Chapter Six

Drainage and Stormwater Management

6.1 INTRODUCTION

Adequate drainage is essential in the design of highways since it affects the highway's serviceability and usable life, including the pavement's structural strength. If ponding on the traveled way occurs, hydroplaning becomes an important safety concern. Drainage design involves providing facilities that collect, transport and remove stormwater from the highway. The design must also consider the stormwater reaching the roadway embankment through natural stream flow or manmade ditches.

This chapter deals with drainage policies, procedures and guidance to be followed in achieving cost-effective design and construction within DelDOT's Highway System. The information contained herein is compiled from various federal and national publications, textbooks, and drainage manuals. The information provided is of a general nature with the inclusion of methods, criteria and references specifically applicable to DelDOT projects.

Source documents are listed in the introduction to each section. It is presumed that the designer is familiar with the basic theory and methods of analysis and design in both hydrology and hydraulics. The information provided herein will have to be supplemented with hard copies or on-line access to the referenced documents.

The regulatory environment related to drainage design is ever changing and continues to grow in complexity. Engineers responsible for the planning and design of drainage facilities must be familiar with Federal, state, county and local regulations, laws, and ordinances that may impact the design of storm drain systems.

Many federal laws have implications that affect drainage design. These include laws concerning:

- Flood insurance and construction in flood hazard areas,
- Navigation and construction in navigable waters,
- Water pollution control,
- Environmental protection,
- Protection of fish and wildlife, and
- Coastal zone management.

Federal agencies formulate and promulgate rules and regulations to implement these laws. Highway hydraulic engineers should keep informed regarding proposed and final regulations.

Some of the more significant federal laws affecting highway drainage are:

- **The Department of Transportation Act** established the Department of Transportation and sets forth its powers, duties, and responsibilities to establish, coordinate, and maintain an effective administration of the transportation programs of the Federal Government.
- **Federal-Aid Highway Acts** provide for the administration of the Federal-Aid Highway Program. Proposed Federal-aid projects must meet existing and probable future traffic needs and conditions in a manner conducive to safety, durability, and economy of maintenance, and must be

designed and constructed according to standards best suited to accomplish this and to conform to the needs of each locality.

- **Federal-Aid Highway Act of 1970** provides for the establishment of general guidelines to ensure that possible adverse economic, social and environmental effects relating to any proposed Federal-aid project have been fully considered in developing the project. In compliance with the Act, the Federal Highway Administration (FHWA) issued process guidelines for the development of environmental action plans as contained in the Federal-Aid Highway Program Manual (FHPM) and in 23 CFR 795 et seq.
- **Federal-Aid Highway Act of 1966 amended by the Act of 1970** requires the issuance of guidelines for minimizing possible soil erosion from highway construction. In compliance with these requirements, the FHWA issued guidelines that are applicable to all Federal-aid highway projects. These guidelines are included in the FHPM. Regulatory material is found in 23 CFR 650.201.

All Federal-aid projects shall conform to FHWA guidelines. FHWA policy includes the following:

- **FHWA Policy 23CFR635 Subpart D—General Material Requirements**
- **FHWA Policy 23CFR650 Subpart A—Location and Hydraulic Design of Encroachments on Floodplains** establishes policy affecting any project that includes an encroachment on a base floodplain.
- **FHWA Policy 23CFR650 Subpart C—National Bridge Inspection Standards** defines the national standards for the proper safety inspection and evaluation of all highway bridges including the evaluation of bridges for scour susceptibility in accordance with the guidance in FHWA Technical Advisory T5140.23.
- **FHWA Policy 23CFR650 Subpart H—Navigational Clearances for Bridges** requires coordination with the United States Coast Guard (USCG) and United States Army Corps of Engineers (USACE) in providing adequate vertical and horizontal clearance for navigation on navigable waterways.

Other federal laws affecting hydraulic tasks, analyses, design, or construction include those formulated under the following legislative acts:

- The Coastal Zone Management Act of 1972
- The Federal Water Pollution Control Act (Clean Water Act)
- The Fish and Wildlife Act of 1956
- The Fish and Wildlife Coordination Act
- The Flood Disaster Protection Act of 1973
- The National Environmental Policy Act (NEPA) of 1969
- The Rivers and Harbors Act (1899)
- The Safe Drinking Water Act of 1974
- Water Quality Act of 1987
- Wild and Scenic Rivers Act

6.2 DESIGN RESPONSIBILITIES

Responsibilities for drainage design are divided between the Bridge Design and Project Development sections based primarily on the size of the drainage basin. Bridge Design is responsible for all watersheds over 300 ac and locations where the design discharge capacity opening(s) exceeds 20 ft^2. Project Development is responsible for:

- All locations requiring pipe culverts not exceeding 20 ft^2 of waterway opening;
- Closed drainage systems involving storm drains;
- Roadside drainage including swales and ditches;
- Erosion and sediment control for transportation projects;

- Stormwater quantity and quality management.

The section responsible for design of the drainage structure shall also design the related stream bank stabilization and erosion control.

The scope of this chapter is limited to the drainage design responsibilities of the Project Development Section. Refer to Chapter Three of the Bridge Design Manual for information on drainage design for structures. Structural designs are coordinated through the Bridge Design Section.

6.3 DESIGN CRITERIA

The Department has established basic design criteria for design storm frequency depending upon roadway type, and the design of open channel flow, pipe flow, pavement drainage, and storm drains (closed drainage systems). These criteria are summarized in Figures 6-1 through 6-3.

6.4 DESIGN PROCEDURES

The goal of drainage design is to economically design systems that limit the exposure of adjoining upstream and downstream properties, the newly constructed roadway and the traveling public to an acceptable flood risk during high flows. Attachment B has several example problems that illustrate the design procedure for the most commonly used drainage facilities used on a roadway project. AASHTO's *Model Drainage Manua*l is one of the best resources for preparing a project's drainage design. In addition, DelDOT has an Internet Web Site (http://www.deldot.gov/) that has additional drainage references.

Below is a brief description of drainage design steps normally followed when designing roadway facilities. Each of these is expanded in sections referenced in the steps.

Step 1. Data collection (Section 6.5) includes:

a. Establish agency requirements and define design criteria and parameters.

b. Access the applicable online drainage design resources (hard copies, if necessary), other design references, environmental controls, drainage criteria, etc.

c. Prepare a drainage area map using USGS maps, topographic maps and aerial photography.

d. Determine soil characteristics using the county soil survey map from the US Natural Resource and Conservation Service (NRCS).

e. Obtain As-Built or record plans from previous projects in the area.

f. Contact the area maintenance office for existing flooding problems.

g. Request soil borings to establish the typical water table elevation. Saturated soil conditions can adversely impact the drainage system design; require expensive dewatering to build; and require permits. Therefore this condition should be identified early in the process.

h. Obtain existing and future land use information.

i. Visit the anticipated drainage area site to verify collected data, locate existing structures and visually observe any problems with their past performance, signs of past high water levels, stream erosion problems, currently developed areas, existing stormwater management facilities, etc.

j. Request a video inspection of any drainage facility to remain in service.

k. Obtain existing and proposed underground utility locations.

Figure 6-1
Design Criteria ─ Frequency
(Return Period in Years)

Functional Classification	Type of Drainage Installation [1]			
	Pipe Culverts	Storm Drains	Roadside Ditches	Median Drains
Interstate, Freeways and Expressways	50	10 [2]	50	50
Arterials	50	10 [2]	25	25 [2]
Collectors	50 [3]	10 [2]	25 [4]	10 [2]
Local Roads and Streets including Subdivision Streets	25	10 [5]	10	10 [5]

[1] For Stormwater Management see Section 6.10.

[2] Use a 50-yr frequency at sag points, i.e. underpasses or depressed roadways, where ponded water can be removed only through the storm drain system.

[3] Use a 25-yr frequency for rural collectors.

[4] Use a 10-yr frequency for rural collectors.

[5] Use a 25-yr frequency at underpasses or depressed roadways where ponded water can be removed only through the storm drain system.

Figure 6-2
Design Criteria ─ Allowable Spread on the Pavement Cross Section

Functional Classification	*Allowable Water Spread*
Interstate, Freeways, Expressways	Full shoulder width
Arterials with full shoulder or parking lane Arterials with less than full shoulder or parking lane	Full shoulder or parking lane One half of adjacent driving lane
Collectors Design speed < 45 mph Design speed ≥ 45 mph	 One half of driving lane Full shoulder width
Local Roads and Streets including Subdivision Streets	One half of driving lane
Sag Points (all facilities)	Full shoulder width

Figure 6-3
Design Criteria ─ Miscellaneous

Ditches	
Ditch flow line below edge of shoulder Ditch water surface elevation below edge of shoulder Minimum ditch grade	≥2.5 ft preferred 0.5 ft minimum; 1 foot preferred 0.003 ft/ft (preferred 0.005)
Pipes	
Minimum size - cross road pipe / culvert Minimum size - storm drain	18 in 15 in
Minimum full flow velocity	3.0 ft/s
Maximum outlet velocity (determined by design parameters)	Based on scour, erosion, risk potential of discharge channel, and mitigating measures such as energy dissipators
Maximum continuous distance between storm drain structures (without clean-out access)	300 ft
Reinforced Concrete Pipe Bedding	Class C, unless specified otherwise
Personnel Grate for Pipe Inlet	All pipes 12 in and larger with an open inlet that is not a straight run to the outlet without full daylight visible when looking through the pipe to the other end
Minimum Pipe Cover	3 ft preferred at profile grade line, 1 ft at the shoulder, depending upon the structural requirements of the pipe material, bedding type and fill height at the installation location; see supplemental figures on DelDOT's web site and Table 12.6.6.3-1 of the *AASHTO LRFD Bridge Design Specifications*
General Criteria - Storm Drain System	
Hydraulic Grade Line (HGL)	≥ 1 ft below top elevation of all manhole covers or top of any inlet
Required manhole or inlet locations	Intersection of two or more storm drains Pipe size change Alignment change Grade changes and sag points in curbed sections
Outfall pipe invert elevation	≥ 0.2 ft below lowest incoming pipe elevation (preferred)
Discharge pipe	Invert higher than the outfall elevation
Inlet clogging factor of safety	1.5 with curb and 2.0 without curb 1.0 for curb opening inlet

Step 2. Hydrology (Section 6.6)

The principles of a hydrologic study for a watershed are common to most design methods. The watershed is defined and divided into subareas contributing to proposed stormwater outlets. Water flows through the watershed as sheet flow, shallow concentrated flow, swales, open channels, pavement drainage, roof drains, and storm drains The existing and future surface runoff characteristics of these subareas are determined and a coefficient or runoff curve number is assigned. The flow parameters are than determined including the hydraulic length of the reaches, the types of flow, and average slope. Using this data flow velocities are determined and the flow times of each type of flow is determined and combined into a time of concentration (t_c) for the watershed. A design rainfall frequency and intensity is selected. Using this data the peak discharge is determined.

Available methods of hydrological analysis include the Rational Method; TR-20 Project Formulation Hydrology Program System; TR-55 Urban Hydrology for Small Watersheds and its Windows version WINTR-55; and the U.S. Geological Survey (USGS) Method which has developed regression equations for Delaware. Each of these procedures has its own methodology, assumptions, limitations, and applications.

The Rational Method is most frequently used for estimating small, homogenous or highly impervious drainage areas which are smaller than 50 acres, including gutter flow, drainage inlets, stormwater systems, small ditches, swales and culverts. This method is not recommended for storage design, i.e., stormwater management.

TR-20 is a computer hydrological model simulating runoff hydrographs using natural and synthetic rainfall events over a large complex watershed up to 2000 acres with multiple subareas, channel reaches, and varied confluence points that are routed through downstream reaches and structures.

Data generated by TR-20 has been used to develop a more user-friendly method for analysis of small watersheds (1 to 300 acres) referred to as TR-55 and WINTR-55. Except for pavement drainage, these are commonly used when analyzing stormwater flow for watersheds and their subareas under both pre- and post-development conditions. TR-55 can be applied to designing most of a project's drainage design features, including stormwater management facilities.

USGS has developed regression equations for Delaware based upon years of data from stream gauging stations. This method is most applicable for estimating the magnitude and frequency of flood-peak discharges and flood hydrographs for large streams, channels and culverts. These equations relate peak discharge to independent variables describing the physical and climatic basin characteristics. Refer to the Bridge Design Manual for more details.

Step 3. Hydraulics

Using the hydrologic data and other acquired information, lay out an initial drainage system to accommodate this flow. Drainage system may include various types and shapes of open channels (i.e., ditches, swales, and gutters), storm drains, median drains, and cross road pipe culverts. The system profile should begin at the farthest discharge point downstream, i.e., a natural or artificially created channel, or an existing drainage system. To minimize the expense of pipe excavation, pipe slopes should conform to the surface slope wherever possible.

Roadside ditches and culverts pose a roadside hazard and should be designed considering safety as a design parameter. Use the principles found in AASHTO's *Roadside Design Guide* to minimize their potential as roadside hazards.

a. Open Channel Design (Section 6.7): There are several types of open channels used in drainage design including roadside ditches, toe-of-slope ditches, intercepting ditches, chutes (stone-lined or pipes) used on step embankments, and gutters used in curbed sections.

i. For design of roadside channels, hydraulic conditions usually are assumed to be steady and uniform. Flow under these conditions has an energy line approximately equal to the average ditch grade, and a flow depth that changes gradually over time. This allows for determining normal flow values where velocity, depth of flow, surface roughness and slope is constant over the design section.. The capacity or discharge of this open channel is determined by using the modified Manning's equation (which combines Manning's and the continuity equations). Resources are referenced in Section 6.7 that detail open channel design procedures.

ii. For non-uniform flow where either the depth of flow, velocity, or cross section changes, the principles and equations for unsteady, non-uniform flow are used, the computer program HEC-RAS is recommended.

iii. Filter strips, biofiltration swales, bioretention areas and infiltration ditches are designed using DURMM, DNREC's green technology computer model based on a combination of computer routines from TR-22 and TR-55.

b. Pavement (Storm) Drainage System Design (Section 6.8): There are several reasons to provide adequate pavement drainage design, including maintaining the service level during stormy conditions; protecting the user from ponding water that could cause loss of vehicle control; protecting adjacent natural resources, development and property; and maintaining the structural integrity of the pavement section. A typical pavement/storm drainage system consists of curb, gutter, inlets, drainage structures (i.e., junction boxes and manholes), storm drains (pipes), and an outfall structure into an acceptable discharge point.

The design of a project's drainage system may require installing new pipe culverts, or upgrading existing pipe culverts. In this instance see Step 4.

i. The **hydrologic analysis** should be consistent with Step 2. The rational method is used to calculate the peak discharge to be carried by the gutter and storm drains.

ii**. Gutters** are designed using the principles for open channel flow by establishing the rate of flow or discharge, the depth of flow, the velocity, the surface roughness and the slope using a modified form of Manning's equation.

iii**. Inlets** can be grate inlets, curb-opening inlets, (a combination inlet involving a curb opening combined with a grate inlet), or occasionally slotted inlets. Refer to DelDOT's *Standard Construction Details* for the detailed description of their shape and dimensions. See Figure 6-2 and 6-3 for the recommended allowable spread on the pavement and clogging capacity (efficiency). Inlet design consists of determining the interception capacity of the type of inlet under consideration and its location on a continuous slope or in a sag point.

The locations of inlets are a function of the roadway geometrics and cross section, in addition to hydraulic considerations. The underground connecting pipes have to be accessible for maintenance. The physical constraints from using available inspection and cleanout equipment are a limiting factor in the maximum distance between inlets. The volume of inlet bypass flow combined with the normal flow calculated to reach each downstream inlet affect the inlet spacing. Other factors include features such as curb ramps, crosswalks, driveways, intersecting streets, points of superelevation transition, and roadway profile low points or sag vertical curves. Refer to Section 6.8 and Example 5, Attachment B, for a description of manually determining inlet spacing. It is important to note that inlet design in sag locations (both for vertical curves and superelevated sections) is critical due to ponding that might encroach on the roadway.

iv. **Storm drains (pipes)** are used in areas of limited right-of-way and in curbed sections, normally in combination with inlets, junction boxes, manholes and outfall structures. Sections of smaller pipe are referred to as segments or runs. The shorter runs usually discharge into a larger main storm drain called a trunk line discharging into an outfall. Locating storm drains, particularly manholes or other drainage appurtenances, outside of the roadway (especially the traveled way) should be considered for safer maintenance operations and minimizing traffic disruption. In addition, if there is pavement settlement due to compaction problems or erosion of the subgrade from joint separation, the repairs can be made with a reduced effect on the flow of traffic.

Pipe criteria includes the minimum allowable pipe size, the material type based on the allowable service life for the project and location being designed, and any special installation or structural requirements specified by the *Standard Specifications* or manufacturer. Refer to DelDOT *Design Guidance Memorandum Number 1-20R*.

Placement with respect to underground and above ground utilities is to be considered throughout the system design. Utility relocations and/or their protection are expensive. The cost may be fully paid by the project or shared with the utility provider and their customers.

Storm drain capacity design is based on the principles of open channel flow, i.e., steady, uniform flow with a constant average velocity within each pipe run. This is accomplished by assuring the HGL is below the top of pipe. To complete this analysis the designer needs to have a working plan showing the layout and profile of the storm drainage system with the location of storm drain runs and trunk lines, direction of flow, location and numbering of drainage structures and manholes, and location of utilities. Starting with the upstream storm drain run, the designer performs the hydrological calculations for the system. Storm drains are sized using Manning’s equation, varying the slope and size needed for the design discharge. Losses in the system at the various drainage structures, bends, etc. are calculated. The final step is determining the system HGL to ensure open channel flow exists throughout the system. The design should be evaluated and adjusted to avoid pressure flow conditions. See Section 6.8 for a more detailed discussion.

Median drainage is an important consideration on divided roadways, especially with high-speed lanes. Ponding water and its spread onto the travel lane or shoulder need to be minimized and the pavement’s structural integrity protected from saturation leading to failure. This is difficult since the shape and width of the median is primarily controlled by safety considerations and not by the preferred hydraulic characteristics. Therefore medians are usually wide, shallow, and depressed with a swale with relatively flat side slopes and drainage inlets to control the flow.

The principles of open channel flow are used for locating inlets to limit the depth of water to an elevation of 0.5 ft minimum, one ft preferred, below the outside edge of shoulder. Inlets in sags may function either as a weir or orifice depending upon the type of grate and depth of water, so sag locations must be given special consideration. The preferred method of ensuring adequate drainage in a sag location is to install inlets (flanker basins) upstream from the low point.

Drainage structures include manholes, junction boxes and outlet structures. Each of these has a particular function in the drainage system and a role in the efficiency, capacity and future maintenance of the system. Refer to the *Standard Construction Details* for the size, shape, etc. of these structures. The designer should check the structure dimensions versus the storm drain size to ensure they are compatible. Of primary concern is allowing for the losses in flow capacity these features introduce into design of the system. HDS-5 and HEC-22 discuss this subject.

Designing a storm drainage system for the roadway section is normally an iterative process with several data inputs; the use of available drainage software is recommended. Available references include HEC-22 *Urban Drainage Design Manual*; HEC-12 *Drainage of Highway Pavements; The State-of–the Art, Storm Drain Systems* (Volume IX of the AASHTO *Highway Drainage Guidelines*); and the AASHTO *Model Drainage Manual*.

Step 4. Culverts (Pipe Culverts Section 6-9)

Culverts are closed conduits (pipes) placed through an embankment as a part of the drainage system to convey runoff collected by roadside channels or natural watercourses such as streams.

Frequently a roadway project will involve installation of a new cross pipe culvert and/or upgrading an existing culvert. For culverts requiring an area of 20 square feet or more, refer the design to the Bridge Design Section.

Hydraulic design considerations include establishing the design storm frequency, the allowable headwater, the allowable tailwater, the minimum allowable pipe diameter, and the type of pipe material (considering allowable service life and location with respect to location under the pavement or the embankment, and the anticipated design load). Headwater and tailwater levels depend upon the roadway typical section and profile, acceptable upstream and downstream flooding, stormwater management interests and other environmental concerns.

Refer to the resources listed in Section 6.9 for detailed design procedures for culverts. Culvert placement and end treatments affect roadside safety.

Step 5. Erosion and Sediment Control (Section 6-10)

Erosion and Sediment Control is an integral part of designing and constructing a drainage system. The erosion and sediment plan has to be prepared concurrently with the drainage plan. The requirements of this plan are in DelDOT's *ES$_2$M Design Guide* and the *Delaware Sediment and Stormwater Regulations*.

Step 6. Stormwater Management (Sections 6-11 and 6-12)

Managing highway runoff involves designing systems to address stormwater quantity and quality. The design of these facilities is covered in more detail in Sections 6.11 and 6.12; the *ES$_2$M Design Guide*; and the *Delaware Erosion & Sediment Control Handbook;* and DNREC's *Green Technology*: *The Delaware Urban Runoff Management Approach DURMM: The Delaware Urban Runoff Management Manual.*

The goal of runoff quantity management is to minimize excessive downstream flooding and property damage. The basic principle is that any proposed land development (including roadways) should not increase the downstream flow above what existed prior to the changes. Two methods used to control this flow are:

1. Retention facilities are used to control stormwater runoff and dissipate it through evaporation and infiltration. They are commonly used when there is no nearby natural watercourse or body of water for the release of the stored water.

2. Detention facilities are used to reduce the peak discharge and slowly release this runoff. Normally they are designed to release all of the stored water after the storm has passed. Frequently detention and retention facilities are combined to: manage pollution and sediment; control stormwater quantity discharge; and as an aquifer recharge basin.

A second goal of stormwater management is to maintain water quality. Runoff from developed areas can degrade water quality. Designing for water quality is referred to as Non-Point Source Pollution Control. Legislation referred to as the National Pollutant Discharge Elimination System (NPDES) requires that the pollutants in runoff from storm systems be reduced. Pollutions are reduced using Green Technology BMP's including biofiltration grass swales, biorentention areas, infiltration ditches and area filter strips, forebays at pipe outfalls and/or retention ponds with slow release outlets.

