

2. Bottom Width

The bottom width shall be consistent with the maximum depth and velocity criteria.

3. Maintenance Easements

A maintenance corridor shall be provided for all channels. Channels with a top width less than 20 feet wide shall be provided with a maintenance corridor on one side of the channel that is at least 20 feet wide and accessible to maintenance equipment. Channels with top widths 20 feet wide and greater shall have a 20 feet corridor on both sides. No permanent structures, fences, or access barriers shall be placed within the maintenance corridor.

4. Side Slopes

Side slopes shall be 4H:1V or flatter; the minimum side slope shall be 3H:1V. Steeper slopes may be used in existing developed areas subject to additional erosion protection and approval from the City.

5. Vegetation

The vegetation species chosen to line channels must be sturdy, drought resistant, easy to establish. A thick root structure is necessary to control weed growth and erosion. The NRCS can provide assistance selecting appropriate grass mixtures and recommending seeding and maintenance methods.

Newly constructed channels must be stabilized immediately after completion and provided with a protective cover of mulch or an appropriate temporary liner sufficient to hold soil and seed in place until vegetation is established. If possible, disturbed areas should be seeded with a permanent vegetation seed mix. To provide quick ground cover the seed mix a season appropriate nurse crop. Vegetation such as rye grain, millet, etc., germinates quickly and will not compete with the sod-forming grasses. When the immediate seeding of permanent vegetation is not practical, an annual crop may be planted and the perennial vegetation seed may be planted later in the stubble or residue.

6.6.3 Concrete-Lined Channels

The criteria for the design and construction of concrete lined channel are presented below:

A. Freeboard

Adequate channel freeboard above the designed water surface shall be provided and should be not less than that determined by Equation 6-11:

$$H_{FB} = 2.0 + 0.025V\,(d)^{0.67} \tag{6-11}$$

where:

H_{FB}	=	freeboard height, in feet
V	=	average channel flow velocity, in feet per second
d	=	depth of flow, in feet

Freeboard shall be provided above superelevation, standing waves, and/or other water surface disturbances. Concrete side slopes should be extended to provide freeboard. Freeboard should not be obtained by the construction of levees.

B. Superelevation

Superelevation of the water surface shall be determined at all horizontal curves and design of the channel section adjusted accordingly.

C. Velocities

Flow velocities should not exceed 8 feet per second or result in a Froude Number greater than 0.9 for non-reinforced linings. Flow velocities should not exceed 18 feet per second for reinforced linings.

6.6.4 Rock-Lined Channels

Channel linings constructed from ordinary riprap, or wire encased rock (gabions) to control channel erosion can be cost effective. Situations for which riprap linings might be appropriate are: (1) where major flows, such as the 100-year flood are found to produce channel velocities in excess of allowable non-eroding values; (2) where channel side slopes must be steeper than 3H:1V; (3) for low flow channels; and (4) where rapid changes in channel geometry occur, such as at channel bends and transitions. State and federal rules can also govern the use of hard linings such as riprap in intermittent and perennial streams.

A. Riprap Channel Linings

Many factors govern the size of the rock necessary to resist the forces tending to move the riprap. For the riprap itself, this includes the size and weight of the individual rocks, the shape of the stones, the gradation of the particles, the blanket thickness, the type of bedding under the riprap, and the slope of the riprap layer. Hydraulic factors affecting riprap include the velocity, current direction, eddy action, and waves. Riprap channel linings should be designed according to <u>North Carolina Erosion and Sediment Control Planning and Design Manual</u>, Appendix 8.05 or Section 6.6.4.

1. Roughness Coefficient

The Manning's roughness coefficient for ordinary riprap and grouted riprap should be selected using Table 6-3. The "n" value is dependent on the predominant rock size.

2. Rock Size

The design should adhere to the North Carolina Department of Transportation (NCDOT) <u>Standard Specifications for Roads and Structures</u> for gradation requirements for riprap.

3. Toe Protection

Where only the channel sides are to be lined, additional riprap is needed to provide for long term stability of the lining. In this case, the riprap lining should extend at least 3 feet below the existing channel bed and the thickness of the blanket below the existing channel bed increased to at least three times d_{50} to accommodate possible channel scour during floods. The term d_{50} means the rock size for which 50 percent of the sample is finer and can be determined by a sieve analysis from a material sample.

4. Channel Bends

The potential for erosion increases along the outside bank of a channel bend due to the acceleration of flow velocities on the outside part of the bend. Thus, it is often necessary to provide erosion protection in channels which otherwise would not need protection.

The minimum allowable radius for a riprap lined bend is 1.2 times the top width of the design flow water surface and in no case less than 50 feet. Riprap protection should be placed along the outside of the bank and extend downstream from the bend a distance equal to the length of the bend.

Where the mean channel velocity exceeds the allowable non-eroding velocity in straight channel sections, the rock size in the bends must be 3 to 6 inches greater around bends having a radius less than the greater of the following: two times the top width, or 100 feet. The minimum allowable radius for a riprap lined bend in this case is also 1.2 times the top width of the design flow water surface.

5. Transitions

Scour potential is amplified by turbulent eddies in the vicinity of rapid changes in channel geometry such as transitions and bridges. Riprap protection for transitions shall be designed in accordance with the Federal Highway Administration's (FHWA) HEC-11, <u>Design of Riprap Revetment</u>.

B. Wire Enclosed Rock (Gabions)

Wire enclosed rock refers to rocks that are bound together in a wire basket so that they act as a single unit, usually referred to as a gabion. One of the major advantages of wire enclosed rock is that it provides an alternative in situations where available rock sizes are too small for ordinary riprap. Another advantage is the versatility that results from the regular geometric shapes of the wire-enclosed rock. The rectangular blocks and mats can be fashioned into almost any shape that can be formed with concrete. Plastic coated wire should be specified. The design professional should be aware that if the flow contains coarse material, sand or gravel, it may break the wire basket, enabling the smaller rocks within the gabions to be transported downstream.

C. Bedding Requirements for Rock-Lined Channels

Long term stability of riprap and gabion erosion protection is strongly influenced by proper bedding conditions. A large percentage of all riprap failures are directly attributable to bedding failures. A properly designed bed provides a buffer of intermediate sized material between the channel bed and the riprap to prevent piping of channel particles through the voids in the riprap. Two types of bedding are commonly used: (1) a granular bedding filter and (2) filter fabric.

1. Granular Bedding

A bed of mineral aggregate is adequate for most ordinary riprap, grouted riprap, or wire encased riprap applications. The NCDOT <u>Standard Specifications for Roads and Structures</u> should be used for granular bedding.

2. Filter Fabric

Filter fabric has proven to be an adequate replacement for granular bedding in many instances. Filter fabric provides an adequate bedding of channel linings along uniform mildly sloping channels where leaching forces are primarily perpendicular to the fabric.

Filter fabric is not a complete substitute for granular bedding. Filter fabric usually provides filtering action only perpendicular to the fabric and usually has only a single equivalent pore opening between the channel bed and riprap. Filter fabric has a relatively smooth surface, which provides less resistance to stone movement. As a result, filter fabric is restricted to slopes no steeper than 2.5H:1V. Tears in the fabric greatly reduce its effectiveness; therefore, direct dumping of riprap on filter fabric is usually not recommended and care must be exercised during construction.

At drop structures and sloped channel drops, where seepage forces may run parallel with the fabric and cause piping along the bottom surface of the fabric, special care is required in the use of filter fabric. Seepage parallel with the fabric might be reduced by folding the edge of the fabric vertically downward about 2 feet (similar to a cutoff wall) at approximately 12-foot intervals along the installation, particularly at the entrance and exit of the channel reach. Filter fabric should be lapped a minimum of 12 inches at roll edges with upstream fabric being placed on top of downstream fabric at the lap.

Fine silt and clay may clog the openings in the filter fabric, preventing free drainage and increasing the failure potential due to uplift. For this reason, a granular filter is recommended for fine silt and clay channel beds.

6.6.5 <u>Other Channel Linings</u>

The criteria for the design of channels with linings other than vegetation, rock, or concrete will be dependent on the manufacturer's recommendations for the specific product. The design professional will be required to submit the technical data in support of the proposed material. Additional information or calculations may be requested by the City to verify assumptions or design criteria. The following minimum criteria will also apply:

A. Flow Velocity

The maximum normal depth velocity will be dependent on the construction material utilized. The Froude Number shall be less than 0.8.

B. Freeboard

Freeboard shall be calculated using the same equation as that for vegetative-lined channels. The design professional should adjust for horizontal curvature.

C. Curvature

The centerline curvature shall have a minimum radius equal to twice the top width of the design flow but not less than 100 feet.

D. Roughness Coefficient

A Manning's "n" value range shall be established by the manufacturer's data with the high value used to determine depth/capacity requirements and the low value used to determine Froude Number and velocity restrictions.

E. Cross Sections

The same cross section criteria as that used for vegetative-lined channels shall apply.

6.7 WATER SURFACE PROFILE ANALYSIS

For final design, water-surface profiles must be computed for all major channels. Computation of the water-surface profile should utilize the standard backwater analysis and consider all losses due to changes in channel velocity, drops, curves, bridge openings, and other obstructions. Computations must begin at a known point and extend in an upstream direction for subcritical flow.

Backwater computation can be made using the standard step method presented in <u>Open-Channel Hydraulics</u>, by Chow. Many computer programs are available for the computation of backwater curves. The most widely used program is HEC-RAS, Water-Surface Profiles, developed by the United States Army Corps of Engineers (USACE). This program will compute water-surface profiles for natural and manmade channels.

WSPRO, a program developed for the FHWA can also be used to analyze one-dimensional gradually varied steady flow in open channels. WSPRO can analyze flow through bridges and culverts, embankment overflow, and multiple-opening stream crossings.

XP-SWMM, Stormwater Management Model, a program originally developed by the Environmental Protection Agency, can also be selected to analyze open channels. In addition, the model is suitable for analyzing closed conduit systems in both free surface discharge and surcharge conditions.

For prismatic channels, the backwater calculation can be computed manually using the Direct Step Method. For an irregular non-uniform channel, the Standard Step Method is used, which is a more tedious iterative process. The use of HEC-RAS, WSPRO, XP-SWMM, or comparable program is recommended for non-uniform channel analysis.

The effects of superelevation and energy losses due to resistance in open channel bends must be considered in backwater computations. In addition to superelevation on bends, flow separation in the bend creates a backwater effect that must also be considered. More detail on determining these effects may be found in Chow's Open Channel Hydraulics.

6.8 SUPERCRITICAL FLOW

Supercritical flow in an open channel creates certain hazards that the design professional must take into consideration. From a practical standpoint, it is generally not possible to have any curvature in such a channel. Careful attention must be taken to ensure against excessive oscillatory waves resulting from minor obstructions, which may extend down the entire length of the channel. Imperfections at joints of lined channels may rapidly cause a deterioration of the joints, resulting in a complete failure of the channel. Additionally, high-velocity flow entering cracks or joints creates an uplift force by converting velocity head to a negative pressure head, which can damage the channel lining.

6.9 ENERGY DISSIPATORS

Hydraulic structures include energy dissipators, in the form of channel drops, transitions, baffle chutes, riprap, plunge pools, and many other specific drainage works. Their shape, size, and other features vary widely depending upon the function to be served. Riprap aprons and plunge pools are the preferred design options to dissipate energy in this area, and therefore, this Article focuses on these two energy dissipaters. The other mentioned types are highly engineered structures. If their design is required, more information may be found in the Bureau of Reclamation publication Hydraulic Design of Stilling Basins and Energy Dissipaters and the FWHA publication HEC-14, Hydraulic Design of Energy Dissipaters for Culverts and Channels.

6.9.1 Riprap

Constructed riprap may serve as a lining as well as a dissipater since it dissipates the flow energy and slows velocities. The Manning's coefficient for roughness for riprap ranges from 0.023 to 0.036 (see Table 6-3). The riprap dissipater design is governed by the same rules and principals as riprap channel lining and riprap apron design. Riprap aprons should be designed according to North Carolina. Erosion and Sediment Control Planning and Design Manual, Appendix 8.06

Traditionally, riprap is placed at locations prone to channel bed and bank erosion and periodically is used as a retrofit design solution when flow regime changes have caused excessive channel flows and velocities higher than permissible for the channel lining. Placement of riprap on the channel bottom and banks downstream of an outlet structure is often required for alleviating possible undermining of the structure.

Riprap design should take into account the following parameters:

- stone durability,
- stone density,
- stone size,
- stone shape,
- stone gradation,
- velocity of flow against the stone,
- filter bed requirements,
- channel side slopes,
- the design flow Froude Number, and
- the size and type of stone readily available from commercial sources.

Most of the riprap mixture should consist of stones having length, width, and thickness dimensions as nearly equal as practical and should not be flat slabs. The riprap layer should be a minimum of 1 1/2 times, as thick as the dimension of the large stones (curve size), and should be placed over a gravel bedding or permeable geotextile layer.

A primary reason for riprap failure is placement of undersized individual stones in the maximum size range. Failure also occurs because of improper engineering design of riprap gradation, seepage control, and/or bedding filter requirements.

Installation of riprap dissipators at culverts located in perennial and intermittent streams must be performed in accordance with North Carolina Division of Water Quality and USACE criteria and must not interfere with the possible migration of water based life in the stream.

6.9.2 Plunge Pools

A plunge pool is naturally formed by scour when a free-falling flow drops vertically into a pool. It will be scoured to a depth relative to the height of fall, depth of tailwater, and the concentration of flow. The plunge pool dissipater mimics this natural phenomenon with a special emphasis on the design material. Since the depth of scour is influenced by the erodibility of the streambed material, the constructed pool must be heavily protected with adequately large riprap or reinforced concrete to safely absorb the energy and erosive forces of falling water. A plunge pool may only be used in a channel with a continuous low flow because of the health and safety hazards created by a stagnant pool.

6.10 FLOW TRANSITIONS

A flow transition structure is a change of channel cross section designed to allow for a minimum amount of flow disturbance. Several types of transitions are shown on Figure 6-3, on the next page. The abrupt (headwall) and the straight line (wingwall) are the most common.

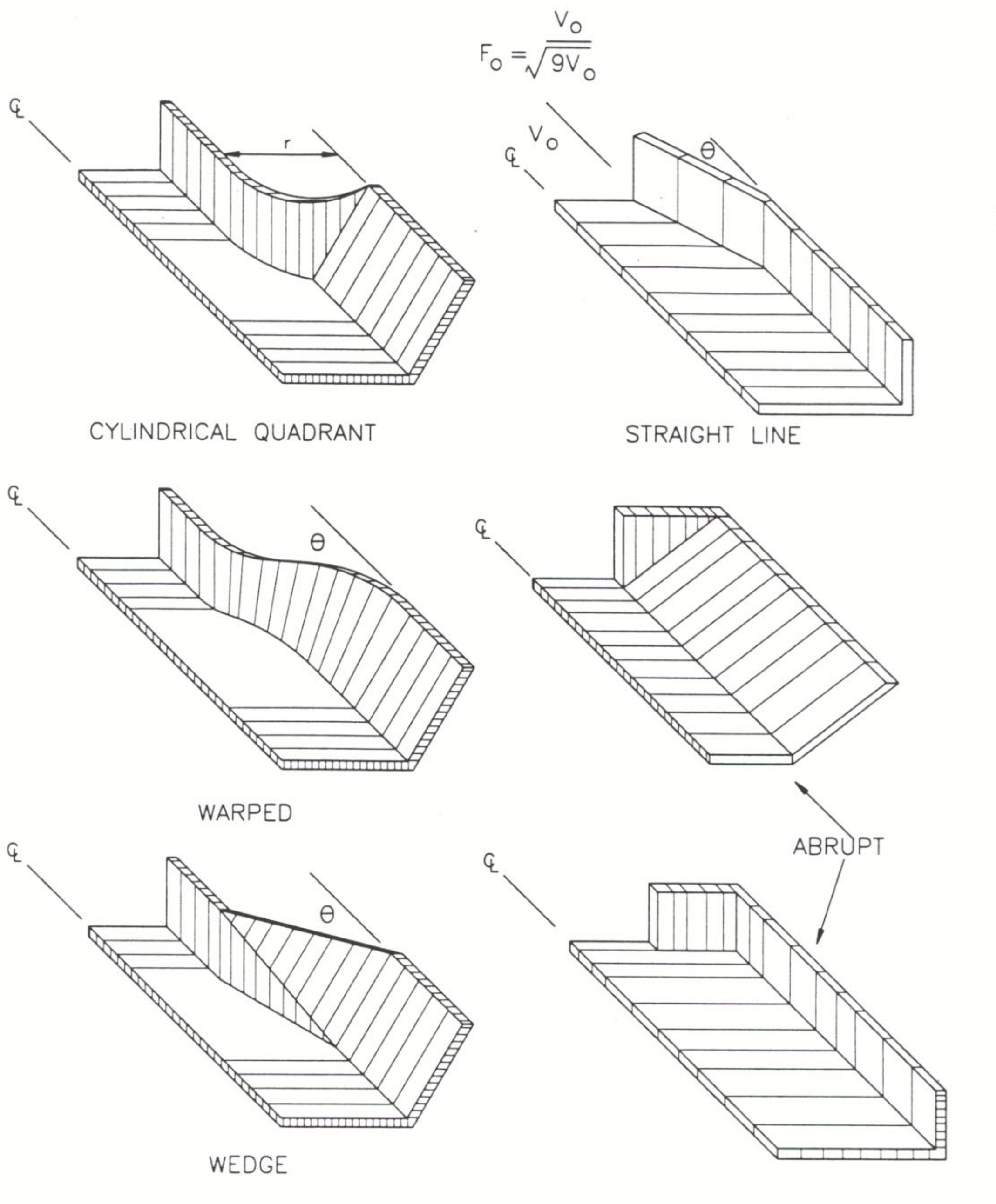
Special inlet transitions are useful when the conservation of flow energy is essential because of allowable headwater considerations. Section 7 Culverts and Bridges includes a discussion on culvert design with improved inlets.

Outlet transitions (expansions) must be considered in the design of all culverts, energy dissipaters, and channel protection. The standard wingwall-apron combinations and expansions downstream of dissipater basins are most common.

6.11 SCOUR

Basically, scour is the net result of an imbalance between the capacity of the flow to transport sediment from an area and the rate of supply of sediment to the area. At a bridge crossing, for instance, the area of interest is the immediate vicinity of the bridge foundation, the piers, and abutments. The imbalance of this capacity and supply can arise from a variety of causes, which can be generally categorized as 1) those characteristics of the stream itself, and 2) those due to the modification of the flow by the bridge piers and abutments.

Because of the overall complexity of the hydrodynamic forces existing in a natural stream channel, the detailed flow pattern in an unobstructed stream cannot be predicted over time with great accuracy. Reasonable estimates can be made based on observations along reaches of similar streams, and in some cases, actual records and measurements for the particular reach of the stream under investigation can be performed. The design professional is encouraged to use the method and procedures in the FWHA's publication HEC-18, Evaluating Scour of Bridges to evaluate scour.



Source: Federal Highway Administration, HEC No. 14, Hydraulic Design of Energy Dissipators for Culverts and Channels.

Figure 6-3 Channel Transition Types

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SECTION 7
CULVERTS AND BRIDGES

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7.1 INTRODUCTION

The function of a drainage culvert or bridge is to pass the design storm flow under a roadway, railroad, or other features within the design criteria parameters without negative impacts on the upstream or downstream channel, adjacent areas, or structures. Symbols used in this unit are defined throughout and also in Section 7.8 List of Symbols.

7.2 DESIGN CRITERIA

7.2.1 Design Frequency

Culverts and bridges shall be designed such that stormwater does not overtop the associated roadway for the minor storm event. The minor storm recurrence intervals for various roadway types are presented in Table 1-1 in Section 1.3 Design Policy. In addition, during the 100-year storm event, stormwater shall not overtop any street crossing over a Federal Emergency Management Agency (FEMA) regulated streams; otherwise, for streams not regulated by FEMA overtopping of alley or local streets shall not exceed 12 inches. Overtopping of collector, thoroughfare, and freeway roadways is not permitted. Residential dwellings public, commercial, and industrial buildings shall not be inundated at the lowest finished floor, which must be 2 feet above the elevation generated by 100-year storm event, unless the building is flood proofed.

7.2.2 Culvert Discharge Velocity

The culvert design must consider effects of discharge velocity, eddies, or turbulence on the natural channel, downstream property, and roadway embankment. The maximum permissible velocity in the downstream channel shall be based on criteria discussed in Section 6.5 Design Consideration.

A minimum velocity of 2.5 feet per second must be maintained for the design flow in all culverts to prevent sedimentation. Deviation from this minimum velocity is permitted where the construction of a culvert is regulated by the United States Army Corps of Engineers (USACE) for the purpose of protection of habitat for aquatic wildlife. Areas prone to siltation or scour must be protected commensurate with the value of the structure and surrounding property and installed to protect against damage or structure failure.

7.2.3 Culvert Material Type

Culverts shall be made of either concrete or corrugated metal. Structural plate products are also acceptable. Culverts must comply with North Carolina Department of Transportation requirements for material and performance criteria.

7.2.4 Bridge Openings

Bridge openings should be designed to have as little effect on the flow characteristics as possible, and be consistent with good bridge design and economics. Bridge structures must not protrude into the cross section of the design flow.

7.2.5 Floodplain Management

Certain areas within the City have been designated by the FEMA as flood hazard areas. All work impacting or adjacent to these areas shall be in compliance with all current FEMA regulations and permit requirements. The City has designated Floodplain Protection Overlay Districts (FPOD) consistent with Article 4.14 of its Unified Development Ordinance. All developments within a FPOD that apply for a Stormwater Permit must document that the development is in full compliance with the FPOD regulations.

7.3 CULVERT TYPES

Culverts shall be selected based on hydraulic principles, with a size and shape that creates a headwater depth that will not cause damage to adjacent property. The range of operating conditions must be known to properly design a culvert. Two major types of culverts exist, based on the hydraulic characteristics of flow: inlet and outlet control. For each type of control, different factors and equations are used to compute the hydraulic capacity of a culvert.

A culvert barrel may flow full or partially full. Full flow throughout the length of pipe is rare, and generally at least part of the barrel flows partially full. Water is flowing under pressure in a full flow condition and the capacity of the culvert is affected by the upstream and downstream conditions and the hydraulic characteristics of the culvert. Partially full or free surface flow can be categorized as subcritical, critical, or supercritical.

A dimensionless parameter known as the Froude Number is calculated to help categorize the flow. The Froude Number is discussed in Section 6.4.3 Critical Flow. The different flow regimes are summarized in Table 7-1.

TABLE 7-1
FLOW CATEGORIES

Flow Type	Depth	Velocity	Froude Number
Subcritical	$y > y_c$	$V < V_c$	$Fr < 1$
Critical	$y = y_c$	$V = V_c$	$Fr = 1$
Supercritical	$y < y_c$	$V > V_c$	$Fr > 1$

7.3.1 <u>Inlet Control</u>

Under inlet control, only the headwater and the inlet configuration affect the hydraulic performance. The headwater depth is measured from the invert of the culvert entrance to the water surface at the culvert entrance. This depth of water at the culvert entrance supplies the energy necessary to force flow through a culvert. The inlet face area is the same as the barrel area for non-improved inlets. The inlet edge configuration plays an important role in the hydraulic efficiency of culverts. Inlet types include projected, mitered, squared edges in a headwall, and beveled edge configurations. Figure 7-1, on the next page, shows several types of flow hydraulics under inlet control conditions.

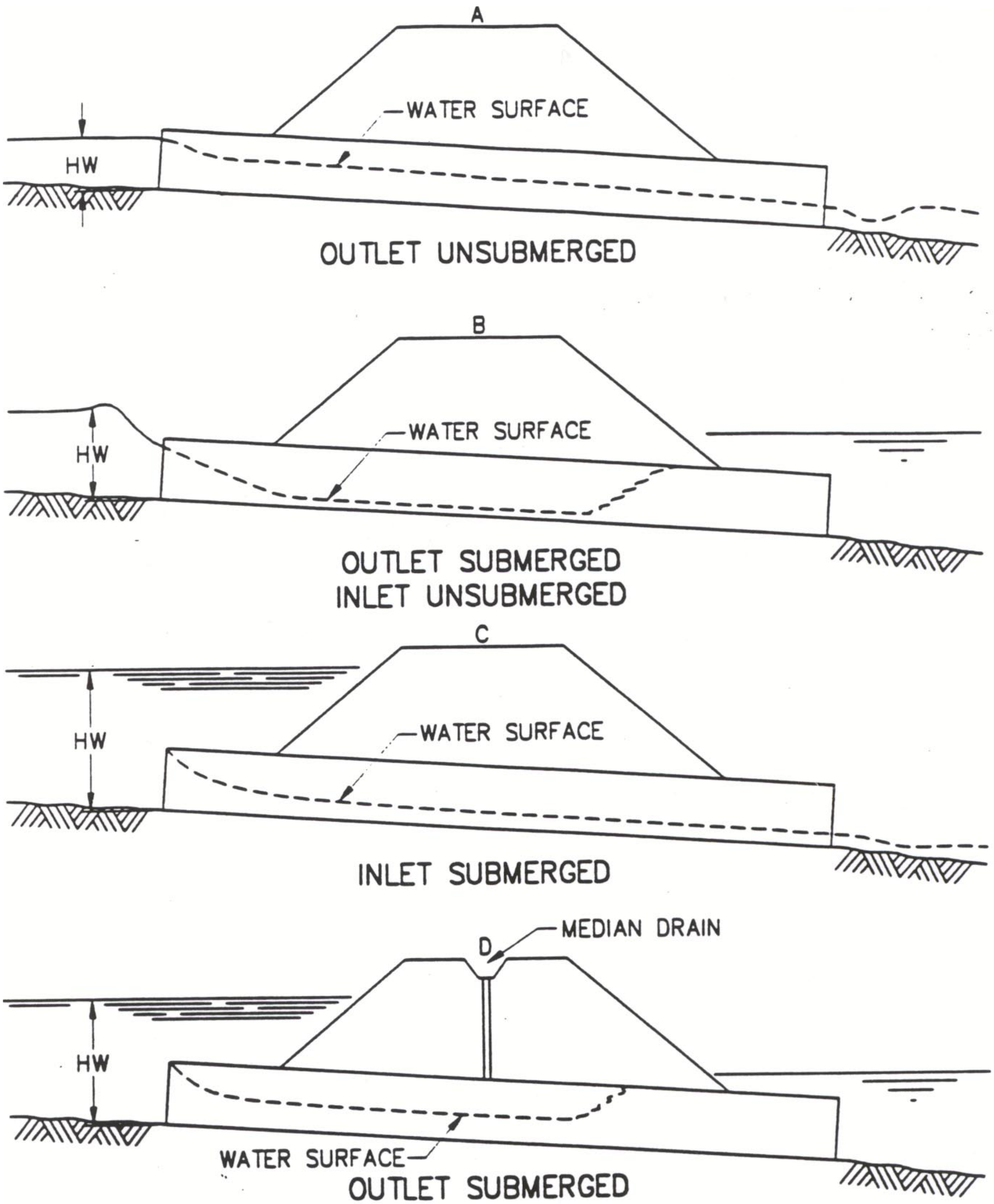


Figure 7-1 Types of Inlet Control

In a condition where neither the inlet nor the outlet ends of the culvert are submerged, the flow passes through critical depth just downstream of the culvert entrance and the flow in the barrel is supercritical. The barrel flows partially full over its length, and the flow approaches normal depth at the outlet end.

If critical flow occurs near the inlet, the culvert is operating under inlet control. The maximum discharge through a culvert flowing partially full occurs when flow is at critical depth for a given energy head. To assure that flow passes through critical depth near the inlet, the culvert must be laid on a slope equal to or greater than critical slope for the design discharge. Placing culverts that flow partially full on slopes greater than critical slope will increase the outlet velocities but will not increase the discharge. Discharge is limited by the section near the inlet where critical flow occurs.

The capacity of a culvert flowing partially full with inlet control is governed by the following equation when the approach velocity nears zero (see Figure 7-1A).

$$HW = d_c + \frac{V_2^2}{2g} + h_e \tag{7-1}$$

where:

HW	=	headwater depth, depth of water above the invert at the upstream end of the culvert in feet
d_c	=	critical depth of flow, in feet
V_2	=	critical velocity at entrance of culvert, in feet per second
g	=	acceleration of gravity, 32.2 feet per second squared
h_e	=	entrance head loss, in feet
	=	$k_e\left[\frac{V_2^2}{2g}\right]$
k_e	=	entrance loss coefficient (Table 7-2)

The submergence of the outlet end of the culvert does not assure outlet control as shown in Figure 7-1B. In this case, the flow just downstream of the inlet is supercritical and a hydraulic jump forms in the culvert barrel.

Figure 7-1C is a more typical design situation. The inlet end is submerged and the outlet end flows freely, the flow is supercritical and the barrel flows partially full over its length. Critical depth is located just downstream of the culvert entrance, and the flow is approaching normal depth at the downstream end of the culvert.

Figure 7-1D illustrates the fact that submergence of both the inlet and the outlet ends of the culvert does not assure full flow. In this case, a hydraulic jump will form in the barrel. A median inlet is required to ventilate the culvert barrel. If the barrel were not ventilated, sub-atmospheric pressures

TABLE 7-2
ENTRANCE LOSS COEFFICIENTS

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

[1] "End sections conforming to fill slope" are the sections commonly available from manufacturers. From limited hydraulic tests, they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections incorporating a closed taper in their design have superior hydraulic performance.

Source: Federal Highway Administration, Hydraulic Design Series No. 5, Hydraulic Design of Highway Culverts.

could develop and create an unstable condition during which the barrel would alternate between full flow and partially full flow.

7.3.2 Outlet Control

All the factors affecting the hydraulic performance of a culvert in inlet control also influence culverts in outlet control. In addition, culvert performance under outlet control conditions is affected by the barrel characteristics (roughness, area, shape, length, and slope) and the tailwater elevation.

Barrel roughness is a function of the culvert material and is represented by Manning's "n" coefficient. The barrel length is the total length extending from the entrance to the exit of the culvert. The barrel slope is the actual slope of the culvert and is often equivalent to the slope of the stream. The tailwater elevation is based upon the downstream water surface elevation measured from the outlet invert. Backwater calculations or normal depth approximations, when appropriate, are two methods used to determine the tailwater elevation. Figure 7-2, on the next page, depicts several examples of culvert hydraulics under outlet control.

Figure 7-2A represents the classic full flow condition, with both inlet and outlet submerged. The barrel is conveying pressurized flow throughout its length. This condition is often assumed in calculations, but seldom presents itself in non-laboratory conditions.

In Figure 7-2B, the outlet is submerged with the inlet open to the atmosphere. For this case, the headwater is shallow so that the inlet crown is exposed as the flow contracts into the culvert.

Flow at the outlet of most culverts is open to atmospheric pressure, but, depending on topography or downstream constraints, a tailwater pool of a depth sufficient to submerge the outlet may form. For an outlet to be submerged, the depth at the outlet must be equal to or greater than the diameter of pipe or height of box plus any elevation difference between the invert out and the stream bed. The capacity of a culvert flowing full with a submerged outlet is governed by the following equation when the approach velocity nears zero. Outlet velocity is based on full-pipe flow at the outlet.

$$HW = H + TW - S_o L \tag{7-2}$$

where:

HW	=	headwater depth, in feet, depth of water above the invert at the upstream end of the culvert.
H	=	head for culvert flowing full, in feet
TW	=	tailwater depth, in feet
S_o	=	slope of culvert, in feet per foot
L	=	length of culvert, in feet

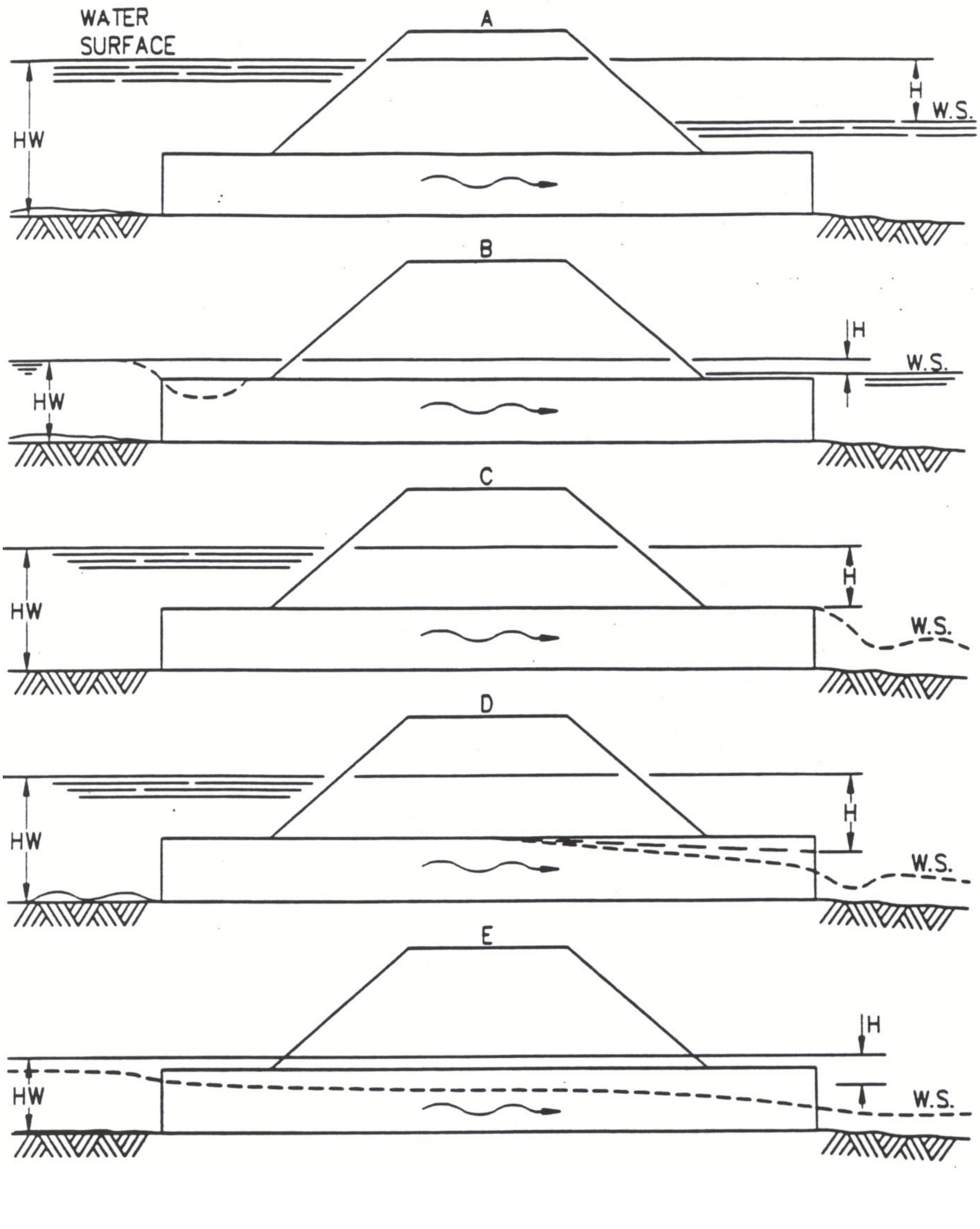


Figure 7-2 Types of Outlet Control

Figure 7-2C shows the entrance submerged to such a degree that the culvert flows full throughout its entire length while the exit is open to the atmosphere. This rare condition requires an extremely high headwater to maintain full barrel flow without tailwater. Outlet velocities are usually high under this condition.

Figure 7-2D represents a more typical situation. The culvert entrance is submerged due to high headwater and the outlet end flows freely with a low tailwater. For this condition, the barrel flows partially full over at least part of its length (subcritical flow) and flow reaches a critical depth just upstream of the outlet.

The capacity of a culvert flowing full over at least part of its length with a submerged entrance ($HW \geq 1.2\ D$) is governed by the following equation when the approach velocity nears zero. Outlet velocity is based on critical depth if the tailwater depth is less than critical depth ($TW < d_c$). If the tailwater depth is greater than critical depth ($TW > d_c$), outlet velocity is based on the tailwater depth.

$$HW = H + P - S_o L \tag{7-3}$$

where:

HW	=	headwater depth, in feet, depth of water above the invert at the upstream end of the culvert
H	=	head for culverts flowing full, in feet
P	=	pressure line height, in feet
	=	$(d_c + D)/2$
d_c	=	critical depth, in feet
D	=	diameter or height of structure, in feet
S_o	=	slope of culvert, in feet per foot
L	=	length of culvert, in feet

In the condition where neither the inlet nor the outlet end of the culvert are submerged as in Figure 7-2E, the barrel flows partially full over its entire length, and the flow profile can be subcritical or supercritical. The tailwater depth can be above or below critical depth.

If the headwater pool elevation does not submerge the culvert inlet ($HW < 1.2D$), the design discharge is subcritical ($S_o < S_c$), the tailwater depth is above critical depth ($TW \geq d_c$), and the control occurs at the outlet. The capacity of the culvert is governed by the following equation when the approach velocity is considered zero.

$$HW = TW + \frac{V_{TW}^2}{2g} + h_e + h_f - S_o L \tag{7-4}$$

where:

HW	=	headwater depth above the invert of the upstream end of the culvert in feet (headwater depth must be equal to or less than 1.2D)
TW	=	tailwater depth above the invert at the downstream end of the culvert, in feet
V_{TW}	=	culvert discharge velocity, at tailwater depth, in feet per second
h_e	=	entrance head loss, in feet
	=	$k_e\left[\frac{V_E^{\ 2}}{2g}\right]$
V_E	=	velocity just inside the culvert, in feet per second
k_e	=	entrance loss coefficient (Table 7-2)
g	=	acceleration of gravity, 32.2 feet per second squared
h_f	=	friction head loss, in feet
	=	$\frac{29n^2L}{R^{1.33}}\left[\frac{V^2}{2g}\right]$
n	=	Manning's roughness coefficient
L	=	length of culvert barrel, in feet
V	=	average culvert velocity, in feet per second
	=	Q/A
Q	=	discharge, in cubic feet per second
A	=	cross sectional area of flow, in square feet
R	=	hydraulic radius, in feet
	=	A/WP
WP	=	wetted perimeter, in feet
S_o	=	slope of culvert, in feet per foot

The capacity of a culvert flowing partially full with outlet control and a tailwater depth below critical depth ($TW < d_c$) is governed by the following equation when the approach velocity nears zero. The entrance is open to the atmosphere ($HW < 1.2D$), and the design discharge is subcritical ($S_o < S_c$).

$$HW = d_c + \frac{V_c^{\ 2}}{2g} + h_e + h_f - S_o L \qquad (7\text{-}5)$$

where:

HW	=	headwater depth above the invert of the upstream end of the culvert, in feet (headwater must be equal to or less than 1.2D, or entrance is submerged).
d_c	=	critical depth, in feet
V_c	=	critical velocity occurring at critical depth, in feet per second
h_e	=	entrance head loss, in feet
	=	$k_e \left[\frac{V_c^{\ 2}}{2g}\right]$
k_e	=	entrance loss coefficient (Table 7-2)
g	=	acceleration of gravity, 32.2 feet per second squared
h_f	=	friction head loss, in feet
	=	$\frac{29n^2L}{R^{1.33}}\left[\frac{V^2}{2g}\right]$
V	=	average pipe velocity, in feet per second
	=	Q/A
Q	=	discharge, in cubic feet per second
A	=	cross sectional area of flow, in square feet
n	=	Manning's roughness coefficient
L	=	length of culvert barrel, in feet
R	=	hydraulic radius, in feet
	=	A/WP
WP	=	wetted perimeter, in feet
S_o	=	slope of culvert, in feet per foot

7.4 CULVERT END TREATMENTS

Properly designed headwalls and endwalls for culverts provide gradual flow transition, anchor the culvert to prevent movement, control erosion and scour resulting from excessive flow velocities and turbulence, and prevent adjacent soil from sloughing into the waterway opening.

Headwalls shall be constructed of reinforced concrete. Shapes, such as, straight parallel headwalls, flared headwalls, or warped headwalls, outfitted with or without aprons, depend on local site requirements. All corrugated metal culverts shall have a headwall and endwall with the exception of driveway culverts for roadside ditches, unless otherwise directed by the City.

7.4.1 Entrance Conditions

It is important to recognize that the operating characteristics of a culvert may be completely changed by the shape or condition at the inlet or entrance. Culvert designs must consider energy losses that may occur at the entrance. The entrance head losses may be determined by the following equation.

$$h_e = k_e \left[\frac{V^2}{2g} \right] \tag{7-6}$$

where:

h_e	=	entrance head loss, in feet
k_e	=	entrance loss coefficient (Table 7-2)
V	=	velocity of flow in culvert, in feet per second
g	=	acceleration of gravity, 32.2 feet per second squared

7.4.2 Headwall/Endwall Treatment

In general, the following guidelines should be used in the selection of the headwall or endwall type:

Straight Headwall and Endwall

A. Approach velocities are low (below 6 feet per second).
B. Backwater pools may occur.
C. Approach channel is undefined.
D. Sufficient right-of-way or easement is available.
E. Downstream channel protection is not required.

Flared Headwall and Endwall

A. Channel is well defined.
B. Approach velocities are between 6 and 10 feet per second.
C. Medium amount of debris is expected.

Warped (Twisted) Headwall and Endwall

A. Channel is well defined and concrete lined.
B. Approach velocities are between 8 and 20 feet per second.
C. Medium amount of debris is expected.

Warped headwalls outfitted with drop-down aprons effectively accelerate flow through the culvert and are effective endwalls where flow transitions from closed conduit flow to open-channel flow. This type of headwall design is appropriate only where the drainage structure is large and right-of-way or easement is limited.

7.4.3 Improved Inlets

Several types of improved inlets have been developed. The use of these inlets may provide substantial savings by allowing for a barrel size reduction in the proposed structure. The use of these inlets is optional and should be based on an economic analysis by the designer. For box culverts, reinforced concrete structures, and structures using headwalls, the use of beveled inlets or tapered inlets is required. Inlet improvements are reflected in the flow capacity calculations through the entrance loss coefficient, k_e presented in the Table 7-2. For more information and the design procedure, the designer must refer to HDS-5, Hydraulic Design of Highway Culverts, by the Federal Highway Administration (FHWA).

7.5 CULVERT DESIGN WITH STANDARD INLETS

The information and publications recommended to design culverts according to the procedure presented in this Section can be found in HDS-5, Hydraulic Design of Highway Culverts. For special cases and larger structure sizes, the FHWA publication HDS-10, Capacity Charts for the Hydraulic Design of Highway Culverts should be used. In addition, a PC compatible computer program (HY-8) is available from the FHWA to perform these calculations.

7.5.1 Culvert Sizing

Figures 7-3 through 7-10 contain a series of curves that show the discharge capacity per barrel for each of several sizes of similar type culverts for various headwater depths. Headwater is measured from the invert of the culvert, which is the low point of the culvert's cross section.

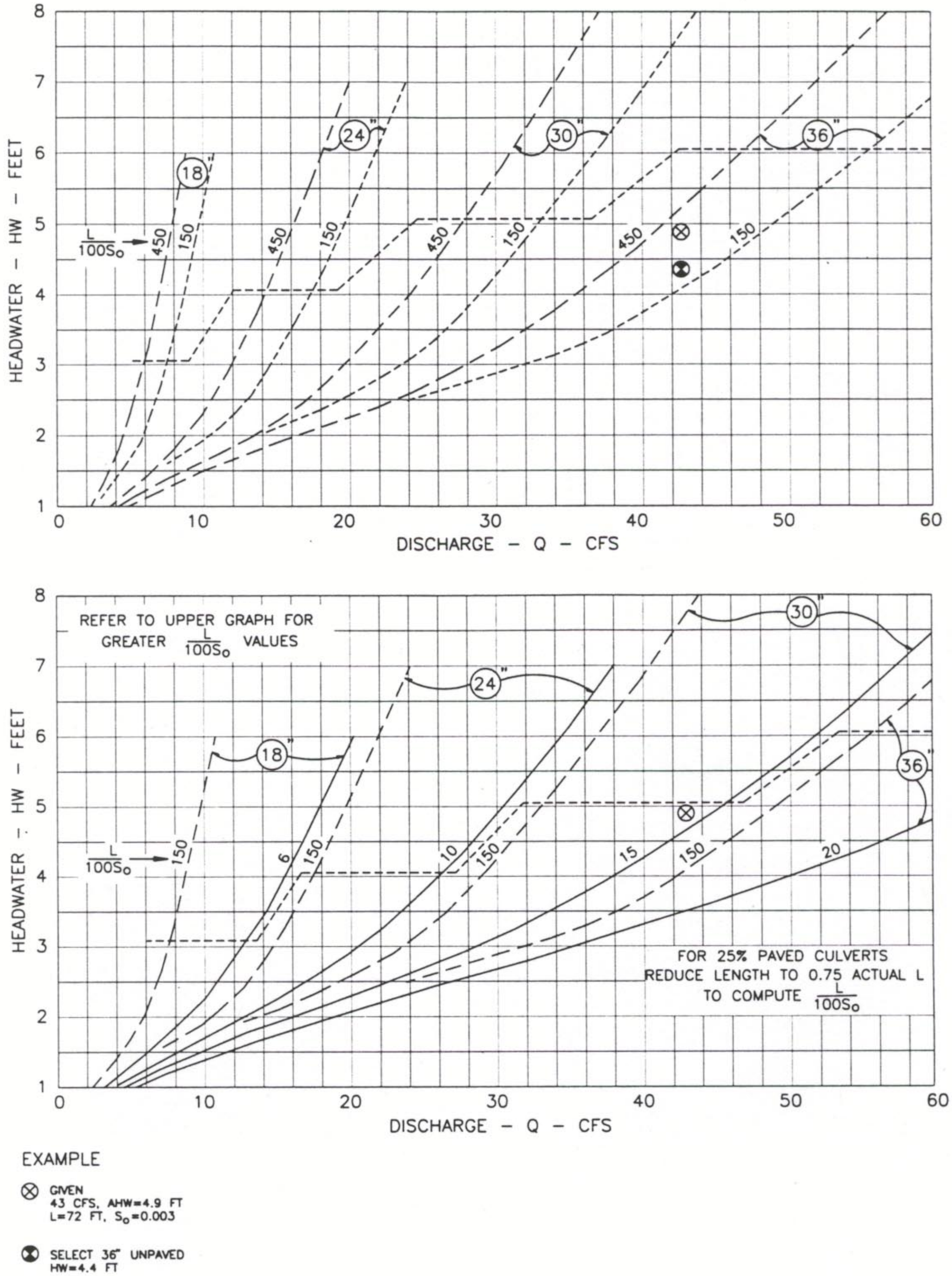
Each culvert size is described by two lines, one solid and one dashed. The numbers associated with each line are the ratio of the length, L, in feet, to the slope, $100S_o$, in percent. The dashed lines represent the maximum L/(100S_o) ratio for which the curves may be used without modification. The solid line represents the division between outlet and inlet control. For values of L/(100S_o) less than that shown on the solid line, the culvert is operating under inlet control and the headwater depth is determined from the L/(100S_o) value given on the solid line. The solid-line inlet control curves are plotted from model test data. The dashed-line outlet control curves are computed for culverts of various lengths with relatively flat slopes. It should be noted that unrestricted flow at the outlet was assumed (tailwater depth is assumed to not influence the culvert performance).

For culverts flowing under outlet control, the head loss at the entrance was computed, and the hydraulic roughness of the various culvert materials was taken into account in computing resistance loss for full or partly full flow. The Manning's "n" values used for each culvert type ranged from 0.012 to 0.032. Table 7-3 lists typical roughness coefficients for various types of culverts.

TABLE 7-3
MANNING'S ROUGHNESS COEFFICIENTS "n" FOR CULVERTS

Construction Materials	Design Coefficient[1]					
Concrete Pipe		0.012				
Corrugated-Metal Pipe*	2-2/3"x1/2" Corrugations					3"x1" Corrugations
Unpaved		0.024				0.027
25% Paved		0.021				0.023
	Diameter					
Structural Plate Pipe*	5 ft		7 ft		10 ft	15 ft
Unpaved	0.033		0.032		0.030	0.028
25% Paved	0.028		0.027		0.026	0.024
Helically Corrugated Pipe*	2-2/3"x1/2" Corrugations					
	Diameter					
	12"	18"	24"	36"	48"	60" and Larger
Unpaved	0.011	0.014	0.016	0.019	0.020	0.021
25% Paved	--	--	0.015	0.017	0.020	0.019
	3"x1" Corrugations					
	Diameter					
	48"	54"	60"	66"	72"	78" and Larger
Unpaved	0.023	0.023	0.024	0.025	0.026	0.027
25% Paved	0.020	0.020	0.021	0.022	0.022	0.023

[1] Designer may select a single representative "n" for design purposes.
* Fully Paved All Types 0.012
Source: American Iron and Steel Institute, Modern Sewer Design



Source: Federal Highway Administration, HEC No. 10, Capacity Charts for the Hydraulic Design of Highway Culverts.

Figure 7-3 Culvert Capacity Standard Circular CMP, Headwall Entrance 18" to 36"

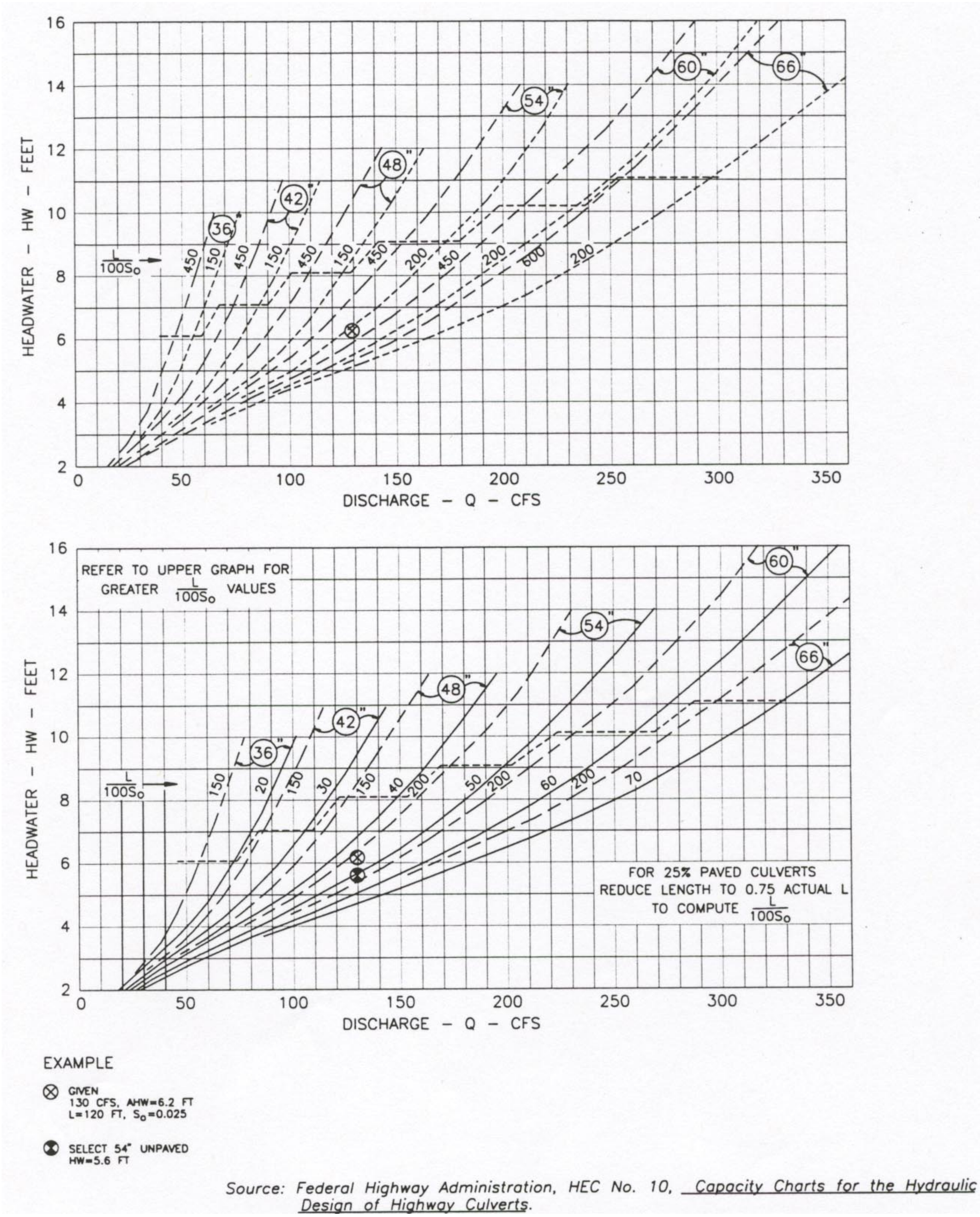
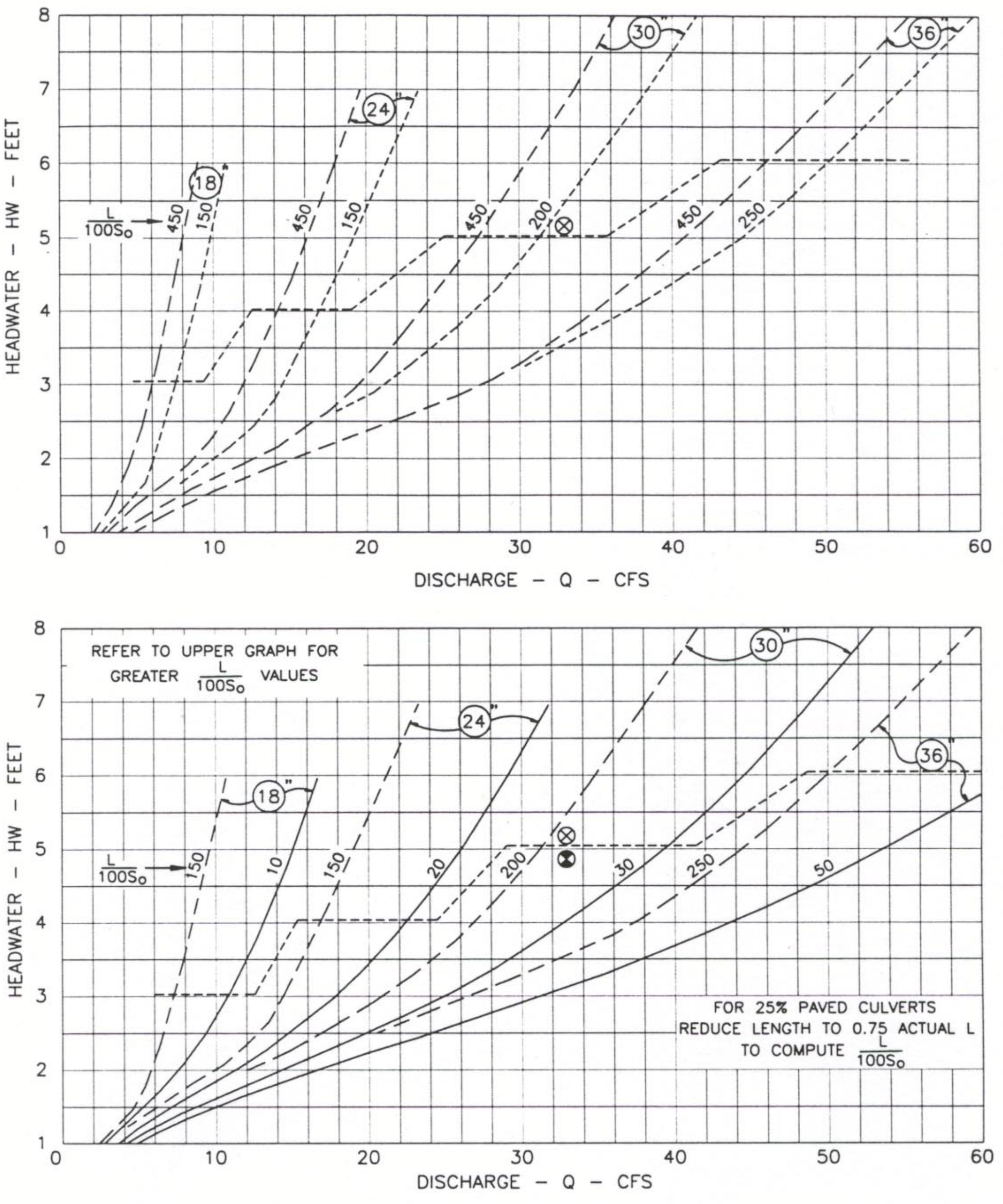