Step 7. Drainage Report:

The final product of the drainage design is the H&H (Hydrology and Hydraulic) Report (Stormwater Management Report). This report should clearly compare the analysis results to the approved design criteria including whether or not the proposed design meets the appropriate criteria.

This report should include the following:

- **Executive Summary** with an overview of the project intent, drainage methodology and stormwater management approach.
- **Project Description** with detailed information of project scope and proposed construction.
- **Hydrological and Hydraulic Criteria** including methodology used for the drainage design of open channels, pavement drainage and culverts.
- **Stormwater Management Approach** with geographical description of project and design criteria for stormwater quality and quantity management.
- **Watershed Descriptions** discussing the various flow characteristics of drainage areas noting existing drainage facilities and assessment of offsite drainage (beyond survey data collection). Each area is given an identification code as shown on the drainage plans.
- **Conclusions** describing the proposed drainage for each area with a summary comparing the pre- and post-project conditions.
- **Special Conditions** requiring any special design measures to meet permit requirements, water quality standards, etc.
- **Backup Data** plans, maps, quadrangles, assumptions, hand calculations, computer output, etc.

The report is normally prepared in two phases. The preliminary report would include an initial hydrologic analysis, a preliminary drainage plan and, most importantly, the proposed design criteria and methodology to be used in performing the detailed drainage analysis. The final report would contain this data along with the final drainage calculations and final drainage plan. Refer to DelDOT's *ES_2M Design Guide* for the required contents of a stormwater management report. Additional guidance and samples are available from the Quality Section.

6.5 DESIGN PROCESS

The following describes the process for performing an acceptable drainage design.

6.5.1 LEGAL REQUIREMENTS AND AGENCY COORDINATION

Highway construction encroaching flood plains, subaqueous lands, wetlands, lakes, ponds, and natural streams shall conform to prevailing governmental regulations. Strict attention shall be given at the planning stage of the project to avoid potential legal problems. Design activities shall be closely coordinated with DelDOT's Environmental Studies Section and other agencies for obtaining necessary permits complying with the regulations and the project schedule.

Sediment and stormwater regulations were enacted into State Law in 1990 and 1991. The designer should be familiar with the provisions and ensure that the design conforms to the requirements.

6.5.2 DATA COLLECTION

6.5.2.1 INITIAL PHASE

The initial phase of the design is to acquire the relevant data and information. This information is available from various sources including federal, state, and county agencies. See Attachment A for resource references.

Site investigations and field surveys are absolutely necessary and should include:

- Watershed characteristics, soil surveys, U.S.G.S. map topography, and land use;
- Stream course data—profile and cross-sections within the right-of-way and in the

vicinity. Note high water marks when possible. Obtain FEMA maps (http://msc.fema.gov), if available, for 100-year flood plain location and elevation;

- Existing drainage facilities with design details, if available, and field data;
- Current aerial photographs;
- Wetlands locations from field survey data and DelDOT's Environmental Studies Section; and
- Any affected resource protection areas from available maps.

When evaluating existing facilities within the project area, the designer should:

- Video and investigate the need for replacing existing drainage facilities due to inadequate strength or functional deficiency;
- Check utilities for possible impact and constraints on the drainage design; and
- Verify the ditch and stream bank elevations.

The designer should plan on removing and replacing any existing damaged pipe. It is also important that all pipes are relatively free of silt and debris in order to properly inspect and ensure the bottom of the pipe is intact.

6.5.3 DRAINAGE PLANS

The drainage plans are prepared in two phases—preliminary and final. The preliminary drainage plans are included in the preliminary construction review package. The final drainage plans are included in the semi-final construction plan review package. Based upon the comments from the two reviews, the final H&H Report is prepared and submitted.

6.5.3.1 PRELIMINARY DRAINAGE PLAN

The preliminary drainage plans include the following information related to drainage:

- Cut areas with beginning and end stations indicating depths of cut;
- Fill areas with beginning and end stations indicating depths of fill;
- Stations and elevations of high points;
- Stations and elevations of low points;
- Limits of relatively flat sections (grades flatter than 0.5%);
- Drainage areas and subareas indicating basin divides from cross sections and topographic sheets;
- Preliminary locations of proposed drainage facilities;
- Limits of steep profiles where erosion protection is anticipated;
- Locations of existing utilities which may impact highway drainage;
- Potential drainage storage and outfall locations; and
- Sediment and stormwater management project review and design checklist items.

6.5.3.2 FINAL DRAINAGE PLAN

The preliminary drainage plan and previously gathered information are reviewed in the field in developing the final drainage plans.

In fill areas the designer should:

- Investigate the need for roadside ditches to carry surface runoff and indicate their approximate locations on the working drawings;
- Estimate the extent of possible erosion qualitatively and indicate where preventive measures may be needed.

In cut areas, the designer should:

- Indicate where benching may be necessary,
- Delineate approximate ditch locations and/or inlets and storm drains; and
- Evaluate potential erosion problems and indicate what measures should be taken to minimize them.

In flat sections, the designer should:

- Indicate where the runoff can be disposed; and

- Evaluate the impact of ditch or storm drain slopes.
- At intersecting roads and superelevation transitions the designer should:
- Establish and verify inlet locations; and
- Note changes in cross slopes, warping and crowning of roadways, and the means of effective interception of surface runoff.
- Field reviews should include gathering information required by the sediment and stormwater management checklist.

In all cases, check the boundaries of drainage areas, describe the topography, determine the ground cover, and verify soil types against the published soil survey maps from the county conservation district, to aid in determining the coefficient of runoff, runoff curve number, and time of concentration.

6.5.4 DRAINAGE REPORT

Drainage design and therefore the report is divided into several topic areas including, hydrology; roadside ditches; open channels; pavement and storm drains; cross road culverts; erosion and sedimentation control; stormwater quantity management; and stormwater quality management.

Each of these topic areas has multiple resource publications, Internet sites, and software for design as well as design aides such as charts, nomographs, tables, and figures. In addition, some topic areas have several acceptable methods for performing the design calculations. In order for a reviewer to check the report, it is necessary that all the resources or applicable portions of each be included or identified.

The analysis performed during design is documented in a comprehensive report of the hydrological characteristics of the drainage areas and hydraulic calculations relating to drainage and stormwater management. The basics of the drainage report are developed after defining federal, state and local requirements, data collection, a preliminary design and a comprehensive field review. A preliminary H&H Report is necessary for producing an acceptable preliminary plan package. Obtaining written approval from the Project Manager of this report and proposed methodology is required.

For the semi-final plan submittal, a final H&H Report is prepared which also includes the final analysis for drainage and stormwater management with verification that the approved criteria have been met or exceeded along with construction detail. The final drainage report describes the drainage characteristics within the project area, the various design criteria, data collected, data sources and references, anticipated computer software for preliminary analysis, a proposed analysis format, etc. This report may need minor revisions after its final review. Refer to DelDOT's *ES_2M Design Guide* for the required contents of a stormwater management report. Additional guidance and samples are available from the Quality Section.

6.6 HYDROLOGY

Hydrology studies the effect of rainfall intensity, duration, and runoff at prescribed frequencies on a watershed. The design of most drainage facilities requires the peak rate of flow at a specified frequency; others, such as drainage systems involving detention storage, depend on runoff hydrographs that estimate the pre- and post-development peak discharge.

6.6.1 REFERENCES

Publications

The following references are needed in order to prepare a hydrological study. Because of the availability of computer software to perform the calculations, updated versions of these publications frequently delete the basic reasoning and understanding of principles used to develop the software. Therefore, some references also include the original publications.

- *Urban Hydrology for Small Watersheds*, Technical Release No. 55 (TR-55), NRCS, Revised 2003

- *Hydraulic Design of Highway Culverts*, Highway Design Series 2 (HDS-2), FHWA, 2002
- *Introduction to Highway Hydraulics*, Hydraulic Design Series No. 4 (HDS-4), FHWA, 1983 and 2001
- *Hydrology*, Hydraulic Engineering Circular 19 (HEC-19), FHWA, 1984
- *Urban Drainage Design Manual*, Hydraulic Engineering Circular 22 (HEC-22), FHWA, 2001
- *Magnitude and Frequency of Floods on Nontidal Streams in Delaware*, U.S. Geological Survey Scientific Investigations Report 2006-5146, (http://pubs.usgs.gov/sir/2006/5146/) - regression equations

Tables and Figures

The following tables and figures are required to perform a hydrological study of a watershed:

- DelDOT's Criteria for Design Frequency
  - Figure 6-1
- Frequency Factor, C_f
  - Section 3.2.2.1, HEC-22
- Intercept Coefficient (k) for Velocity vs. Slope Relationship
  - Figure 6-4
  - Table 3-3, HEC-22
- Delaware's Rainfall Intensity Estimate
  - Figures 6-5 to 6-7
- Runoff Coefficients for Rational Method
  - Figure 6-8
  - Table 5.7, HDS-2
  - Appendix B, Tables 11 and 12, HDS-4
  - Table 3-1, HEC-22
- Nomograph for Determining Velocities Using the Upland Method of Estimating T_{t1} and T_{t2}
  - Figures 37 and 52, HEC-19
  - Figure 3-1, TR-55
- Soil Group Descriptions for TR-55 Method
  - Figure 6-9
  - Appendix A, TR-55
- Hydrologic Soil Groups for Delaware
  - Figure 6-10
  - Exhibit A, TR-55
- Delaware's 24-Hour Rainfall Depths
  - Figure 6-11
- Runoff Curve Numbers
  - Table 5.4, HDS-2
  - Table 2-2a thru d, TR-55
- Manning's Roughness Coefficients (n)
  - Table 2.1, HDS-2
  - Table 3-4, HEC-22
  - Table 3-1, TR-55
- Unit Peak Discharge (q_u) for NRCS Type II Rainfall Distribution
  - Exhibit 4-II, TR-55
- Adjustment Factor (F_p) for Pond & Swamp Areas Spread Throughout the Watershed
  - Table 5.6, HDS-2
  - Table 3-9, HEC-22
  - Table 4-2, TR-55

Computer Software

- TR-20
- TR-55
- WINTR-55
- PondPack
- WMS (Watershed Modeling System)
- HEC-HMS

6.6.2 DESIGN FREQUENCY

Since it is not economically feasible to design drainage facilities for the maximum potential runoff, a limit on the runoff for the respective design frequency must be specified. The frequency at which the corresponding flood can be expected to occur is the reciprocal of the probability that the expected flood will be equaled or exceeded in a given year. The frequency period is also known as the recurrence interval or return period. The risk of flooding and economy in construction are the two factors that govern the design storm frequency to be used for designing any drainage facility. See Figure 6-1 for design frequency criteria.

6.6.3 PEAK DISCHARGE

Peak discharge is the maximum rate of flow that results from a storm corresponding to the selected design frequency for a particular drainage location being analyzed or designed. There are numerous methods available for determining peak discharge. For roadway drainage DelDOT uses the Rational Method for homogenous drainage areas such as pavement and stormwater drain system design. This method is most accurate for watersheds smaller than 50 acres, but can be used for watersheds up to 200 acres. The NRCS TR-55 Method is used for complex watersheds up to 300 acres having several subareas that may or may not be homogenous. This method also can be used for stormwater management analysis such as detention, retention and control structures. Areas larger than this normally require drainage facilities that fall within the responsibility of the Bridge Design section. Both of these methods are briefly discussed in following sections. Example 1, Attachment B, illustrates the use of the rational method; Example 2 uses TR-55.

6.6.3.1 THE RATIONAL METHOD

The rational method employs an empirical equation relating runoff to rainfall intensity. It estimates the peak rate of runoff using: the drainage area; a weighted runoff coefficient that considers existing and proposed land use and types of soils and surfaces; and the rainfall intensity.

The Rational Method's equation is expressed as:

$$Q = C_f CIA \qquad (6.1)$$

where:

Q = Peak flow (ft^3/s)

C_f = Dimensionless frequency factor for design frequencies greater than 10-yr

C = Dimensionless runoff coefficient, a function of the watershed's ground cover

I = Design storm rainfall intensity (in/hr)

A = Drainage area (ac)

This equation is most reliable for surfaces that are smooth, uniform and impervious such as pavements. The equation's reliability is subject to three basic assumptions: (1) the peak runoff at any point is a direct function of the average rainfall intensity for the time of concentration to that point; (2) the recurrence interval of the peak discharge is the same as the recurrence interval of the average rainfall intensity; and (3) the time of concentration is the time required for the runoff to become established and flow from the most distant point of the drainage area to the point of discharge.

For a drainage area consisting of different subareas with varying soil characteristics and cover, different runoff coefficients are assigned and a weighted average is developed using the equation:

$$C = \frac{\sum C_x A_x}{A_{total}} \qquad (6.2)$$

For more detailed information about the rational formula refer to HDS-2 and HEC-22.

PROCEDURE

Step 1. Obtain the following information for each design segment:

a. Drainage area;

b. Land use (percentage of impermeable area such as pavement, sidewalks, roofs, etc.);

c. Soil types (highly impermeable or impermeable soils);

d. The hydraulically most distant point of the drainage area to the point of discharge with significant contribution;

e. Difference in elevation from the farthest point of the drainage area to the point of discharge.

Step 2. Determine time of concentration (t_c)

The time of concentration (t_c) is the maximum time required for the runoff to flow from the most remote point in a drainage area or subarea to the proposed point of discharge. Among a number of alternative paths that runoff could take from far distant points of a drainage area, the time of concentration represents the longest possible travel time (which is not necessarily the longest distance). The path of runoff that governs the time of concentration is the longest hydraulic length of the drainage area. It may be necessary to check several watercourses to determine the longest flow path with significant contribution.

The type and length of flow vary over the hydraulic length. The characteristics of these flow types divide them into three flow patterns: overland (sheet) flow (T_{t1}), shallow concentrated flow (T_{t2}), and open channel flow (T_{t3}). The time of concentration is the summation of the travel times for each type of flow over each of their hydraulic lengths. The minimum value of t_c is 6 minutes (0.1 hr) and the maximum is 600 minutes (10 hr).

The factors affecting the time of concentration and the time of flow are:

1. **Surface roughness:** Development in the watershed changes the flow velocity retardance from very slow shallow overland flow through vegetation, redirecting the flow to impervious areas such streets and gutters, to storm drains that transport the runoff more rapidly. Therefore, the travel time is usually decreased.
2. **Channel shape and flow pattern:** Travel time in small watersheds is influenced by the resulting upstream overland flow. In developing or developed areas, the runoff is directed to defined channels as soon as possible. This results in increased flow velocity and decreased travel time.
3. **Slope:** Drainage designs have a variety of site grading solutions to manage water flow. If there is extensive use of swales, ditching or storm drains, the slopes within the watershed can be modified dramatically. Generally, swales and ditches increase slopes, and storm drains decrease slopes.

Overland (Sheet Flow) takes place on plane surfaces, nonconverging irregular surfaces, usually at the uppermost areas of watersheds, and is less than 1.5 inches deep. The hydraulic length for sheet flow is normally assumed to be a maximum 100 feet for unpaved surfaces and 150 feet for paved surfaces. The equation for sheet flow is a version of the kinematic wave equation and is:

$$T_{t1} = \frac{0.93}{I^{0.4}}\left(\frac{nL_1}{\sqrt{s}}\right)^{0.6} \qquad (6.3)$$

where:

T_{t1} = Travel time (min)

n = Roughness coefficient from Table 3-1, TR-55 or Table 3-2, HEC-22

L_1 = Length of flow, (ft)

I = Average rainfall excess intensity for a storm duration = t_c (in/hr)

s = Surface slope (ft/ft)

I depends on t_c and is initially assumed from Figures 6-5, 6-6 or 6-7. t_c is calculated from equation 6.4 and checked against the initial assumed value. This is repeated until the two successive values are the same.

NRCS (TR-20), has also developed a method for determining T_{t1} for overland flow based on a 24-hr Type II rainfall distribution, eliminating the need for trial and error calculations until T_t equals storm duration. The equation is:

$$T_{t1} = \frac{0.42(nL_1)^{0.8}}{{P_{24}}^{0.5}s^{0.4}} \qquad (6.4)$$

where:

T_{t1} = Travel time (min)

n = Roughness coefficient from Table 3-1, TR-55, Table 3-2, HEC-22 or Table 2-1, HDS-2.

L_1 = Length of flow, (ft)

P_{24} = 2-yr Type II 24-hr rainfall (in)

s = Surface slope (ft/ft

The **Upland (Velocity) Method** is used to determine the travel time for shallow concentrated and open channel flow segments by dividing their respective hydraulic lengths by each segment's average flow velocity:

$$T_t = \frac{L}{60V} \qquad (6.5)$$

where:

T_t = Segment travel time (min)

L = Segment length (ft)

V = Segment velocity (ft/s)

Each segment's velocity depends upon the watercourse's average slope and surface roughness.

Shallow Concentrated Flow takes over from sheet (overland) flow as very shallow channels or gutters along the hydraulic length, 1.5 to 6.0 inches in depth. Limit sheet flow to 100 feet for unpaved areas and 150 feet for paved areas. The equation for determining the average velocity for shallow concentrated flow is:

$$V = 32.8ks^{0.5} \qquad (6.6)$$

where:

V = Average velocity (ft/s)

k = velocity intercept coefficient based on slope and surface type, Figure 6-4

s = average slope of the watercourse (ft/ft)

V can be obtained graphically from Figures 37 or 52, HEC-19; Figure 4b, HDS-4; or Figure 3-1, TR-55. The travel time (T_{t2}) is determined by using equation 6.5.

Figure 6-4
Intercept Coefficient (k) for Velocity vs. Slope Relationship for Rational Method

Land Cover and (Type of Flow)	k
Forest with heavy ground litter; hay meadow (overland flow)	0.076
Trash fallow or minimum tillage cultivation, contour or strip cropped; woodland (overland flow)	0.152
Short grass pasture (overland flow)	0.213
Cultivated straight row (overland flow)	0.274
Nearly bare and untilled (overland flow)	0.305
Grassed waterway (shallow concentrated flow)	0.457
Unpaved (shallow concentrated flow)	0.491
Paved area (shallow concentrated flow); small upland gullies	0.619

Open Channel Flow includes natural streams, ditches, pipes and gutters in the lower stretch of the watershed along the hydraulic length to the point of termination at an outfall. Determining when channel flow begins is based on several indicators, including survey data defining the channel, visible signs on aerial photography of the watershed, on area topographic maps, and USGS quadrangle sheets.

Determine the velocities by using Manning's equation:

$$V = \frac{1.49(R)^{2/3}(s)^{1/2}}{n} \qquad (6.7)$$

where:

V = Average velocity (ft/s)

R = Hydraulic radius (ft) = A/P_w (6.7a)

A = Cross sectional flow area (ft^2)

P_w = Wetted perimeter (ft)

s = Energy grade line (channel or pipe flow line) (ft/ft), and

n = Manning's roughness coefficient for open channel flow

The travel time for open channel flow, T_{t3}, is obtained from equation 6.5. In developed areas there may be a variety of open channel facilities including storm drains, swales, ditches, gutters. The travel time for each one of these is computed by using Manning's formula and then combined to provide T_{t3} for the complete open channel flow in the watershed.

The time of concentration, t_c, is equal to the sum of the travel times derived for all three types of flow:

$$t_c = T_{t1} + T_{t2} + T_{t3} \qquad (6.8)$$

Step 3. Determine the rainfall intensity (I)

Rainfall intensity is the amount of rainfall in inches per hour based on a duration that equals the time of concentration and the design storm frequency. The rainfall intensity can be obtained from Figures 6-5 to 6-7 provided that the time of concentration (duration) is known.

Step 4. Select the appropriate runoff coefficient (C)

The rational method uses a runoff coefficient that is representative of the drainage area being studied. This coefficient averages the area ambient moisture content, infiltration/ evaporation potential, average slope, existing and future ground cover, and developed surfaces.

The drainage area is divided into its various pre- and post-developed subareas. Using Figure 6-8, a runoff coefficient is assigned and a weighted coefficient is calculated. Higher values of C are used for steeply sloped areas and longer return periods because infiltration and other losses have a reduced effect on the runoff.

Step 5. Compute the design peak flow (Q)

Determine the peak discharge using the Rational Method equation, $Q = C_f CIA$.