Figure 7-4 Culvert Capacity Standard Circular CMP, Headwall Entrance 36" to 66"



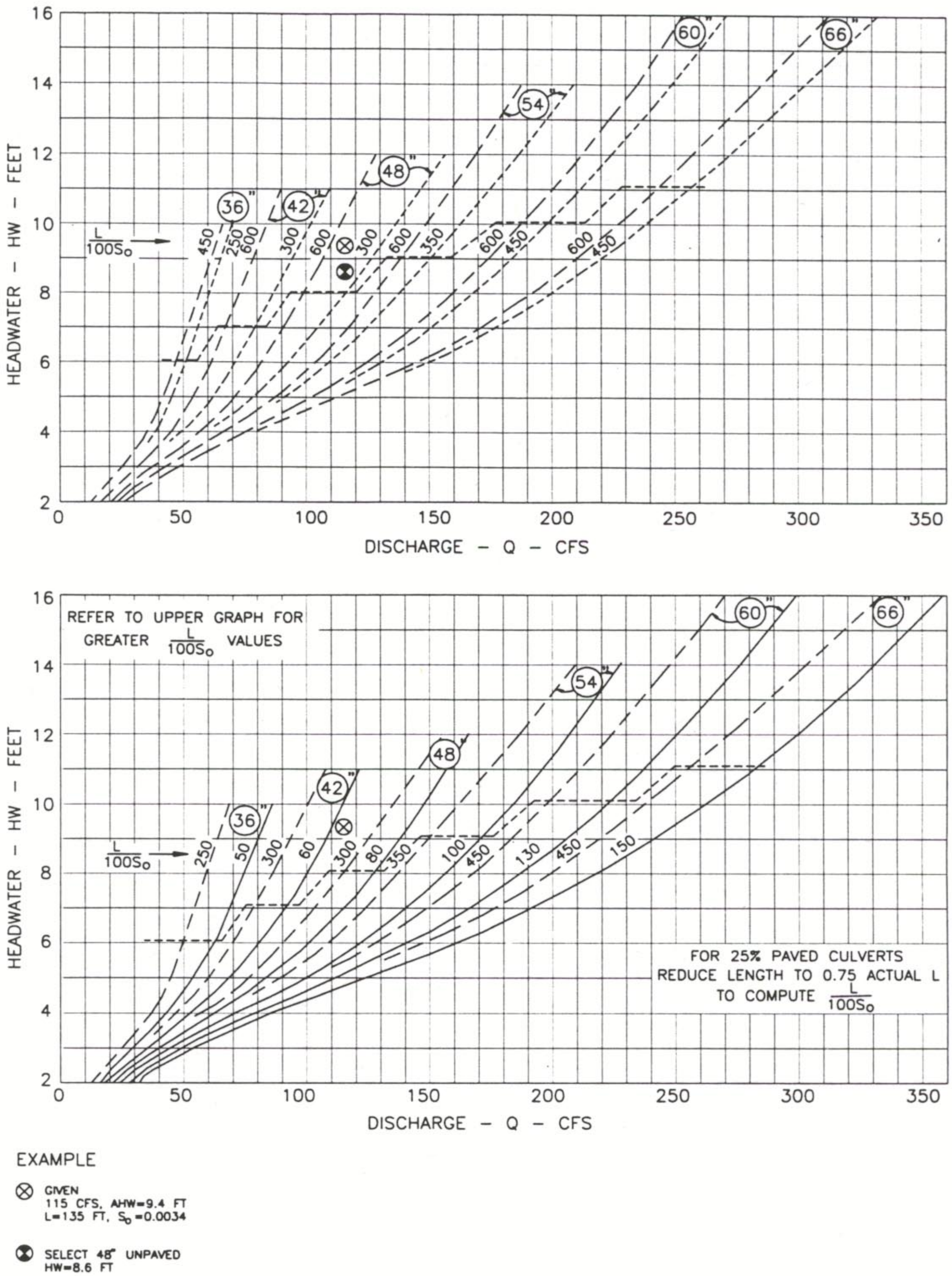
EXAMPLE

⊗ GIVEN
43 CFS, AHW=5.2 FT
L=70 FT, S_o=0.005

SELECT 30" UNPAVED
HW=4.9 FT

Source: Federal Highway Administration, HEC No. 10, Capacity Charts for the Hydraulic Design of Highway Culverts.

Figure 7-5 Culvert Capacity Standard Circular CMP, Projecting Entrance 18" to 36"



Source: *Federal Highway Administration, HEC No. 10, Capacity Charts for the Hydraulic Design of Highway Culverts.*

Figure 7-6 Culvert Capacity Standard Circular CMP, Projecting Entrance 36" to 66"

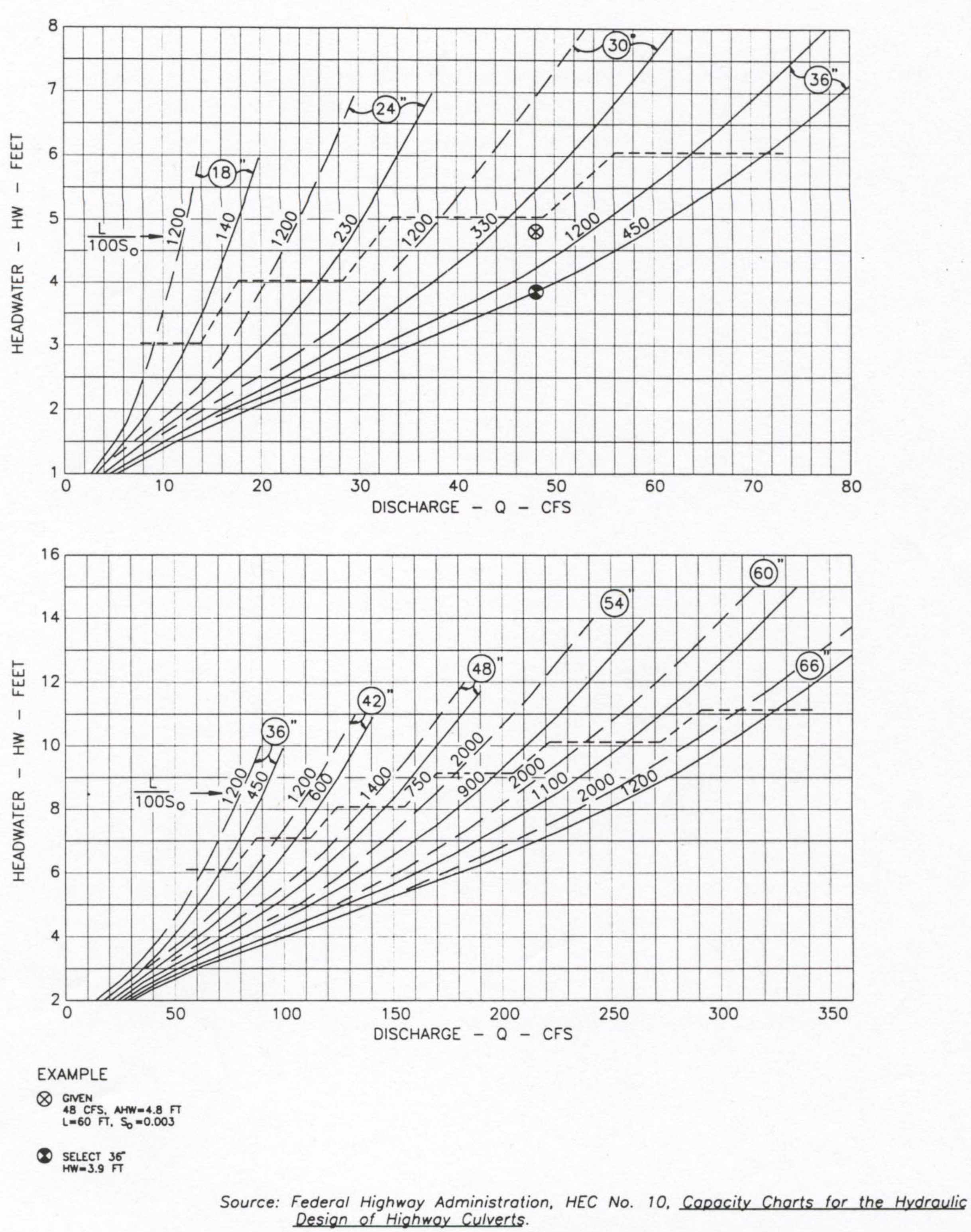
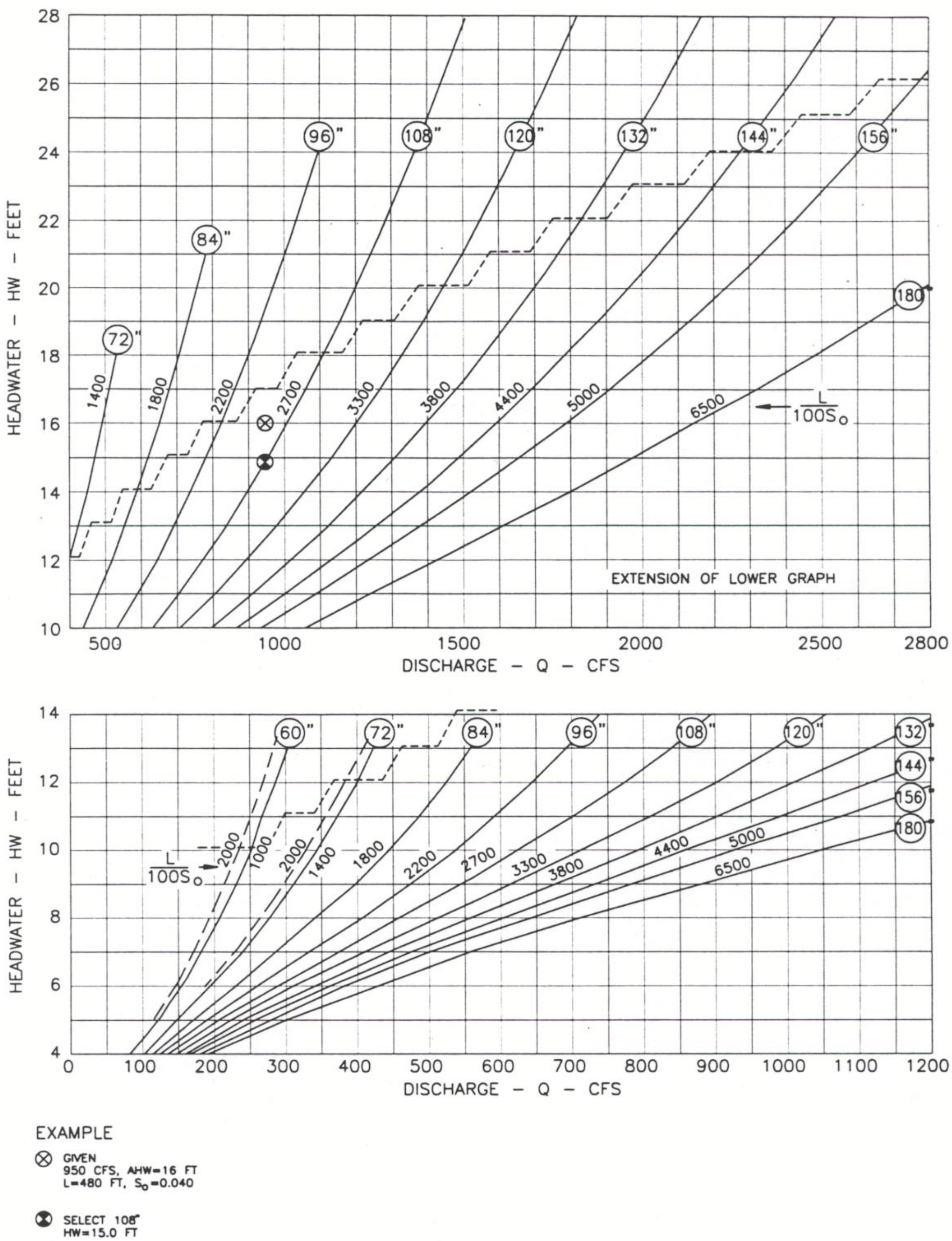
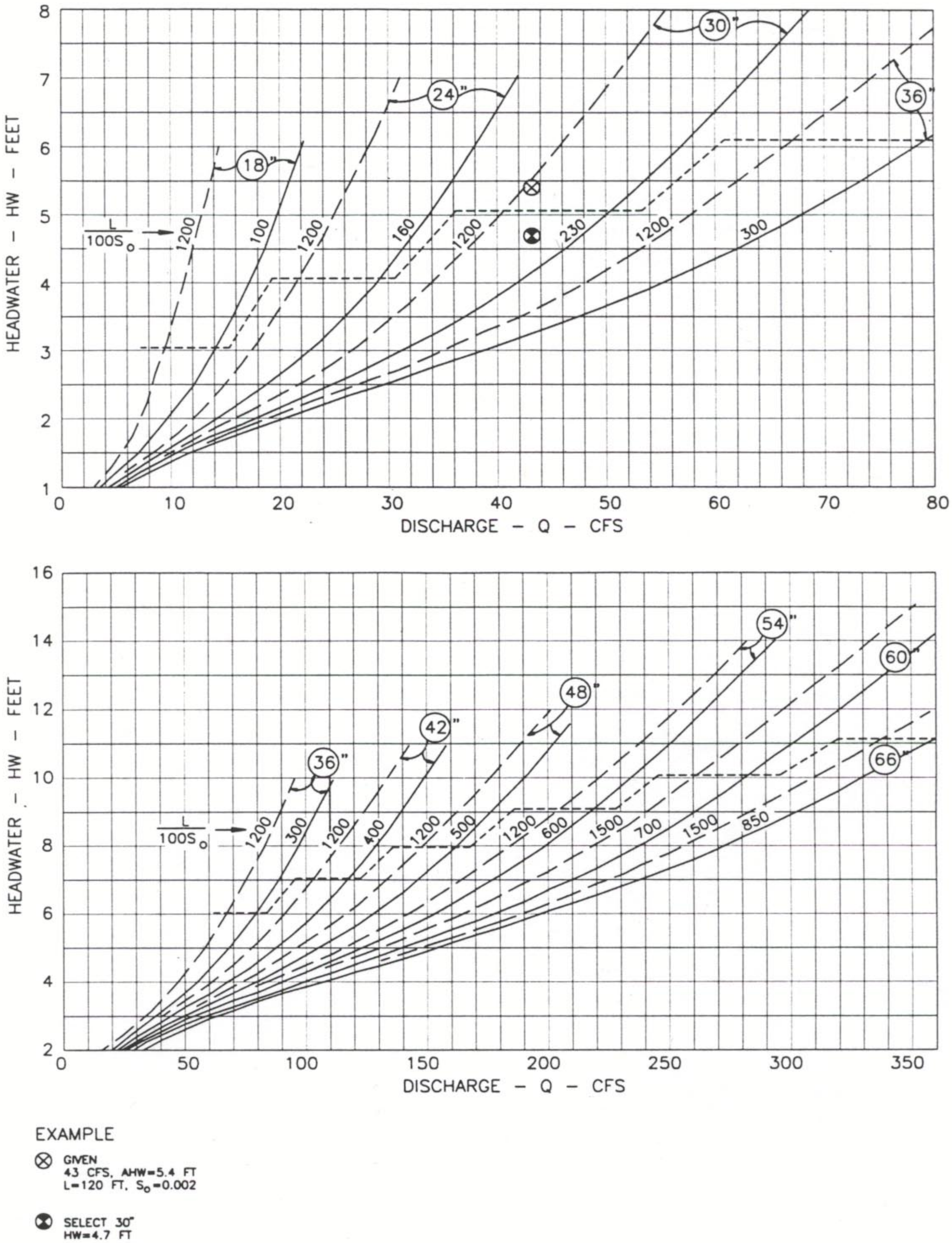


Figure 7-7 Culvert Capacity Standard Circular RCP, Square Entrance 18" to 66"



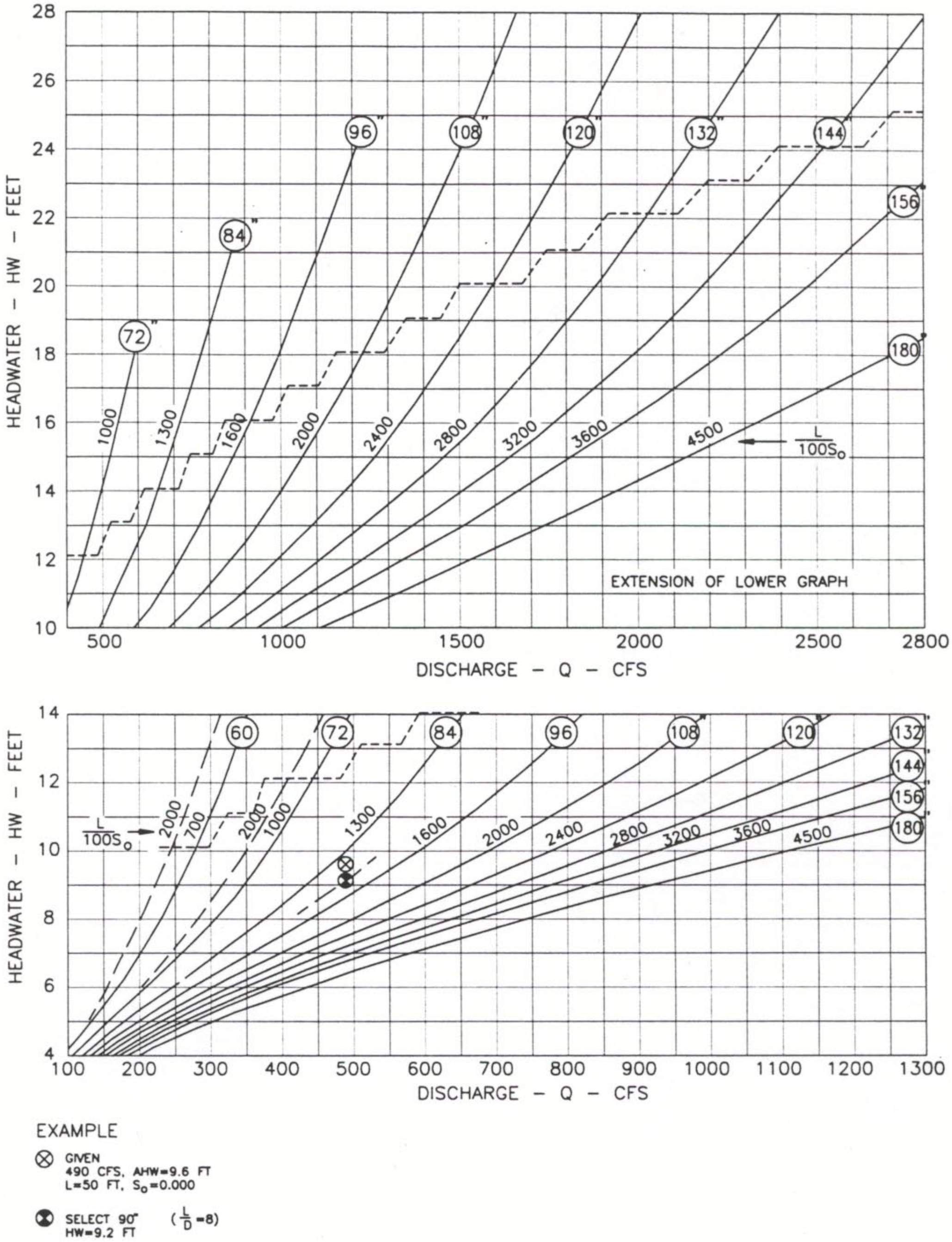
Source: Federal Highway Administration, HEC No. 10, Capacity Charts for the Hydraulic Design of Highway Culverts.

Figure 7-8 Culvert Capacity Standard Circular RCP, Square Edged Entrance 60" to 180"



Source: Federal Highway Administration, HEC No. 10, Capacity Charts for the Hydraulic Design of Highway Culverts.

Figure 7-9 Culvert Capacity Circular RCP, Groove-Edged Entrance 18" to 66"



Source: Federal Highway Administration, HEC No. 10, Capacity Charts for the Hydraulic Design of Highway Culverts.

Figure 7-10 Culvert Capacity Circular RCP, Groove-Edged Entrance 60" to 180"

Except for large pipe sizes, headwater depths on the charts extend to three times the culvert height. Pipe arches and oval pipe show headwater up to 2.5 times their height since they are generally used in areas of low fill. The dotted line, stepped across the charts, shows headwater depths of approximately twice the barrel height and indicates the upper limit of restricted use of the charts. Above this line, the headwater elevation should be checked with the standard inlet and outlet control nomographs found in Figures 7-11 through 7-18.

The headwater depth given by the charts is actually the difference in elevation between the culvert invert at the entrance and the total head, that is depth plus velocity head, for flow in the approach channel. In most cases, the water surface upstream from the inlet is close to this level, since velocities are low in the area upstream of the culvert, and the chart determination may be used as headwater depth for practical design purposes. Where the approach velocity is in excess of 3.0 feet per second, the velocity head must be subtracted from the curve determination of headwater to obtain the actual headwater depth.

Proper use of the capacity charts, Figures 7-3 through 7-10, will minimize problems of scour or sedimentation. The procedure for sizing a culvert is summarized below.

A. List design data: Q (cubic feet per second), L (feet), allowable HW (feet), S_o (feet per foot), type of culvert barrel and entrance.

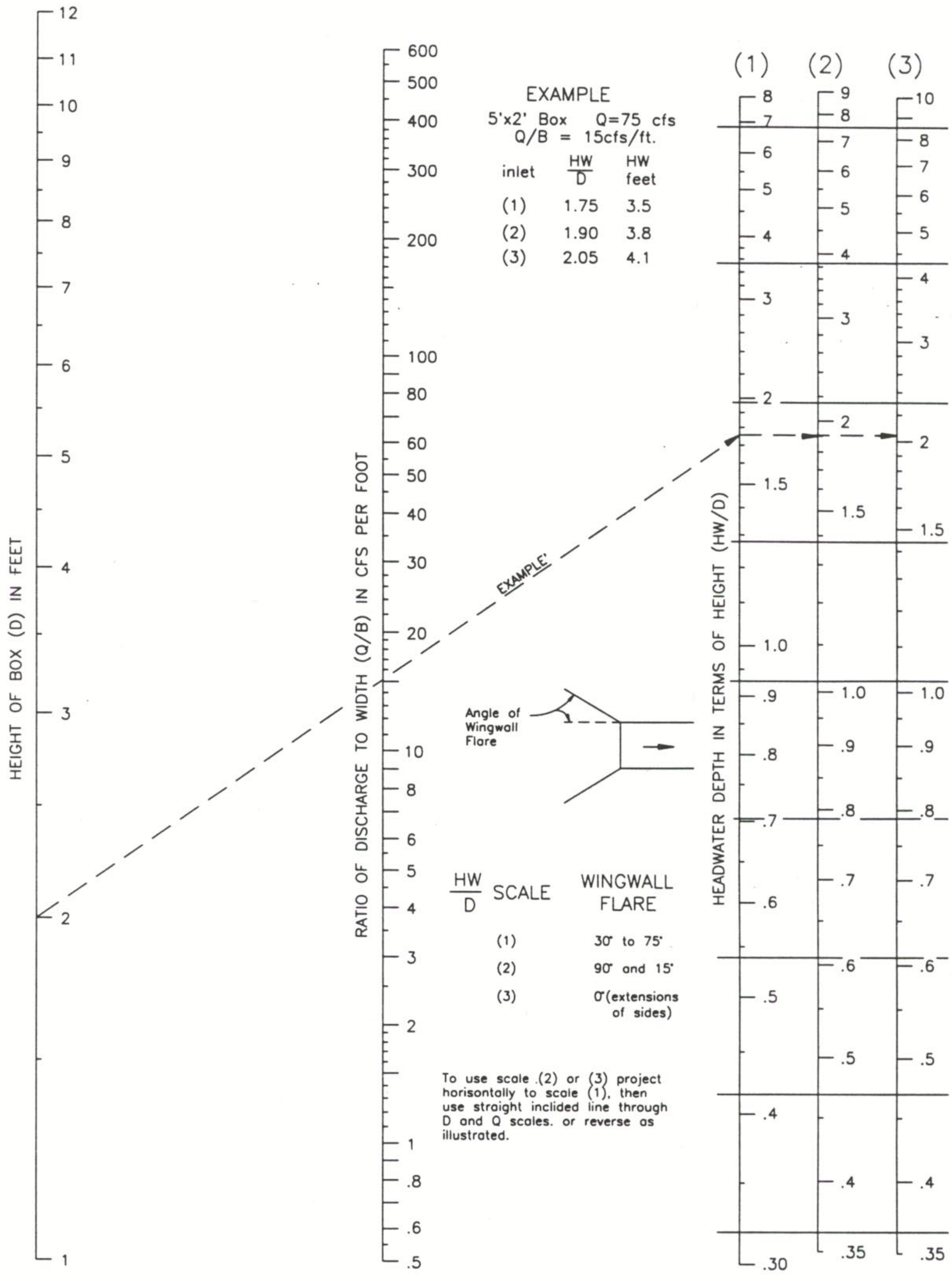
B. Compute $L/(100S_o)$.