Figure 6-5
Rainfall Intensity Estimates (in/hr) for Rational Method - New Castle County

Frequency (yr)	Duration (min)									
	5	10	15	30	60 (1 hr)	120 (2 hr)	180 (3 hr)	360 (6 hr)	720 (12 hr)	1440 (24 hr)
2	4.97	3.97	3.33	2.30	1.44	0.87	0.62	0.38	0.23	0.13
5	5.83	4.67	3.94	2.80	1.79	1.08	0.78	0.48	0.29	0.17
10	6.42	5.13	4.33	3.13	2.04	1.24	0.89	0.55	0.34	0.20
25	7.13	5.68	4.80	3.55	2.37	1.45	1.05	0.66	0.41	0.25
50	7.60	6.05	5.10	3.84	2.60	1.61	1.18	0.74	0.47	0.29
100	8.06	6.40	5.40	4.13	2.85	1.77	1.30	0.83	0.53	0.34
200	8.44	6.69	5.63	4.38	3.07	1.93	1.43	0.92	0.60	0.39
500	8.88	7.02	5.89	4.69	3.36	2.14	1.60	1.05	0.70	0.46

Figure 6-6
Rainfall Intensity Estimates (in/hr) for Rational Method - Kent County

Frequency (yr)	Duration (min)									
	5	10	15	30	60 (1 hr)	120 (2 hr)	180 (3 hr)	360 (6 hr)	720 (12 hr)	1440 (24 hr)
2	5.06	4.05	3.40	2.34	1.47	0.90	0.65	0.40	0.24	0.14
5	6.01	4.81	4.06	2.88	1.85	1.13	0.82	0.50	0.30	0.18
10	6.68	5.35	4.51	3.27	2.13	1.31	0.95	0.59	0.36	0.21
25	7.54	6.01	5.08	3.76	2.50	1.56	1.14	0.71	0.44	0.27
50	8.15	6.49	5.48	4.13	2.79	1.76	1.29	0.81	0.51	0.32
100	8.76	6.96	5.86	4.49	3.09	1.96	1.45	0.92	0.59	0.37
200	9.92	7.39	6.22	4.84	3.39	2.17	1.62	1.04	0.67	0.43
500	10.02	7.93	6.65	5.29	3.80	2.45	1.85	1.21	0.80	0.52

Figure 6-7
Rainfall Intensity Estimates (in/hr) for Rational Method - Sussex County

Frequency (yr)	Duration (min)									
	5	10	15	30	60 (1 hr)	120 (2 hr)	180 (3 hr)	360 (6 hr)	720 (12 hr)	1440 (24 hr)
2	5.06	4.04	3.39	2.34	1.47	0.91	0.66	0.40	0.24	0.14
5	6.02	4.83	4.07	2.89	1.85	1.16	0.84	0.52	0.30	0.19
10	6.76	5.40	4.56	3.30	2.15	1.35	0.99	0.61	0.36	0.22
25	7.67	6.11	5.15	3.82	2.54	1.61	1.19	0.74	0.45	0.28
50	8.32	6.62	5.59	4.21	2.85	1.83	1.35	0.85	0.52	0.33
100	8.96	7.12	6.00	4.59	3.16	2.05	1.53	0.97	0.61	0.38
200	9.60	7.61	6.40	4.98	3.49	2.28	1.71	1.10	0.70	0.45
500	10.38	8.21	6.88	5.48	3.93	2.59	1.97	1.28	0.84	0.54

Note: Interpolation shall be used for rainfall for intermediate durations**.**

Figure 6-8
Recommended Runoff Coefficients (C) for Rational Method

Type of Drainage Area Surface			Runoff Coefficient, C
Earth Surfaces	Sand, from uniform grain size with no fines to well graded with clay or silt	Bare	0.15
		Light Vegetation	0.10
		Heavy Vegetation	0.05
	Loam, from sandy or gravelly to clayey	Bare	0.20
		Light Vegetation	0.10
		Dense Vegetation	0.05
	Gravel, from clean gravel & gravel sand mixtures with no silt or clay to high clay or silt contents	Bare	0.25
		Light Vegetation	0.15
		Dense Vegetation	0.10
	Clay, from coarse sandy to pure colloidal clays	Bare	0.40
		Light Vegetation	0.30
		Dense Vegetation	0.25
Lawns	Sandy soil	Slope, Flat to 2%	0.05
		Average Slope 2% to 7%	0.10
		Steep Slope Over 7%	0.15
	Clayey soil	Slope, Flat to 2%	0.13
		Average Slope 2% to 7%	0.18
		Steep Slope Over 7%	0.25
Pavements	Asphalt		0.95
	Concrete		0.95
	Compacted graded aggregate or gravel		0.70
Railroad yard areas			0.30
Parks, golf courses and cemeteries			0.15
Playgrounds			0.25
Unimproved Areas			0.15
Cultivated areas			0.30
Swamp or marsh areas			0.08
Roofs			0.90
Drives and walkways			0.90
City business areas			0.85
City with dense residential areas and varying soil and vegetation conditions			0.70
Residential areas, single family units			0.40
Residential areas, duplexes and twins			0.50
Residential areas, multi-units			0.70
Suburban residential areas (lots 1/2 ac or more)			0.35
Apartment complexes			0.60
Light industrial areas			0.70
Heavy industrial areas			0.80

6.6.3.2 THE NRCS METHOD

NRCS Technical Release 55 (TR-55, 1986; WINTR-55, 2003), *Urban Hydrology for Small Watersheds* is one of the methods commonly used to determine a watershed's peak discharge. TR-55 is a computer application based on data and routines developed using TR-20. This method is particularly useful for determining pre- and post-development runoff rates and the subsequent design of any required stormwater control structures.

The two most commonly used design applications for roadway drainage in TR-55 are: (1) the Graphical Peak Discharge method and (2) the Tabular Hydrograph method. The Graphical method is limited to a single homogenous watershed where land use, soil, and cover are uniformly distributed throughout the watershed. The Tabular method uses hydrographs for analyzing a larger more complex watershed having: multiple homogenous subareas with several converging reaches; changing land uses within any of the existing homogenous watershed; and storage and control structures. Worksheets for each method are provided in TR-55.

The two methods use the same approach to determine the basic design data necessary to for calculating the watershed('s) peak discharge: the runoff depth, time of concentration and travel time within the watershed(s) and an initial abstraction factor (I_a) based on the CN to determine the unit peak discharge per square mile per inch of runoff. Equations convert the runoff depth, the unit peak discharge, the drainage area, and a storage adjustment factor to calculate the peak watershed discharge or generate a pre- and post-hydrograph of the peak discharge.

The study begins by defining the watershed and dividing it into homogenous subareas using existing and proposed land uses. Land use determines the percentage of pervious and impervious surfaces. The drainage runoff characteristics are also based upon the hydrological soil classification. A runoff curve number (CN) is assigned to these subareas. CN represents the hydrologic characteristics of the drainage area including the average slope, soil group type, plant cover, amount of impervious surfaces, interception rates, and surface storage. Using the runoff curve number and the NRCS 24-hour rainfall data, the design storm(s) runoff depth for the watershed(s) is determined as follows:

Runoff Depth (q_d)

The runoff depth equation is:

$$q_d = \frac{(P_{24} - 0.2S))^2}{P_{24} + 0.8S} \qquad (6.9)$$

where

q_d = Runoff depth for a specified return period (in)

P_{24} = 24-hour rainfall using Figure 6-11 for the corresponding return period (in)

S = Potential maximum retention after runoff begins, inches

S is related to the soil and cover conditions through the runoff curve number for the various subareas and defined by the equation:

$$S = \frac{1000}{CN} - 10 \qquad (6.10)$$

where

CN = Weighted average runoff curve number using a ratio of assigned subarea CN's to their area as related to the entire watershed area. CN must 40 or greater.

The hydrologic soil group(s) in each subarea is based on the soil classification (denoted as A, B, C and D) based on their rainfall infiltration rates. A represents soils with the maximum rate of rainfall infiltration and D the soils with the lowest rate of infiltration.

Refer to Table 2-2a-d, TR-55, for the runoff curve numbers for urban and rural areas. These tables were developed assuming an Initial abstraction (I_a) of 20% of the potential maximum retention after runoff begins (S) in the drainage area. I_a represents losses prior to runoff and includes initial infiltration storage in surface depressions, evapotranspiration, leaf and vegetation retention, and other factors. Cover types represented by vegetation, bare soil, and impervious surfaces are usually determined from field reconnaissance, aerial photographs, land use maps, and soil survey maps. Hydrologic condition indicates infiltration and runoff; it is estimated from the density of plant and residue cover in rural areas.

A soil survey for each county is available from the United States Department of Agriculture. These publications contain aerial photographic maps of Delaware with detailed soils information. Figure 6-10 lists the hydrologic soil groupings commonly found in Delaware. Soil survey maps for Kent and Sussex counties are available online at the web site http://www.udel.edu/FREC/spatlab for the University of Delaware Spatial Analysis Lab.

After determining CN and the 24-hour rainfall for the watershed, the runoff depth (q_d) is calculated using Worksheet 2, figure 2-1 and table 2-1 in TR-55 or equations 6.9 and 6.10.

Next, the travel time for each type of flow and the ultimate time of concentration to the point of interest must be determined.

Time of Concentration and Travel Time (t_c)

The flow times for the three types of flow within each watershed (as described in Section 6.4) are calculated as follows.

Sheet (Overland) Flow

The travel time when using TR-55 is obtained from the modified Manning's-kinematic solution and is determined in hours, modified from equation 6.4:

$$T_{t1} = \frac{0.007(nL_1)^{0.8}}{P_2^{\;0.5} s^{0.4}} \qquad (6.11)$$

where

T_{t1} = Travel time for overland flow (hr)

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

n = Manning’s roughness coefficient for sheet flow (Table 3-1, TR-55)

L_1 = Portion of hydraulic length on which sheet flow takes place (ft)

s= Average slope of the hydraulic length L_1 (ft/ft)

The maximum for L_1 is 100 feet for unpaved surfaces and 150 feet for paved surfaces.

Shallow Concentrated Flow

For shallow concentrated flow, the time of concentration (T_{t2}) is computed using equation 6.5. V_2 is obtained from Table 3-1, TR-55, or the equations:

$$V = 16.10(s)^{0.5} \text{ Unpaved surfaces } (6.12)$$

$$V = 14.99(s)^{0.5} \text{ Grassed waterway } (6.12a)$$

$$V = 20.30(s)^{0.5} \text{ Paved surfaces } (6.13)$$

where

V = Average velocity (ft/s), and

s = Slope of the hydraulic gradeline (watercourse slope) (ft/ft)

Open Channel Flow

The travel time for open channel flow (T_{t3}) is determined by using Manning’s equation, and equations 6.3 and 6.5 When using equation 6.5 use 3600 instead of 60 to calculate hours instead of minutes.

The time of concentration for the drainage basin (t_c) is the sum of the travel time for each flow segment. The minimum value for this method is 6 minutes (0.1 hr) and a maximum of 10 hours. See Worksheet 3, TR-55, for the procedure.

At this point the designer must decide on the scope of the watershed analysis. If the watershed is hydrologically homogenous with land use, soils, and cover uniformly distributed with one main watercourse, then the Graphical Peak Discharge Method can be used. If the watershed has nonhomogeneous areas that can be divided into homogenous areas, land use changes are proposed for a portion of the watershed and stormwater management facilities requiring hydrographs are necessary, then the Tabular Hydrograph Method is used.

There are other regulatory controls that influence the selected method. State sediment and stormwater policy is that all hydrologic computations must use the NRCS Type II-24 hour rainfall event and for all projects south of the Chesapeake and Delaware Canal, the Delmarva Unit Hydrograph will be incorporated into the design process.

GRAPHICAL PEAK DISCHARGE METHOD

The peak discharge at the site under consideration is obtained from the equation:

$$Q_i = q_u\, A_m\, q_d\, F_p \tag{6.14}$$

where,

- Q_i = Peak discharge for the *i*'th year frequency (ft^3/s)
- q_u = Unit peak discharge (ft^3/s /sq mi/in)
- A_m = Drainage area (sq mi)
- q_d = Runoff depth (in) (equation 6.9)
- F_p = Pond and adjustment factor (Table 4-2, TR-55)

The only unknown variable in this equation is q_u. The steps to find the value of q_u are:

1. Using the weighted CN determine the initial abstraction (I_a) from Table 4-1, TR-55.
2. For the design frequency, select the 24-hour rainfall (P) from Figure 6-11.
3. Calculate the ratio of I_a/P.
4. Peak discharge per square mile per inch of runoff (q_u) is obtained from exhibit 4-II for Type II Rainfall Distribution, the previously defined t_c and the I_a/P ratio.

Worksheet 4, Appendix D, TR-55, provides an easy guide for calculating Q_i. Also, the equations used to generate the figures and tables in TR-55 can be found in Appendix F, TR-55.

TABULAR HYDROGRAPH METHOD

First, use worksheet 5a to develop a summary of basic watershed data by subarea. The total t_c for each subarea is the total travel time for the subarea reach. Find the hydrograph coordinates for selected t_c's using exhibit 5. The flow time is found using the equation:

$$q = q_t A_m Q \tag{6.15}$$

where:

- q = Hydrograph coordinate (cfs) at hydrograph time (t)
- q_t = Tabular hydrograph unit discharge from exhibit 5 (csm/in)
- A_m = Individual subarea drainage area (mi^2)
- Q = Runoff (in)

The data needed to model the present and future conditions of the watershed using hydrographs for each subarea and routing these hydrographs to the watershed control structures and outfall are:

1. Drainage area for each homogenous subarea. (mi^2)
2. Time of concentration for each subarea in hours. See the previous discussion and Chapter 3, Worksheet 3, TR-55 for the procedure.
3. Time of travel for each routing reach in hours. See the previous discussion and Chapter 3, Worksheet 3, TR-55 for the procedure
4. The weighted CN for each subarea. See Table 2-2 and Worksheet 2, TR-55 for the procedure.
5. 24-hour rainfall for the design frequency using the Delmarva Unit Hydrograph
6. Runoff, in inches, for each subarea. See the previous discussion and Chapter 2, Worksheet 2, TR-55 for the procedure.
7. Initial abstraction (I_a) from Table 5-1, TR-55 .

8. Calculate the ratio of I_a/P for each subarea.
9. Develop a composite hydrograph by summing prerouted individual subarea hydrographs using Worksheet 5b, TR-55.

Chapter 5 in TR-55 describes the procedure for developing a watershed's composite flood hydrograph with the limitations that may apply.

Figure 6-9
Soil Group Descriptions for TR-55 Method

Soil Group	Hydrologic Description	Soil Type
A	High infiltration and low runoff, very well drained sands or gravels	Sand, loamy sand or sandy loam
B	Moderately well drained soils with fine to moderately coarse textures	Silt loam or loam
C	Low infiltration rate, moderately fine to fine texture	Sandy clay loam
D	Very low infiltration rate, high runoff potential, predominately clayey soil	Clay loam, silty clay loam, sandy clay, silty clay or clay

Figure 6-10
Hydrologic Soils Descriptions for TR-55 Method

Aldino	C	Elsinboro	B	Keyport	C	Pepperbox	B
Assawoman	D	Evesboro	A	Kinkora	D	Plummer	D
				Klej	B	Pocomoke	D
Bayboro	D	Fort Mott	A			Portsmouth	D
Berryland	D	Fallsington	D	Mulica	D	Pone	D
Butlertown	C			Manor	B		
		Greenwich	B	Metapeake	B	Rumford	B
Calvert	D	Gleneig	B	Matawan	C	Rutlege	D
Chester	B			Mattapex	C	Rosedale	A
Codorus	C	Hammonton	C	Montalto	C		
Collington	B	Hatboro	D			Sassafras	B
Comus	B	Hurlock	D	Nanticoke	D		
				Neshaminy	B	Talleyville	B
Delanco	C	Johnston	D				
				Osier	D	Watchung	D
Elioak	C	Kalmia	B	Othello	D	Woodstown	C
Elkton	D	Kenansville	A				

Other common hydrologic soils

Borrow Pits	Extremely variable	Ingleside	A
Coastal Beaches	A	Swamp	D
Made land	C	Broadkill, Westbrook	D
Mixed alluvial	C-D	Urban Land	C-D
Mixed silts and clay	C		

Figure 6-11
24-Hour Rainfall Depths for Delaware for TR-55 Graphical Method

NRCS Type II, 24-Hour Duration			
Storm Event	**County**		
	New Castle	**Kent**	**Sussex**
1-yr	2.7	2.7	2.8
2-yr	3.2	3.3	3.4
5-yr	4.1	4.3	4.4
10-yr	4.8	5.2	5.3
25-yr	6.0	6.5	6.7
50-yr	6.9	7.6	7.9
100-yr	8.0	8.9	9.2
500-yr	10.9	12.6	13.0

6.7 OPEN CHANNEL FLOW

Open channels are the most commonly used component of a drainage system. They may occur naturally within the drainage basin or be artificially introduced in the design process. Open channels include natural/manmade ditches, streams, median swales and gutters. They may be unlined or lined with artificial or natural materials to protect against erosion.

In drainage design, the term “open channel” is frequently used to describe the design and flow characteristics of all drainage systems (ditches, storm drains or culverts) that have their flow controlled by atmospheric pressure and gravity. The primary design control is the difference in elevation between the various open channel sections. However, the open channel’s shape, surface type, coefficient of friction, and alignment are considered in determining its flow.

In open channel design, the final geometric parameters are selected to ensure that the flow does not exceed critical depth or critical velocity. This is an iterative process to reach the appropriate design.

Open channels are used:

- Longitudinally as intercepting channels at the top of a cut section;
- In cut sections of roadways to remove stormwater;
- In medians to covey roadway runoff to inlets:
- At the bottom and toe of embankment slopes to convey stormwater to a discharge point.

6.7.1 REFERENCES

Publications:

The primary references for understanding the principles and analysis of open channel flow are:

- *Design Charts For Open-Channel Flow*, Hydraulic Design Series No. 3, (HDS-3) FHWA, 1973
- *Introduction to Highway Hydr*aulics, Hydraulic Design Series No. 4, (HDS-4) FHWA, 1983
- *Design of Riprap Revetment*, Hydraulic Engineering Circular 11 (HEC-11), FHWA, 1989
- *Hydraulic Design of Energy Dissipaters for Culverts and Channels*, Hydraulic

Engineering Circular 14 (HEC-14), FHWA, 1975

- *Design of Roadside Channels with Flexible Lining*, Hydraulic Engineering Circular 15 (HEC-15), FHWA, 1986 and 2005
- *Urban Drainage Design Manual*, Hydraulic Engineering Circular 22 (HEC-22), FHWA, 2001
- *Open-Channel Hydraulics*, Ven Te Chow, 1959

Tables and Charts:

Access to the following tables and charts is needed for analyzing open channel flow:

- Open Channel Flow Schematic
  - Figures 6 and 7, HDS-4
- Manning's Roughness Coefficients *n* for Open Channels
  - Figure 6-16
  - Table 1, HDS-3
  - Tables 12 and 14, HDS-4
  - Tables 2.1 and 2.2, HEC-15
- Channel Geometry Equations
  - Appendix B, HEC-15
  - Table 2-1, Chow
- Nomograph for Solution of Manning's Equation
  - Chart 83, HDS-3
  - Appendix C, Chow
- Solution of Manning's Equation for Channels of Various Side Slopes
  - Chart 16, HEC-12
- Capacity of Trapezoidal Channels
  - HDS-3 and HDS-4
- Geometric Design Chart for Trapezoidal Channels
  - HDS-3
  - Appendix B, Chow
- Uniform Flow in Trapezoidal Channels by Manning's Equation
  - Table B.1, HEC-14
- Open Channel Design Procedure
  - Figure 6-12
- Maximum Non-Scouring Ditch Grades with Grass Lining
- Permissible Velocities for Channels Lined with Grass
  - Figure 6-14
- Maximum Allowable Velocities for Open Channels
  - Figure 6-15
- Channel Lining Design Procedure
  - Figure 6-17
- Channel Lining Design Computation Chart
  - Figure 6-18
- DelDOT's Roadside Ditch Design Form
  - Example 4, Attachment B
- K_b Factor for Maximum Stress on Channel Bends
  - Chart 21, HEC-22
- Stability Factor Selection Criteria
  Chart 3, HEC-11
- Correction Factor for Riprap Size
  - Chart 2, HEC-11
- Riprap Size Relationship
  - Chart 1, HEC-11

Computer Software

- HYDRAIN
- HY7 WSPRO
- HEC-RAS

6.7.2 OVERVIEW

Most roadside drainage design is assumed to be steady, uniform flow. The designer assumes that the channel cross section, slope, alignment and surface roughness are reasonably constant over a sufficient length. For steady, uniform flow in open channels, Manning's equation (Equation 6.7) is used to determine the velocity.

The capacity of the channel is determined by:

$$Q = VA \qquad (6.16)$$

Combining the two equations results in:

$$Q = A\frac{1.49}{n}R^{2/3}s^{1/2} \qquad (6.17)$$

In the above equations:

V = Mean velocity (ft/s)

n = Manning's roughness coefficient

R = Hydraulic Radius (ft), the ratio of flow area of cross section (A) to the wetted perimeter (P), A/P_w

s = Slope of the energy grade line (ft/ft). Usually it is parallel to the water surface or the profile of the channel bottom.

Q = Rate of flow (ft^3/s)

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

Manning's roughness coefficient n may vary with the depth of flow. A range of n values for different types of open channels is provided in the referenced resources.

To simplify solving the equations a conveyance factor (K) can be developed in a tabular or curve form for the more commonly used channel cross sections based on the following equation:

$$K = \frac{1.49}{n} AR^{2/3} \qquad (6.18)$$

As a result, substituting K into Manning's equation the rate of flow, Q, would be:

$$Q = Ks^{1/2} \qquad (6.19)$$

6.7.3 ROADSIDE DITCHES

Roadside ditches are trapezoidal or V-shaped open channels that usually parallel the roadway and are lined with grass or non-erodible materials required for erosion protection. Even in curb sections the designer may need to include a roadside ditch in the drainage design. Median ditches and other small-excavated channels are designed using the same principles as for roadside ditches.

Roadside ditches provide an opportunity to reduce runoff pollutants with infiltration through permeable (vegetative) surfaces that allow the pollutants to be absorbed into the underlying soils and reduce the peak flow velocity. The swales and ditches include stone check dams, vegetated beds, and banks with shallow longitudinal slopes and relatively shallow side slopes. One of the controlling factors is the design flow.

Roadside ditches serve several purposes:

- To collect runoff from the highway and adjacent areas which may sometimes extend outside the right-of-way, and dispose of the accumulated runoff at suitable outlet locations,
- To drain the pavement structure and the top layer of supporting subgrade to prevent saturation and loss of support for the pavement, and to protect it from damage from freeze-thaw cycles, and
- To control the quantity and quality of roadway runoff while minimizing any impact to adjacent properties. Draining runoff onto adjacent properties should only be considered when absolutely necessary; in this case, temporary and/or permanent easements may be needed.

6.7.3.1 DESIGN CRITERIA

In additional to any controls set in other chapters of this manual, the following criteria is to be used in the design of roadside ditches:

1. The rational method is used to compute the design runoff, and Manning's equation for capacity. Figure 6-1 has design frequencies.
2. Roadside ditches shall conform to clear zone criteria and legal requirements. Safety of motorists, minimum maintenance requirements, aesthetic values, and avoiding potential damage to abutting properties as well as adverse environmental impacts are essential considerations in design. Refer to AASHTO's *Roadside Design Guide*.
3. Maintain a minimum freeboard of 0.5 ft minimum, 1 ft preferred, below the edge of shoulder.
4. For adequate subbase and pavement drainage, the bottom of the roadside ditch should be at least 2.5 feet below the shoulder edge of the traveled way.
5. Side slopes for grass-lined ditches shall be 3:1 or flatter to facilitate mowing operations. The designer should review the current roadside vegetation to ensure the proper selection of a roughness coefficient.

6. At the top of cut slopes, an intercepting ditch should be investigated. Every effort shall be made to prevent sediment and debris from filling in the ditch.
7. All roadside ditches are to be lined with grass or special materials for scour protection. The ditch linings are to be designed to withstand potential erosion.
8. The minimum ditch grade is 0.3%, but 0.5% is preferred.
9. Gradual transitions should be used for cross section changes. Normally, transition rates are 25 ft for every foot change in the ditch bottom width and 100 ft for increasing or decreasing the value of *z* (horizontal component of the side slope) by 1.
10. After completing the design, the designer should ensure that the proposed right-of-way and easements are adequate to construct the selected ditch sections.

6.7.3.2 DESIGN PROCEDURE

Since the channel's *n* value changes with time, typically (but not always) becoming higher as vegetation is established, the design section shall be checked for velocity constraints based on the lowest *n* value for the newly constructed channel that will yield the highest velocity. The channel shall also be checked with a higher *n* factor for the mature growth stage to establish the minimum capacity of the channel. The vegetative retardance factor is directly related to the selected *n* value. Curves for these relationships can be found in Chow's *Open Channel Hydraulics*, HEC-15, and the NRCS *Engineering Field Handbook.*

The design procedure consists of three phases:

1. Design the ditch for stability. Determine the dimensions using a low retardance factor, since the permanent stand of grass has not been established.
2. Design for the maximum capacity by determining the increase in depth of flow necessary to provide the maximum capacity under the assumed final ditch lining conditions.
3. After checking these two conditions and deciding on the final dimensions, the design criteria freeboard is added.

Refer to Sections 6.10, 11 and 12 for a discussion on designing roadside ditches as a part of stormwater management. These sections also more fully describe the phased approach to ditch design.

The following design procedure is used for roadside ditches.

1. A preliminary layout of the roadside ditch system with flow patterns shall be drawn on the construction plans along with other drainage facilities (existing and proposed), utilities, right-of-way and other specific details that may influence or restrict the ditch design.
2. In general, the grade of the roadside ditch should closely follow the grade line of the highway. Deep ditches and frequent breaks in grade should be avoided. The distance between break points should preferably be at least 100 ft. Use Figure 6-13 as a guide for limiting the maximum grade of roadside ditches. Working plans showing the ditch layout should also include the ditch profile and all features above and below ground that may be potential obstructions to the proposed ditch.
3. Highway cross sections, usually at 50-ft intervals, showing the natural ground and the proposed highway section including all roadside ditches used in establishing the construction limits within the proposed right-of-way.
4. Schematic diagrams of the ditch plan, including geometric elements of the ditch, topographic features, and basin divides shall be drawn to facilitate the computation of peak discharges at the design frequency.
5. An approved spreadsheet (such as the one in Example 4, Attachment B, should be used for hydraulic computations. Manning's roughness coefficient can be

assumed to be 0.05 for the grass surface of the channel.

6. If the computed velocity is lower than the permissible velocity for soil with grass linings as stipulated in Figure 6-14, then permanent grass seeding and mulching per DelDOT's *Standard Specifications* should suffice.
7. If the computed velocity exceeds the permissible velocity on grass surface, then the designer must increase the ditch cross section and flatten the profile to lower the velocity of flow below the permissible limit or use a special channel lining for permanent erosion control as described in Section 6.7.4.

6.7.4 DITCH EROSION CONTROL

6.7.4.1 OVERVIEW

When the flow velocity of a roadside ditch exceeds the allowable velocity on bare earth of the channel (see Figure 6-15) during construction, then a temporary ditch lining, such as an erosion control blanket, is needed to prevent erosion of the ditch. For surface erosion due to overland flow, the permissible velocities of Figure 6-14 should also apply.

See DelDOT's *ES_2M Design Guide, Standard Construction Details,* and the *Delaware Erosion & Sediment Control Handbook* for the requirements and construction details for erosion and sediment control items to be used during construction.

In preparing an H&H report, the post-construction ditch section is analyzed. Usually most surfaces will have grass or some other type of vegetation. As a result, the permissible velocity increases on all types of soil used in construction. In spite of the growth of vegetation, the permissible velocity may be exceeded in certain cases that would necessitate the use of a permanent lining in lieu of seeding and mulching. This permanent ditch lining, which is capable of withstanding erosion, may be either rigid or flexible. The rigid lining usually consists of cast-in-place concrete paving, fabric formed concrete revetment mattress, bituminous (asphaltic) concrete paving or grouted riprap. For flexible lining, alternatives include riprap with or without geotextile fabric and wire enclosed riprap (gabions).

Rigid linings can sustain high velocities up to 20 ft/s, and should be considered when the channel bed is not expected to undergo appreciable settlement. The cost of construction is a very important factor in the selection of this type of lining. Soil reinforcing mats are a cost-effective alternative to rigid linings.

Section 6.7.4.2 briefly discusses the procedure for design a flexible lining. HEC-11 and HEC-15 contain detailed information on the design, example problems and specifications for flexible lined channels. The following design procedures are based on those publications.

Figure 6-12
Open Channel Design Procedure

Open Channel Design using $Q = A\frac{1.49}{n}R^{2/3}s^{1/2}$			
Step 1	Select channel cross section (typically trapezoidal)	Step 5	Determine required depth[1]
Step 2	Determine slope (normally the centerline profile grade line ≤ 3%)	Step 6	Determine subcritical or supercritical flow[2]
Step 3	Select lining (initial trial assumed to be grass; slopes greater than 3% may need different lining)	Step 7	Calculate design velocities for erodible lining sections-check criteria[3] (reevaluate section by section as necessary)
Step 4	Determine roughness coefficient *n* (use an average value from Figure 6-16)	Step 8	Add minimum free board

[1] Use charts from HDS-3, nomograph solution for Manning's equation, and/or charts for hydraulic elements of channel sections, partial area of flow in a circular channel and depth of flow in a trapezoidal channel (also available on DelDOT's web site).

[2] Avoid supercritical flow by adjusting design parameters.

[3] Use a conservative retardance, i.e., C for channel capacity and D for flow velocity.

Figure 6-13
Maximum Non-Scouring Ditch Grades with Grass Lining[1]

Soil Type	Flow Depth (in)	Grade (%)
Erosive	6	3.0
	12	1.5
	18	0.8
Average	6	6.0
	12	3.0
	18	1.6
Non-Erosive	6	10.0
	12	5.0
	18	3.0

[1] Slopes greater than 5% are not recommended.

Figure 6-14
Permissible Velocities for Open Channels Lined with Grass

Cover Type	Slope Range (%)	Permissible Velocity (ft/s)	
		Erosion-resistant soils	Easily eroded soils
Bermuda grass	0-5 5-10 >10	8 7 6	6 5 4
Buffalo grass, Kentucky bluegrass, Smooth brome and Blue grama	0-5 5-10 >10	7 6 5	5 4 3
Grass mixture	0-5 5-10 (max)	5 4	4 3
Lespedeza sericea, Weeping lovegrass, Ischaemum (Yellow bluestem), Kudzu, Alfalfa and Crabgrass	0-5 (max)	3.5	2.5
Annuals-used for temporary erosion control such as Common Lespedeza or Sudangrass	0-5 (max)	3.5	2.5

Figure 6-15
Maximum Allowable Velocities for Open Channels

Soil type	AASHTO Classification	Maximum velocity (ft/s)	
		Bare Earth	Grass Lining
Fine sand	A-3, Beach Sand	2.0	—
Sandy loam	A-2-4 Non-plastic	2.0	4.0
Silt loam	A-4 Non-plastic	2.5	5.0
Ordinary firm loam	A-7-5 Plastic, silt clay sandy	3.0	5.0
Fine gravel	Granular	4.0	—
Alluvial silt (colloidal)	Plastic topsoil	4.0	6.0
Stiff clay	A-7-6 Clay	4.5	7.0
Graded loam to cobbles	Non-plastic soil and rock	4.5	7.0
Graded silt to cobbles	Plastic soil and rock	5.0	8.0
Coarse gravel	Creek Gravel	6.0	—
Cobbles and shingles	Soft rock	7.0	—
Shale and hardpan	Medium rock	8.0	—
Bedrock	Rock	20.0	—
Paved lining			
Concrete		18.0	
Asphalt		15.0	
Grouted riprap		18.0	

Figure 6-16
Manning's Roughness Coefficients (*n*) for Open Channels

Type of Channel and Surface Description					Range of *n* values	
					Normal	Maximum
Natural Streams						
1. Fairly regular section						
a. Some grass and weeds, little or no brush					0.030	0.035
b. Dense growth of weeds, depth of flow materially greater than weed height					0.035	0.050
c. Some weeds, light brush on banks					0.035	0.050
d. Some weeds, heavy brush on banks					0.050	0.070
e. Some weeds, dense willows on banks					0.060	0.080
f. For trees within channel, with branches submerged at high stage, increase all the above values by					0.010	0.020
2. Irregular sections, with pools, slight channel meander Increase respective values above by					0.010	0.020
3. Mountain streams, with pools, no vegetation in channel, bank usually steep, trees and brush along banks submerged at high flood stage						
a. Bottom gravel, cobbles, and few boulders					0.040	0.050
b. Bottom cobbles with large boulders					0.050	0.070
Excavated of Dredged Channels						
	Normal	Maximum				
1. Earth, straight and uniform			2. Earth, winding and sluggish			
a. Clean, recently completed	0.018	0.020	a. No vegetation		0.025	0.030
b. Clean, after weathering	0.022	0.025	b. Grass, some weeds		0.030	0.033
c. Gravel, uniform section, clean	0.025	0.030	c. Dense weeds or aquatic plants in deep channels		0.035	0.040
d. With short grass, few weeds	0.027	0.033	d. Earth bottom and rubble sides		0.030	0.035
			e. Stony bottom and weedy sides		0.035	0.050
			f. Cobble bottom and clean sides		0.040	0.050
3. Dragline — excavated or dredged	0.028	0.033	4. Rock cuts			
a. No vegetation	0.050	0.060	a. Smooth and uniform		0.035	0.040
b. Light brush on banks			b. Jagged and uniform		0.040	0.050
5. Channels not maintained, weeds and brush uncut						
a. Dense weeds, high as flow depth					0.080	0.120
b. Clean bottom, brush on sides					0.050	0.080
c. Dense brush, high flood stage					0.100	0.140
Lined Open Channels						
1. Temporary Erosion Protection			2. Permanent Erosion Protection			
a. Soil reinforcing mat	0.025	0.036	a. Concrete		0.013	0.015
b. Excelsior blanket	0.035	0.066	b. Asphalt		0.016	0.018
c. Jute net	0.022	0.028	c. Compacted gravel		0.033	0.044
			d. Plain riprap		0.035	0.045
			e. Grouted riprap		0.030	0.040
Highway channels & Swales with maintained Vegetation (Values are for velocities of 6 and 2ft/s)						
	Bermuda, Kentucky bluegrass and Buffalograss		Good stand, any grass		Fair stand, any grass	
	Normal	Maximum	Normal	Maximum		
1. Depth of flow up to 0.7 foot						
i. Mowed to 2 in	0.045	0.070				
ii. Length 4 to 6 in	0.050	0.090				
iii. Length about 12 in			0.090	0.180	0.080	0.140
iv. Length about 24 in			0.150	0.300	0.130	0.250
2. Depth of flow 0.7 — 1.5 feet						
i. Mowed to 2 in	0.035	0.050				
ii. Length 4 to 6 in	0.040	0.060				
iii. Length about 12 in			0.070	0.120	0.060	0.100
iv. Length about 24 in			0.100	0.200	0.090	0.170
3. Grass linings (general)					0.030	0.060

6.7.4.2 DESIGN PROCEDURE FOR FLEXIBLE LININGS

The normal treatment for reducing the chance of future erosion is to use stone lining (riprap) in the erodible ditch section. The general design procedure involves the following steps:

1. The depth of flow (*d*) and velocity (*V*) in the channel corresponding to the known peak discharge are computed with the Manning's roughness coefficients for riprap from Figure 6-16 as illustrated in Example 4, Attachment B.

2. The required D_{50} riprap size is obtained from the equation:

$$D_{50} = C\,\frac{0.001V_a^{\,3}}{\left(d_{avg}^{\,0.5}\,K_1^{1.5}\right)} \qquad (6.20)$$

where

D_{50} = the size of stone (ft) such that 50% of all stones in the riprap are smaller than D_{50}

C = a correction factor for the specific gravity of stone and the stability factor to be applied

V_a = Average velocity (ft/s)

d_{avg} = Average depth of flow (ft)

K_1 = Bank angle correction factor

C can be calculated by using the equation:

$$C = 1.61\left[\frac{SF}{S_s - 1}\right]^{1.5} \qquad (6.21)$$

where

SF = Stability factor to be applied, Table 1, HEC-11 (Default = 1.2)

S_s = Specific gravity of stones in the riprap (Default = 2.65)

K_1 can be calculated form the equation:

$$K_1 = (1 - 2.27\,\sin^2\theta)^{1/2} \qquad (6.22)$$

where

θ = Angle of the bank with the horizontal axis

The value of C may alternatively be obtained from Chart 2, HEC-11. The remaining expression of equation 6.21 can be obtained from Chart 1, HEC-11.

The corresponding weight W_{50} (lbs.) for D_{50} (ft) stone is given by:

$$W_{50} = 32.67 D_{50}^3 S_s \qquad (6.23)$$

However, for rock riprap, Manning's roughness coefficient varies with the mean size D_{50} as given in the equation:

$$n = 0.0395 D_{50}^{1/6} \qquad (6.24)$$

For a more accurate solution, the value of n corresponding to D_{50} should be obtained from equation 6.25 and used in Steps 1 and 2 for a refined value of D_{50}.

3. With the required mean stone size (D_{50}), the type of riprap can be specified from the gradation data in DelDOT's *Standard Specifications*. Also refer to the *Standard Construction Details*.

4. In addition, the following criteria shall be taken into considered when specifying the layer thickness:

   a. The layer thickness should not be less than the spherical diameter of D_{100} stone or less than 1.5 times the spherical diameter of D_{50} stone whichever is the greater thickness. D_{100} denotes that 100% of the riprap stones are smaller than this size.

   b. The layer thickness should not be less than 12 inches for practical placement.

   c. The thickness determined by the above criteria shall be increased by 50% when the riprap is placed under water due to uncertainties associated with this type of placement.

   d. An increase in thickness of 6 to 12 inches accompanied by an appropriate increase in stone sizes should be provided where riprap revetment will be subjected to floating debris, ice or waves.

To prevent the riprap from settling, a geotextile filter fabric is placed between the riprap and the underlying soil. This prevents migration of fine particles through voids in the riprap, distributes the weight of the riprap units to provide a more uniform settlement, and permits relief of hydrostatic pressures within the soil.

Please refer to the *Standard Construction Details* for more information on the layers of the ditch section.

Geotextile Filter Fabrics. The design of filter fabric is based on specifications conforming to its functional requirements. DelDOT's *Standard Specifications* specify filter fabrics for use under riprap for erosion protection. For good performance, a properly selected filter fabric should be installed as recommended by the manufacturer, the *Standard Specifications* and the *Standard Construction Details*.

Figure 6-17
Channel Lining Design Procedure

Step 1	Hydrologic computations: includes determining the drainage area, rainfall design year, rainfall intensity and runoff coefficient	Step 7	Determine hydraulic resistance using HEC-11 charts with known *R*, *A* and d_{max} or calculate
Step 2	Design flow - temporary and/or permanent	Step 8	Determine velocity using HEC-11 charts with known *R* and S_o
Step 3	Soil erodibilty; retardance	Step 9	Determine the allowable flow rate, *Q*
Step 4	Channel description including geometry; includes bottom width, side slopes, flow line slope, top width and area	Step 10	If $Q_{allowable} >> Q_{design}$, the channel is over designed If $Q_{allowable} << Q_{design}$, the channel is under designed See Figure 6-18 for a useful computation chart.
Step 5	Trial lining type	Step 11	Based on above results, if necessary, perform Steps 3 thru 10 with new parameters
Step 6	Determine permissible maximum depth of flow, d_{max}, (using HEC-11 Charts)	Step 12	If a temporary lining is to be used until establishment of permanent cover, it may be prudent to repeat steps 6 thru 9 using temporary lining characteristics

Figure 6-18
Channel Lining Design Computation Chart

Lining Type	d_{max}	*B*	d_{max}/B	*A/Bd*	*A*	*R/d*	*V*	*Q = AV*	*T*	Remarks

6.8 PAVEMENT DRAINAGE AND STORM DRAINS

In certain situations, especially for urban highways, it may not be practical to construct roadside ditches (open channels) to collect pavement runoff and convey it to an outfall point. The options are to provide curbs or curb and gutter combinations along the shoulder edges for containing the runoff and channelizing the flow into inlets through which the pavement runoff is removed.

Gutter flow along the pavement is confined to a width and depth that will not obstruct nor cause a hazard to traffic. The ponding width will have a corresponding depth. The gutter flow capacity depends upon the contributing drainage area, gutter width, cross slope, longitudinal slope, and roughness.

Usually the stormwater flowing along median swales or in open areas of parking lots is also removed through inlet structures. A storm drain receives the runoff from inlets, conveys it through a closed conduit, and discharges at a suitable outfall. This is referred to as a closed drainage system as opposed to open channels/roadside ditches. The design of storm drains involving curbed drainage and inlets will be generally described in this section.

6.8.1 REFERENCES

PUBLICATIONS

- *Design Charts For Open-Channel Flow*, Hydraulic Design Series No. 3 (HDS-3), FHWA 1973 (Reprint 1980)
- *Introduction to Highway Hydr*aulics, Hydraulic Design Series No. 4 (HDS-4), FHWA, 1983 and 2001
- *Drainage of Highway Pavements*, Hydraulic Engineering Circular 12, (HEC-12), FHWA, 1984
- *Urban Drainage Design Manual*, Hydraulic Engineering Circular 22, (HEC-22), FHWA, 2001
- DelDOT *Design Guidance Memorandum Number1-20R.*

TABLES, CHARTS AND FIGURES

Access to the following references will be needed to perform this analysis, most of which are in HEC-12 or HEC-22. HEC-12 has been superseded by HEC-22 but still may be useful in design.

- Manning's Roughness Coefficients *n* for Pavements and Gutters
  - Table 4-3, HEC-22
- Allowable Water Spread
  - Figure 6-2
- Inlet and Gutter Sections
  - Figure 6-19
- Nomograph for Velocity in Triangular Gutter Sections
  - Chart 4B, HEC-22
- Flow in Triangular Gutter Sections
  - Chart 29, HDS-3
  - Chart 1B, HEC-22
- Frontal Flow to Total Gutter Flow Ratio
  - Chart 2B, HEC-22
- Flow in Composite Gutter Sections
  - Chart 5, HEC-12
- Curb-Opening and Slotted Drain Inlet Length for Total Interception
  - Figure 35b, HDS-4
  - Chart 9, HEC-12
- Curb-Opening and Slotted Drain Inlet Interception Efficiency
  - Figure 36, HDS-4
  - Chart 10, HEC-12
  - Chart 8B, HEC-22
- Grate Inlet Capacity in Sump Conditions
  - Chart 11, HEC-12
  - Chart 9B, HEC-22
- Depressed Curb Opening Inlet Capacity in Sump Conditions
  - Chart 12, HEC-12
  - Chart 10B, HEC-22
- Orifice Flow in Depressed Curb-Opening Inlet
  - Chart 14, HEC-12
  - Chart 12B, HEC-22
- Slotted Drain Inlet Capacity in Sump Locations
  - Chart 15, HEC-12

  - Chart 13B, HEC-22
- Ratio of Frontal Flow to Total Flow in Trapezoidal Channel
  - Chart 17, HEC-12
  - Chart 15B, HEC-22
- Inlet Clogging Factor of Safety
  - Figure 6-3
- Gutter and Inlet Design using HEC-22
  - Figure 6-20
- Frontal Flow Interception factor, R_f
  - Figure 6-21
- Side Flow Interception Factor, R_s
  - Figure 6-22
- Min. Pipe Slope for Full Flow at 3 ft/s
  - Figure 6-24
  - Table 7-7, HEC-22
- Circular Pipe Conveyance Factor (K)
  - Figure 6-25
- Wall Thickness and Approximate Weight of Circular Concrete Pipe
  - Figure 6-26
- Manning's Roughness Coefficients (n) for Pipe
  - Figure 6-27
- Inlet Spacing Computation Form
  - Example 5, Attachment B

COMPUTER SOFTWARE

- HYDRAIN using the HYDRA module
- InRoads Storm & Sanitary
- HY22 Urban Drainage Design Programs

6.8.2 PAVEMENT DRAINAGE

The primary source for this section is HEC-22. The flow capacity of the curb and gutter section of a roadway pavement depends upon:

1. Manning's roughness coefficient of the surface,
2. Longitudinal slope of the roadway,
3. Cross slope,
4. Allowable spread, and
5. Inlet spacing.

6.8.2.1 MANNING'S ROUGHNESS COEFFICIENT

Manning's roughness coefficient varies slightly for different types of pavement and gutter surfaces. For simplicity, use n = 0.016 for all surfaces of pavements and gutters.

6.8.2.2 LONGITUDINAL SLOPE

The longitudinal roadway slope S_L (ft/ft) is initially established by following the ground profile. The roadway profile is adjusted to balance the cut and fill sections to maximize use of existing soil. Excavation required for drainage also enters into this decision. Flat slopes create flooding problems in a curbed drainage system. Therefore, a minimum gradient of 0.30% within 50 ft of the PI station in the sag or crest of vertical curves should be used in design. Warping the shoulder by gradually changing its cross slope shall be considered when the profile grade falls below 0.3% on vertical curves. Also for a curbed drainage system, the roadway profile shall have a minimum gradient of 0.3% on tangent sections.