C. Enter the appropriate capacity chart with the design discharge, Q.

D. Find the $L/(100S_o)$ value for the smallest pipe that will pass the design discharge. If this value is above the dotted line in Figures 7-3 through 7-10, use the nomographs to check headwater conditions.

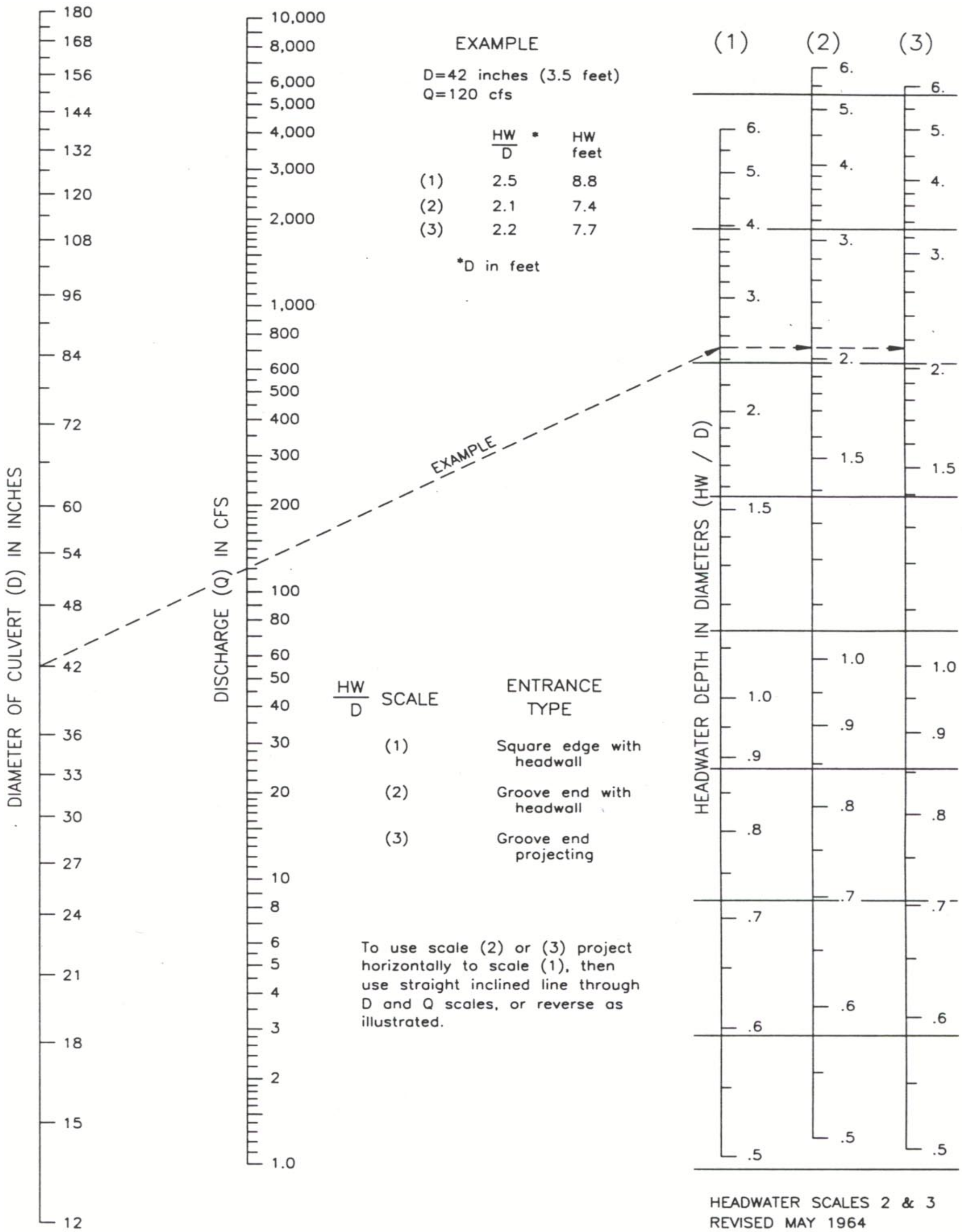
E. If the computed $L/(100S_o)$ is less than the value of $L/(100S_o)$ given for the solid line, then the value of HW is the value obtained from the solid line curve. If the computed L/(100S) is larger than the value for the dashed outlet control curve, then special measures must be taken, and the reader is referred to the FHWA publications listed in the bibliography. Check the HW value obtained with the allowable HW. If the indicated HW is greater than the allowable HW, then try the next larger pipe size.

The advantage of the capacity charts (Figures 7-3 through 7-10) over the standard inlet nomographs (Figures 7-11 through 7-18) is that the capacity charts are direct where the nomographs are trial and error. The capacity charts can be used only when the flow reaches critical depth at the outlet.



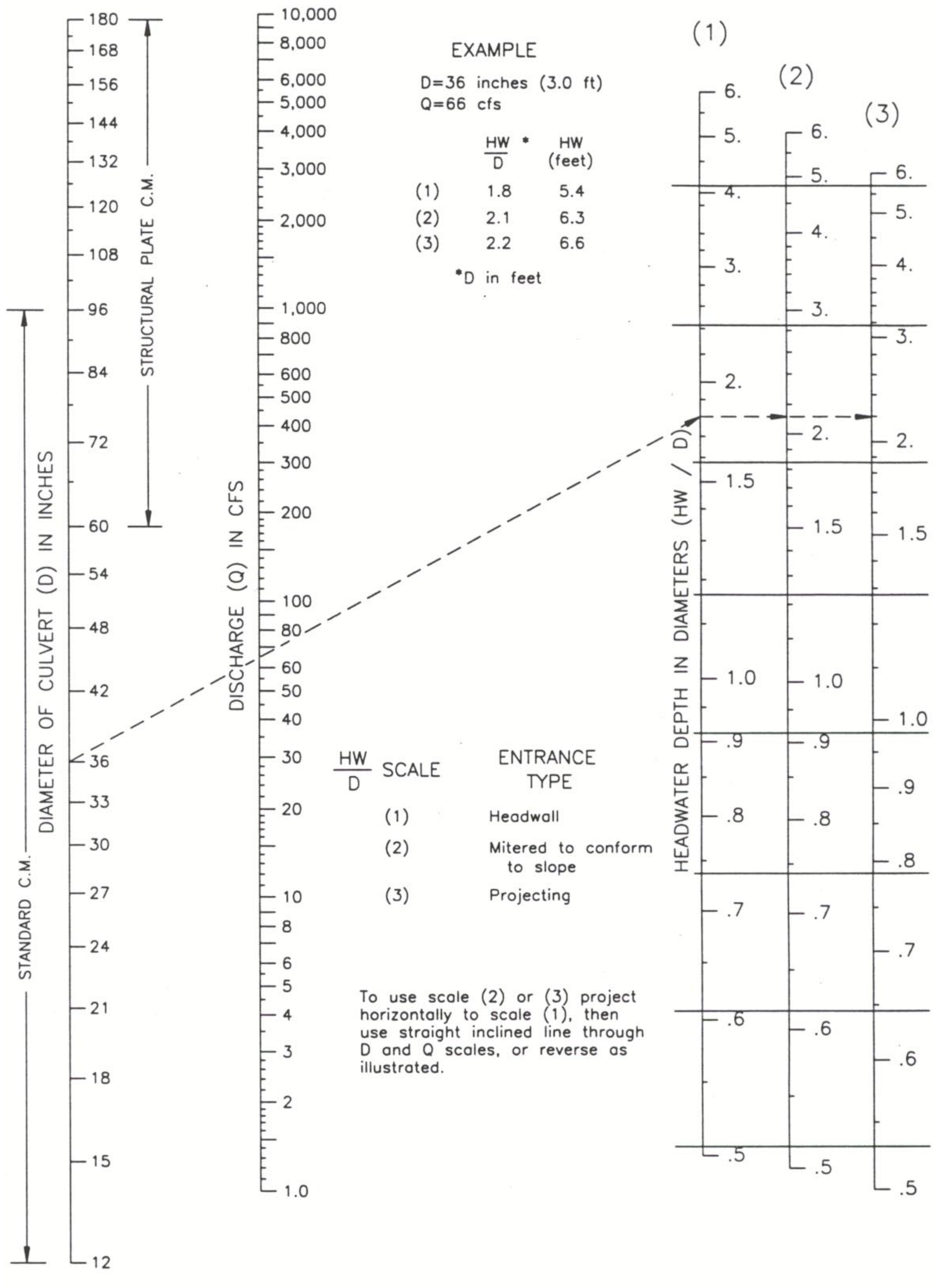
Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Figure 7-11 Headwater Depth for Box Culverts with Inlet Control



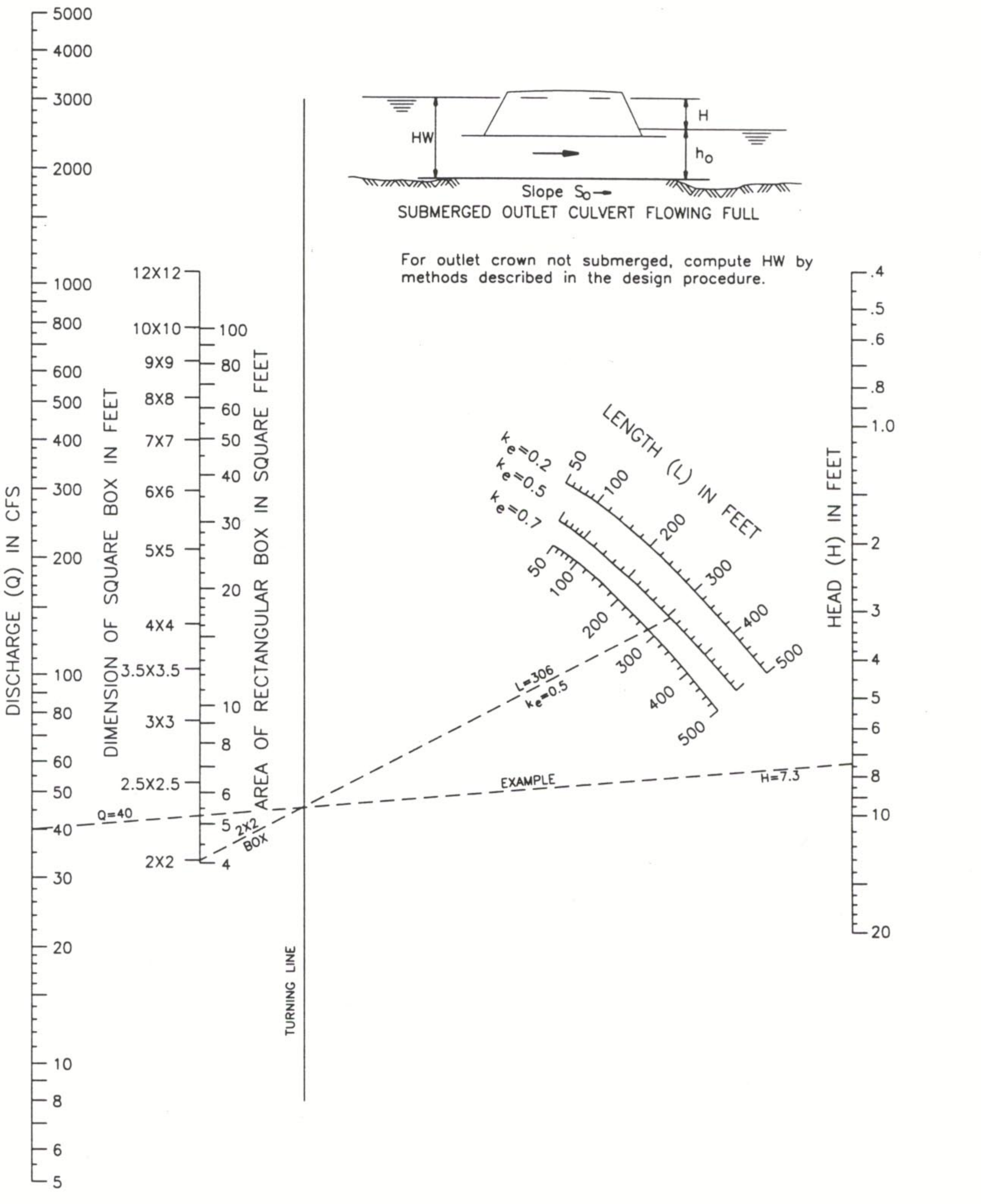
Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Figure 7-12 Headwater Depth for Concrete Pipe Culverts with Inlet Control



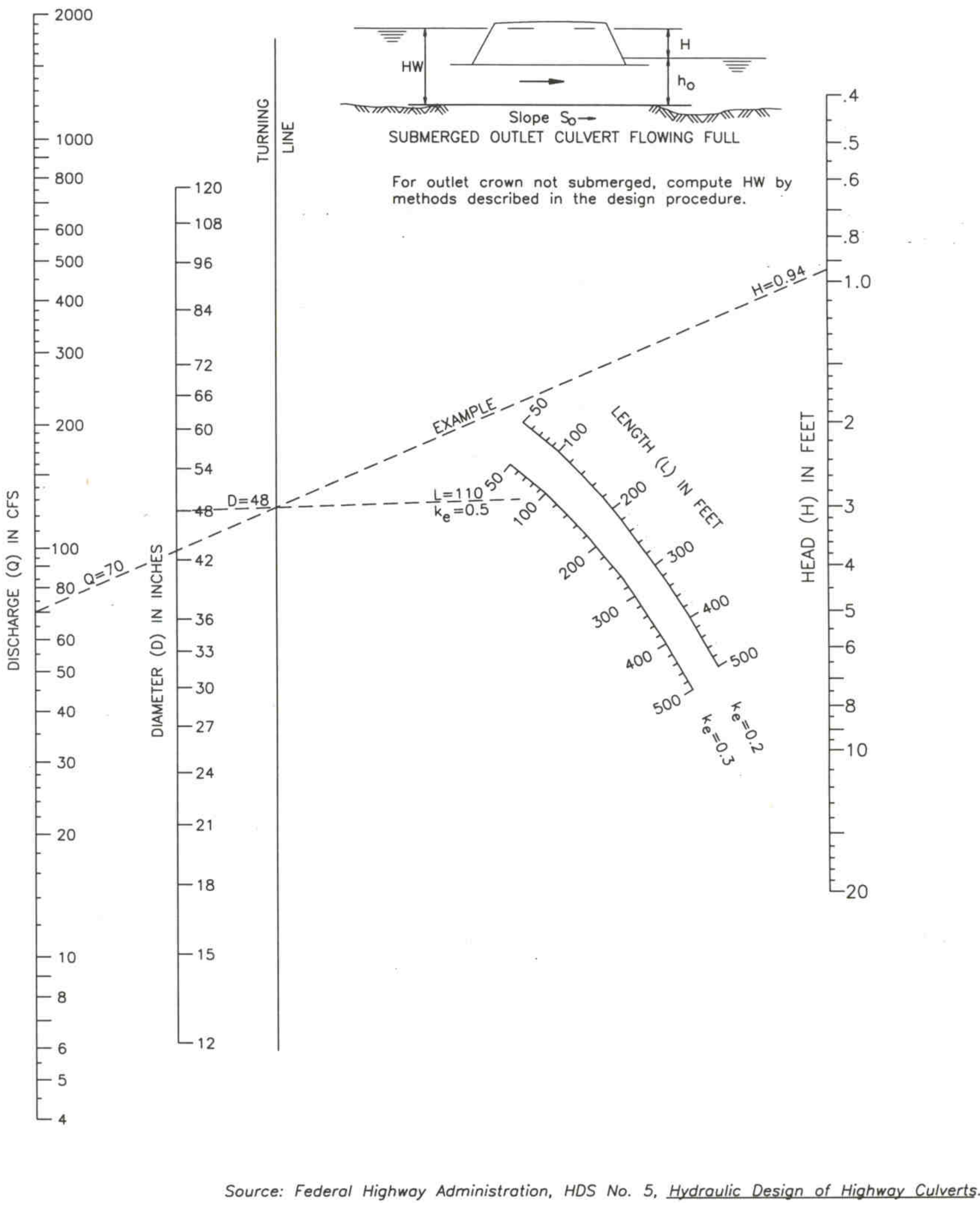
Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Figure 7-13 Headwater Depth for Corrugated Metal Pipe Culverts with Inlet Control



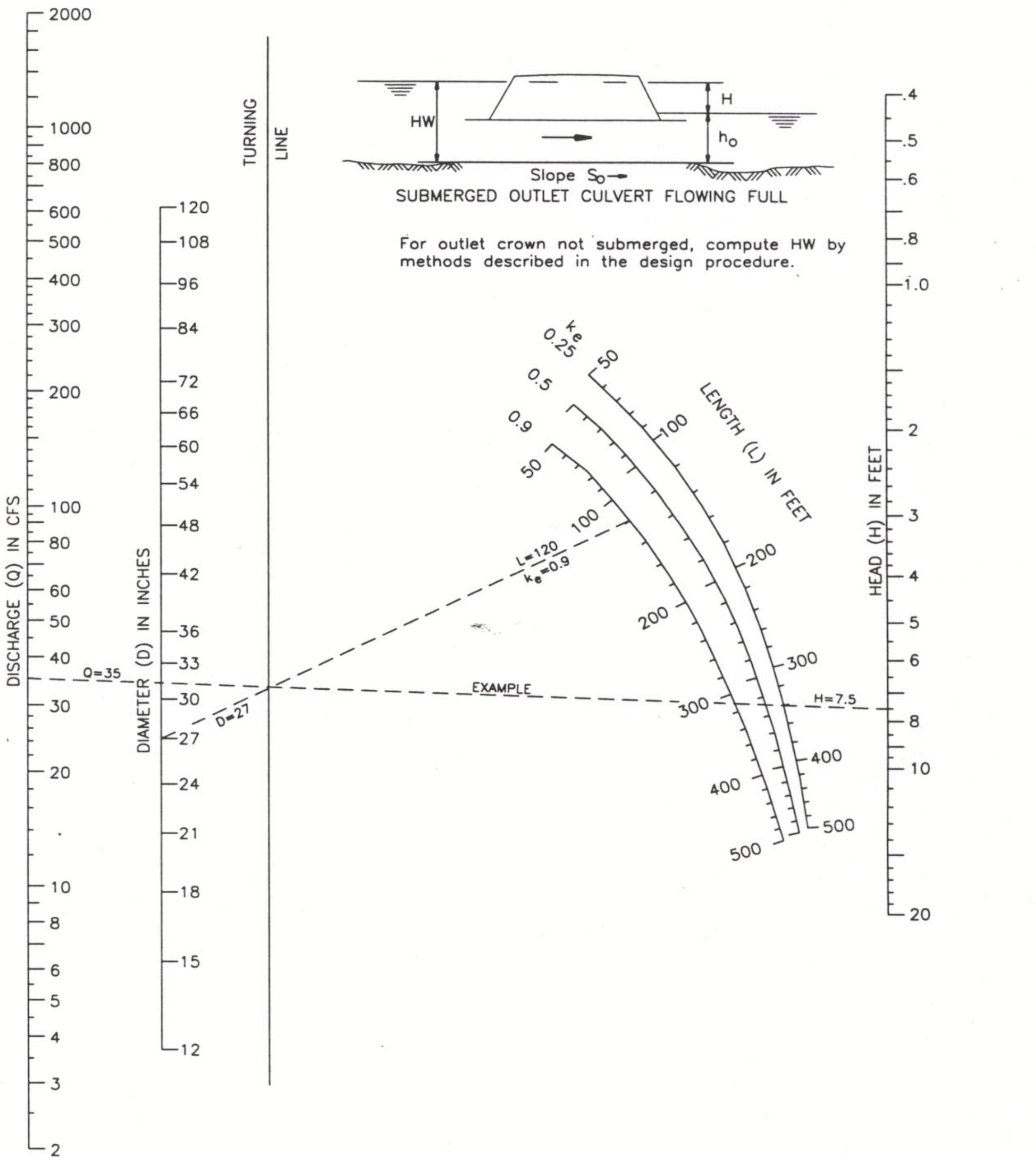
Source: *Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.*

Figure 7-14 Head for Outlet Control Concrete Box Culverts Flowing Full n = 0.012



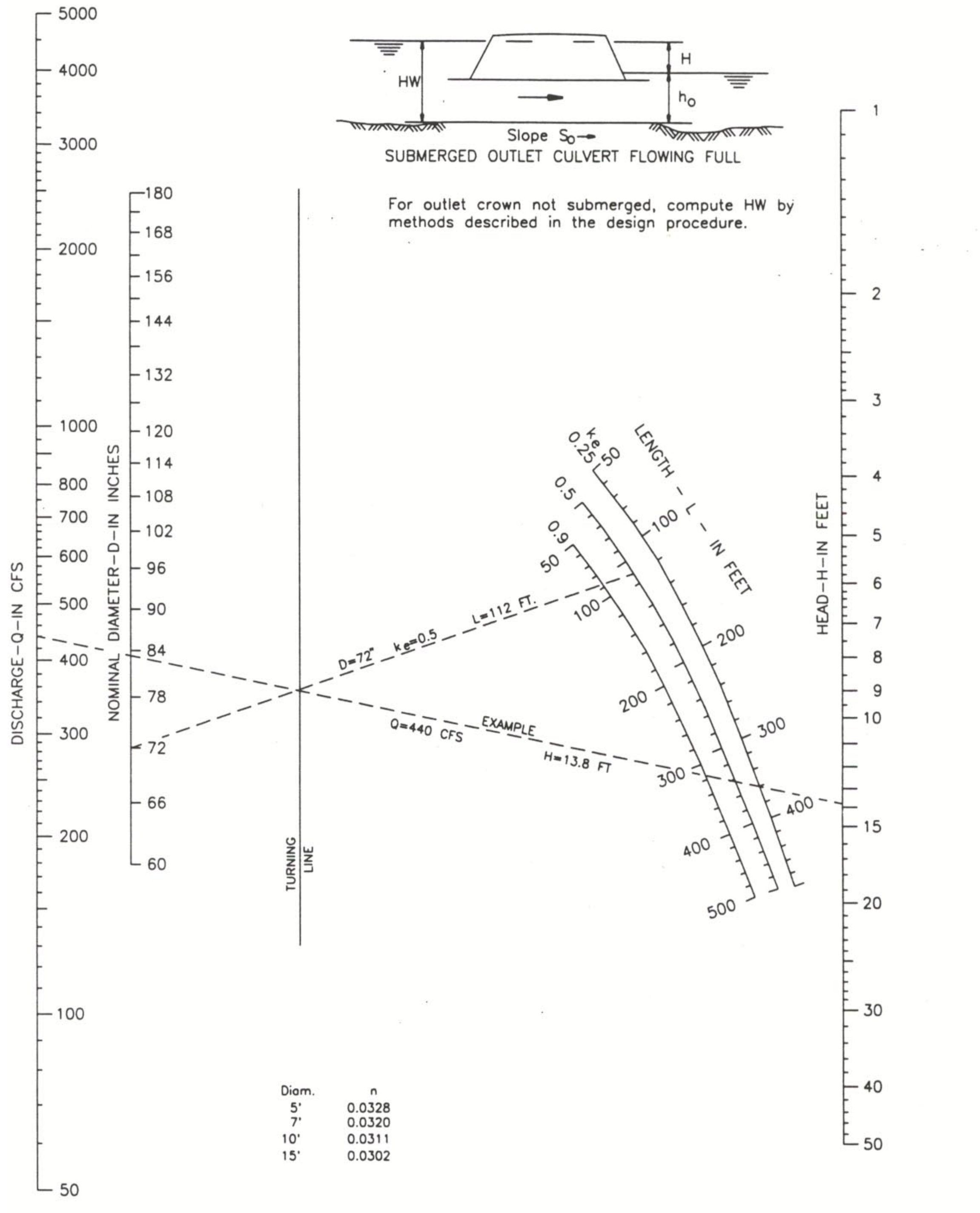
Source: *Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.*

Figure 7-15 Head for Outlet Control Concrete Pipe Culverts Flowing Full n = 0.012



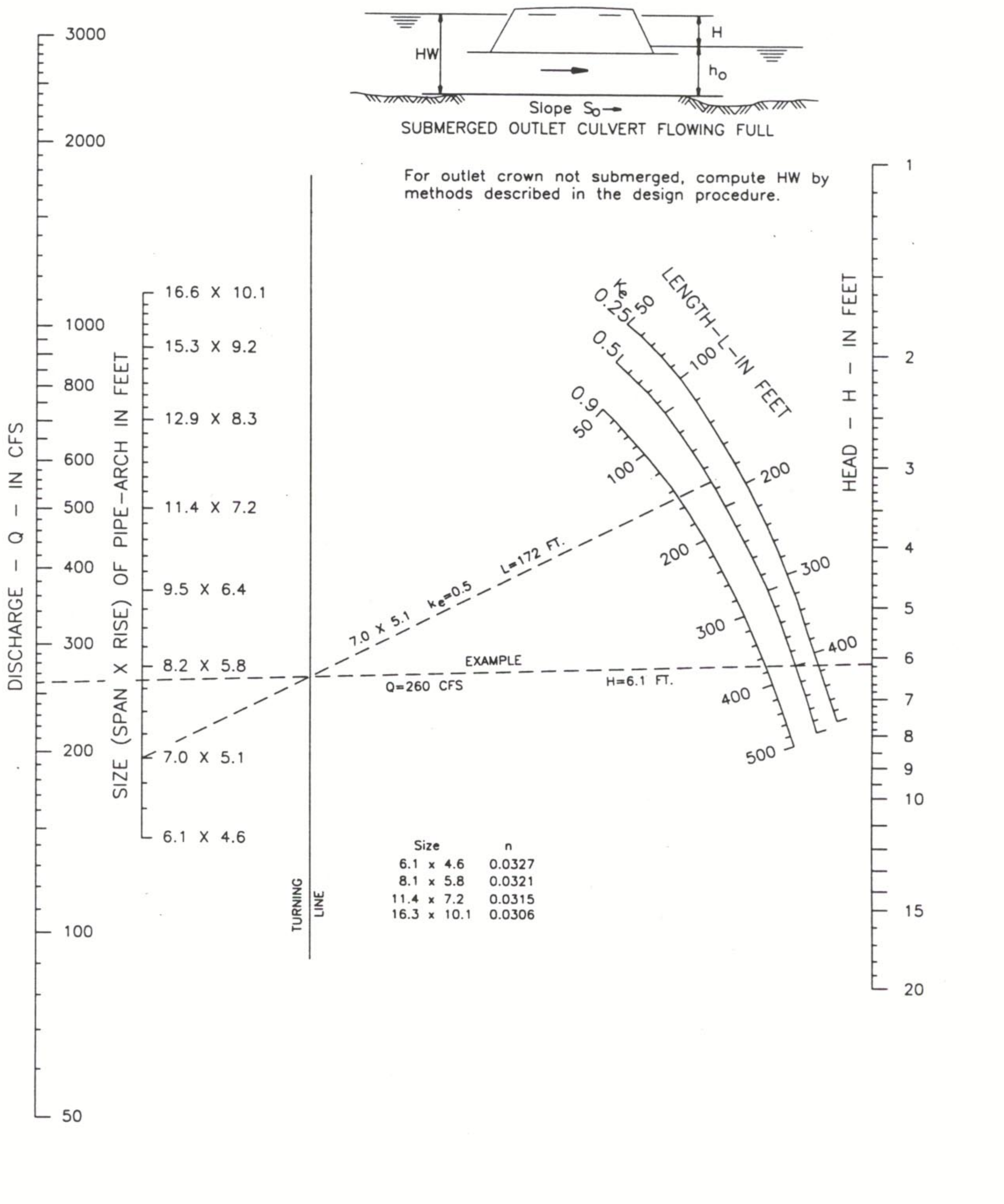
Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Figure 7-16 Head for Outlet Control Standard Corrugated Metal Culverts Flowing Full n = 0.024



Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Figure 7-17 Head for Outlet Control Structural Plate Corrugated Metal Pipe Culverts Flowing Full n = 0.0328 to 0.0302



Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Figure 7-18 Head for Outlet Control Structural Plate Corrugated Metal Pipe Arch Culverts, 18" Corner Radius Flowing Full n = 0.0327 to 0.0306

Critical depth for various culvert sections can be determined using the appropriate curves on Figures 7-19 and 7-20. When the critical depth at the outlet is less than the tailwater depth, the nomographs must be used. However, both methods will provide the same results where either of the two methods is applicable.