In the design of roadways, equal tangent parabolic vertical curves are commonly used for setting the profile. Several basic equations apply when designing drainage for this type of profile. To determine an elevation at a desired point on the vertical curve use the equation:

$$E_x = E_a + \frac{g_2 - g_1}{2L} x^2 + g_1 x \qquad (6.26)$$

where

E_x = Elevation of a point at a distance x from the PVC (ft)

E_a = Elevation PVC (ft)

g_1 = Entering grade (%)

g_2 = Exiting grade (%)

x = Distance measured from PVC (ft) divided by 100

L = Length of vertical curve (ft) divided by 100

Note: g_1 and g_2 are positive for ascending grade and negative for descending grade

To determine the turning point on a vertical curve (highest point for a crest curve or lowest point for a sag curve) use the equation:

$$x_t = \frac{g_1 L}{g_1 - g_2} \qquad (6.27)$$

where

x_t = Location of the turning point from PVC (ft)

Other notations are as previously defined. The elevation of the turning point can be obtained from equation 6.30 by setting x equal to x_t.

The slope of a point (S) on a vertical curve is determined using the equation:

$$S = \frac{g_2 - g_1}{L} x + g_1 \qquad (6.28)$$

S is the slope in (%) at a point on the curve that is a distance x from the PVC. The other notations are as previously defined.

6.8.2.3 CROSS SLOPE

Adequate cross slope on roadways is essential for rapid movement of surface water to reduce sheet flow and potential hydroplaning. On a normal section, the cross slope on a traffic lane is 2% and the shoulder cross slope is 4%. Where these minimum cross slopes are not provided (as in transitions between superelevated and normal sections, or on cross streets and near curb ramps) all efforts shall be made to minimize the sheet flow by providing additional drainage appurtenances and/or with suitable geometric adjustments.

6.8.2.4 ALLOWABLE WATER SPREAD

It is imperative that the computed width of water spread does not exceed the allowable water spread as stipulated in Figure 6-2. The drainage report shall include a comparison of the two values to ensure compliance with the criteria.

6.8.2.5 CURB AND GUTTER FLOW

There are two types of gutter sections used to control drainage along curbed roadways. One section has a uniform cross slope and the other has a composite cross slope. Their design is considered a shallow open channel. Refer to Figure 6-19, DelDOT's *Standard Construction Details*, and Figure 4-1, HEC-22, for a description of the shape, details, dimensions and terminology used in designing gutter and inlet drainage.

6.8.2.5.1 GUTTER WITH UNIFORM CROSS SLOPE

As illustrated in Figure 6-19, triangular open channels with uniform cross slope are characterized by:

- T = Width of water spread (ft)
- d = Depth of flow at curb (ft)
- S_x = Cross slope (ft/ft)
- S_L = Longitudinal roadway slope (ft/ft)
- n = Manning's roughness coefficient
- A = Cross-sectional area of flow (ft^2)

Manning's equation for flow rate in an open channel has to be modified for use with gutter sections because the depth of flow is very shallow and wide. The governing equations for average velocity (V, ft/s) and flow (Q, ft^3/s) in a triangular gutter section are:

$$V = \frac{1.12}{n} S_L^{\,0.5} S_x^{0.67} T^{0.67} \qquad (6.29)$$

$$Q = \frac{0.56}{n} S_L^{\,0.5} S_x^{1.67} T^{2.67} \qquad (6.30)$$

Also for a uniform triangular gutter section,

$$d = TS_x \qquad (6.31)$$

$$A = 0.5T^2 S_x = 0.5 \, d^2 / S_x \qquad (6.32)$$

Therefore, the flow rate is given by:

$$Q = \frac{0.56}{n} S_L^{\,0.5} \frac{d^{2.67}}{S_x} \qquad (6.33)$$

See Charts 1B and 4B, HEC-22, for the nomographs to solve these equations. From Chart 4B, HEC-22, the flow velocity (*V*) can be determined when the width of water spread (*T*) is given (and vice versa) with other known parameters.

The width of water spread (*T*) varies between inlets in a curbed drainage system and so does the velocity of flow (*V*). If T_1 is the spread at the trailing end of an inlet, T_2 is the spread at the approach end of the downstream inlet, and T_a represents the average width of spread between the two inlets on the triangular gutter section, then T_a can be obtained from Table 4-4, HEC-22.

T_a is useful in estimating the travel time of surface water flow on a triangular gutter section with drainage inlets. The velocity V_a corresponding to T_a is the average velocity that can be obtained from Chart 4B, HEC-22. Once V_a is known, the average travel time is determined by dividing the length of travel (distance between the two inlets) by V_a.

Chart 1B, HEC-22, provides the solution for the rate of flow *Q* in a triangular gutter section. When *Q* is known, *T* can be found and vice versa, provided *n*, *S*, and S_x are also known. This chart can also be used for solving the flow rate of V-shaped gutter sections by combining the two cross slopes of the V-sections into a single cross slope as illustrated on the chart.

6.8.2.5.2 GUTTER WITH COMPOSITE CROSS SLOPE

Composite cross slope results when a break exists in the cross slope of the gutter section. See Figure 6-19 for typical gutter sections and nomenclature. A composite cross slope is normally associated with the use of Integral P.C.C. Curb & Gutter, Type 1. The hydraulic efficiency of a gutter section with composite cross slope is significantly higher than one with uniform cross slope. Therefore, the use of Integral P.C.C. Curb & Gutter, Type 1, is preferred in the design of a curbed drainage system.

Composite curb and gutter sections are characterized by the following terms that are also used in the gutter flow equations:

- *T* = Width of water spread (ft)
- T_s = Width of section with normal cross slope, equal to the total spread minus the gutter pan width
- S_x = Cross slope (ft/ft) of adjacent traveled way or shoulder
- *d'* = Depth of flow at cross section break
- *W* = Gutter pan width of greater slope
- S_w = Slope of gutter section
- *d* = Depth of flow at curb (ft)
- S_L = Longitudinal roadway slope (ft/ft)
- *n* = Manning's roughness coefficient
- *A* = Cross-sectional area of flow (ft^2)

Figure 6-19
Inlet and Gutter Sections

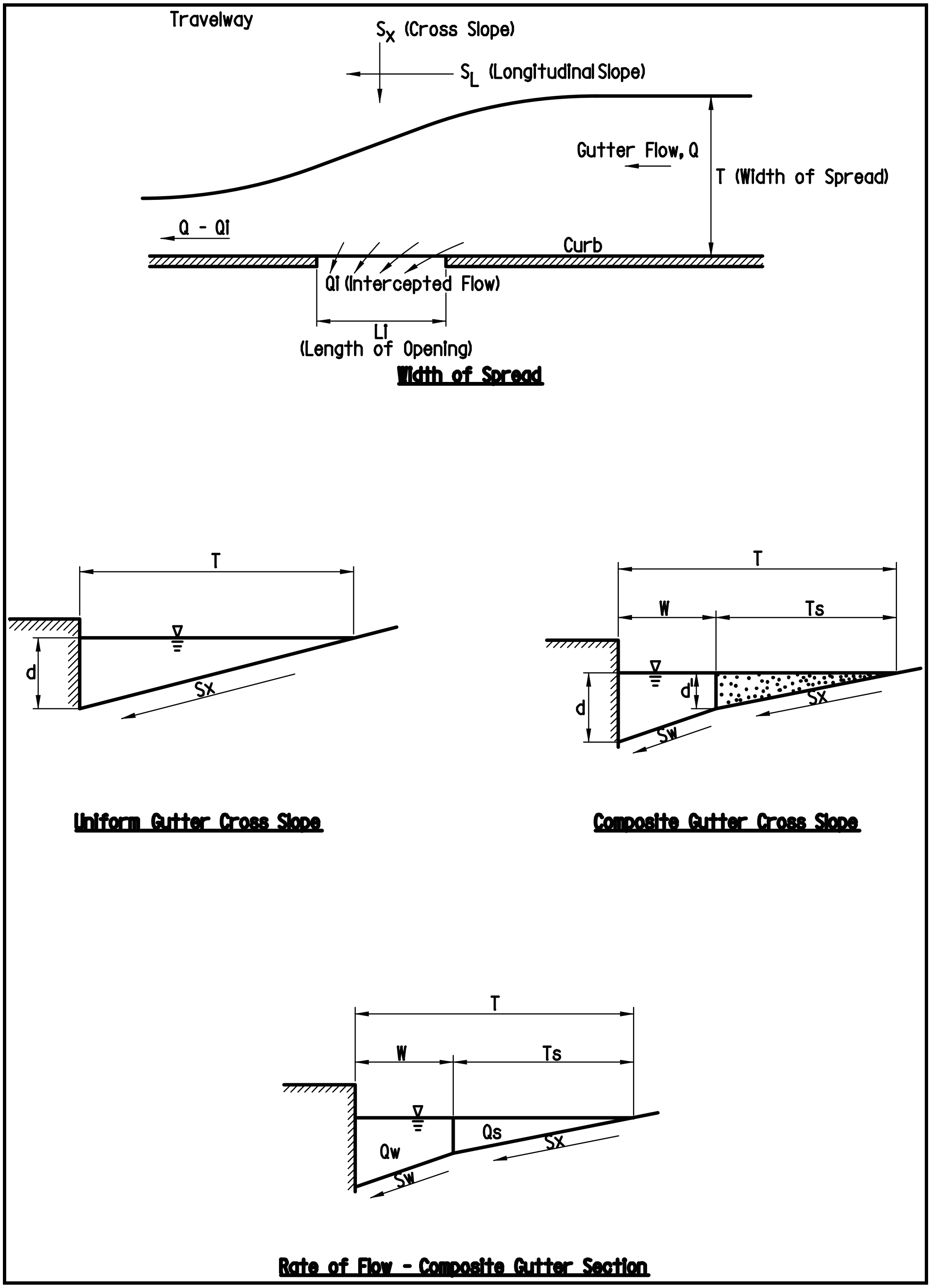


Figure 6-20
Gutter and Inlet Design using HEC-22

Capacity (Gutter and Inlet)			
Step 1	Determine design parameters		
L = Length of design strip A = Area of contribution C = Runoff coefficient of contributing area t_c = Time of concentration I = Design year rainfall intensity Q = Peak flow using $Q = CIA$[1]		S_x = Slope of adjacent shoulder or pavement S_w = Slope of gutter section n = Roughness coefficient (0.016 for concrete) T = Allowable spread width of flow W = Proposed gutter width L_{E1} = Length of grate	
Step 2	Calculate the following ratios needed for using charts: $Z = 1/S_x$, W/T and S_w/S_x	Step 9	Determine E (the total efficiency of the inlet) using: $E = (R_f E_o) + R_s(1-E_o)$
Step 3	Find Qn, the flow in the shoulder or pavement (area outside of W) from Chart 1B	Step 10	Find Q_i (the inlet interception flow) using: $Qi = EQ$
Step 4	Find gutter velocity V from Chart 4B using S_w/S_x ratio	Step 11	Determine Q_b (the inlet flow bypass) from $Q_b = Q - Q_i$
Step 5	Find E_o from Chart 2B using factors S_w/S_x and W/T	Step 12	Using the design flow values, determine the depth of water at the curb line for any curb overflow[2] and actual spread.
Step 6	Find Q_w (the gutter flow rate over the grate) using: $E_o = Q_w/Q$	Step 13	If Step 12 results do not meet the criteria, than repeat the procedure until the correct parameters and results are met.
Step 7	Find R_f (frontal flow interception rate) from Figure 6-21 or Chart 5B.	Step 14	Add the inlet flow bypass to the next Q for the next inlet downstream.
Step 8	Find R_s (side flow interception factor) from Figure 6-22 or Chart 6B.	Step 15	Determine the location of the first inlet by iteratively finding the area generating a flow equal to Q_i.

1. The flow to the all inlets is the sum of flow from all contributing areas that will reach the inlet being analyzed.

2. Don't overflow the curb.

3. See Example 5 for complete form and instructions, also HEC-22 Figure 4-19.3.

In the hydraulic analysis of a composite section, the area is broken into three triangular sections. The modified Manning's equation is applied to each section. The rate of flow in a gutter section with a composite cross slope is given by the equation:

$$Q = \frac{0.56}{n} S_L^{0.5} \left(\frac{d^{2.67}}{S_w} + \left[\frac{d'^{\,2.67}}{S_x} - \frac{d'^{\,2.67}}{S_w} \right] \right) \quad (6.33)$$

where

$d = T_s S_x + W S_w$ (6.35)

$d' = T_s S_x$ (6.36)

$A = 0.5[T_s^2 S_x + 2\, T_s\, W\, S_x + W^2 S_w]$ (6.37)

Charts 1B and 2B, HEC-22, can be used in conjunction with each other to obtain Q for a composite gutter section. The notations are defined in the respective charts.

Chart 5, HEC-12, provides the hydraulic solution of composite gutter sections. When Q, n, S_L, W, S_x and S_w are known, T can be found from this chart. Likewise, Q can be determined when T is given.

6.8.2.5.3 GUTTER FLOW DESIGN TABLES

The hydraulic analysis of gutter sections for both uniform and composite cross slopes can be performed in accordance with the equations and charts referred to in this section as well as available software. For performing gutter flow design by hand, drainage tables for pavements of varying geometric characteristics that are usually incorporated in road design are provided on DelDOT's web site; the variables in these tables are the width of water spread (T), the flow rate (Q), and the average velocity (V).

6.8.2.6 DRAINAGE INLETS

Drainage inlets for intercepting surface runoff should be properly designed with respect to their location and capacity so that the drainage system functions efficiently. They are used on roadways, median swales, roadside ditches and parking areas.

6.8.2.6.1 INLET TYPES

There are a variety of inlets available for use in drainage. Many have a unique shape and application. The Department's basic inlets and grate types are shown in the *Standard Construction Details*. These details have a series of inlet top unit designs (drainage inlet assembly) to fit the available types of curbs and for swale locations.

Inlet capacity is improved by combining a grate inlet with a curb opening, particularly at sag points, and is relatively free from clogging by debris. This type of inlet is commonly used on projects.

Curb-inlets have no grate opening and are used particularly on narrow paved medians and curbed sections were the driver might shy away from a grate. They work best where grades are flatter than 3 percent. The inlet is depressed 2 inches below the gutter flow line with transitions at both ends. It is best to use Integral P.C.C. Curb and Gutter, Type 3, with curb-inlets since it only has a one-foot gutter pan. The minimum length of these inlets is 5 ft; the total length shall be specified in 5 ft multiples.

6.8.2.6.2 INLET GRATES

The types of inlet grates used on projects are shown in DelDOT's *Standard Construction Details*. All the grates are 20 in by 36 in. A description of the grates follows.

Type 1 grate has an opening area of 320 in^2, approximately 44% of the total area. The rounded bars intercept flow more efficiently. It is used adjacent to curb with or without integral gutter where bicycle traffic can be anticipated.

Type 2 grate has an opening area of 370 in^2, approximately 51% of the total area. This grate is used adjacent to a curb in controlled access highways or in median swales where bicycle traffic is restricted.

Type 3 grate has an opening area of 295 in^2, approximately 41% of the total area. This type of grate is used in open parking areas, median swales, and along roadsides where bicycle traffic can be expected. These grates are intended to intercept the surface runoff in sump conditions and shall not be used beside curbs.

Type 4 vane grate has an opening area of 215 in^2, approximately 30% of the total area. This type of grate has a higher hydraulic capacity and lower weight than the other types. It may be used where bicycle traffic can be expected. It is not recommended for use in sump locations.

6.8.2.6.3 HYDRAULIC CHARACTERISITICS OF INLETS

In addition to size and shape, the interception capacity and efficiency of the inlet depend upon its location. The intended hydraulic function is also a selection and design consideration.

The hydraulic analysis of inlets is described for their three most common uses. These are to intercept flow:

- Along a curb on a continuous grade,
- Along a curb in a sag or surface runoff in a sump location, and
- In a swale (ditch).

6.8.2.6.4 INLET INTERCEPTION ON CONTINUOUS GRADE

HEC-22 is the primary resource to be used for designing inlet interception on a continuous grade. The following is a brief discussion of this procedure. The chart references throughout Sections 6.8.2.6.2 and 6.8.2.6.3 are found in HEC-22. For design purposes, the capacity of a combination inlet is analyzed based on the grate efficiency and the curb opening is considered a relief for clogging conditions.

The three elements of runoff (Figure 6-19) associated with the inlet capacity on a continuous grade are:

- The rate of flow of stormwater approaching the inlet (the total gutter flow, Q).
- The rate of flow intercepted by the inlet (Q_i).
- The carryover or bypass rate of flow (Q_b).

Therefore,

$$Q_b = Q - Q_i \qquad (6.38)$$

The efficiency (E) of the inlet is:

$$E = \frac{Q_i}{Q} \qquad (6.39)$$

An inlet on continuous grade can be a grate inlet, a curb inlet or a slotted drain inlet.

Grate Inlet The total gutter flow Q consists of two components:

$$Q = Q_w + Q_s \qquad (6.40)$$

where

Q_w = Frontal flow over width W of the grate, and

Q_s = Side flow.

The ratio of frontal flow to total gutter flow (E_o) is defined by the equation:

$$E_o = \frac{Q_w}{Q} = 1 - (1 - W/T)^{2.67} \qquad (6.41)$$

Where:

Q = Total gutter flow (ft^3/s)

Q_w = Flow over width W of grate (ft^3/s)

W = Width of depressed gutter or grate (ft)

T = Total spread of water in the gutter (ft)

Solutions for E_o are presented in Chart 2B for various $\frac{W}{T}$ and $\frac{S_w}{S_x}$ ratios.

Let: $R_f = \dfrac{Q_{wi}}{Q_w}$ (6.42)

where

R_f = Frontal flow Interception factor,

Q_{wi} = Frontal flow Intercepted by the inlet.

Also,

$R_s = \dfrac{Q_{si}}{Q_s}$ (6.43)

where

R_s = Side flow interception factor,

Q_s = Side flow intercepted by the inlet.

The efficiency E is given by,

$$E = R_f E_o + R_s (1 - E_o) \quad (6.44)$$

Therefore,

$$Q_i = EQ = Q\,[\,R_f\ E_o + R_s\,(1 - E_o)\,] \quad (6.45)$$

The value of E_o from Chart 2B is used in the above equations. R_f and R_s depend upon the approach velocity V that corresponds to Q for the total gutter flow. Whether the flow takes place on either a uniform cross slope or a composite cross slope, V can be obtained by solving the equations or from the nomographs. Having known V, the values of R_f and R_s can be obtained from Figures 6-21 (for the Type 1 grate only) and 6-22, or Charts 5B and 6B, respectively.

Figure 6-21
Type 1 Grate Frontal Flow Interception Factor, *R_f**

V	< 6	7	8	9	10	11	12	13	14	15
R_f	1.0	0.92	0.83	0.73	0.63	0.56	0.47	0.38	0.28	0.19

*Adapted from Chart 5B, HEC-22

Figure 6-22
Side Flow Interception Factor,* R_s

V	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
S_x								***R_s***							
0.02	0.63	0.32	0.19	0.12	0.08	0.06	0.05	0.04	0.03	0.03	0.02	0.20	0.02	0.01	0.01
0.03	0.71	0.42	0.26	0.17	0.12	0.09	0.07	0.06	0.05	0.04	0.03	0.03	0.02	0.02	0.02
0.04	0.77	0.49	0.32	0.22	0.16	0.12	0.09	0.07	0.06	0.05	0.04	0.04	0.03	0.03	0.02
0.05	0.81	0.55	0.37	0.26	0.19	0.14	0.11	0.09	0.07	0.06	0.05	0.05	0.04	0.03	0.03
0.06	0.83	0.59	0.41	0.29	0.22	0.17	0.13	0.11	0.09	0.07	0.06	0.05	0.05	0.04	0.04

*Using equation $R_s = \dfrac{1}{\left[1 + (0.15)(V^{1.8}) / S_x L^{2.3}\right]}$

Curb-Opening Inlet. For uniform cross slope, the length of curb opening for total interception of gutter flow is given by the equation:

$$L_T = 0.6\, Q^{0.42}\, S_L^{\ 0.3} \left(\frac{1}{n\, S_x} \right)^{0.6} \qquad (6.46)$$

where

L_T = Length of curb opening in feet for total interception of gutter flow (100% efficiency of interception).

Other notations are as defined previously.

The efficiency E for inlets shorter than L_T is expressed by the equation:

$$E = 1 - \left(1 - L/\, L_T \right)^{1.8} \qquad (6.47)$$

This equation is also valid for depressed curb-opening inlets.

Charts 7B and 8B provide graphical solutions of these equations.

However, curb-opening inlets must be depressed to significantly increase their interception of gutter flow. This results in a composite cross slope as illustrated on Chart 7B. The equation is:

$$S_e = S_x + S'_W\, E_o \qquad (6.48)$$

Where,

S_e = Equivalent cross slope for depressed opening (ft/ft)

S_x = Pavement cross slope (ft/ft)

S'_w = Cross slope of the gutter measured from the edge of pavement (ft/ft)

E_o = Ratio of flow over the depressed width to the flow over the total gutter section (Q_w/Q)

As shown in Chart 7B, W is the depressed width or gutter width (ft) and a equals the gutter depression (in).

Therefore,

$$S'_W = \frac{a}{(12W)} \qquad (6.49)$$

Thus for depressed curb-opening inlets:

$$L_T = 0.6\, Q^{0.42}\, S_L^{\ 0.3} \left(\frac{1}{n\, S_e} \right)^{0.6} \qquad (6.50)$$

Slotted Drain Inlet The equations for solving for L_T and E and Chart 7B for curb-opening inlet along with the nomograph in Chart 5B for interception efficiency are applicable to slotted drain inlets on a continuous grade.

6.8.2.6.5 INLET INTERCEPTION IN SAG LOCATIONS

Inlets on sag vertical curves of roadway pavement, in open parking areas, and on median swales, operate in a sump condition. As the surface runoff approaches from some or all sides of the inlet, the interception takes place as if the flow occurs over a weir or an orifice depending on the depth of flow. The inlet operates as a weir with low head and as an orifice with relatively high head.

Note that when using Chart 9B the following applies:

- Inlet Type 1─Opening ratio = 0.44
- Inlet Type 2─Opening ratio = 0.51
- Inlet Type 3─Opening ratio = 0.41
- Inlet Type 4─Opening ratio = 0.30

Inlet grates in sag locations are analyzed for two conditions:

Condition 1 - Inlet Grate Operating as a Weir

The flow rate intercepted by a grate inlet is

$$Q_i = C_w\, P\, d^{1.5} \qquad (6.51)$$

where

Q_i = Flow intercepted by the inlet (ft^3/s)

C_w = Weir coefficient = 3

P = Perimeter of the sides of inlet approached by runoff (ft)

d = Average depth of water adjacent to the inlet (ft) (see Figure 4-17, HEC-22)

Chart 9B has the graphical solution to this equation. As indicated on the chart, if the flow is hindered on one side of the inlet due to the curb, then that side has to be omitted in calculating *P*. For practical purposes *W* and *L* shall be taken as the outside dimensions of the grate inlet.

Generally, the Type 1 grate operates as a weir in a curbed drainage design. The permissible depth of flow at the curb is limited to the height of curb and the allowable spread. However, when used in parking areas and swales, the Type 1 grate may operate as an orifice because a greater depth of flow is likely to be accommodated.

Condition 2 - Inlet Grate Operating as an Orifice

This condition exists when the depth of water exceeds 8 in. The flow rate intercepted by the grate inlet is given by:

$$Q_i = C_o\, A\left(2gd\right)^{0.5} \qquad (6.52)$$

Where,

C_o = Orifice coefficient = 0.67

A = Clear area of grate opening (ft^2)

g = Acceleration due to gravity (32.2 ft/s^2),

d = Average depth of water adjacent to the inlet (ft) (see Figure 4-17, HEC-22)

Chart 9B presents the solution for equation 6.52 using the given inlet opening ratios.

When the depth of water is in transition between weir and orifice flow, the perimeter line on the lower half of the figure should be connected by a smooth curve with the corresponding clear area line of the upper half for Q_i in transition.

Depressed Curb-Opening Inlet in Sag Location

Case 1 - Depressed Curb-Opening Inlet Operating as a Weir

The inlet interception Q_i (ft^3/s) is given by:

$$Q_i = 2.3\left(L + 1.8W\right)d^{1.5} \qquad (6.53)$$

where

L = Length of depressed curb opening (ft)

W = Lateral width of depression (ft)

d = Depth of water (ft) at curb measured from normal cross slope

In this case,

$$d = T\,S_x \qquad (6.54)$$

where

T = Water spread

S_x = Slope of adjacent pavement

For the depressed inlet to operate as a weir:

$$d \le [\ h + \frac{a}{12}] \qquad (6.55)$$

where

h = Height of curb-opening inlet (ft)

a = Depth of depression (in)

Case 2 - Depressed Curb-Opening Inlet Operating as an Orifice

The inlet interception Q_i (ft^3/s) is given by:

$$Q_i = 0.67\, h\, L\left(2\, gd_o\right)^{0.5}$$
$$= 0.67\, A\left[2g\left(d_i - \frac{h}{2}\right)\right]^{0.5} \qquad (6.56)$$

where

d_o = Effective head on the center of orifice throat (ft)

A = Clear area of the opening (ft^2)

h = Height of depressed curb-opening inlet (ft)

d_i = Height of depressed curb-opening orifice (ft), expressed as:

$$d_i = TS_x + \frac{a}{12} \qquad (6.57)$$

a = Depth of depression (in)

For the depressed curb-opening inlet to operate as an orifice, the depth of flow should be greater than 1.4 *h*.

Chart 10B gives the graphical solution of equations for depressed curb-opening inlets. Chart 12B gives the solution for inclined and vertical orifice throats.

Slotted Drain Inlet

The slotted drain inlet in sump condition operates as a weir when the depth of flow is up to 0.2 ft and as an orifice when the depth of water exceeds 0.4 ft. Chart 13B provides the solution both for weir and orifice conditions with dashed lines for transition. Note the multiplying factor involving the width of slot at the top of the figure.

6.8.2.6.6 INLET IN OPEN CHANNEL

Grate inlets are installed in median swales or roadside ditches to intercept the flow of these open channels. They can be either on a continuous grade or in a sump depending on the profile of the channel. For the sump condition, the methodology outlined in Section 6.8.2.6.5 applies. For installations on a continuous grade, the analysis for interception by grate inlets in Section 6.8.2.6.4 is valid, except for E_o that was defined by equation 6.41.

E_o for trapezoidal channels can be obtained from Chart 15B. The intercepted flow Q_i is determined by using equation 6.45.

6.8.2.6.7 FACTOR OF SAFETY FOR CLOGGING OF GRATE INLETS

Because grate inlets may become clogged with debris, the theoretical interception capacity should be properly reduced in drainage design.

Generally a grate inlet on continuous grade is placed beside a curb that has an opening. This opening, disregarded in the interception analysis, should alleviate the effect of clogging that seldom occurs on a continuous grade. Therefore, a factor of safety to reduce the theoretical interception capacity of grates on continuous grade may be deemed unnecessary.

However, in sump conditions where clogging is anticipated, the factors of safety to be applied are:

- for curb on one side use 1.5 (S_{f1})
- for no curb, i.e., the interception takes place from all four sides, use 2.0 (S_{f2}).

For weir flow, the clogging shall be assumed to occur lengthwise, so that a reduced width of grating is taken into consideration to account for the factor of safety. For example, consider a standard grate that operates as a weir. If it is placed alongside a curb, then the perimeter to be considered for computing the interception with factor of safety is given by:

$$P = 2\left(\frac{W}{S_{f1}} + L\right)/12 \qquad (6.58)$$

$$P = 2\left(\frac{20}{1.5} + 36\right)/12 = 5.22\,ft$$

If the curb does not exist, then the interception would occur from all four sides. Therefore,

$$P = 2\left(\frac{W}{S_{f2}} + L\right)/12 \qquad (6.59)$$

$$P = 2\left(\frac{20}{2} + 36\right)/12 = 7.67\ \ ft$$

When a grate inlet in a sump condition operates as an orifice, then its clear open area (A) should be divided by the factor of safety specified in the foregoing to compute the actual interception capacity.

6.8.2.6.8 INLET LOCATIONS

6.8.2.6.8.1 DESIGN CRITERIA

1. Drainage inlets shall be designed for the runoff frequency and spread as defined in DelDOT's design criteria. The design involves determining the spacing of inlets based on their interception capacity.

2. Aside from inlet locations as required by their hydraulic design, additional inlets shall be provided in strategic areas to avoid the

concentration of sheet flow by intercepting the runoff in advance. Some of those strategic locations are:

- Upstream of median breaks, entrance and exit ramp gores, curb ramps, crosswalks and street intersections.
- Upstream and downstream of bridge approaches.
- Upstream of superelevation tangent runout and cross slope reversal.
- End of channels in cut sections.
- Behind curb and sidewalk in low areas.
- At the low point of a sag vertical curve.

The inlet(s) at the low point should be designed to keep the water spread within the allowed limit. The preferred method is to design flanking inlets to act in relief of the low point inlet if it gets clogged or the water spread exceeds the limit due to a less infrequent storm. The flanking inlets shall be located where the gutter elevation is approximately 0.2 feet higher than the low point or where allowed spread would be exceeded if a sump drain clogged. The method described in HEC–22 is used to design flanking inlets.

Equations for elevation and low point on a sag vertical curve are included in Section 6.8.2.2. The distance of each flanking inlet from the low point x (feet) is given by:

$$x = \left(4000L/A\right)^{0.5} \qquad (6.60)$$

Where:

L = Length of vertical curve

A = Algebraic difference in approach grades

3. The maximum spacing of inlets shall be 300 ft.

4. The efficiency of inlets on continuous grade shall be at least 70%. However, inlets provided at strategic locations should intercept 100% of the gutter flow.

5. The factor of safety for potential clogging of inlet grates in sump conditions are specified in Section 6.8.2.6.7. In approved locations, parallel bar grates are preferred in sump conditions.

6. The criteria for using curb-opening inlets are in Section 6.8.2.6. When the roadway profile is steeper than 3%, curb-opening inlets should not be used.

7. Gutter inlets should not be used to intercept runoff from areas adjacent to the roadway, neither within nor outside the right-of-way. The surface runoff from those areas may be more efficiently intercepted in advance, and sometimes disposed of by other means such as roadside ditches.

6.8.2.6.8.2 SPACING OF DRAINAGE INLETS

Inlet spacing begins at a required distance from the crest of roadway subject to previously defined location criteria. The flow rate of stormwater intercepted by the first inlet and the bypass flow rate are computed and its efficiency is checked. If the inlet efficiency is found to be lower than the prescribed minimum, then the inlet location is moved upstream to achieve the required 70% efficiency. Then the computation proceeds to the next inlet where the discharge from its watershed (downstream of the first inlet) is added with the bypass from the first inlet. This total gutter flow rate is used to compute the water spread, which cannot exceed the design criteria, and the efficiency of this inlet. The procedure is repeated as the roadway profile moves into a sag curve where the storm water is ultimately removed from the inlet at the low point, which operates in a sump condition at 100% efficiency. The following equations are used for spacing inlets on grade. It should be noted that these equations are based on flow data obtained using the procedure outlined in Figure 6-20.

***Spacing of First Inlet from Crest*.**

$$L_1 = \frac{Q_{1T} \; x \; 43{,}560}{C_1 \, I_1 \, \overline{W}_1} \qquad (6.61)$$

where

L_1 = Distance of the first inlet from crest (ft)

Q_{1T} = Total gutter flow capacity upstream of the first inlet within the width of permissible water spread that conforms to the required efficiency

C_1 = Weighted average runoff coefficient of the drainage area of the first inlet

I_1 = Rainfall intensity corresponding to the time of concentration for the first inlet (in/hr)

$\overline{W}_1$ = Mean width of the first inlet's drainage area (ft)

Spacing of Subsequent Inlets

$$L_n = \frac{\lfloor Q_{nT} - Q_{B,n-1} \rfloor x\ 43{,}560}{C_n\, I_n\, \overline{W}_n} \quad (6.62)$$

where

L_n = Distance of *n*'th inlet from the *(n-1)*'th inlet (ft)

Q_{nT} = The total gutter flow capacity upstream of n'th inlet within the width of permissible water spread that conforms to the required inlet efficiency (ft^3/s)

$Q_{B,n-1}$ = Bypass flow rate from the *(n-1)*'th inlet (ft^3/s)

C_n= Weighted average runoff coefficient of the drainage area of the *n*'th inlet

I_n= Rainfall intensity corresponding to the time of concentration for the *n*'th inlet (in/hr)

$\overline{W}_n$ = Mean width of the *n*'th inlet's drainage area (ft)

For non-uniform drainage areas, the solutions to these equations are obtained by trial and error. The spacing distances are assumed and the mean widths of drainage areas are computed along with other parameters until the equations are satisfied with the assumed spacing of the drainage inlets.

All inlets of the drainage system shall be numbered and the computed results shall be presented in a tabular format. Use a procedure such as described in Example 5, Attachment B.

6.8.3 STORM DRAINS

The hydraulic design of a system is performed after the preliminary layout locating inlets, storm drains, and outfalls. The Rational Formula is used to determine the peak discharge for sizing storm drains. The hydraulic design is a two step process: (1) select a preliminary pipe size based on hydrology and simplified hydraulic computations and (2) compute the hydraulic gradeline (HGL) for the system. The second step refines the preliminary pipe size based on the hydraulic losses in the system.

HEC-22 is the primary resource for the design of storm drains**.** Since this is a trial and error process, the use of approved computer software is recommended. The designer should check that the output is in an approved format for inclusion in the H&H Report. It is particularly important that the results are compared to DelDOT’s drainage criteria in the output data where applicable.

Each pipe segment of the storm drain system is designed individually for size and slope from inlet to inlet or junction to junction. The Rational Formula is used to compute the peak discharge at the inlet in the beginning of each segment. From the uppermost end of the storm drain system the design begins and continues downstream for each segment to the outfall point. When all pipe segments are designed, it may be necessary to determine the hydraulic gradeline elevation to check the operation of the system under the design storm frequency by computing head losses in the system starting from the design high water elevation at the outfall and proceeding upstream (backwards) considering every pipe segment up to the beginning inlet. This may be a trial and error procedure that is simplified by using computer software.

The storm drain design process begins with the collection of the following data:

1. Hydrologic data of the watershed including the outfall data.
2. Locations of underground utilities, existing or proposed.
3. Topographic data (including runoff basin divides, streets, developments, and lot lines) of the entire watershed draining into the storm drain system with arrows on the maps denoting all directions of flow from each lot and street. This information should be superimposed on the working drawings of preliminary plans.
4. Preliminary roadway plans for the project including inlet locations, roadway typical sections, profiles, and cross sections.

Choosing inlet locations is nearly complete before the actual storm drain design. The next step is to develop a layout plan for the conveyance of the intercepted runoff through main storm drains and laterals that connect all outlets to a suitable outfall point. This system layout plan shall indicate locations of inlets, manholes and junction boxes; all pipe segments with directions of flow; and existing or proposed underground utilities such as water, gas, cable and sanitary sewer. Then a rough profile gradeline from inlet to inlet of the storm drain system leading to the outfall is established, the pipe segments are designed, potential utility conflicts (if any) are resolved, the hydraulic gradeline is checked, and final adjustments are made for approval of the design.

6.8.3.1 DESIGN CRITERIA

Storm drains –– The following general criteria and those in Figure 6-3 shall apply to storm drain design:

1. The storm drain trunk line should not be under a traveled way but in the median, a shoulder area or behind the curb.
2. The minimum velocity should be 3 ft/s flowing full to prevent the deposition of sediments and debris inside the storm drain system. See Figure 6-24.
3. In order to prevent damage to the pipe surface, the maximum velocity at full flow should not exceed 15 ft/s. Slopes flatter than 10% are normally used.
4. To prevent water bubbling out of inlets and manholes, particularly at low points, the hydraulic gradeline shall remain at least 1 ft below the gutter elevation at any inlet or top of manhole.
5. For concrete pipe, Class IV is most commonly used along with Class V for greater load bearing locations, such as where there is minimal cover. Other pipe materials (such as corrugated metal and High Density Polyethylene (HDPE)) must also be considered as per federal regulations. The designer has to ensure that the pipe material can bear site-specific loads, perform well under the site conditions and comply with the project's expected service life criteria.
6. The preferred minimum cover over a storm drain is 3.0 ft. Refer to DelDOT's *Standard Construction Details, Standard Specifications, AASHTO LRFD Bridge Design Specifications,* and manufacturer's recommendations for proper bedding and cover requirements under roadway pavements.
7. A simple initial approach for preliminary design and evaluation to ensure the system will operate as open channel flow is to lay out each pipe segment at its friction slope under full flow condition and use equations 6.64 and 6.65.
8. The layout of pipe junctions at inlets, manholes and junction boxes shall be carefully planned for smooth flow with minimum head loss. Pipes entering or exiting a junction cannot exceed an angle of 45^o. The designer shall ensure that the pipes fit into the proposed structure, horizontally and vertically. Refer to the *Standard Construction Details* for minimum clearances to the drainage structure walls.
9. At changes in pipe size, the soffits (top inside surfaces of the two pipes) should be

maintained at the same level by dropping the invert of the larger pipe.

10. The size of any pipe segment shall not decrease in size in the downstream direction even if a change in design such as increasing the slope would permit the use of a smaller sized pipe.
11. For cleanout and maintenance inspection, access to storm drains shall be provided through drainage inlets or manholes within the 300 ft maximum distance.
12. If a storm drain or ditch carrying highway runoff extends beyond the right-of-way, then a permanent easement or additional right-of-way to the outfall shall be acquired for future maintenance.
13. Curved alignment of storm drains following the general alignment of the roadway may be permitted. The manufacturer's recommendations for pipe layout should be followed.
14. The design criteria for strength and hydraulic analysis of pipes shall conform to the provisions of Section 6.9.
15. The designer's initial site evaluation should include the prevailing ground water elevation level for estimating dewatering costs.
16. Pressure flow design should not be considered.

Outfalls — The following general criteria should be considered when determining storm drain outfalls.

1. Compliance with the NPDES.
2. Compliance with the Delaware Erosion and Sediment Control regulations.
3. Coordinated with the locations of any required stormwater quantity and quality management systems.
4. Preferably discharges into a natural drainage course.
5. Ensuring that overland flow occurs before entering any wetlands.
6. The storm drain outfall flow should not scour the downstream reach.
7. The outfall should desirably operate at the design storm flow level under inlet control and as a free (non- submerged) outfall.
8. The design of the system should be refined to avoid adverse hydraulic conditions such as excessive velocity, erosion, or excessive scouring requiring the need for expensive energy dissipaters, or bank overflow protection.
9. Embankment slopes and headwall designs for the outfall shall be in conformance with the *Roadside Design Guide.*

Figure 6-23
General Guidelines for Culvert Outfall Treatment*

Pipe outfall design velocity	Channel treatment required
Max. allowable velocity above those in Figures 6-14 and 6-15 up to 10 ft/s	Sod or stabilization
10 ft/s to 14 ft/s	Dumped riprap for a distance of 15 to 25 feet
Greater than 14 ft/s	Special treatment, such as a plunge pool, stilling basin or energy dissipater

*Note: The outfall design is determined by the stormwater control regulations and design procedure.

Figure 6-24
Minimum Pipe Slope to Ensure a 3.0 ft/s Velocity in a Storm Drain Flowing Full

Inner Pipe Diameter (in)	Full Pipe Flow (ft^3/s)	Minimum Slope (%)				
		n = 0.011	n = 0.012	n = 0.013	n = 0.020	n = 0.024
12	2.4	0.32	0.38	0.44	1.04	1.49
15	3.7	0.24	0.28	0.32	0.78	1.11
18	5.3	0.18	0.22	0.26	0.60	0.87
21	7.2	0.15	0.18	0.21	0.49	0.71
24	9.4	0.12	0.15	0.17	0.41	0.59
27	11.9	0.11	0.13	0.15	0.35	0.51
30	14.7	0.09	0.11	0.13	0.30	0.44
36	21.2	0.07	0.09	0.10	0.24	0.34
42	28.9	0.06	0.07	0.08	0.20	0.28
48	37.7	0.05	0.06	0.07	0.16	0.23
54	47.7	0.04	0.05	0.06	0.14	0.20
60	58.9	0.04	0.04	0.05	0.12	0.17
66	71.3	0.03	0.04	0.05	0.11	0.15
72	84.8	0.03	0.03	0.04	0.09	0.14

6.8.3.2 DESIGNING STORM DRAINS

Open channel design shall be used for the design of a storm drain system. For the rare occasions when pressure flow design is used due to the topographic constraints of the site, the engineering justification and calculations for pressure flow design shall be included in the project's drainage report and must be approved by DelDOT's Assistant Director - Design.

6.8.3.2.1 OPEN CHANNEL DESIGN

Under most conditions a roadway's storm drain system flows under atmospheric pressure and gravity. The various pipe segments are sized assuming that they will flow practically full under design discharge such that the surface of the design flow in the pipe is open to atmospheric pressure. One of the controlling design criteria for pipe design is the minimum slope necessary to maintain 3 ft/s flowing full to reduce silting.

The design depth of flow should be lower than the full depth (diameter) of the pipe. A circular pipe flowing at a depth of 93% of its diameter is able to carry the flow equal to its full depth capacity.

The design method consists of computing the design discharge to be carried by each pipe segment and determining the size and slope of pipe necessary for carrying this discharge. This is accomplished by proceeding step by step from the beginning of a storm drain line downstream to its terminus, which may be a junction point where one trunk line discharges into another line or the outfall point. It is essential that the soffit elevation of the outlet pipe at the point of discharge is at or above the design tailwater elevation at the outfall.

The outfall point is a discharge point into a natural watercourse, excavated channel, retention pond or another storm drain system for disposing of the stormwater collected from the highway. Estimating the design tailwater elevation is based on hydrologic considerations relating to the downstream channel characteristics as well as the ratio of watershed area of the storm drain system (tributary) to the total drainage area of the outfall (main stream). The tailwater elevation can be affected by differences in the design frequencies and peak discharge rates of the watersheds, including downstream structures.

One simple initial approach for preliminary design and evaluation to ensure the system will operate as open channel flow is to lay out each pipe segment at its friction slope under full flow condition and use equations 6.64 and 6.65; also see Section 6.8.3.2.4. Placing the pipe at its friction slope results in determining the normal depth of uniform flow under constant discharge. Essentially this is the slope at which the friction and gravity forces in the direction of flow are equal but acting in different directions. Hydraulically this means that at normal depth the slope of the pipe, the slopes of the HGL and EGL are numerically equal and parallel to each other. Normal depth is a function of the discharge flow rate, the channel geometry, the channel/pipe slope, and the resistance due to friction. The equations in Section 6.8.3.2.4 use these factors to find the correct pipe size and slope. Figure 6-28 shows pipe flow capacity for various pipe sizes at its friction slope using Manning's equations for *n* equals 0.012. Available software maximizes the hydraulic efficiency of the system based upon the input parameters selected for the best and most economical final design.

The open channel design of a storm drain system can be performed following the procedure and filling out a form similar to that in Example 6, Attachment B.

6.8.3.2.2 HYDRAULIC GRADE LINE PROCEDURE

Since the design criteria specifies that the HGL elevation at any structure be one foot below the structure top elevation, it is necessary to compute the elevations of the HGL in a storm drain system whether under open channel or pressure flow conditions. The computations are based on the design frequency discharges.

Normally the HGL is assumed to be outlet control. The HGL is determined by first establishing the design tailwater elevation at the outfall and comparing it to the average critical depth (d_c) and the height of the pipe (d_c+D)/2. The larger of these values is selected as the beginning elevation. Depending upon the outfall channel, establishing the tailwater elevation could involve analysis of joint or coincidental occurrence of storm events. Normally, the ratio of the upstream and downstream watersheds is large enough that the tailwater depth would be the normal flow depth of the receiving channel.