Inlet Control

Inlet control calculations determine the headwater elevation required to pass the design flow through the selected inlet control culvert configuration. The approach velocity head may be included as part of the headwater if desired. The inlet control nomograph for standard inlets (Figures 7-11 through 7-13) is used in the design process. Steps A through F outline the proper culvert selection procedure.

A. Locate the selected culvert size and flow rate on the appropriate scales of the inlet control nomograph (note that for box culverts, the flow rate per foot of barrel width is used).

B. Using a straightedge, carefully extend a straight line from the culvert size through the flow rate and mark a point on the first headwater/culvert height (HW/D) scale. The first HW/D scale is also a turning line.

C. If another HW/D scale is required (reflects Entrance Type), extend a horizontal line from the first HW/D scale (the turning line) to the desired scale and read the result.

D. Multiply HW/D by the culvert height, D, to obtain the required headwater (HW) from the invert of the control section to the energy grade line. If the approach velocity is neglected, HW equals the required headwater depth (HW_i). If the approach velocity is included in the calculations, deduct the approach velocity head ($V^2/2g$) from HW to determine HW_i.

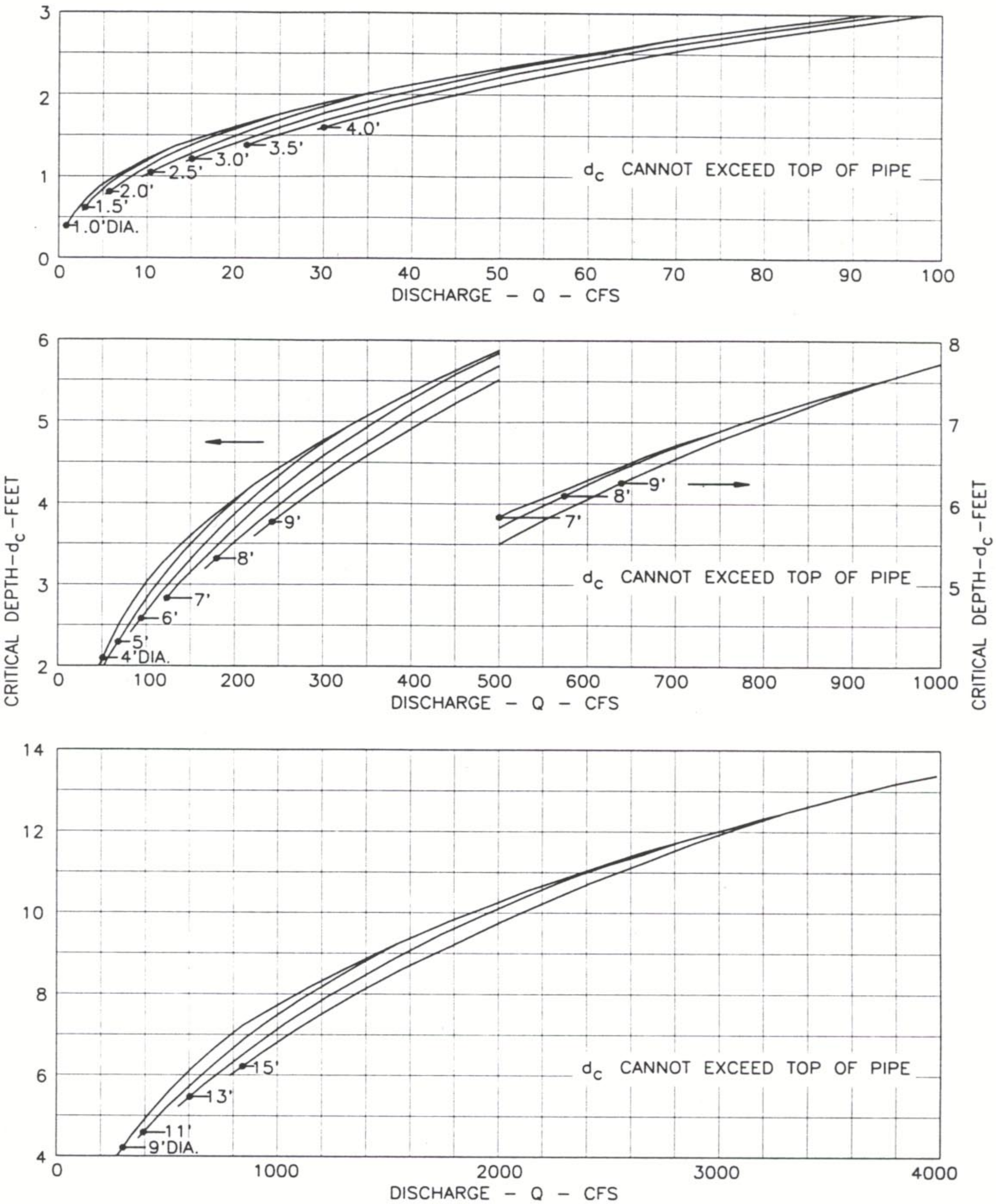
E. Calculate the required depression (FALL) of the inlet control section below the stream bed as follows:

$$HW_d = EL_{hd} - EL_{sf} \tag{7-7}$$

$$FALL = HW_i - HW_d \tag{7-8}$$

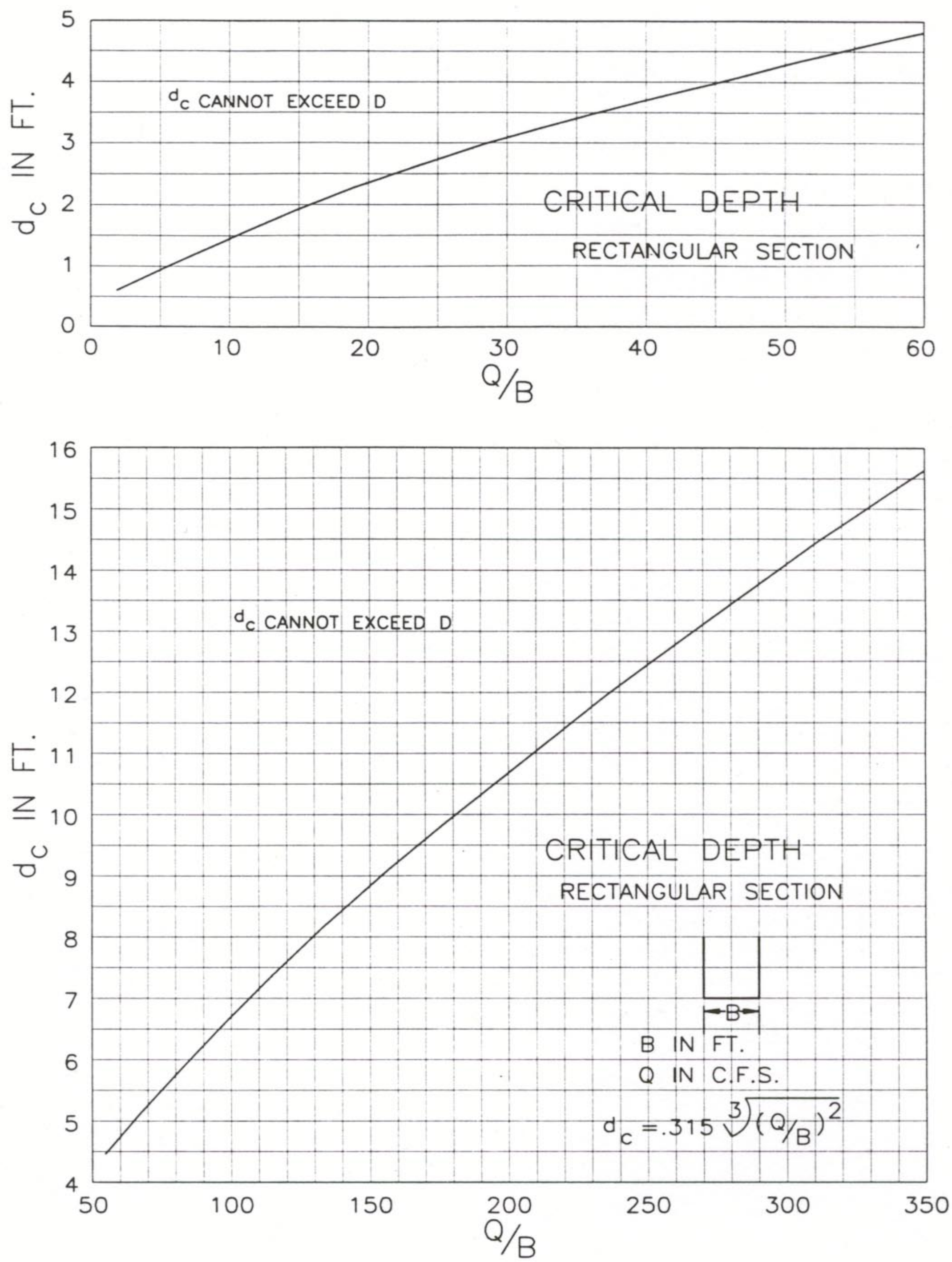
where:

HW_d	=	design headwater depth, in feet
EL_{hd}	=	design headwater elevation, in feet
EL_{sf}	=	elevation of the stream bed at the face, in feet
FALL	=	required depression below the stream bed, in feet
HW_i	=	allowable headwater depth at the inlet, in feet



Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Figure 7-19 Critical Depth, d_c Circular Pipe



Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Figure 7-20 Critical Depth, d_c Rectangular Section

Possible results and consequences of this calculation are:

1. If the FALL is negative or zero, set FALL equal to zero and proceed to step F.

2. If the FALL is positive, the inlet control section invert must be depressed below the streambed at the face by a distance equal to the value of the FALL. If the FALL is acceptable, proceed to step F.

3. If the FALL is positive and greater than is judged to be acceptable, select another culvert configuration and begin again at step A.

F. Calculate the inlet control section invert elevation as follows:

$$EL_i = EL_{sf} - FALL \tag{7-9}$$

where:

EL_i	=	the invert elevation at the face of the culvert (EL_f) or at the throat of the culvert with a tapered inlet (EL_t), in feet
EL_{sf}	=	elevation of the stream bed at the face, in feet
FALL	=	required depression below the stream bed, in feet

Outlet Control

Outlet control calculations result in the headwater elevation required to convey the design discharge through the selected outlet control culvert configuration. The approach and downstream velocities may be included in the design process, if desired. Outlet control nomographs and critical depth charts (Figures 7-14 through 7-20) are used in the design process. Steps A through H outline the proper culvert selection procedure.

A. Determine the tailwater depth (TW) above the outlet invert at the design flow rate. This depth may be obtained from backwater or normal depth calculations, or from field observations.

B. Locate the flow rate on the appropriate critical depth chart (Figure 7-19 or Figure 7-20) and read the critical depth (d_c). Critical depth, d_c, cannot exceed the culvert diameter, D. The d_c curves are truncated for convenience when they converge. If an accurate d_c is required for d_c > 0.9D, consult the Handbook of Hydraulics, by King and Brater or other hydraulic references.

C. Calculate $(d_c + D)/2$

D. Determine the depth from the culvert outlet invert to the hydraulic grade line elevation (h_o).

$$h_o = TW \text{ or } (d_c + D)/2, \text{ whichever is larger}$$

E. From Table 7-2, obtain the appropriate entrance loss coefficient, k_e, for the culvert inlet configuration.

F. Determine the losses through the culvert barrel, H, using the outlet control nomograph.

1. If the Manning's "n" value given in the outlet control nomograph is different than the Manning's "n" for the culvert, adjust the culvert length using the formula:

$$L_1 = L\left[\frac{n_1}{n}\right]^2 \qquad (7\text{-}10)$$

where:

L_1	=	adjusted culvert length, in feet
L	=	actual culvert length, in feet
n_1	=	desired Manning's "n" value
n	=	Manning's "n" value from the outlet control chart

Then, use L_1 rather than the actual culvert length when using the outlet control nomograph.

2. Using a straightedge, connect the culvert size with the culvert length on the appropriate k_e scale. This intersection defines a point on the turning line.

3. Again using the straightedge, extend a line from the discharge through the point on the turning line to the Head Loss (H) scale. Read H. H is the energy loss through the culvert, including entrance, friction, and outlet losses.

Note: Careful alignment of the straightedge is necessary to obtain good results from the outlet control nomograph.

G. Calculate the required outlet control headwater elevation.

$$EL_{ho}=EL_o+H+h_o \tag{7-11}$$

where:

EL_{ho}	=	required outlet control headwater elevation, in feet
EL_o	=	invert elevation at the outlet, in feet
H	=	total head loss, in feet
h_o	=	depth from the culvert outlet invert to the hydraulic grade line, in feet
	=	TW or $(d_c+D)/2$, whichever is larger

H. If the outlet control headwater elevation exceeds the design headwater elevation, a new culvert configuration must be selected and the process repeated. Generally, a larger barrel will be necessary since inlet improvements are of limited benefit in outlet control.

Evaluation of Results

Compare the headwater elevations calculated for inlet and outlet control for a specific culvert size. The higher of the two is designated the controlling headwater elevation. The culvert can be expected to operate with that higher headwater for at least part of the time.

If the controlling headwater is based on inlet or outlet control, determine the area of flow at the outlet based on the barrel geometry and the following:

A. Critical depth if the tailwater is at or below critical depth;

B. Tailwater depth if the tailwater is between critical depth and the top of the barrel; and

C. The height of the barrel if the tailwater is above the top of the barrel.

Table 7-4 can be used to determine the cross sectional area of a circular pipe flowing partially full. The ratio of d/D is the depth of water to the diameter of the pipe. The cross sectional area can be calculated with Equation 7-12.

$$A = C_a D^2 \tag{7-12}$$

where:

A	=	cross sectional area of pipe flowing partially full, in square feet
C_a	=	cross sectional area coefficient, Table 7-4
D	=	diameter of pipe, in feet

TABLE 7-4
CROSS SECTIONAL AREA COEFFICIENTS, C_a,
FOR A CIRCULAR CONDUIT FLOWING PARTIALLY FULL

$\frac{d}{D}$	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.0000	0.0013	0.0037	0.0069	0.0105	0.0147	0.0192	0.0242	0.0294	0.03500.
0.1	0.0409	0.0470	0.0534	0.0600	0.0668	0.0739	0.0811	0.0885	0.0961	1039
0.2	0.1118	0.1199	0.1281	0.1365	0.1449	0.1535	0.1623	0.1711	0.1800	0.1890
0.3	0.1982	0.2074	0.2167	0.2260	0.2355	0.2450	0.2546	0.2642	0.2739	0.2836
0.4	0.2934	0.3032	0.3130	0.3229	0.3328	0.3428	0.3527	0.3627	0.3727	0.3827
0.5	0.393	0.403	0.413	0.423	0.433	0.443	0.453	0.462	0.472	0.482
0.6	0.492	0.502	0.512	0.521	0.531	0.540	0.550	0.559	0.569	0.578
0.7	0.587	0.596	0.605	0.614	0.623	0.632	0.640	0.649	0.657	0.666
0.8	0.674	0.681	0.689	0.697	0.704	0.712	0.719	0.725	0.732	0.738
0.9	0.745	0.750	0.756	0.761	0.766	0.771	0.775	0.779	0.782	0.784

d = Depth of water, in feet
D =Diameter of pipe, in feet
Source: King and Brater, Handbook of Hydraulics.

Repeat the design process until an acceptable culvert configuration is determined. Once the culvert is selected, it must be fitted into the roadway cross section. The culvert must have adequate cover and the headwalls and wingwalls dimensions must be designed.

If outlet control governs and the headwater depth (referenced to the inlet invert) is less than 1.2D, it is possible that the barrel flows partially full through its entire length. In this case, caution should be used in applying the Approximate Method of setting the downstream elevation based on the greater of tailwater or $(d_c + D)/2$. If an accurate headwater depth is necessary, backwater calculations should be used to check the result from the Approximate Method. If the headwater depth falls below 0.75D, the Approximate Method should not be used.

If the selected culvert will not fit the site, return to the culvert design process and select another culvert. If neither tapered inlets nor flow routing methods are applied, document the design process in detail. Designs shall be accompanied by a performance curve that displays culvert behavior over a range of discharges.

7.5.2 Design Procedure

Due to challenges arising from topography and other considerations, the actual design of a culvert is more complex than a simple culvert sizing. This procedure is intended to guide and streamline the design, even though the situations encountered are too varied and unique to be generalized. However, the presented procedure should be utilized to ensure that some special issue is not overlooked or omitted.

Design Computation Forms

The Culvert Design Form, Figure 7-21, has been formulated to guide the design process. Summary blocks are provided at the top of the form for the project description, and the designer's identification. Summaries of hydrologic data are also included. At the top right of the page is a small sketch of the culvert with blanks for inserting important dimensions and elevations.

The central portion of the design form contains lines for inserting the trial culvert description and calculating the inlet control and outlet control headwater elevations. Space is provided at the lower center for comments and at the lower right for a description of the culvert barrel selected.

The first step in the design process is to summarize all known data for the culvert at the top of the appropriate design form. This information should be collected or calculated prior to performing the actual culvert design. The next step is to select a preliminary culvert material, shape, size, and entrance type. The designer then enters the design flow rate and proceeds with the inlet control calculations.

Invert Elevations

After determining the allowable headwater elevation, the tailwater elevation, and the approximate length, invert elevations must be determined. Scour is not likely to occur in an artificial channel when the culvert has the same slope as the channel. To reduce the chance of failure due to scour, invert elevations corresponding to the natural grade should be used first. The flow velocity in the upstream channel should be determined to support the scour analysis.

Culvert Size and Shape

After the invert elevations have been established, proceed with culvert size and shape design. Use the computation forms, the capacity charts, and the nomographs to determine the barrel size (pipe diameter or size of box culvert) that will meet the headwater requirements. The smallest diameter that appears in the nomographs and capacity charts is 12 inches.

CULVERT DESIGN FORM

PROJECT: ____________________

STATION: ____________________

SHEET ______ OF ______

DESIGNER/DATE: ________ / ________

REVIEWER/DATE: ________ / ________

HYDROLOGICAL DATA

SEE ADD'L SHTS.

☐ METHOD: ____________________

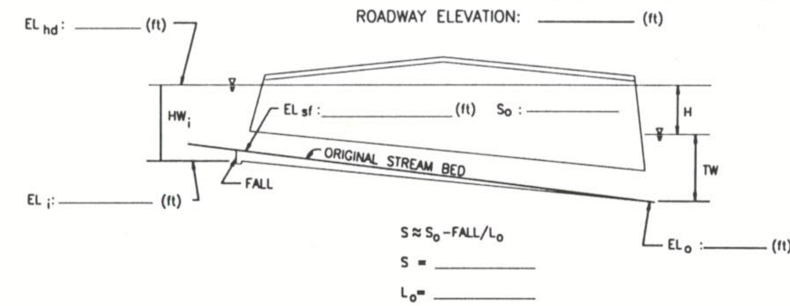
☐ DRAINAGE AREA: ________ ☐ STREAM SLOPE: ________

☐ CHANNEL SHAPE: ____________________

☐ ROUTING: ________ ☐ OTHER: ________

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW(cfs)	TW(ft)



CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) HW_i/D = HW/D OR HW_i/D FROM DESIGN CHARTS

(3) FALL = $HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) h_o = TW or $(d_c+D/2)$ (WHICHEVER IS GREATER)

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

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

SUBSCRIPT DEFINITIONS:

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ____________________

SHAPE: ____________________

MATERIAL: ____________ n ________

ENTRANCE: ____________________

Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Figure 7-21 Culvert Design Form

Limited Headwater

If there is insufficient headwater elevation to obtain the required discharge, increase the size of the culvert barrel, lower the inlet invert, use an irregular cross section, or use any combination of the above.

If the inlet invert is lowered, special consideration must be given to scour. The use of gabions, concrete drop structure, riprap, and headwall with apron and toe walls should be investigated and compared to design a culvert with stable inlet structure.

Culvert Outlet

The outlet velocity must be checked against the permissible velocities of the downstream channel to determine if excessive scour is likely to occur. If scouring is likely, then riprap, an expanding end section, or an engineered energy dissipater should be used.

Minimum Slope

To prevent sediment from obstructing the culvert, the culvert slope must be equal to or greater than the slope required maintaining a minimum velocity of 2.5 feet per second for design flow unless it is otherwise governed by other regulations, such as those mentioned Section 7.2.2 Culvert Discharge Velocity. The design slope should be checked, and, if the required minimum velocity is not obtained, the design must be adjusted. The suggested adjustment methods include decreasing the pipe diameter, increasing the culvert slope, selecting a smoother pipe material, or any combination of the above.

Example 1: Pipe Culvert

Given: Design Discharge, Q_{100} = 200 cubic feet per second

Allowable Headwater Elevation = 108.0 feet

Shoulder Elevation = 111 feet

Elevation Inlet Invert = 100 feet

Culvert Length, L_a = 200 feet

Downstream channel approximates a 5-foot wide trapezoidal channel with 1.5H:1V side slopes, a Manning's "n" of 0.04, and S_o = 0.01 feet per foot

Find: Design a circular corrugated metal pipe with standard $2^2/_3$–inch by $^1/_2$-inch corrugations and a concrete pipe with a groove end. Set the inlet invert at streambed elevation.

Note: Use Figures 7-12, 7-13, 7-15, 7-16 and 7-19 and Design Form, Figure 7-21.

PROJECT: EXAMPLE PROBLEM No. 1

STATION: 1+00

SHEET 1 OF 1

CULVERT DESIGN FORM

DESIGNER/DATE: LAC, 7/18

REVIEWER/DATE: DES, 7/19

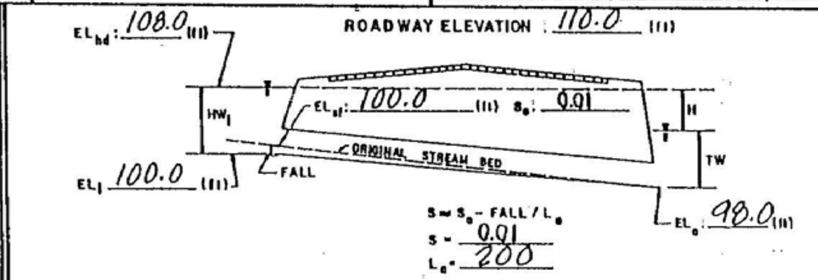
HYDROLOGICAL DATA

SEE ADD'L SHTS.

- ☐ METHOD: RATIONAL
- ☐ DRAINAGE AREA: 125AC. ☐ STREAM SLOPE: 1.0%
- ☐ CHANNEL SHAPE: TRAP, bw=5', n=0.04, 15H:1V
- ☐ ROUTING: N/A ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
100	200	3.5



CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
C.M.P.-CIRC.-72IN.-SQUARE EDGE	200	200	1.01	6.1	–	106.1	3.5	3.9	4.9	4.9	0.5	2.8	105.7	106.1	11.2	TRY 60" C.M.P.
" – " – 60IN. – " "			1.50	7.50	–	107.5		4.0	4.5	4.5	0.5	6.8	109.3	109.3	15.1	TRY 60" CONC.
CONC. – " – 60IN. – GROOVE END			1.36	6.8	–	106.8			4.5	4.5	0.2	2.9	105.4	106.8	17.4	TRY 54" CONC.
" – " – 54IN. – "			1.77	7.97	–	108.0			4.3	4.3	0.2	4.7	107.0	108.0	19.2	OK

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) FALL = $HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) h_o = TW or $(d_c + D/2)$ (WHICHEVER IS GREATER)

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

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

SUBSCRIPT DEFINITIONS:

- a. APPROXIMATE
- f. CULVERT FACE
- hd. DESIGN HEADWATER
- hi. HEADWATER IN INLET CONTROL
- ho. HEADWATER IN OUTLET CONTROL
- i. INLET CONTROL SECTION
- o. OUTLET
- sf. STREAMBED AT CULVERT FACE
- tw. TAILWATER

COMMENTS / DISCUSSION:

HIGH OUTLET VELOCITY - OUTLET PROTECTION OR LARGER CONDUIT MAY BE NECESSARY

CULVERT BARREL SELECTED:

SIZE: 54 IN.