Then working upstream along the entire pipe system, compute friction loss for each individual pipe segment, losses due to curvature or bends, and junction losses at the various structures to determine the HGL.

For pressure flow design, refer to Example 6, Attachment B, for a form and procedure for tabulating computed results. Computer programs such as HYDRA make the computation of a system's EGL and HGL simple. However, it is important that the designer understand the process to judge if the input and output are valid.

6.8.3.2.3 SAG POINTS

DelDOT's design criteria require that at sag points where the only drainage available is the storm drain, the pipe draining the sag point and inlet(s) should be sized for a 50-yr frequency rainfall. Although software is available for most of these calculations, they can be done by computing the bypass flow occurring at each inlet during a 50-yr frequency rainfall and accumulating it at the sag point. For sizing the downstream storm drain, one method of determining the additional bypass flow is to convert the flow to an equivalent using the product of the weighted runoff coefficient and contributing area, *CA*, that can be added to the design *CA*; see discussion on the rational formula and the inlet design procedure in Example 5, Attachment B. This equivalent *CA* is found by dividing the 50-yr bypass by the I_{10} in the pipe at the sag point.

Sag point drainage is an important design consideration due to flooding of the roadway and adjacent properties as well as the potential of creating a hydroplaning area. In order to prevent this condition from occurring, the use of flanking inlets is recommended.

6.8.3.2.4 HYDRAULIC PROCEDURES

The hydraulic capacity of a storm drain system is determined with equations 6.16

through 6.19.For circular storm drains flowing full and R = D/4, and the equations become:

$$V = \frac{0.590}{n} D^{2/3} S^{1/2} \quad (6.63)$$

$$Q = \frac{0.463}{n} D^{8/3} S^{1/2} \quad (6.64)$$

where

V = Velocity (ft/s)

D = Pipe diameter (ft)

S = Storm drain slope (ft/ft)

n = Roughness coefficient

Q = Pipe flow (ft^3/s)

The references listed in Sections 6.8 and 6.9 contain nomographs and charts that can be used to solve these equations during design.

Another approach to finding pipe flow is to develop a conveyance factor, K, which represents the physical characteristics of the channel/pipe; see equations 6.18 and 6.19.

Using a combination of these equations, the friction slope, S_f, of the conveyance surface can be used to determine the values for the velocity and flow capacity of the pipe. The equations developed are:

$$S_f = \left(\frac{Vn}{1.49R^{2/3}}\right)^2 = 29\frac{n^2}{R^{4/3}}\left(\frac{V^2}{2g}\right) \quad (6.65)$$

For full flow conditions:

$$S_f = \frac{184n^2}{D^{4/3}}\left(\frac{V^2}{2g}\right) \quad (6.66)$$

Or,

$$S_f = \left(\frac{Q}{K}\right)^2 \quad (6.67)$$

where K is defined as

$$K = \frac{1.49}{n} AR^{2/3} \quad (6.68)$$

Or,

$$K = \frac{0.463}{n} D^{2.67} \quad (6.69)$$

For concrete pipes (n = 0.012) flowing flow:

$$S_f = \frac{Q^2}{1490\left(D^{8/3}\right)^2} \quad (6.70)$$

The head loss due to friction H_f is determined by the equation:

$$H_f = S_f L \quad (6.71)$$

where

L = Length of pipe

However, most design will be done using software. In the references there are tables that show of the results of solving these equations for different pipe materials, slopes, and roughness coefficients. These tables allow for a preliminary evaluation of pipe size and slope as well as checking software output. The full flow capacity is calculated by multiplying the square root of the slope times the K value obtained from Figure 6-25.

6.8.4 FLARED END SECTIONS

Flared end sections (FES's) are to be used outside of the clear zone at intakes requiring personnel safety grates and outfalls. (Personnel safety grates are only placed on the intake end of a pipe run that is not linear; refer to DelDOT Design Guidance Memorandum No. 1-15.) **They are not recommended for roadside drainage installations or on the ends of driveway pipes where crashworthy treatments are needed**; please refer to AASHTO's *Roadside Design Guide*. During installation, the FES shall be placed at the flow line grade of the pipe as with any pipe section. FES's are not manufactured to fit skewed slopes. For a skewed pipe culvert, the FES is placed in line with the pipe, and the fill slope is warped to fit the FES. When calculating the quantity of pipe, the FES is not included in the length of pipe, and is a separate pay item.

Figure 6-25
Circular Pipe Conveyance Factor (*K*)

D Pipe Diameter (in)	*A* Area (ft²)	*R* Hydraulic Radius (ft)	$K = \frac{1.49}{n} AR^{2/3}$		
			n =0.011	n =0.012	n = 0.013
12	0.785	0.250	42.2	38.7	35.7
15	1.227	0.313	76.6	70.2	64.8
18	1.767	0.375	125	114	105
21	2.405	0.438	188	172	159
24	3.142	0.500	268	246	227
27	3.976	0.563	367	337	311
30	4.909	0.625	486	446	411
33	5.940	0.688	627	575	531
36	7.069	0.750	790	725	669
42	9.621	0.875	1192	1093	1009
48	12.566	1.000	1702	1560	1440
54	15.904	1.125	2330	2136	1972
60	19.635	1.250	3086	2829	2611

Figure 6-26
Wall Thickness and Approximate Weight of
Circular Concrete Pipe Class IV with Type B Wall Thickness

Size (in)	Wall Thickness (in)	Approximate Weight (lbs. per ft)	Size (in)	Wall Thickness (in)	Approximate Weight (lbs. per ft)*
12	2.0	93	36	4	524
15	2.25	127	42	4.5	686
18	2.5	168	48	5	867
21	2.75	214	54	5.5	1068
24	3	264	60	6	1295
30	3.5	384	66	6.5	1542
33	3.75	451	72	7	1811

* Pipe sections are 8 ft in length.

Figure 6-27
Manning's Roughness Coefficients (*n*) for Pipe

Type of Pipe	Recommended *n*
Concrete - Round and elliptical	0.012
Corrugated Metal - Annular (2 2/3 x ½ in) Helical (2 2/3 x ½ in) Spiral Rib	 0.024 0.020 0.012
Plastic - Polyvinyl High Density Polyethylene	 0.011 0.012
Polyethylene-Double Walled	0.012

Figure 6-28
Friction Slope (ft/ft) for *n* = 0.012, Full Flow

Q (cfs)	Pipe Diameter (in)											
	12	15	18	21	24	27	30	36	42	48	54	60
2	0.003											
4	0.011	0.003										
6	0.024	0.007	0.003									
8	0.043	0.013	0.005									
10	0.067	0.020	0.008	0.003								
12	0.097	0.029	0.011	0.005								
14	0.132	0.040	0.015	0.007	0.003							
16		0.052	0.020	0.009	0.004							
18		0.066	0.025	0.011	0.005	0.003						
20		0.082	0.031	0.014	0.007	0.004						
24		0.118	0.044	0.020	0.010	0.005						
26		0.138	0.052	0.023	0.011	0.006						
25			0.048	0.021	0.010	0.006	0.003					
28			0.061	0.027	0.013	0.007	0.004					
30			0.069	0.031	0.015	0.008	0.005					
32			0.079	0.035	0.017	0.009	0.005					
34			0.089	0.039	0.019	0.010	0.006					
35			0.095	0.042	0.020	0.011	0.006					
36			0.100	0.044	0.022	0.012	0.007					
38			0.111	0.049	0.024	0.013	0.007					
40				0.054	0.027	0.014	0.008	0.003				
42				0.060	0.029	0.016	0.009	0.003				
44				0.066	0.032	0.017	0.010	0.004				
45				0.069	0.034	0.018	0.010	0.004				
46				0.072	0.035	0.019	0.011	0.004				
48				0.078	0.038	0.020	0.012	0.004				
50				0.085	0.042	0.022	0.013	0.005				
55				0.103	0.050	0.027	0.015	0.006	0.003			
60				0.122	0.060	0.032	0.018	0.007	0.003			
65					0.070	0.038	0.021	0.008	0.004			
70					0.082	0.044	0.025	0.009	0.004			
75							0.028	0.011	0.005			
80							0.032	0.012	0.005	0.003		
85							0.037	0.014	0.006	0.003		
90							0.041	0.016	0.007	0.003		
95							0.046	0.017	0.008	0.004		
100							0.051	0.019	0.008	0.004		
105							0.056	0.021	0.009	0.005		
110							0.061	0.023	0.010	0.005	0.003	
115							0.067	0.025	0.011	0.005	0.003	
120							0.073	0.028	0.012	0.006	0.003	
125							0.079	0.030	0.013	0.006	0.003	
130							0.086	0.032	0.014	0.007	0.004	
135							0.092	0.035	0.015	0.008	0.004	
140							0.099	0.038	0.016	0.008	0.004	
145							0.106	0.040	0.018	0.009	0.005	0.003
150								0.043	0.019	0.009	0.005	0.003
160								0.049	0.022	0.011	0.006	0.003
170								0.055	0.024	0.012	0.006	0.004

6.9 PIPES AND PIPE CULVERTS

6.9.1 REFERENCES

Publications

The primary publications for understanding the principles for the analysis of pipes and pipe culverts are:

- *Design Charts For Open-Channel Flow*, Hydraulic Design Series No. 3 (HDS-3), FHWA 1973 (Reprint 1980)
- *Introduction to Highway Hydr*aulics, Hydraulic Design Series No. 4 (HDS-4), FHWA 1983 and 2001
- *Hydraulic Design of Highway Culverts*, Hydraulic Design Series No. 5 (HDS-5), FHWA, 2005
- *Hydraulic Charts for the Design of Highway Culverts*, Hydraulic Engineering Circular No. 5 (HEC-5), FHWA, 1980
- *Capacity Charts for the Hydraulic Design of Highway Culverts,* Hydraulic Engineering Circular No. 10 (HEC-10), FHWA, 1972
- *Drainage of Highway Pavements*, Hydraulic Engineering Circular No. 12, (HEC-12), FHWA, 1984
- *Hydraulic Design of Improved Inlets for Culverts*, Hydraulic Engineering Circular No. 13 (HEC-13), FHWA, 1972
- *Hydraulic Design of Energy Dissipaters for Culverts and Channels*, Hydraulic
- Engineering Circular No. 14 (HEC-14), FHWA, 1975
- *Urban Drainage Design Manual*, Hydraulic Engineering Circular No. 22, (HEC-22), FHWA, 2001

In addition to these references there are manuals available from the different pipe manufacturers, including the *Concrete Pipe Design Manual* (viewable online at http://www.concrete-pipe.org/techdata.htm) and the *Handbook of Concrete Culvert Pipe Hydraulics* by the American Concrete Pipe Association, *Modern Storm Sewer Design* by the American Iron and Steel Institute, and the design manual of the Plastics Pipe Institute. All of these give pipe size data, general hydrology and hydraulic design, structural design, durability and recommended installation methodology. Much of this information is not available in other publications.

Tables and Charts

Access to the following tables and charts are needed. Most of these can be found in the referenced publications.

- Defining Energy Concept in Pipe Flow
  - Figure III-8 and III-9, HDS-5
  - Figure 7-8, HEC-22
- Manning's Roughness Coefficient *n* for Pipes
  - Figure 6-27
  - Table 14, HDS-4
  - Table 4, HDS-5
  - Tables 3-4 and 7-1, HEC-22
- Friction Slope for Circular Pipe, Full Flow
  - Figure 6-28
  - Charts 53 and 54, HDS-3
- Hydraulic Elements of Circular Sections for Various Flow Depths
  - Figure 40, HDS-4
  - Charts 26 and 27, HEC-22
- Uniform Flow in Circular Sections Flowing Partly Full
  - Table B.2, HEC-14
- Velocity in Pipe Conduits
  - Charts 35 thru 51, HDS-3
- Velocity in Elliptical Pipes
  - Chart 74, HDS-3
- Critical Depth of Flow - Circular Pipes
  - Chart 4B, HDS-5
  - Appendix B, HEC-14
- Critical Depth for Elliptical Pipes
  - Chart 31B, HDS-5
  - Figure B.2, HEC-14
- Entrance Loss Coefficients
  - Table 12, HDS-5
  - Table 7-5b, HEC-22

- Headwater Depth for Concrete Pipe Culverts with Inlet Control
    - Chart 1B, HDS-5
    - Chart 28B, HEC-22
- Head for Concrete Pipe Culverts Flowing Full
    - Chart 5B, HDS-5
- Headwater Depth for C.M. Pipe Culverts with Inlet Control
    - Chart 2B, HDS-5
    - Chart 29B, HEC-22
- Head for Standard C.M. Pipe Culverts Flowing Full
    - Chart 6B, HDS-5
- Headwater Depth for C.M. Pipe-Arch Culverts with Inlet Control
    - Chart 34B, HDS-5
- Head for Standard C.M. Pipe-Arch Culverts Flowing Full
    - Chart 39B, HDS-5
- Headwater Depth for Oval Concrete Pipe Culverts with Inlet Control
    - Chart 29B, HDS-5
- Head for Elliptical Concrete Pipe Culverts Flowing Full
    - Chart 33B, HDS-5
- Allowable Fill Heights for RCP Pipe
    - DelDOT's web site
- Allowable Fill Heights for Round Corrugated Steel Pipe
    - DelDOT's web site
- Allowable Fill Heights for Pipe Arch Corrugated Steel Pipe
    - DelDOT's web site
- Allowable Fill Heights for Round Corrugated Aluminum Pipe
    - DelDOT's web site
- Allowable Fill Heights for Pipe Arch Corrugated Aluminum Pipe
    - DelDOT's web site
- Typical Cross-Sections Elliptical Concrete Pipe
    - Concrete Pipe Design Manual, Illustrations 5.1 and 5.3
- Minimum Slopes for Full Flow in Pipes at 3 ft/s
    - Figure 6-24
    - Table 7-7, HEC-22
- Determining Culvert Size with HEC-5
    - Figure 6-29
- Culvert Design Form
    - Figure 7-7, HEC-22

Computer Software

- WinHY-8 Hydraulic Design of Highway Culverts
- HEC-RAS
- NFF

6.9.2 HYDRAULIC DESIGN OF PIPE CULVERTS

Culverts addressed in this chapter are pipe culverts and are defined as a separate category in drainage design. Their primary function is to carry a watershed's peak discharge flow from one side of a roadway embankment to an acceptable downstream outfall. The important considerations in locating and designing a culvert are:

- Topography
- Debris potential
- Design storm frequency
- Allowable upstream water depth
- Allowable downstream water depth
- Allowable outfall velocity
- Minimum culvert flow velocity
- Risk of overtopping the roadway
- Geometric and safety criteria
- Erosion and sediment criteria

For a project, pipe culverts may be one or a combination of several designated materials depending upon the service life criteria for the project's highway functional classification, the project scope, location within the roadway section, environmental factors, manufacturer's installation requirements, and economics.

Culverts may be single or multiple barrels with the following shapes:

1. Box
2. Circular

3. Elliptical (for concrete pipes only, also called oval concrete pipes), and
4. Pipe arch

Factors that determine the performance, capacity and required culvert size include the following:

1. D = Height of inside barrel opening (ft)
2. HW = Headwater depth at the culvert entrance
3. AHW = Allowable headwater depth at the inlet permitted by design criteria or other safety considerations.
4. d_c = Critical depth (ft) is the depth of flow with minimum specific energy. The specify energy is the sum of the depth plus velocity head for a given discharge.
5. k_e = Entrance loss coefficient for full flow
6. L = Length of culvert
7. S_f = Head loss per length of culvert due to friction (ft)
8. n = Coefficient for surface roughness of the pipe wall (see Figure 6-27)
9. S_o = Slope of pipe
10. TW= Tailwater depth at the culvert outfall
11. V = Mean velocity at any depth (ft/s)

The design method for sizing pipe culverts is based on procedures in HEC-5, HEC-10 and other publications. The following discussion describes the basic concept for culvert design. Computer programs such as HEC-RAS, NFF and HY-8 automate the design methods. However, the designer should have an understanding of the basis used to develop the software.

When using any approved procedure, the results must be summarized to include the projected flow capacity, the pipe size, the required slope, discharge velocity and a comparison of the calculated values with the design criteria.

6.9.3 DESIGN CRITERIA

The design frequency is selected from Figure 6-1. The allowable headwater shall remain at least 6 inches below the edge of roadway embankment, preferably one foot. However, in flood plain areas, the headwater shall conform to the applicable regulations. Since the outlet velocity is invariably greater than the velocity in the natural channel, erosion control provisions for the outlet velocity should be used. Altering the hydraulic characteristics of the culvert can reduce the outlet velocity.

The allowable headwater and tailwater depths are significant factors in culvert design. Some the concerns inaccurately determining these heights are:

- Damage to upstream and downstream properties.
- Flooding of travel lanes.
- Relationship to low point(s) in the roadway profile.
- $HW / D \leq 1.5$
- Proposed/probable future land use affects on the drainage watershed.
- Possible damage to the roadway support structure.
- FEMA 100-yr flood plain regulations.
- Potential for debris clogging and possible bypassing of culvert to the sag point.
- Effect on wetlands and other environmentally sensitive areas.
- Stability of the downstream discharge area beyond the point of outfall.

Figure 6-29
Culvert Size Determination Using HEC-5

Step 1	Enter known data, i.e., Q, L, S_o, and *AHW*, in an acceptable culvert design form. For the initial trial, use the friction slope for $S_{o.}$	Step 6	If $TW \geq D$, $h_o = TW$. If $D > TW$, then $h_o = \frac{d_c + D}{2}$ or *TW*, whichever is larger. Obtain d_c from the charts.
Step 2	Using the principles of open channel flow; determine the design storm depth of flow; assume this is the *TW* unless downstream conditions control this depth.	Step 7.	Compare the two values of *HW*. The higher value indicates what the flow condition is, i.e., outlet or inlet control
Step 3	Select a trial size assuming inlet control, $HW/D = 1.5$; use appropriate *HW* nomograph. If $HW > AHW$, try larger size pipe until $HW < AHW$.	Step 8.	If inlet control prevails, the type of culvert, the size and entrance type being evaluated are correct. Other combinations of pipe materials and entrance types could be checked if appropriate
Step 4	Assume that outlet control prevails.	Step 9	The values of *HW* are compared with *AHW*. The culvert size is adjusted and reevaluated until $HW = AHW$
Step 5	Find *HW* using: $HW = h_o - LS_o$. See HEC-5 for procedure and chart references.		

6.9.4 DESIGN PROCEDURE SUMMARY

Flow Controls

Flow through the culvert is determined by the available energy differential between the inlet and outlet. Depending upon this relationship, the culvert may flow under two types of flow. The design must consider each type of flow to select a culvert type and size.

- **Inlet Control:** The culvert size, inlet geometry and headwater depth determine the flow capacity. Inlet control exists as long as water can flow through the pipe at a greater rate than it enters. This type of flow is not affected by downstream conditions nor the roughness, length and slope of the culvert. Primarily the barrel shape, cross-sectional area, inlet edge and headwater depth affect inlet control. Culverts flowing under inlet control will always flow partially full.

- **Outlet Control:** Outlet control exists as long as water can enter the culvert at a greater rate than the water can flow through it. Capacity is affected by energy losses beginning at the outlet with the backwater affect of the tailwater depth, proceeding upstream with the losses due to culvert barrel characteristics including the size, shape, type of material, roughness, slope and length. At the inlet, losses due to the inlet geometry and headwater depth affect the capacity. The culvert can flow either with partial or full flow.

The following is a brief overview of the two most commonly used methods for determining culvert sizes. One method is based on a series of culvert capacity charts and the other is based on a series of nomographs using inlet and outlet control as the primary parameters. These methods involve manually determining the required culvert size by trial and error. The designer may choose to use the HY8 Culvert Analysis Microcomputer

Program, HEC-RAS or other approved software.

The steps for design are easy and provide a reliable design and/or a check on computer generated designs. The methods are limited and do not address hydrograph routing or velocity concerns involving energy dissipators or outlet scour potential.

Critical depth is an important consideration in the design of culverts. For the available energy head, the maximum discharge occurs at critical flow and the depth of flow at this point is defined as the critical depth. A culvert under inlet control does not have the hydraulic losses associated with outlet control and has its maximum capacity at critical flow.

For inlet control the critical depth occurs in the barrel near the culvert entrance and normal flow and depth is reached before the outfall. Conversely under outlet control the critical depth occurs near the outfall and the tailwater depth becomes an important factor.

Capacity Chart Method - HEC-10

Define Design Data

1. Establish the design *Q*.

2. Approximate the length *L* of the culvert.

3. Select a roughness factor *n* and preliminary slope.

4. Determine the allowable headwater depth.

5. Determine the allowable outfall velocity based on the proposed erosion characteristics of the discharge point.

6. Select a trial culvert, including a barrel cross sectional shape and entrance type. Approximate the initial trial pipe size by dividing the allowable headwater depth by a factor of two.

Determine culvert size

Select the appropriate capacity chart for the trial culvert size. Following the chart directions, find the headwater depth using inlet control. If this exceeds the allowable headwater depth, then try a larger pipe. Conversely, if it is less than the allowable, then try a smaller pipe. Next compare the headwater depths for outlet and inlet control. The control with the higher headwater governs and is used to continue the design.

Determine outlet velocity

A. Outlet Control—Outlet velocity equals the flow divided by the flow cross sectional area at the outlet. If the outlet is not submerged, the flow area is usually based on a depth of flow equal to the average of the critical depth and the pipe's vertical height.

B. Inlet Control—The outlet velocity is calculated using Manning's formula. Capacity charts for solving this equation for full flow and partial flow values are found in the references.

Selection Report

An important step is to summarize the results showing the selected pipe size, pipe material, required headwater and outlet velocity. The design criteria shall be compared with the design results to ensure compliance.