SHAPE: CIRCULAR

MATERIAL: CONC. n 0.012

ENTRANCE: GROOVE END

7.6 BRIDGE HYDRAULIC DESIGN

Bridges are often required to cross open urban channels; therefore, sizing the bridge openings is of paramount importance. When large culverts are used in lieu of bridges, the design approach often differs. Open channels with improperly designed bridges are susceptible to either excessive scour or deposition, and may not be able to convey the design flow.

7.6.1 Design Approach

Designing bridge openings includes water surface profile and hydraulic gradient analyses of the channel during the major storm. Once this hydraulic gradient is established without the bridge, the maximum allowable effect of the bridge (backwater) on the channel flow and current floodplain regulations should be determined.

Scour is the result of the erosive action of running water loosening and transporting material from the streambed and banks. Local scour is restricted to a minor part of the width of a channel and occurs around piers, abutments, spurs, and embankments. It is caused by the acceleration of the flow and the development of vortexes induced by the obstruction to the flow. The FHWA publication, HEC-18, Evaluating Scour at Bridges, outlines the procedure to evaluate scour at bridges.

7.6.2 Bridge Opening Freeboard

The distance between the design flow water surface elevation and the bottom of the bridge deck (bridge low cord) is called freeboard, which varies for all bridges according to the roadway and bridge design. The bridge design may require the freeboard to be several feet high. Potential obstructions must be considered in setting the freeboard height. Rules cannot simplify the design process; every bridge must be studied individually.

In certain cases, the design professional might choose to intentionally cause ponding upstream from the bridge to reduce downstream peak flows during the storms generating flows greater than the major design storm event. In these cases, no freeboard would be allowed. This design approach is used when downstream areas are highly developed, and the upstream areas have open space or parks adjacent to the channel. However, the elevation due to the ponding behind the bridge shall not come within 2 feet of the finished floor elevation of upstream buildings.

7.7 BRIDGE HYDRAULIC ANALYSIS

Backwater caused by the bridge must be calculated during the bridge design process. The bridge-affected water surface elevations may be higher than water surface elevations for unobstructed flow in the natural channel profile. The standard step method for backwater computations can be used to compute the water surface profile. The computations begin at one end of the study reach and proceed cross section by cross section to the other end of the study reach. The standard step method involves the solution of the dynamic equation of gradually varied flow. This method is discussed in Open Channel Hydraulics by Chow. At bridge crossings where the flow hydraulics are more complex, momentum and other equations may be used to compute the changes in water surface elevation.

Many computer programs may be used to compute water surface profiles. Two widely used programs are HEC-RAS and WSPRO. HEC-RAS, Water Surface Profiles was developed by the USACE. Water Surface Profile Computations (WSPRO) was developed by the United States Geological Survey (USGS) for the FHWA. The use of HEC-RAS or WSPRO is recommended for bridge hydraulic analysis.

7.7.1 HEC-RAS Model

The HEC-RAS, Water Surface Profiles, computer program is used to analyze backwater effects from bridge waterways. The methodology incorporated into HEC-RAS is based on several simplifying assumptions, but the model produces satisfactory results in many applications. The assumptions are as follows: (1) steady flow; (2) gradually varied flow; (3) one-dimensional flow with correction for horizontal velocity distribution; (4) small channel slope; (5) friction slope (averaged) constant between two adjacent cross sections, and; (6) rigid boundary conditions.

HEC-RAS is intended for calculating water surface profiles for steady state gradually varied flow in natural or man-made channels. Both subcritical and supercritical flow profiles can be calculated. The effects of various structures such as bridges, culverts, weirs, and objects in the floodplain may be considered in the computations. The computational procedure is based on the solution of the one-dimensional energy equation with energy loss due to friction evaluated with Manning's Equation. HEC-RAS is also designed for application in floodplain management and flood insurance studies to evaluate floodway encroachments. Also, the program is a useful tool for assessing the effects of channel improvements and levees on water surface profiles.

HEC-RAS computes energy losses caused by structures such as bridges and culverts in two parts. One part consists of the losses that occur in reaches immediately upstream and downstream from the bridge where contraction and expansion of the flow is taking place. The second part consists of losses at the structure itself and is calculated with the normal bridge method, special bridge method, or the special culvert option.

The bridge routines in HEC-RAS allow the modeler to analyze a bridge with several different methods without changing the bridge geometry. The bridge routines have the ability to model low flow (Class A, B, and C), low flow and weir flow (with adjustments for submergence), pressure flow (orifice and sluice gate equations), pressure and weir flow, and high flows with the energy equation only. The model allows for multiple bridge and/or culvert openings at a single location.

The culverts hydraulics in HEC-RAS is based on the FHWA standard equations from the publication HDS-5, Hydraulic Design of Highway Culverts. The culvert routines include the ability to model circular, box, elliptical, arch, pipe arch, low profile arch, high profile arch, and semi circular culverts. The HEC-RAS program has the ability to model multiple culverts at a single location. The culverts can have different shapes, sizes, elevations, and loss coefficients. The user can also specify the number of identical barrels for each culvert type.

The culvert option is a new feature in Version 4.5. The FHWA standard equations for culvert hydraulics are used to compute losses through single or multiple barrel structures.

7.7.2 WSPRO Model

The FHWA contracted with the USGS to develop an improved water surface profile computation program. WSPRO is a digital model for water surface profile computations for open-channel flow and is compatible with conventional techniques used in existing step-backwater analysis models. WSPRO incorporates several desirable features from existing models. Profile computations for free-surface flow through bridges are based on relatively recent developments in bridge backwater analysis and recognize the influence of bridge geometry variations. Pressurized flow situations (girders partially or fully inundated) are computed using existing Federal Highway Administration techniques. Embankment overtopping flows, in conjunction with either free-surface or pressurized flow through the bridge, can be computed. WSPRO is also capable of computing profiles at stream crossings with multiple openings (including culverts). WSPRO is recommended when performing scour computations.

Although specifically oriented towards hydraulic design of stream highway crossings, WSPRO is equally suitable for water surface profile computations unrelated to highway design.

7.8 LIST OF SYMBOLS

The following is a list of symbols used in Section 7 of this Article, their corresponding units and a brief description of the symbol.

TABLE 7-5

LIST OF SYMBOLS

Symbol	Units	Description
A_b	sq ft	Area of bend section of slope-tapered inlets
H_f	ft	Depth of pool, or head, above the face section invert
H_t	ft	Depth of pool, or head, above the throat section invert
HG Line	ft	Hydraulic grade line
HW	ft	Headwater elevation; subscript indicates control section (HW, as used in HDS-5, is a depth and is equivalent to H_f)
HW_c	ft	Headwater elevation required for flow to pass crest in crest control
HW_f	ft	Headwater elevation required for flow to pass face section in face control
HW_o	ft	Headwater elevation required for culvert to pass flow in outlet control
HW_t	ft	Headwater elevation required for flow to pass throat section in throat control
h_e	ft	Entrance head loss
h_f	ft	Friction head loss
h_o	ft	Elevation of equivalent hydraulic grade line referenced to the outlet invert
k_e		Entrance energy loss coefficient
k_b		A dimensionless effective pressure term for bend section control
k_t		A dimensionless effective pressure term for inlet throat control
L_a	ft	Approximate total length of culvert, including inlet face section control
L_1, L_2, L_3, L_4	ft	Dimensions relating to the improved inlet as shown in sketches of the different types of inlets
N		Number of barrels
n		Manning's roughness coefficient
P	ft	Length of depression
Q	cfs	Rate of flow
R	ft	Hydraulic radius = Area/Wetted Perimeter
S	ft/ft	Slope of culvert barrel
S_c	ft/ft	Slope of natural channel producing critical discharge
S_e	ft/ft	Slope of embankment
S_f	ft/ft	Slope of FALL for slope-tapered inlets (a ratio of horizontal to vertical)
S_n	ft/ft	Friction slope
S_o	ft/ft	Slope of natural channel
T	ft	Depth of the depression
Taper	ft/ft	Sidewall flare angle (also expressed as the cotangent of the flare angle)
TW	ft	Tailwater depth at outlet of culvert referenced to outlet invert elevation
V	ft/sec	Mean velocity of flow
V_c	ft/sec	Critical velocity
W	ft	Width of weir crest for slope-tapered inlet with mitered face
W	ft	Top width of depression
y	ft	Difference in elevation between crest and face section of a slope-tapered inlet with mitered face

SECTION 8
HYDRAULICS OF DETENTION

Page

LIST OF FIGURES

8.1 INTRODUCTION

On-site runoff detention is one option for managing urban stormwater. Detention involves collecting, storing, and slowly releasing increased runoff due to development before it enters the main drainage system. This stormwater management technique can often be an effective and economical means of reducing peak flow rates (preventing flooding and channel erosion) and mitigating water quality problems (removing pollution and preventing siltation). Large detention facilities that serve multiple developments are generally preferable to small individual on-site facilities that only serve one subdivision or office complex.

The Stormwater Management Plan for the City identifies detention alternatives to manage runoff under various development conditions. For specific types of development, special requirements may necessitate the modification of detention facility design. These facilities will have some water quality aspects that will need to be considered. Depending upon the measure chosen, the detention method may need to combine other treatment methods to achieve water quality goals. Proposed detention structures that are not addressed in the Stormwater Management Plan must be approved by the City. Section 10 Water Quality provides additional background on the adaptation of detention facilities for water quality concern. The design professional should review Section 10 Water Quality and its references for design guidelines for the incorporation of water quality control into a detention facility.

This section emphasizes the quantitative benefits of the use of detention structures to control the peak discharge to downstream areas. Related design methods and procedures for these structures are included.

8.2 DETENTION BASIN DESIGN CRITERIA

The detention basin is the most widely used detention measure for attenuating peak discharges from urbanizing areas. Basins can be designed to fit a variety of sites and incorporate multiple outlets to meet requirements for multi-frequency control of flow. Figure 8-1, on page 8-3, shows a schematic of a typical stormwater detention basin.

If the stormwater management facility includes a permanent pool of water, it is commonly known as a wet detention basin. The permanent pool provides water quality control through sedimentation and the additional pool provides for peak flow control of stormwater runoff. Wet detention basins should have a minimum contributing drainage area of 10 acres. If groundwater is present, the design can be adapted for wet detention. Wet detention basin should be constructed offline of any stream to avoid regulatory requirements and subsequent mitigation.

Safety

Detention basins shall be designed to minimize the chance of accidental injury to any person coming in contact with the system. Safety considerations for stormwater drainage system components must be evaluated during the design process. The use of fencing around detention basins shall be considered by the design professional on case-by-case basis. A safety bench shall be incorporated around the perimeter of the detention basin to prevent accidental falls into the basin and for stability of facility and ease of maintenance. The safety bench shall consist of a 10-foot width with slopes no greater than 6H:1V. The area may be temporarily inundated during major storms but should remain dry for minor storms.

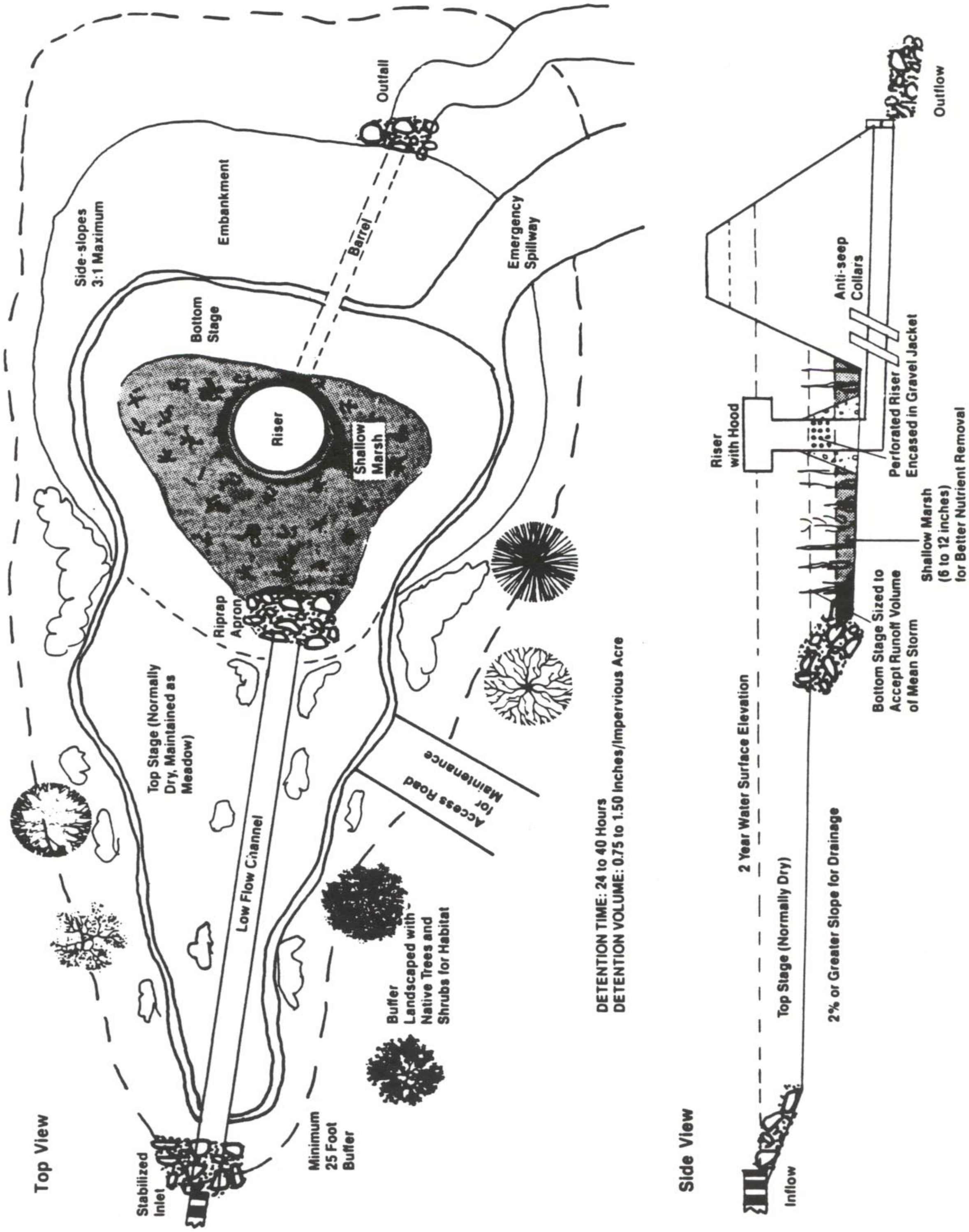


Figure 8-1 Schematic of Stormwater Detention Pond Design Features

Embankment

Depending on the site conditions, the detention volume may be created by excavating the required volume. The embankment is most frequently comprised of an earthen dam, but rock and concrete structures are also used. The earthen dam is preferred for the City and use of any other method requires the City approval prior to design. The maximum slope of 3H:1V should be used on the embankments to allow maintenance equipment to maintain ground cover. The construction of the embankment, depending on storage volume, dam height, and downstream hazard potential, may be classified as a dam, and therefore, a subject to the Dam Safety Law from 1967 (Dam Safety). It is the design professional's responsibility to determine if the detention structure is subject to this law. Refer to North Carolina Department of Environment and Natural Resources, Division of Land Resources Dam Safety Program Administrative Rules for more information.

If the dam is over 10 feet tall the top width shall be 8 feet minimum. If the dam is less than 10 feet tall the top width can be less than 8 feet with City approval but in no case shall it be less than 5 feet.

Embankment design should incorporate the following considerations:

- Foundation preparation and treatment
- Control of seepage
- Embankment stability
- Construction considerations
- Subsidence
- Maintenance
- Access
- Vegetation

Low growing turf grasses which are periodically mowed are recommended for the dam and the emergency spillway to prevent erosion, control woody growth and inhibit burrowing.

If the detention facility is not a subject to the Dam Safety, the City may require the design professional to evaluate the proposed design using more stringent design requirements/criteria assigned by the City, including a dam breach analysis, if failure could potentially cause loss of life or substantial damage to property.

Freeboard

A minimum of 1-foot of freeboard shall be added to the design water surface elevation. It is the responsibility of the design professional to determine if additional freeboard is necessary. The City Engineer reserves the right to require additional freeboard if deemed necessary for safety or other considerations.

8.2.1 Design Storm and Outlet Flow Limitations

Stormwater detention is required to attenuate flow peaks such that the post-development peak discharge rate does not exceed the pre-development peak discharge rate for the 1- and 10-year 24-hour storm events. Runoff volume drawdown time for the 1-year 24-hour storm must be a minimum of 24 hours, but not more than 120 hours. Design storms and other design requirements are provided in Sections 1.3 Design Policy of this Article.

8.2.2 Storage Volume Required

The volume resulting from the difference in pre- and post-development peaks or hydrographs must be determined. The Rational Formula may only be used for sizing detention basin for small areas, as described in Section 2.3 Hydrologic Methods. Methods which include a runoff hydrograph shall be used for watersheds larger than 20 acres. Runoff hydrographs must be developed as part of the evaluation of drainage system performance during the design storm and major storm events. Computations of runoff hydrographs which do not rely on a continuous accounting of antecedent moisture conditions shall assume antecedent moisture condition II. The permanent pool should not be included in the stage-storage relationship.

8.2.3 Principal Outlet Works

Where the discharge structure consists of a single pipe outlet, the pipe shall have a minimum inside diameter of 18 inches, since maintenance of outlets smaller than 18 inches requires much more effort. If design release rates call for outlets smaller than this, release structures, such as perforated risers or flow control orifices are recommended.

Depending on the geometry of the outlet structure, discharge for various headwater depths can be controlled by the inlet crest (weir control), the riser or barrel opening (orifice control), or the riser or barrel pipe (pipe control). The hydraulic performance of these flow controls shall be evaluated when determining the rating curve of the principal outlet. Weir, orifice, and pipe flow equations shall be used to evaluate a single-opening outlet structure.

Weir Flow

Weir flow may be computed for a standard, non-contracted, horizontal weir by the following equation:

$$Q = CLH_w^{3/2} \tag{8-1}$$

where:

Q	=	discharge, in cubic feet per second
C	=	weir coefficient, use 3.08 for broad crested weir with a rounded leading edge
L	=	length of the weir, in feet; for circular riser pipes, L is the pipe circumference
H_w	=	the depth of flow over the weir crest, in feet

Another common weir is the V-notch, whose equation is as follows:

$$Q = C\tan\left(\frac{\theta}{2}\right)H^{2.5} \tag{8-2}$$

where:

Q	=	discharge, in cubic feet per second
C	=	weir coefficient, use 2.5 v-notch veir
θ	=	angle of the notch at the apex, in degrees
H	=	total energy head, in feet

The weir coefficient is a function of various hydraulic properties and dimensional characteristics of a weir. Experiments have been conducted on various types of weir configurations and formulas have been developed to determine the "C" value. Available empirical formulas are numerous. The design professional is urged to solicit hydraulic textbooks such as Handbook of Hydraulics by Brater and King and use engineering judgment for adjustments in aforementioned weir coefficients. The effects of submergence must be considered when designing or evaluating weir flow. A simple check on submergence can be made by comparing the tailwater to the weir crest elevation.

Orifice Flow

Orifice flow may be computed by the following equation:

$$Q = CA\sqrt{2gH_o} \tag{8-3}$$

where

Q	=	discharge, in cubic feet per second
C	=	orifice coefficient, use 0.61 for small submerged orifices
A	=	cross-sectional area of the orifice, in square feet
g	=	acceleration of gravity, 32.2 feet per second squared

H_o = effective head on the orifice, feet, see Figure 8-2.

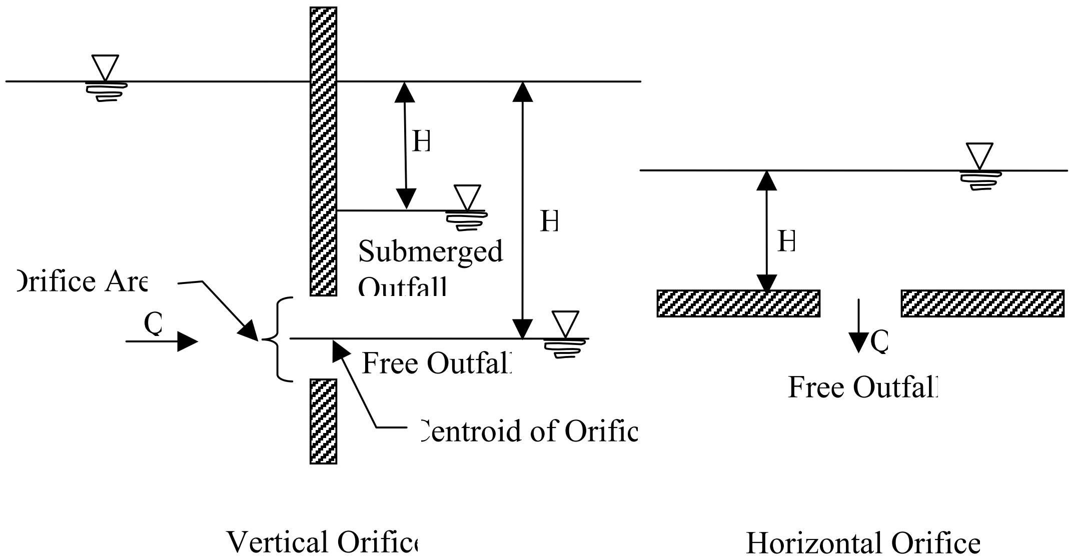


Figure 8-2 Effective Head on Orifice

The orifice coefficient is a function of various hydraulic properties and dimensional characteristics of an orifice. The design professional is urged to solicit hydraulic textbooks such as Handbook of Hydraulics by Brater and King and use engineering judgment for adjustment in aforementioned orifice coefficient.

Pipe Flow

Pipe flow may be computed by the following equation:

$$Q = A\left[\frac{2gH}{1+k_b+k_e+k_f L}\right]^{0.5} \qquad (8\text{-}4)$$

where:

Q	=	discharge, in cubic feet per second
A	=	cross-sectional area of the pipe, in square feet
g	=	acceleration of gravity, 32.2 feet per second squared
H	=	the difference between headwater and tailwater elevations, in feet
k_b	=	bend loss coefficient, use 0.6 for a long radius 90 degree bend
k_e	=	entrance loss coefficient, use 0.5 for sharp edged perpendicular entrance

k_f	=	friction loss coefficient, see paragraph below
L	=	length of pipe, in feet

The friction loss coefficient, k_f, is function of various hydraulic properties and dimensional characteristics of the pipe. The design professional is urged to solicit Natural Resources Conservation Service's Engineering Handbook, Section 5 Hydraulics for friction loss coefficient values and use engineering judgment for adjustment in other aforementioned loss coefficients.

8.2.4 Emergency Spillways

The emergency spillway control structure is independent of the principal outlet works, and is usually designed as a weir. The weir capacity of the emergency spillway should be designed to discharge the water above the design water surface elevation accumulated from the design storm in a manner that satisfies the required minimum freeboard. The spillway control structure and the spillway channel entrance shall be configured to provide for a smooth transition to avoid turbulent flow over the spillway crest. The emergency spillway, along with the capacities of the other outlets, must be able to pass the runoff from the 50-year storm and other design storms such as the 100-year storm and the Probable Maximum Precipitation event, if required.

Discharge from the emergency spillway shall be directed to the receiving channel without causing erosion along the downstream toe of the dam. The emergency spillway shall be constructed in full cut undisturbed soil, if possible, to avoid flows against constructed fill.

The position, profile, and length of the spillway channel are influenced by geologic and topographic features of a site. The cross section dimensions are governed by site conditions and required hydraulic capacity of the channel. Detention pond emergency spillways are designed as stabilized open channels, following principles in Section 6 Open Channels. The spillway should be stabilized with turf grasses and periodically mowed to prevent woody growth from becoming established. The side slopes of the excavated spillway channel shall be no steeper than 4H:1V for ease of maintenance. Where site limitations prevent a full channel cut, a wing dike shall be designed to direct spillway flows away from the downstream toe of the dam. Ready access to the emergency spillway system shall be provided.

The slope of the spillway channel usually follows the configuration of the abutment. However, slopes should not exceed 10 percent. In cases of highly erodible soils, it may be necessary to use other means of protection such as riprap, or concrete channel lining. As an alternative, detention storage can be increased to reduce the frequency or duration of use of the emergency spillway and thereby reduce potential erosion problems.

The emergency spillway channel shall convey flow to the receiving channel with a minimum negative impact on the receiving channel. If necessary to prevent erosion, the receiving channel must be protected by an energy dissipating feature, as described in Section 6.9 Energy Dissipators.

8.3 ON-SITE DETENTION

Potential advantages and disadvantages of on-site detention structures should be considered by the design professional in the early stages of development. Discharge rates and outflow velocities are regulated to conform to the capacities and physical characteristics of downstream drainage systems and the criteria outlined in Section 1.3 Design Policy of this Article. Energy dissipation and flow attenuation resulting from on-site storage can reduce soil erosion and pollutant loading. On-site detention may not necessarily assist in pollutant removal; however, by controlling release flow rates, the required volume of downstream water quality controls may be minimized.

8.3.1 Parking Lots

Two general types of stormwater detention are applicable to parking lot surfaces. One type involves the runoff storage in depressions constructed at drain locations. The stored water is drained into the storm sewer system slowly, using restriction devices such as orifice plates in the drain. Proper design of such paved areas will restrict the ponding to the limited areas. This "planned ponding" will cause the least amount of inconvenience to the users of the parking areas. For example, the parking lot of a shopping center will have the ponding areas located in the least-used portions of the lot, allowing customers to walk to their vehicles in areas of no ponding, except when the entire lot is filled with vehicles. Drainage of ponded water would be fairly rapid to prevent customer inconvenience. In most cases, the water ponding depth should not exceed 6 inches for automobile parking lots, 10 inches for additional parking areas, and 15 inches in truck storage or loading areas. The ponding area should be drained within 1 hour or less after the design rainfall. Computation of the storage volume needed would be similar to the analysis used in designing standard detention facilities.

Another type of stormwater detention on parking lots consists of using the paved areas of the lot to channel the runoff to grassed areas or gravel-filled seepage pits (see Figure 8-3). Water from pavement should run through at least 30 feet of grass before entering an infiltration swale, trench or basin. Soil conditions and infiltration rate reduction due to siltation must be considered. Minimum slopes of one-half (0.5) percent are recommended in parking lot detention areas.

8.3.2 Recreational Areas

Generally, recreational areas such as outdoor athletic fields have a substantial area of grass cover, which often has a relatively high infiltration rate. Generally, stormwater runoff from such fields is minimal. Grassed recreational fields can be utilized for the temporary detention of stormwater peak runoff without adversely affecting their primary function.

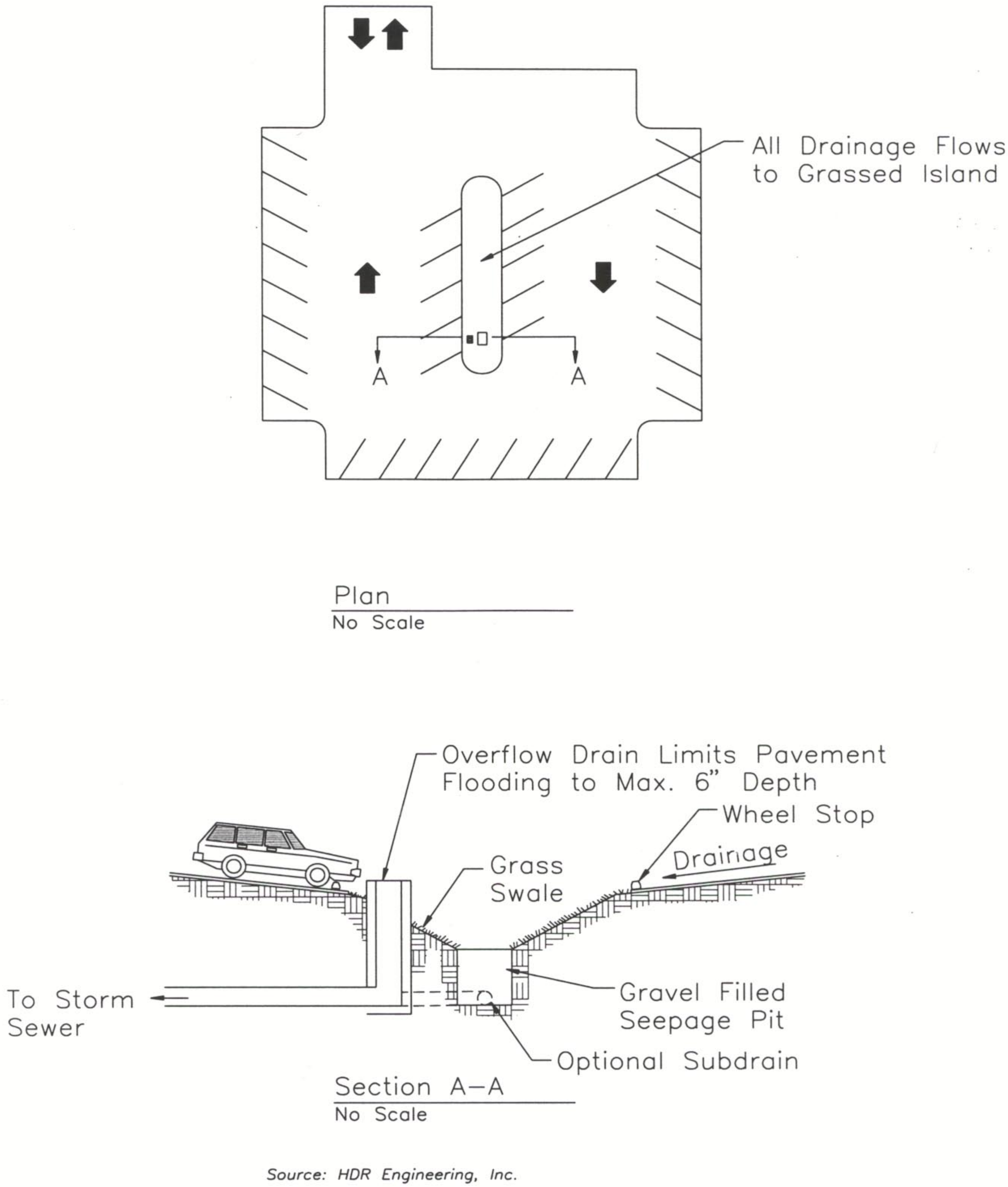


Figure 8-3 Parking Lot Detention

Parks, like recreational areas, generate little runoff of their own; however, parks provide excellent detention storage potential for runoff from adjacent areas.

Refer to the Open Space Regulations in the Unified Development Ordinance for limitations on the use of open space for stormwater detention.

8.3.3 Property Line Swales

Planning and grading new development requires adequate surface drainage away from buildings. When possible, the layout should call for a grassed swale to be located along the back and/or side property line which then drains through the block of properties (see Figure 8-4). Such drainage should be guided away from storm sewers and towards natural channels. If storm sewers are the only point of discharge, the route should be as long as possible to allow infiltration. The final grading plan for the lot layout may allow up to 6 inches of temporary ponding along the property lines. Easements shall be provided along property lines to allow maintenance access. Maintenance of the swales is responsibility of the developer and may be passed to the owner.

8.3.4 Road Embankments

The use of road embankments for temporary storage is an efficient method of attenuating the peak flows from a drainage basin. The design criteria to be used for the temporary detention of water behind road embankments shall include consideration of the major storm runoff. Planning for the usage of embankments must be done with thorough consideration to avoid damage to the embankment, the road structure, and adjacent property. Designs utilizing road embankments shall conform to North Carolina Department of Transportation design criteria, including the slope protection measures.

8.3.5 Pipes, Tanks, and Vaults

Underground facilities such as pipes, tanks, and vaults equipped with hydraulic control structures may be utilized for quantitative stormwater runoff control. Control structures may include orifices and weirs. Facility design shall provide for the storage of the required volume generated by design storm event. Overflow weirs within the facility shall bypass the fully developed design storm. Site grading design shall include overflow provisions for major storm runoff.

8.3.6 Combinations

In many instances, one on-site detention method cannot conveniently or economically satisfy the required amount of stormwater storage. Limitations in storage capacity, site development conditions, soils limitations, and other related constraints may require that more than one method be implemented. For example, parking lot and surface pond storage might both be required to compensate for increases in runoff due to development of a particular site. Whatever storage combinations are suitable for the particular site should be incorporated into the site development plan.

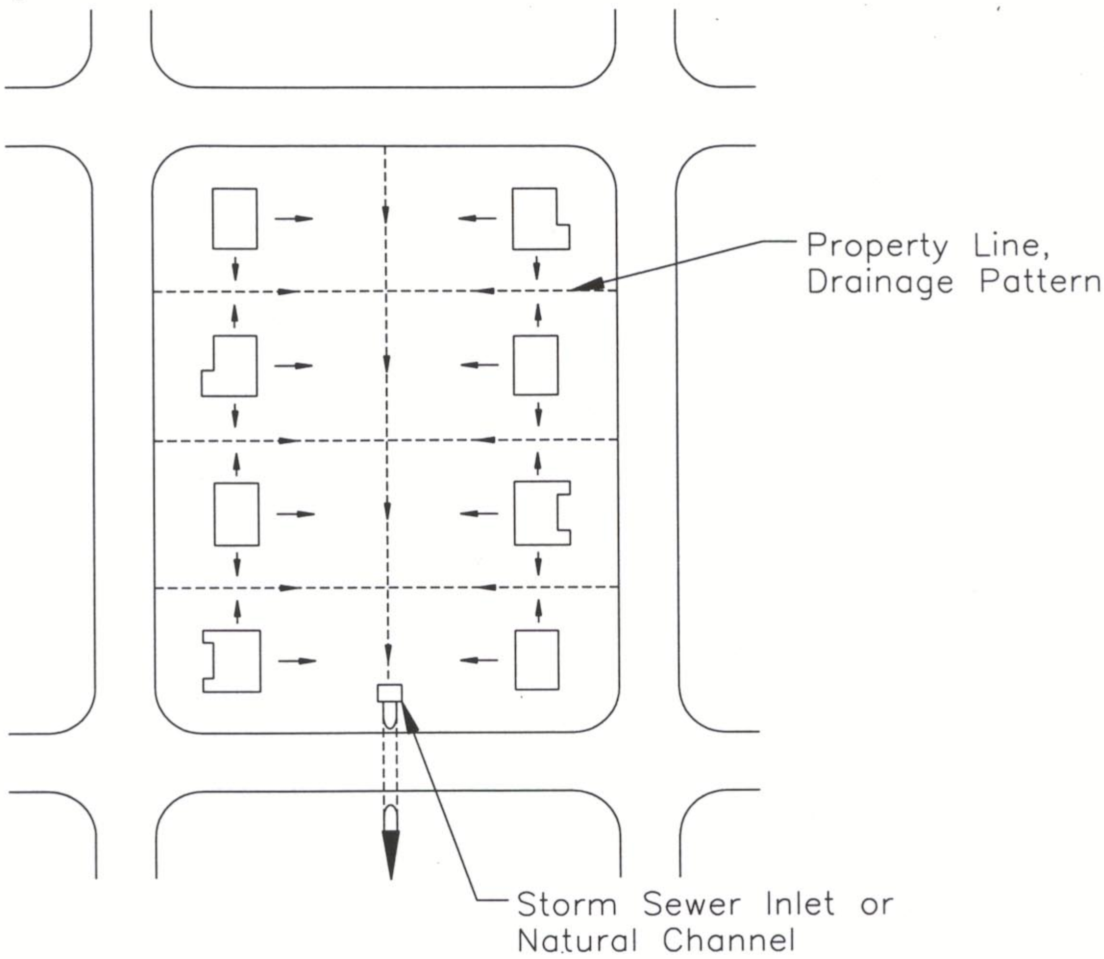


Figure 8-4 Property Line Swales

8.4 HYDRAULIC DESIGN METHODS

8.4.1 Modified Rational Method Analysis

The Modified Rational Method Analysis is a procedure for manipulating the basic Rational Method to reflect the fact that storms with durations greater than the time of concentration for a basin will result in a larger runoff volume even though the peak discharge is reduced. This larger runoff volume must be determined to size detention facilities properly.

The method becomes more valid for progressively smaller basins, eventually reaching a size for which this numerical method closely approximates the natural conditions. The method should, therefore, be limited to relatively small areas such as rooftops, parking lots, or other upstream areas with contributing basins less than 5 acres, or when runoff from this basin is less than one percent of the total flow from the watershed. The consideration of this method minimizes potential major damage which could result from overtopping or failure of the proposed detention facility.

Figure 8-5 presents a series of curves for a theoretical basin described in the following example. These hydrographs are developed by using the basic Rational Method assumptions of constant rainfall intensity, time of concentration (t_c) for the longest flow path, and runoff coefficient. The typical Rational Method hydrograph with the peak discharge coinciding with the time of concentration for the basin is first calculated using the formula, $Q_p = C_fCiA$. Following this, a series of hydrographs representing storms of greater duration than t_c are developed. The rising and falling limb of the hydrograph are, in each case, equal to t_c for the basin. The area under the hydrograph is also equal to the peak discharge rate for that particular rainfall multiplied by the duration of the rainfall.

Example 1: Modified Rational Method

Given: Area: A = 2.0 acres
Type of development: commercial parking lot, fully paved, C = 0.95 (Table 2-5)
Time of concentration: t_c = 8 minutes
Design Frequency = 10 years
Use Intensity-Duration-Frequency Curves, Figure 2-2.

Find: Develop family of curves representing Modified Rational Method hydrographs for the 8-, 10-, 15-, 20-, 30-, and 40-minute storm durations.

Solution:

$$Q_p = C_fCiA$$

The table below summarizes the calculations for this example.

Table of Calculations for Example #1		
Storm Duration (minutes)	**Rainfall Intensity (inches/hour)**	**Peak Runoff Rate (cfs)**
8	6.20	10.9
10	5.74	10.1
15	4.85	8.5
20	4.40	7.7
30	3.50	6.2
40	3.09	5.4

The resulting storm hydrographs are depicted in Figure 8-5, on page 8-15.

The next step in determining the necessary storage volume for the detention facility is to set a release rate and determine the volume of storage necessary to accomplish this release rate.

To determine the storage volume required, a reservoir routing procedure should be accomplished for each of the hydrographs, with the critical storm duration and required volume being determined. The importance of the particular project should govern the type of routing utilized. For small areas requiring repetitive calculations, such as parking lot bays, an assumed release curve is normally satisfactory. For larger areas, such as a pond in a small open area, a hydrograph procedure is more appropriate.

In normal flood routing, the maximum release rate will always occur at the point where the outflow hydrograph crosses the receding limb of the inflow hydrograph. For this reason, the design release rate is forced to coincide with that point on the falling limb of the hydrograph resulting from the storm duration equal to the time of concentration for the basin. The release rate is held constant past this point. The storage volume is then found by determining the area between the inflow and outflow hydrographs. Example 2 continues the calculations performed in Example 1 to determine the required storage volume.

The equation for the storm runoff volume, V_r, can be simplified as:

$$V_r = 60DQ_p \tag{8-6}$$

where:

V_r	=	storm runoff volume, in cubic feet
D	=	storm duration, in minutes
Q_p	=	peak runoff rate of the inflow hydrograph, in cubic feet per second

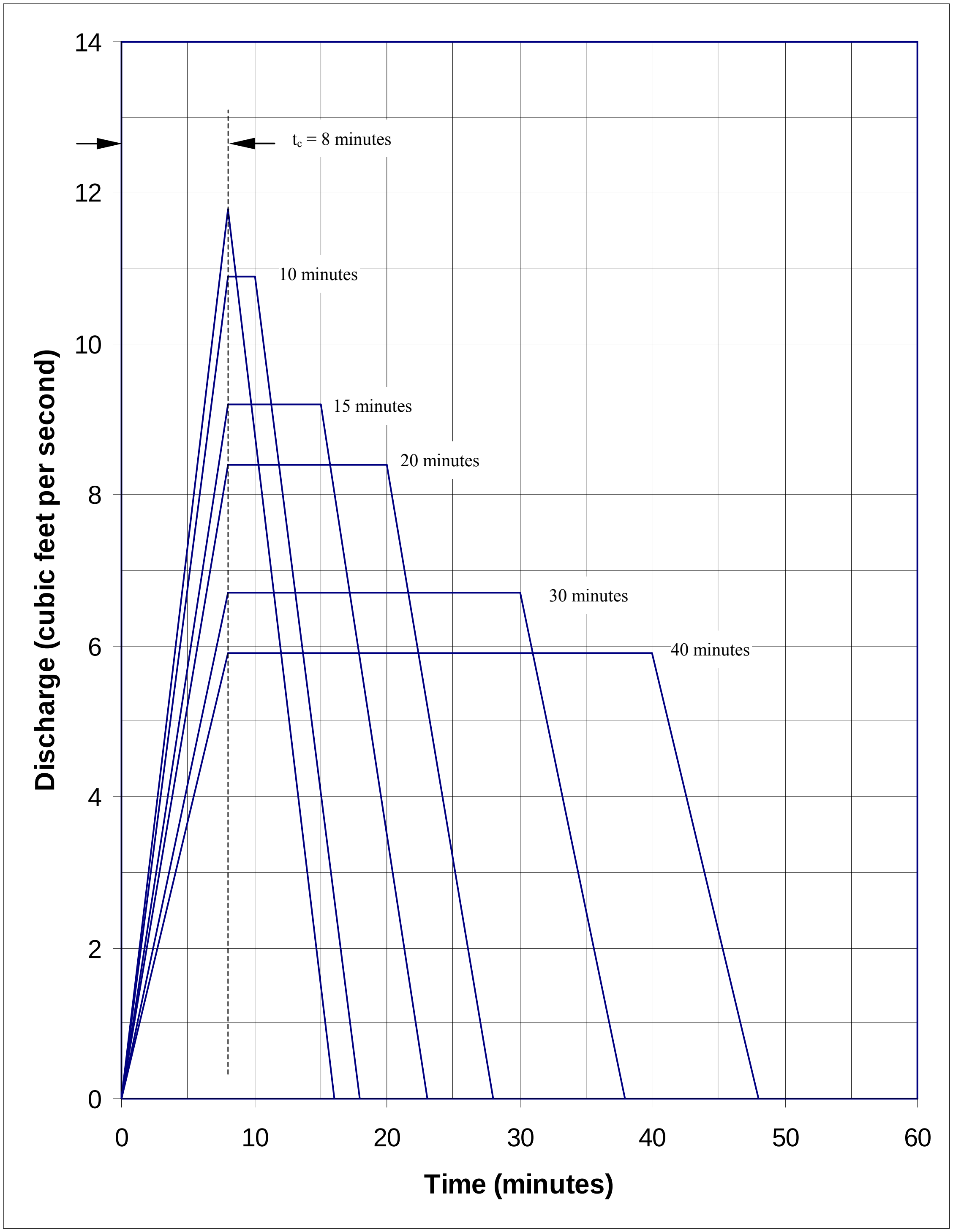


Figure 8-5 Modified Rational Method Analysis - Example 1

The equation for the storm runoff volume, V_r, can be simplified as:

$$V_r = 60DQ_p \tag{8-6}$$

where:

V_r	=	storm runoff volume, in cubic feet
D	=	storm duration, in minutes
Q_p	=	peak runoff rate of the inflow hydrograph, in cubic feet per second

The equation for the required storage volume, V_s, can also be simplified as:

$$V_s = 60D(Q_p - Q_o) \tag{8-7}$$

where:

V_s	=	required storage volume, in cubic feet
D	=	storm duration, in minutes
Q_p	=	peak runoff rate of the inflow hydrograph, in cubic feet per second
Q_o	=	maximum release rate, in cubic feet per second

Example 2: Critical Storage Volume

Given: Drainage basin and other hydrologic information presented in Example 1.

Allowable release rate: $Q_o = 4.0$ cubic feet per second

Find: Determine the critical storage volume

Solution:

$V_r = 60DQ_p$

$V_s = 60D(Q_p - Q_o)$

The table below summarizes the calculations for this example.

Table of Calculations for Example #2		
Storm Duration (minutes)	**Runoff Volume (cubic feet)**	**Required Storage Volume (cubic feet)**
8	5,232	3,312
10	6,060	3,660
15	7,650	4,050
20	9,240	4,440
30	11,160	3,960
40	12,960	3,360

The shaded area in Figure 8-6, on the next page, shows the storage required for this example. The critical storage volume is 4,440 cubic feet for a 20-minute rainfall duration.

These examples describe a method for small area detention analyses. The assumed release curve approximates a formal reservoir routing in much the same way the Rational Method Hydrograph approximates a true storm hydrograph. The curve allows for the low release rate at the beginning of a storm and an increasing release rate as the storage volume fills up.

8.4.2 Hydrograph Procedure for Storage Analysis

The unit hydrograph procedure develops a hydrograph that provides a more reliable solution for detention storage effects. The procedure provides the design professional with greater flexibility to represent the actual modeled conditions. This procedure can be used for any size drainage area. For detention basin design, minimum design storm duration of 24 hours should be used.

The development of the stormwater runoff hydrograph is presented in Section 2.3 Hydrologic Methods of this Article. The hydrograph presented by dashed line in Figure 8-7, on page 8-19 represents inflow to a reservoir by routing the peak over a side channel spillway from the channel into an adjacent ponding area. The analysis for the reservoir storage must take into consideration the characteristics of the outlet structure. The discharge curve for the outlet structure is shown in Figure 8-7 as a solid line. The shape of the solid line reflects the carrying capacity of the outlet with various headwater elevations. The higher the elevation of the water surface in the reservoir, the greater is the discharge through the outlet. The area between the dashed line of inflow hydrograph and the solid line of outflow hydrograph represents the volume of storage required to reduce channel flow from 200 to 100 cubic feet per second.

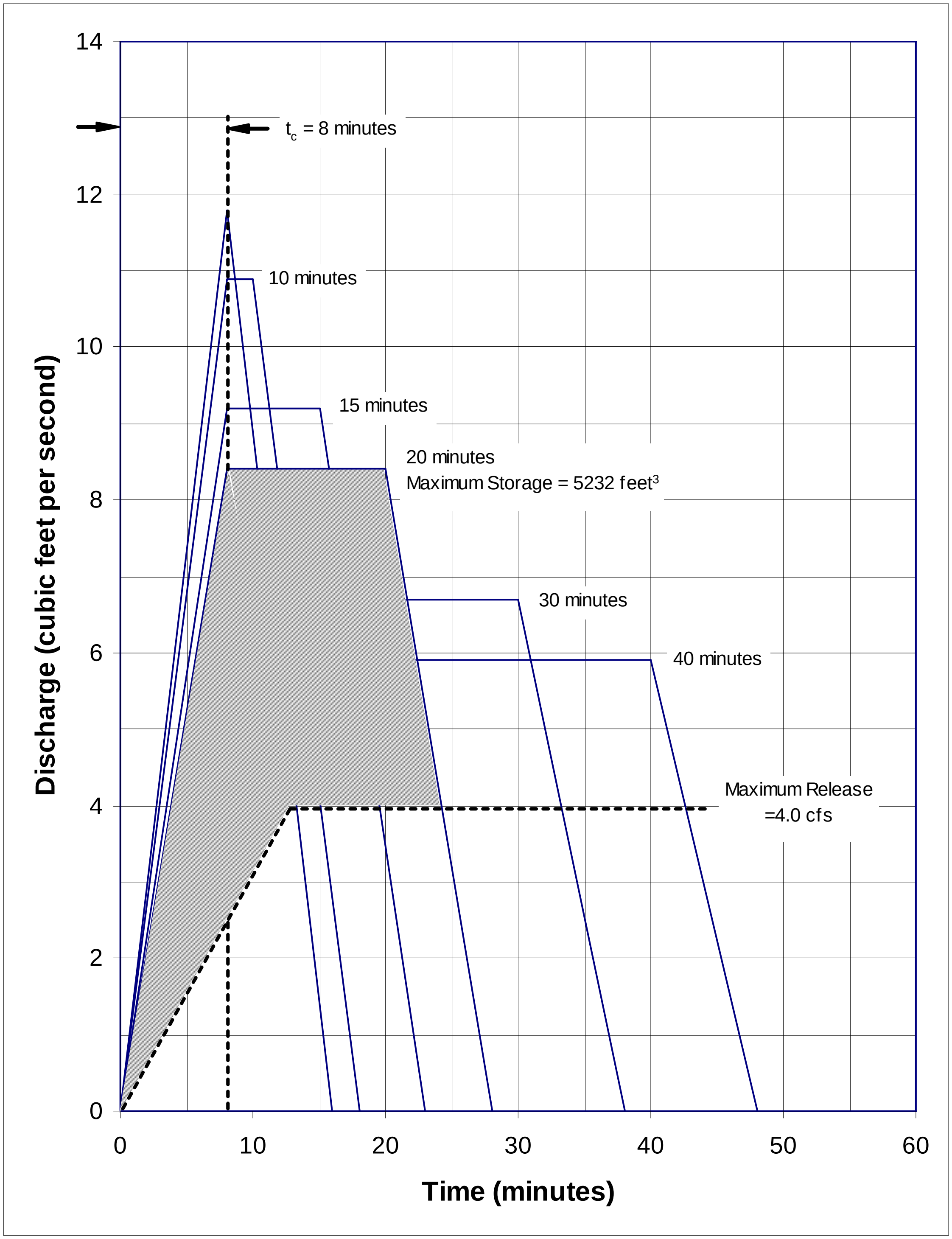


Figure 8-6 Modified Rational Method Analysis - Example 2

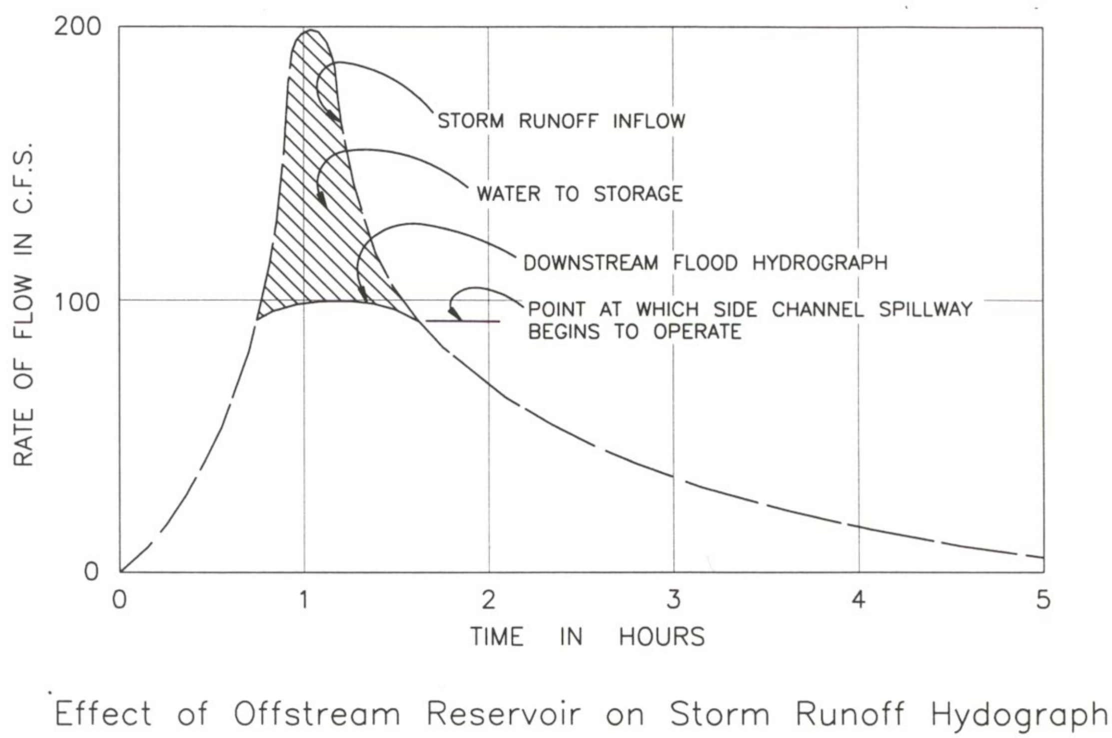


Figure 8-7 Effect of Offstream Reservoir on Storm Runoff Hydrograph

8.4.3 <u>Modified Puls Routing Procedure</u>

Modified Puls Routing Procedure may be used to determine the required volume of the detention basin. Other procedures available in published texts may be used as well. The data needed for this routing procedure include the inflow hydrograph, the physical dimensions of the storage basin, the maximum outflow allowed, and the hydraulic characteristics of the outlet structure and/or spillway.

This procedure involves the creation of the inflow hydrograph, depth-storage relationship, and depth-outflow relationship, which are combined in a routing routine. The results of the routing are the ordinate of the outflow hydrograph, the depth of storage, and the volume of storage at each point in time of the flood duration.

The time interval, Δt, is selected small enough so that there is a good definition of the hydrograph and the change in the hydrograph during the period Δt is approximately linear. This can be accomplished by setting $\Delta t = 5$ or 10 minutes, depending on size of watershed and hydrograph time to peak.

Several assumptions are made in this procedure and include the following.

A. The entire inflow hydrograph is known.

B. The storage volume is known at the beginning of the routing.

C. The outflow rate is known at the beginning of the routing.

D. The outlet structures are such that the outflow is uncontrolled and the outflow rate is dependent only on the structure's hydraulic characteristics.

The derivation of the routing equation begins with the conservation of mass, which states that the difference between the average inflow and average outflow during some time period Δt is equal to the change in storage during that time period. This can be written in equation form as:

$$\bar{I} - \bar{O} = \Delta S/\Delta t \qquad (8\text{-}8)$$

where:

$\bar{I}$	=	average inflow rate, in cubic feet per second
$\bar{O}$	=	average outflow rate, in cubic feet per second
ΔS	=	change in storage volume, in cubic feet
Δt	=	routing period, in seconds

If inflow during the time period is greater than outflow, then ΔS is positive and the pond becomes deeper. If inflow is less than outflow during the time period, then ΔS is negative and the pond becomes shallower. Using the assumptions made previously, this equation can be rewritten as:

$$\left[\frac{I_1 + I_2}{2}\right] - \left[\frac{O_1 + O_2}{2}\right] = \left[\frac{S_2 - S_1}{\Delta t}\right] \qquad (8\text{-}9)$$

where:

I_1	=	inflow rate at time interval 1, in cubic feet per second
I_2	=	inflow rate at time interval 2, in cubic feet per second
O_1	=	outflow rate at time interval 1, in cubic feet per second
O_2	=	outflow rate at time interval 2, in cubic feet per second
S_1	=	storage volume at time interval 1, in cubic feet
S_2	=	storage volume at time interval 2, in cubic feet
Δt	=	routing period, in seconds

Multiplying both sides by two and separating the right-hand side yields:

$$(I_1+I_1) - (O_1+O_2) = \left[2\frac{S_w}{\Delta t} - 2\frac{S_1}{\Delta t}\right] \tag{8-10}$$

Rearranging so that all the known terms are on the left-hand side and all the unknown terms are on the right-hand side yields the final routing equation:

$$(I_1 + I_2) + \left[2\frac{S_1}{\Delta t} - O_1\right] = \left[2\frac{S_2}{\Delta t} + O_2\right] \tag{8-11}$$

However, Equation 8-9 has two unknowns, S_2 and O_2. To obtain a solution for S_2 and O_2, a second equation relating storage and outflow is needed. If outflow is a direct function of reservoir depth (as it is with uncontrolled outflow), there is a direct relationship that exists between reservoir elevation, reservoir storage, and outflow. Therefore, for a particular elevation, there is an answer for storage and outflow (S and O). A relationship between O and $(2S/\Delta t) + O$ must be determined for several elevations and plotted on logarithmic graph paper. The routing equation is solved by adding all the known terms on the left-hand side. This yields a value for $(2S_2/\Delta t) + O_2$. This value is found on the log-log plot of $(2S/\Delta t) + O$ versus O and a value for O_2 can be determined.

Example 3: Depth-Storage/Depth-Outflow

The $(2S/\Delta t) + O$ versus O relationship is derived by combining the depth-storage relationship and the depth-outflow relationship, as previously discussed. This information is shown in Table 8-2. Columns 1, 2, and 3 are given tabulations of the depth-storage and depth-outflow relationships for a specific detention facility.

In column 4, the units of $2S/\Delta t$ and O must be the same. If O is in cubic feet per second, then $2S/\Delta t$ must be changed to cubic feet per second. For a routing time interval of 5 minutes:

$$\frac{2\,S\ \textit{cubic feet}}{5\ \textit{minutes}} \text{ x } \frac{1\ \textit{minutes}}{60\ \textit{seconds}} = 0.00667\ S$$

Thus,

$$\frac{2S}{\Delta t} + O = 0.00667S + O \text{ for } \Delta t = 5\ \textit{minutes}$$

where:

S has units of cubic feet
O has units of cubic feet per second, and
$2S/\Delta t$ has units of cubic feet per second

Table of Calculations for Example #3			
Depth (feet)	**Storage, S (cubic feet)**	**Outflow, O (cubic feet per second)**	**$(2S/\Delta t) + O$ (cubic feet per second)**
0	0	0.00	0.0
1	438	0.43	3.4
2	1224	0.61	8.8
3	2466	0.75	17.2
4	4272	0.87	29.4
5	6750	0.97	46.0

Plotting O on the Y-axis and $(2S/\Delta T) + O$ on the X-axis on a log-log graph can result in any type of curve.

8.5 DEBRIS AND SEDIMENTATION

The performance and reliability of detention facilities can be reduced by natural and man-made debris. Naturally occurring sedimentation can, over a period of time, reduce the storage capacity of a detention basin and thereby reduce the degree of flood protection provided. The obstruction of low flow conduits by debris can reduce outlet capacity and cause the premature filling of the detention basin with stormwater, again reducing the designed level of flood protection provided by the structure. Consequently, design must provide for adequate protection of the outlet from debris and for the control and removal of sediment collected in the basin.

8.5.1 Trash Racks

All outlet works and low flow conduits shall be equipped with a trash rack for debris control. The maximum spacing of trash rack bars shall not exceed two-thirds of the outlet opening size or diameter. The trash rack shall allow for passage of the design flow with 50 percent of the trash rack area blocked. Calculations for head losses through a trash rack shall be included in the hydraulic evaluation of the outlet. The trash rack should have an area equal to 10 times the area of the outlet to maintain low velocities through the trash rack.

8.5.2 Sedimentation

Sediment removal within a detention facility may be facilitated by the use of a "sediment trap" or forebay at the inlet, which will concentrate the majority of the incoming sediment bed load to a small portion of the facility. Sediment traps should be provided in conjunction with all detention facilities.

To create the forebay, a baffle or berm can be introduced to restrict the hydraulic connection between the inlet and the main body of the wet detention basin. Baffles and berms can be constructed from stone, gabions, or compacted earth. Depth is a very important design criterion if the basin is also used for water quality improvement. The average depth of 3 to 6 feet for the permanent pool is recommended for most wet detention basins.

The following list provides guidelines for the design of efficient forebays:

1. Sedimentation volume should not rise higher than the invert elevation below of the inflow channel in order to facilitate conveyance into the forebay.

2. The length/width ratio of the forebay should be a minimum of 2:1, with the length measured along a line between the inlet and outlet.

3. The basin shape should be configured to prevent flow short-circuiting from the inlet to the outlet. Short-circuiting can be minimized by placing the inlet at the opposite end or installing flow baffles or berms. Longer residence times allow the finer sediments to settle out.

4. Provisions for accumulated sediment removal from the forebay shall be provided. Maintenance access should be designed to accommodate dump trucks and other equipment necessary for removal of accumulated sediment.

SECTION 9
SEDIMENT AND EROSION CONTROL

Page

9.1 STANDARDS FOR EROSION AND SEDIMENT CONTROL

Any land disturbing activity within the City of Concord (City) must comply with the Cabarrus County Sediment and Erosion Control Ordinance and the North Carolina Erosion and Sedimentation Control Act of 1973. If the activity is one acre or greater, it must also comply with the National Pollution Discharge Elimination System (NPDES) General Permit for stormwater discharge from construction sites issued for the activity. Appropriate monitoring of the site's sediment control measures and record keeping of inspections are required as part of the NPDES permit in order to comply with North Carolina Department of Environment and Natural Resources (NCDENR) Water Quality rules. Periodic inspection of the sites by City personnel may also occur to ensure compliance with the conditions of the Phase II stormwater permit issued to the City by the NCDENR.

SECTION 10
WATER QUALITY

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10.1 INTRODUCTION

In general, stormwater runoff and nonpoint source pollution has the potential to significantly impact surface water quality. Because the quality of stormwater runoff is closely tied to proper management of stormwater quantity, it is important to address stormwater quality during conceptual project development. This section discusses stormwater management principles and specific strategies, called Best Management Practices (BMPs), for controlling stormwater.

From a stormwater runoff quality perspective, it is helpful to understand the "first flush" phenomenon. Studies have shown that the portion of stormwater runoff captured during the first fifteen minutes of a storm event contains the highest concentrations of pollutants and is commonly referred to as the first flush. Thus, the pollutant accumulation in intense small runoff events can be more detrimental to water quality than larger flooding events. Because of this, upstream management practices that control small volumes of initial runoff can be very effective in enhancing nonpoint source pollutant removal. The BMPs discussed and referenced in this ection emphasize the strategy of controlling the first flush; however, the design professional should not only consider this strategy as the only goal but as an essential component of total stormwater management. This goal must be integrated with other requirements, such as detention of increased runoff volume and considerations for maintenance and operation.

10.2 DESIGN CRITERIA AND STANDARDS

It is the responsibility of each property owner to meet stormwater runoff quality standards. This may be achieved by implementation of BMPs and other measures to reduce nonpoint source pollution. North Carolina Department of Environment and Natural Resources (NCDENR), Division of Water Quality, issues individual permits to owner's of municipal separate storm sewer systems for stormwater discharges from these systems to the receiving waters of the State. All sites required for permitting by the City of Concord's (City) Stormwater Quality Management and Discharge Control Ordinance shall utilize BMPs to meet the requirements of the Ordinance, also identified in Section 1 Policies and Requirements.

The standards and criteria for construction, operation, and performance of these BMPs are provided by NCDENR's <u>Stormwater Best Management Practices Manual</u>. Design professionals should obtain the most current version available of this manual from NCDENR and use it as the primary guidance document for BMP design and operation. The <u>Stormwater Best Management Practices Manual</u> is a guidance document developed for statewide application, and therefore, may contain design guidance not entirely applicable for use in the City. The design professional should exercise good engineering judgment in the use of the manual. The City Engineer may request additional design considerations or reject some BMP design that may compromise operation and attainment of performance standards due to regional conditions.

Other regulations may require additional stormwater runoff quality standards above and beyond those required for National Pollution Discharge Elimination System (NPDES) Phase II and outlined in the Ordinance. Therefore, it is essential that the design professional consider the future use of a site and account for any predictable water quality problems and standards of performance.

10.3 BEST MANAGEMENT PRACTICES - SOURCE CONTROL

The goal of source control BMPs is to keep pollutants from coming in contact with stormwater. Source control consists of good management and housekeeping practices intended to reduce contact between stormwater and pollutants. The source control strategies should always be considered before structural BMPs.

10.3.1 Source Control Strategies

Specific activities that are considered source control BMPs in order of preference are:

- Alter the activity
- Enclose the activity
- Cover the activity
- Segregate the activity

Alter the Activity

The preferred option of source control is altering the activity to eliminate contact between runoff and the potential pollutant source. This alteration can be accomplished by either eliminating pollutant production or by keeping it from coming into contact with the environment. The recycling of used oil rather than dumping it down a storm drain inlet is an example of altering the activity.

Enclose the Activity

If the activity cannot be altered, the next recommended approach is to enclose the activity in some structure. Enclosure accomplishes two things: first, it keeps rainfall from coming into contact with the activity and second, building drains convey non-stormwater discharges into sanitary or process wastewater sewers, or dead-end sumps, and therefore, contamination of runoff is prevented.

Cover the Activity

Placing activity inside a building may not be feasible or may be extremely expensive. A less expensive approach is placing a roof over the activity. This may be affective for certain activities, although it is unlikely to completely keep all precipitation out of contact with pollutants. Internal drains must be connected to the sanitary sewer to collect water used to wash down the area as well as any rainfall that may enter along its perimeter.

Segregation of Activity

The segregation of an activity that is a source of pollutants from other activities of little or no pollution potential may lower the cost of enclosure or covering and make this approach more effective.

Another method segregates the runoff from small, frequent storm events contributing the majority of the high concentration pollutants and/or targeted pollutants from the runoff from higher magnitude storms with low pollutant concentration. The runoff from the small storm events is collected in dead-end sumps, which must be periodically cleaned-out and pollutants properly discarded.

10.3.2 Typical Source Controls

The following is a list of the various types of Source Control BMPs that can be considered for new development:

- Land use controls:
  - Buffers around water bodies
  - Riparian zones along streams
  - Open spaces
  - Mixed and/or controlled zoning
  - Interrupted impervious area criteria

- Floodplain management practices
  - Preservation of floodplain storage
  - Use of natural, vegetated waterways

- Elimination of illicit connections

- Maintenance of drainage facilities

- Housekeeping practices
  - Street sweeping
  - Household hazardous waste collection
  - Fertilizer/herbicide/pesticide control

- Public education

10.4 BEST MANAGEMENT PRACTICES - STRUCTURAL CONTROL

One of the goals of implementing BMPs is to control nonpoint pollution by directing attention to potential contributing sources. Structural BMPs accomplish this by collecting, concentrating, and/or treating the runoff that is one of the largest pollution nonpoint sources. The techniques are designed to reduce soil loss and prevent surface runoff from carrying heavy sediment and nutrient loads into waterbodies.

Structural BMPs may generally be categorized as:

1. Detention
2. Filtration
3. Infiltration

Detention controls retain stormwater onsite either in a permanent or temporary pool with an outlet structure designed to later release the volume at a predetermined rate. The period of detention allows for a dissipation of velocity and the settlement of solids from the discharge. Filtration type BMPs use biological, physical, and chemical processes that provide a reduction in the concentration of contaminants in stormwater. Vegetative uptake, chemical fixation, and physical impedance are processes that assist in filtration. Infiltration controls provide for a reduction in the quantity of stormwater generated by enhancing the rate of infiltration of stormwater into the soil surface. Some methods for controlling stormwater quantity may have the secondary benefit of improving water quality. Natural processes work to treat stormwater for quality control once stormwater enters the vegetation and soil matrix. Selected BMPs can incorporate several of the aforementioned techniques. These BMPs are highly effective in treating a range of pollutants for a range of efficiencies.

Soils typically found in the City are dominated by clay compositions, and therefore, are not highly infiltrative. BMPs that use infiltration as the primary means for treating stormwater should not be regarded as effective options for stormwater quality control. These BMPs will tend to retain volumes of stormwater longer than expected; therefore, causing localized flooding around the BMP facility. If a design professional considers an infiltrative BMP for construction and operation, additional assurance for effective performance must be demonstrated and performed. This shall include and may not be limited to the following assurances: geotechnical testing for infiltration rates, increase in water quality storage volume, incorporation of pretreatment devices and an under drain system and quality control of construction practices to prevent damage prior to initial operation.

10.4.1 Choosing a Structural BMP

There are four screening criteria that can be used to identify the suitable structural BMP for a given situation. The first category is the physical suitability of a site for constructing and maintaining the BMP. Physical suitability includes the technical feasibility criteria related to physical conditions such as slope, soils, geology, groundwater, and area requirements. The second category is related to mitigation of adverse changes in hydrologic conditions such as increases in peak flow and volume of runoff caused by development. The third category is the pollutant removal capability of the structural BMPs; and the final category deals with the environmental and aesthetic amenities provided by the structural BMPs. Table 10-1 lists the criteria for each of the four screening categories.

TABLE 10-1
SCREENING CRITERIA FOR SELECTING STRUCTURAL BMPs

Screening Category	Criteria
Physical suitability	Drainage area Soil types Slope Distance to water table Depth to bedrock Proximity to foundations and wells Land-area requirement Maximum depth Suitability with other land uses Ability to handle high sediment inputs Thermal impacts on receiving waters
Hydrologic conditions	Peak-discharge control Volume control Groundwater recharge Erosion control Control of streambank erosion
Pollutant removal capability	Sediment Nutrients Oxygen demand Trace Metals Pathogens Temperature
Environmental and aesthetic amenities	Maintenance of low flows for aquatic life Creation of habitat for wildlife and aquatic plants Landscape enhancement and aesthetic value Recreational benefits

Source: Effective Watershed Management for Surface Water Supplies

Physical Suitability Criteria

The screening process of structural BMPs should begin with their physical suitability to local conditions. This screening is the first and most important step in the selection process. Two of the main physical factors to be considered are the total contributing drainage area and the infiltration rate of the soils. The suitability of BMPs with respect to drainage area and soil type is presented in Figures 10-1 and 10-2. Experience has shown that both retention (wet) and extended detention (dry) ponds require a contributing area of at least 10 acres to operate properly. Wet ponds are defined as facilities with permanent pools of water. A permanent flow through the facility during non-runoff periods is preferable to keep the permanent pool from being completely static, which requires a sufficient drainage area and suitable climatic conditions. The land requirements for dry ponds are governed by the size of the discharge structure (usually an orifice). Infiltration trenches, vegetated swales, and filter strips are generally acceptable for areas less than 10 acres due to limitations related to space requirements, economics, or their limited range of flow velocity. Hereinafter, filter strips shall refer to revegetated areas on disturbed, graded slopes and are designed to accept only overland sheet flow. This area limitation does not apply to vegetated buffers along streams and bodies of water where an area limitation would not be applicable. Infiltration BMPs are better suited for coarse-textured soils. Soil types such as sandy-gravely soils eliminate wet ponds as an option because these soils are highly permeable, making it difficult to maintain a permanent pool of water.

Other physical restrictions include the slope of the site, depth of bedrock, proximity of nearby wells and building foundations, contributing area requirements, maximum depth limits, applicability to certain land uses, ability of the structural BMPs to handle large sediment loads without clogging, and thermal impact on receiving waters. Common restrictions for various structural BMPs are presented in Figure 10-3. Examination of the data in this figure shows that steep slopes can prevent the use of infiltration and vegetation BMPs. Furthermore, high water tables and shallow bedrock that impedes downward movement of water also limits use of infiltration BMPs. Care must be taken to avoid locating infiltration BMPs close to buildings or water supplies (wells). Because detention ponds often require large areas, placement in existing developments may not be feasible. In addition, many structural BMPs (ponds, trenches, etc.) have depth limitations. When the depth of ponding exceeds 8 feet, stratification can occur during warm weather resulting in anoxic conditions along the bottom of the pond. Several structural BMPs are limited to certain type of land uses or development densities. Vegetated swales are usually limited to low-density residential development and road right-of-way.

Hydrologic Criteria

Hydrologic criteria focus on the ability of the structural BMPs to reduce the runoff from an area to pre-development levels or some other defined condition. Criteria include peak discharge control, volume control, groundwater recharge, and erosion control. Figure 10-4 compares the ability of various structural BMPs to meet these conditions. Similar to the physical suitability criteria, no single BMP can mitigate all hydrologic modifications caused by urban development.

The City requires all water quality BMPs to be designed to control peak discharges for the 1- and 10-year, 24-hour storm events (refer to Table 1-1 in Section 1.3 Design Policy). Detention BMPs accomplish this by temporarily storing the runoff up to desired volume and using outlet structures that allow only a certain flow to pass while detaining the additional flow until the peak has passed. When the pond inflow falls below the value of maximum allowable outflow, the volume stored in the structure will begin to decrease. Infiltration BMPs often have only limited capacity to reduce peak flows. Likewise, vegetative BMPs usually have almost no peak discharge control.

The volume control method decreases the total runoff volume to downstream areas. Detention ponds are ineffective, since they just provide temporary storage and then eventually release the remainder of the volume. On the other hand, infiltration BMPs are effective in reducing the total volume of runoff, because they are designed to divert water back into the soil. These structures are also an excellent means of groundwater recharge.

As a general rule, natural channels with banks flowing full can contain a 2-year storm. Therefore, BMPs that discharge a 2-year storm or greater event may cause erosion in the channel. The BMP discharge must be kept below the 2-year flow and the frequency of occurrence must be minimized. Extended detention ponds in combination with some infiltration devices provide an effective solution for proper design and maintenance.

Pollutant Removal Criteria

Pollutant removal is a function of three interrelated factors: 1) the removal mechanisms, including physical, chemical, and biological processes; 2) the fraction of runoff to be treated by the BMP; and 3) the pollutant(s) targeted for removal. Those BMPs that use settling and filtering processes are effective in removing sediment and those pollutants (both solid and soluble) that adhere to sediment particles. Case studies have shown that ponds can remove as much as 85 percent or more of sediment. However, infiltration BMPs are not recommended without pre-treatment, where high loadings of sediment are encountered, due to the high potential for clogging. Strategically placed filter strips may remove sediment prior to flow entering the infiltration BMP. The vegetated swales and filter strips often provide limited removal rates because urban stormwater frequently short-circuits through buffer areas. Shallow marshes along the perimeter of wet ponds and extended detention ponds have a moderate to high capability to remove both particulate and soluble pollutants due to settling and biological uptake. Figure 10-5 displays the relative capacity for pollutant removal for varying BMPs.

Environmental and Aesthetic Amenity Criteria

The selection of structural BMPs is often determined by the environmental benefits that can be achieved and by the community's willingness to accept the facility. Environmental and aesthetic amenities may include control of stream bank erosion, creation of aquatic habitat, creation of wildlife habitat, elimination of thermal stratification, landscape enhancement, creation of recreational facilities, and reduction of existing hazards. However, the environmental and aesthetic benefits may not be realized unless community's acceptance of the structure is obtained.

Figure 10-6 provides guidance in understanding the offerings of environmental and aesthetic elements benefits for BMPs.

	1 ACRE	5 ACRES	10 ACRES	25 ACRES	50 ACRES	>100 ACRES
WET (RETENTION) POND	○	○	◑	●	●	●
EXTENDED DETENTION POND	○	○	◑	●	●	●
WETLAND SYSTEM	○	◑	◑	●	●	●
INFILTRATION TRENCH	●	●	◑	○	○	○
BIORETENTION AREA	●	●	◑	○	○	○
FILTER STRIP	●	●	◑	○	○	○
VEGETATED SWALE	●	●	◑	○	○	○
SAND FILTER	●	●	◑	○	○	○
PERMEABLE PAVEMENTS	●	◑	○	○	○	○

○ MAY PRECLUDE THE USE OF THIS BMP
◑ CAN BE OVERCOME BY GOOD SITE DESIGN
● GENERALLY NOT A RESTRICTION

Figure 10-1 Drainage Area Restrictions of Structural BMPs

	SAND	SANDY LOAM & LOAMY SAND	LOAM	SILT LOAM	SANDY CLAY LOAM	CLAY LOAM	SILTY CLAY LOAM	SILTY CLAY & SANDY CLAY	CLAY
WET (RETENTION) POND	○	◑	●	●	●	●	●	●	●
EXTENDED DETENTION POND	●	●	●	●	●	●	●	◑	○
WETLAND SYSTEM	○	◑	●	●	●	●	●	◑	○
INFILTRATION TRENCH	●	●	●	◑	○	○	○	○	○
BIORETENTION AREA	●	●	●	◑	○	○	○	○	○
FILTER STRIP	◑	●	●	●	●	●	●	●	◑
VEGETATED SWALE	◑	●	●	●	●	●	●	●	◑
SAND FILTER	●	●	●	◑	○	○	○	○	○
PERMEABLE PAVEMENTS	●	●	●	◑	◑	◑	◑	◑	◑

○ MAY PRECLUDE THE USE OF THIS BMP
◑ CAN BE OVERCOME BY GOOD SITE DESIGN
● GENERALLY NOT A RESTRICTION

Figure 10-2 Soil Restrictions of Structural BMPs

	SLOPE	HIGH WATER TABLE	HIGH BEDROCK	PROXIMITY TO FOUNDATIONS	SPACE CONSUMPTION	MAXIMUM DEPTH	LAND USE RESTRICTIONS	HIGH SEDIMENT LOADS	THERMAL LOADING
WET (RETENTION) POND	●	●	◑	◑	○	○	●	◑	◑
EXTENDED DETENTION POND	●	●	◑	◑	○	○	●	◑	●
WETLAND SYSTEM	◑	◑	◑	◑	◑	○	◑	◑	○
INFILTRATION TRENCH	○	○	○	○	◑	○	●	○	●
BIORETENTION AREA	◑	○	○	○	●	○	◑	○	◑
FILTER STRIP	◑	◑	●	●	◑	●	●	○	○
VEGETATED SWALE	◑	◑	◑	●	●	◑	●	◑	○
SAND FILTER	○	○	○	○	◑	○	◑	○	●
PERMEABLE PAVEMENTS	○	○	◑	◑	◑	●	◑	○	●

○ MAY PRECLUDE THE USE OF THIS BMP
◑ CAN BE OVERCOME BY GOOD SITE DESIGN
● GENERALLY NOT A RESTRICTION

Figure 10-3 Site Restrictions of Structural BMPs

	1-YEAR STORM	10-YEAR STORM	100-YEAR STORM	VOLUME CONTROL	GROUNDWATER RECHARGE	EROSION CONTROL	STREAMBANK PROTECTION
WET (RETENTION) POND	●	●	●	●	○	◐	●
EXTENDED DETENTION POND	●	●	●	●	○	◐	●
WETLAND SYSTEM	●	◐	○	◐	◐	●	●
INFILTRATION TRENCH	◐	◐	○	◐	●	◐	◐
BIORETENTION AREA	◐	○	○	◐	●	◐	○
FILTER STRIP	◐	○	○	○	◐	●	◐
VEGETATED SWALE	●	◐	○	○	◐	●	◐
SAND FILTER	◐	○	○	●	◐	◐	◐
PERMEABLE PAVEMENTS	◐	○	○	◐	◐	◐	○

○ WILL NOT CONTRIBUTE
◐ CAN CONTRIBUTE WITH GOOD SITE DESIGN
● GENERALLY DOES NOT CONTRIBUTE

Figure 10-4 Hydrologic Benefits of Structural BMPs

	SEDIMENT	PHOSPHORUS	NITROGEN	OXYGEN DEMAND	HEAVY METALS	PATHOGENS	TEMPERATURE
WET (RETENTION) POND	●	●	◐	◐	●	○	○
EXTENDED DETENTION POND	●	◐	◐	◐	◐	○	○
WETLAND SYSTEM	●	◐	○	●	●	○	●
INFILTRATION TRENCH	●	●	●	○	●	●	●
BIORETENTION AREA	●	◐	◐	●	●	◐	◐
FILTER STRIP	◐	◐	◐	○	◐	○	○
VEGETATED SWALE	○	◐	◐	○	○	○	○
SAND FILTER	●	◐	○	○	◐	◐	◐
PERMEABLE PAVEMENTS	◐	◐	●	◐	●	○	●

○ NOT EFFECTIVE
◐ MODERATELY EFFECTIVE
● HIGHLY EFFECTIVE

Figure 10-5 Pollutant Control of Structural BMPs

	STREAM LOW FLOWS	AQUATIC HABITAT	WILD LIFE HABITAT	LANDSCAPING	RECREATION	AESTHETICS
WET (RETENTION) POND	◐	●	◐	◐	●	●
EXTENDED DETENTION POND	○	○	○	◐	●	◐
WETLAND SYSTEM	●	●	◐	●	●	●
INFILTRATION TRENCH	●	○	○	◐	○	○
BIORETENTION AREA	●	○	○	●	○	●
FILTER STRIP	◐	○	◐	●	◐	●
VEGETATED SWALE	●	○	○	○	○	◐
SAND FILTER	◐	○	○	◐	○	○
PERMEABLE PAVEMENTS	◐	○	○	◐	◐	◐

○ WILL NOT CONTRIBUTE
◐ CAN CONTRIBUTE WITH GOOD SITE DESIGN
● GENERALLY DOES CONTRIBUTE

Figure 10-6 Environmental and Aesthetic Amenity Contribution of Structural BMPs

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