Nomograph Method HEC-5

Define Design Data

1. Establish the design *Q*.

2. Approximate the length, *L*, of the culvert.

3. Select a roughness factor *n* and preliminary culvert slope.

4. Determine the allowable headwater depth.

5. Determine the allowable outfall velocity based on the proposed outfall conditions, i.e., erodible or non-erodible discharge point.

6. Select a trial culvert, including a barrel cross sectional shape and entrance type. Divide the allowable headwater depth by a factor of two for the initial trial size using the friction slope for S_o.

Find headwater for trial culvert

A. Inlet Control.

(1) Given the design *Q*, size and type of culvert, follow the use directions and the appropriate inlet control nomograph to find the *HW*.

(2) If the *HW* is greater than the allowable, then try another trial size until HW is acceptable for inlet control.

B. Outlet Control

(1) Given Q, size and type of culvert, and estimated tailwater depth (TW, ft), above the invert at the outlet for the design frequency flood condition in the outlet channel:

(a) Locate the appropriate outlet control nomograph for the type of trial culvert. Find the entrance loss coefficient, k_e.

(b) Find the head (H) following the instructions for using the nomograph.

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

$$HW = H + h_o - S_o L \tag{6.72}$$

(3) For tailwater elevations less than the top of the culvert at the outlet, use $h_o = d_c + D/2$ or TW, whichever is the greater, where d_c (the critical depth in feet) is determined from the appropriate critical depth chart.

C. Compare the headwaters previously found for inlet and outlet control. The higher headwater governs in determining the flow control for the given conditions and the selected trial culvert.

D. If HW is higher than acceptable, select a larger trial culvert size and find HW as previously described.

Determine outlet velocity

A. Inlet control — Determine the outlet velocity using Manning's equation.

B. Outlet control — The outlet velocity is equal to the discharge flow divided by the flow cross sectional area at the outlet.

Selection Report

Record the results including the selected culvert size, type, headwater depth, and outlet velocity. Where necessary, compare with the design criteria and indicate whether or not the criteria have been met.

6.10 STORMWATER MANAGEMENT

The goal of stormwater management is to restrict the peak rate or volumetric rate of stormwater runoff after the project area is developed to the same as or less than it was originally, and to preserve or improve the quality of the runoff.

Restricting the post-development peak flow rate reduces the potential for increased downstream erosion and flooding. Typically this quantity control is accomplished using wet ponds or infiltration. These ponds are designed to provide sufficient storage volume so that a restrictive outlet will release no more than the pre-development rate. Stormwater ponds are also used for stormwater quality management with sediment forebays and permanent non-draining pools that encourage the deposition of runoff-borne pollutants, most of which are either sand sized or are bonded to sand sized particles. This allows the removal of an estimated 80% of generated pollutants by controlling the flow rate and letting them naturally settle to the bottom.

If a higher degree of treatment is required, specialized designs are available but they are expensive and generally require extensive, regular maintenance.

DelDOT's Stormwater Engineer is the responsible authority for ensuring a project's stormwater management control and erosion control features are adequate and in compliance with the latest policies, rules and regulations. This section is a general discussion of these subjects to inform and supplement, but not replace any approved design guides, manuals, memorandums of agreement, or other requirements.

Sections 6.10, 6.11 and 6.12 are an overview of DelDOT's *ES_2M Design Guide*, DNREC's *Green Technology: The Delaware Urban Runoff Management Approach*, *DURMM: The Delaware Urban Runoff Management Manual*, and NRCS's *Pond Code 378*. In addition, there are Internet web sites with extensive information on stormwater management concepts and design methodologies.

In stormwater management, a commonly used term is Best Management Practices (BMP's). BMP's are policies, practices, procedures or structures used to mitigate the adverse impacts on surface water quality resulting from changes in the watershed. BMP's are subject to the available area and site conditions for implementation.

One stormwater management strategy is Low Impact Development (LID). LID emphasizes conservation and the use of on-site natural features integrated with engineered, small-scale hydrologic controls in order to more closely mimic the pre-development hydrologic function of the site. LID strategies and techniques can be used to reduce the impact of runoff on the receiving waters. Some LID principles include:

- Maximize retention of native forest cover and restore disturbed vegetation to intercept, evaporate and transpire precipitation.
- Retain and incorporate topographic features and patterns.
- Create a hydrologically rough landscape that slows storm flows and increases the time of concentration.
- Provide multiple or redundant LID measures in order to increase reliability.

6.10.1 REFERENCES

PUBLICATIONS

- *ES_2M Design Guide*, DelDOT
- *Green Technology: The Delaware Urban Runoff Management Approach*, DNREC
- *Delaware Erosion & Sediment Control Handbook*, DNREC
- *Hydraulic Design of Highway Culverts*, Highway Design Series 2 (HDS-2), FHWA
- *Design of Riprap Revetment*, Hydraulic Engineering Circular 11 (HEC-11), FHWA
- *Hydraulic Design of Energy Dissipaters for Culverts and Channels*, Hydraulic Engineering Circular 14 (HEC-14), FHWA
- *Design of Roadside Channels with Flexible Lining*, Hydraulic Engineering Circular 15 (HEC-15), FHWA
- *Hydrology*, Highway Engineering Circular 19 (HEC-19), FHWA
- HEC-22, *Urban Drainage Design Manual Second Edition*
- *Open-Channel Hydraulics*, Ven Te Chow, 1959
- *National Engineering Handbook*, NRCS
- Various State DOT sites i.e. VDOT, MDOT, WSDOT and IDOT

COMPTER SOFTWARE

- Delaware Urban Runoff Management Model (DURMM)
- HEC-2: Floodplain Determination
- TR-20: Large Drainage basin hydrology
- TR-55: Hydrology, including hydrograph generation
- FLOWMASTER: Hydraulic design for various cross-sections such as trapezoidal, v-shape, rectangular, and closed pipes.
- PondPack Hydraulics including pond routings
- HydroCad
- Other approved software on DNREC's Drainage and Stormwater web site

6.10.2 EROSION CONTROL AND STORMWATER MANAGEMENT

The regulations involving Stormwater Management, Erosion, and Sedimentation Control are intended to minimize the adverse impact of projects that change the existing natural environment as well as manmade features due to increased flow and degraded water quality.

Erosion control measures are used during construction to manage surface scour and washouts created by uncontrolled runoff through the construction site and the subsequent deposit of sediment and other pollutants on natural areas and adjacent properties. Permanent stormwater measures are frequently incorporated into the temporary erosion control plans.

Typically, DelDOT projects affect the time of concentration, rate and volume of stormwater peak runoff from the site due to an increase in the amount of impervious surfaces such as a larger roadway pavement section, sidewalks, and concrete gutters.

Erosion control practices include diversion of water entering the site by the construction of diversion berms and swales on the uphill side to route water around or away from the work area; and/or piping clean water all the way through the project and discharging below the work area. Limitations on total work area allowed to be stripped at any time, and stabilized construction entrances are also a form of erosion control, as is frequent interim mulching, and permanent post construction vegetative stabilization.

6.10.3 SEDIMENT AND STORMWATER MANAGEMENT

Sediment control involves the controlled deposition and removal of materials eroded during various construction activities. Sediment control practices include placing silt fencing at designated locations on the construction site; sediment traps; sediment basins; stone check dams in the flow lines of swales and ditches; and filters around storm drain inlets (catch basins). Early construction phasing of permanent features such as wet and dry stormwater ponds with sedimentation forebays, riprap-stilling basins at culvert outlets, biofiltration swales, and vegetative filtration strips are also used. All permanent facilities are cleared, cleaned and reconstructed as necessary at the end of construction activities.

Stormwater management, erosion, and sediment control, are highly interrelated. DNREC and DelDOT combine these elements into one set of plans termed a Sediment and Stormwater Management Plan that accompanies the Stormwater Management Report.

6.10.4 GENERAL CRITERIA

The following is based on the Delaware Sediment & Stormwater regulations.

- Stone check dams are used in swales, ditches and channels to control velocities and erosion until sufficient cover has been established. .
- Outlet protection is required at all discharge points of pipes, channels and spillways.
- Erosion control matting is required on slopes of 3:1 or greater and all concentrated flow locations.
- Sediment traps and basins shall be utilized and sized to accommodate 3600 ft^3 of storage per acre of contributing drainage area until project stabilization is complete. These structures shall be located at the base of the drainage area. The required minimum length to width ratio is 2:1.
- Drainage calculations provide pre- and post-development velocities, peak rates of discharge, and inflow and outflow hydrographs of stormwater runoff at all existing and proposed points of discharge from the site for the 2-yr and 10-yr and 100-yr frequency storm.
- Hydrological data is to be developed using TR-20 or TR-55 with a storm duration based on the 24-hour rainfall event. For projects south of the C&D Canal, the Delmarva Unit Hydrograph shall be used.
- All ponds are to be designed and constructed in accordance with approved version of NRCS Small Pond Code 378.
- For water quality, ponds are sized to store the runoff from a 2-yr storm with a maximum of 1 inch of runoff released over a 24-hour period.
- Post-development peak rates of discharge for the 2-yr and 10-yr (and the 100-yr frequency storm events for projects north of the C&D Canal) and in some cases post-development volumetric rate shall not exceed the pre-development peak rates of discharge based on the same storm frequency criteria.

- Set aside areas for disposal of sediment removed from the stormwater management facilities must be provided. These areas are to be large enough to accommodate at least 2% of the stormwater management facility volume to the elevation of the 2-yr storage volume elevation, have a maximum depth of one foot, and have a slope not to exceed 5%.
- Wet ponds are to have a forebay to act as a sediment trap and two ten foot (10 ft) wide safety benches, one at one foot above and the other at one foot below the normal pool elevation.

6.10.5 PROJECT DESIGN AND REVIEW CHECKLIST

DelDOT's *ES$_2$M Design Guide* and the *Delaware Erosion & Sediment Control Handbook*, are the primary references for preparing the Erosion, Sediment and Stormwater plans. An approved checklist is submitted to the Stormwater Engineer at each plan review stage for any project disturbing over 5,000 square feet of land. A completed checklist is required for each plan submission. The items that do not apply to the project shall be marked "N/A".

6.11 STORMWATER QUANTITY MANAGEMENT

6.11.1 REFERENCES

For publications and computer software, see Section 6.10.

6.11.2 GENERAL

Stormwater quantity control is based on the requirement that the peak rate of discharge from an area after development cannot exceed the peak rate of discharge from that site before development. Both infiltration techniques and stormwater detention pond with a restrictive outlet structure achieves this goal. The measures to control stormwater quantity also can be valuable in maintaining the quality of project runoff and are usually designed concurrently. One such measure is a wet pond that serves both purposes. The permanent pool of water allows for filtration of contaminants, groundwater recharging, creation of wildlife habitat, and other environmental benefits.

Since much of stormwater design is generated through computer analysis, there are several practical considerations, including the following:

1. Field review of the site to ensure the stormwater can physically and economically be directed to the selected location. Just designating an area on the plans is not adequate without a field check.
2. Locate the water table to ensure the pond will have stormwater capacity. If the pond will be normally filled to capacity with groundwater then it will provide little detention value.
3. Take soil borings to ensure the site can be economically excavated or at least have the proper excavation bid items, e.g., rock excavation.
4. Ensure there are provisions for periodic maintenance, e.g., access.

Another important consideration is that many stormwater management facilities are in or adjacent to developed areas and may not be considered a community asset. Therefore, the design should consider aesthetics and safety. Ponds become an attraction particularly when they are flowing full. It is important that the outlet structure be designed for the safety of those who may use it for recreation. The most economical or hydraulically efficient design may not be the safest alternative for the site conditions.

A landscape plan to accompany each pond design is desirable. Woody vegetation, shrubs, or trees planted on the embankment are not allowed. Fencing is discouraged because of maintenance and perceived safety issues. Methods of vegetating ponds vary.

6.11.3 POND DESIGN

Stormwater ponds fill with water when the rate of inflow exceeds the rate of outflow. The volume of water that is temporarily stored during a runoff event can be expressed graphically as the area between the inflow and outflow hydrographs. Figure 6-30 outlines the steps to design a wet pond.

6.11.3.1 POND SIZING

The hydraulic analysis of detention ponds generally involves the following steps:

Step 1. Define design criteria. Determine the required storm frequency events and other criteria as mandated by DNREC. In addition, follow the design criteria based on the functional highway classification as defined in Figure 6-1.

Step 2. Select sites. Based on the topography of the project area select feasible preliminary pond locations. Select these locations based on: proximity to the project; an acceptable outfall location to drain the water (such as a natural stream); adequate space to construct a structure; moderate to flat slopes so grading steep slopes doesn't consume all the space; suitable geotechnical conditions; outside floodplains and wetlands; etc. Other considerations include setting tentative elevations for the bottom, permanent pool level, principal spillway including the various storm frequency structure(s), and the emergency spillway level.

Step 3. Determine peak flows. Using TR-55, TR-20 or another approved version of the NRCS Soil Cover Complex Method, determine the peak rates of runoff for existing conditions (pre-development) from the drainage areas contributing to each selected pond location. These rates cannot be exceeded by post-construction flows.

Step 4. Determine trial pond size. As a design tool, TR-55 has a manual method of determining an initial trial pond size for the designer to assess possible pond location. The method estimates the storage volume required to allow for the control of the peak outflow discharge. The pre- and post-development flows as developed by TR-55 and using TR-55 Figure 6-1 allows the designer to quickly estimate the storage volume. This value is then geometrically related to the available area and depth at a selected site.

Step 5. Fit pond to available location. Sketch the trial pond on the plans to determine its limits and if it is viable. Establishing the bottom elevation of the pond is the starting point. Selecting this elevation is based on hydraulics and geotechnical considerations. Hydraulic limitations involve locating and setting the invert of a positive discharge outfall such as an existing stream or ditch, figuring back to the pond location at a nominal 2% slope. The pond has to be able to drain, so the outlet elevation has to be above the discharge point. Geotechnical considerations include an analysis of the types of soil or rock that may be encountered. Excavating unsuitable material or rock is expensive. The determination of the normal groundwater level at the proposed pond location is extremely important. First, the storage capacity will be decreased by the continual inflow of groundwater. Second there could be a need to protect the existing aquifer from contamination. This also is not inexpensive.

Depending upon the design, additional space may be required for sedimentation forebays or safety benches. For the design of small to moderate ponds (most serve only a few acres in area), allowing an extra 30 to 50% of surface area would be appropriate for initial sizing. Current regulations may also require buffer areas around the pond for access or wetlands protection.

The grading for the pond involves embankment slopes typically 4:1, but no steeper than 3:1. The pond shape is an important part of its function. Irregular shapes with ratios of length to width of at least 2 to 1 are preferred. For safety purposes, there are two 10 ft wide safety benches located one foot above the normal pool and one foot below the normal pool level. The upper bench can be eliminated if the side slopes are 4:1 or flatter. The recommended depth for a wet pond is 3 to 6 ft. The top of the embankment should be 10 ft wide. A 10 ft buffer for access around the toe is preferred. The design top elevation should allow for future embankment settlement.

Obviously ponds consume considerable land area and early site selection is very important.

Step 6. Verify the trial pond size. The size approximated in Steps 4 and 5 must be checked through the use of inflow and outflow hydrographs, analyzing the discharge capacity of alternative outflow structures and routing the various storm flows through the pond. Because several design storms and types of outfall structures must be routed through the pond, approved stormwater management software is used to generate the data. The input data for the software must comply with the current regulations.

The process is to develop stage-storage relationships using the estimated available storage at various pond elevations, then determine what type of spillway is the best outlet structure. Two typical types are a rectangular concrete box or concrete wall with assorted weirs and orifices to control any water quality flow as well as several storm frequency flows. Computer software quickly determines the discharge characteristics of most of the combinations. The discharge characteristics of the outlet structure constitute the stage discharge curve. Size the outlet structure for water quality in accordance with the current DNREC regulations.

Computer software combines the stage-storage curve and the stage-discharge curve based on their common factor stage. The inflow hydrograph is then routed through the pond for each of the required frequencies.

The routed outflow peak rate is compared with the pre-development peak rate or other mandated criteria. Currently, it must be equal to or less than existing for all frequencies. If the routed outflow is greater or considerably less then allowed, then a change in the pond size and/or outlet structure is evaluated. Since it is recognized that there is a rather large error (25% for TR-55) in developing stormwater management volumes, downsizing ponds should not be done without careful study of both the input and generated data.

Step 7. Check emergency spillway. The maximum stage (elevation) reached in the pond during the storm routing (particularly the 100-yr) is also an important part of the analysis. Regulations require one foot (or more) of freeboard (height remaining) at peak stage. If the freeboard is less than one foot, regrading the pond is needed either by raising the embankment or regrading the pool to increase the storage, lowering the detention elevation. Sometimes the pond is so small that the water overtops the dam; this is unacceptable and must be remedied by regrading to make the pond considerably larger, by about a third as a first approximation. If the maximum routed elevation is significantly under the freeboard requirement, a reduction in pond size might be appropriate.

Step 8. Structural review and hazard classification. After the final embankment details are determined, a structural review and hazard classification as to the potential damage that might occur due to a major breach or failure of the structure is performed. The analysis is done using national dam safety standards based on the ultimate land use that may occur during the life of the structure.

6.11.3.2 TYPES OF OUTLET STRUCTURES

The effectiveness of a storage basin is very dependent upon the outlet structure. An outlet structure consists of the principal spillway with provisions for emergency overflow. The principal spillway should have the capacity to convey the design discharge to predevelopment levels as mandated by DNREC from the storage basin over a 24-hr duration without allowing the flow to pass through the emergency spillway.

Principal spillways include a combination of weirs, orifices, drop inlets, and pipes. The typical principal spillway consists of either a weir that controls the flow over the embankment or a multistage riser structure functioning as a drop inlet with one or more weirs and outlet conduits that are designed to control one or more selected storm flows. The multi-stage riser system uses separate openings of various sizes placed at different elevations to control the rate of discharge from the watershed for the specified design storms.

The type of spillway depends upon topography, the size of the storage basin, and the drainage characteristics of the downstream conveyance system. The principal spillway is sized to carry the design storm without allowing flow to enter the emergency spillway using extended detention devices.

If site conditions do not allow for an emergency spillway, then it would be necessary to design the principal spillway to carry the 100-yr storm flow without overtopping the facility. Sizing an outlet structure is based on hydrologic routing calculations as well as the potential threat to downstream life and property. The crest elevation of the principal spillway must be at least 1.0 ft below the crest elevation of the emergency spillway.

The placement of emergency spillways should be on original ground and avoid downstream features such as residences and other facilities.

6.11.3.3 OUTLET HYDRAULICS

A stormwater management facility has an outlet system that controls water flow and quality. The hydraulic design depends on whether the storage basin is intended to be dry or permanently wet. Frequently it is intended that the facility function as an extended detention system that provides control over both water quantity and water quality. This means it allows the pre-development flow to slowly be released over a 24-hr period into the outfall channel while a permanent pool of water is retained in the pond. Preferred design alternatives and criteria per current regulations should be verified before beginning the design to determine the method(s) of stormwater management. The three most commonly used outlets are discussed briefly in the following sections.

6.11.3.3.1 ORIFICES

Orifices are small openings in an outlet structure. For stormwater quality, a circular opening is typically provided to extend detention flow.

When the water rises above the top of the opening, the discharge through an orifice is determined using:

$$Q_o = C_o A_o \left(2gH_o\right)^{0.5} \qquad (6.73)$$

where

Q_o = Discharge through the orifice (ft^3/s)

C_o = Orifice entrance coefficient

A_o = Cross-sectional area of orifice (ft^2)

g = Acceleration due to gravity (32.2 ft/s^2)

H_o = Total head of water (ft), measured from water surface to the center of the orifice. If the orifice outfall is submerged, then H_o is the elevation difference between the headwater and tailwater surfaces.

C_o is a variable coefficient that depends upon the orifice diameter, edge shape and its wall thickness as follows:

- Sharp-edged orifice with walls thinner than the diameter: C_o = 0.60
- Sharp-edged orifice with walls thicker than the diameter: C_o = 0.80
- Rounded edge orifice: C_o = 0.92

6.11.3.3.2 CONCRETE WALL SPILLWAY WITH WEIR

Weirs are used in lieu of a riser/barrel system for shallow basin spillways. The weir outlet is a channel, not a pipe, which operates more efficiently with less maintenance and allows for greater opportunity for improving water quality. The top of the spillway weir wall is used to control the storage volume and the outlet flow. It is important that weir structures constructed on an embankment be protected from undermining and erosion.

Weirs are also commonly used in a concrete box riser system. (See Section 6.11.3.3.3.) The shape, size and number of the weirs may vary to control the runoff for various storm conditions. In addition, concrete box risers allow for the easy installation of stormwater quality orifices and, if necessary, an opening at the bottom to drain the pond.

Two common types of weirs are the broad crested weir and the V-notch weir.

The discharge over a broad crested weir is:

$$Q_w = C_w L_w H_w^{1.5} \qquad (6.74)$$

where

Q_w = Discharge over the weir (ft^3/s)

C_w = Broad-crested weir coefficient

L_w = Length of the weir (ft)

H_w = Head of water over the weir crest (ft)

C_w varies based upon a relationship between the width of the weir and the head measured at a designated point upstream from the weir. An average value is 3.1 or 3.3.

The discharge over a V-notch weir is given by the equation:

$$Q_{vw} = C_{vw} \tan\left(\frac{\theta}{2}\right) H_{vw}^{2.5} \qquad (6.75)$$

where

Q_{vw} = discharge (ft^3/s)

C_{vw} = weir coefficient, usually 2.5

θ = angle of the notch at the apex (degrees)

H_{vw} = total energy head (ft)

In the design of multi-stage risers, notched or rectangular weirs are frequently used to control the flow from one or more design years. When the depth of flow rises above the top elevation of the weir, the weir may be subject to orifice flow conditions. See Section 6.11.3.3.3.

6.11.3.3.3 DROP INLETS

Drop inlets are used in spillway design and may be square, rectangular or circular. The vertical portion of a drop inlet is referred to as a riser. The inlet is a horizontally positioned opening through which water enters, drops vertically through a shaft, and then flows through a horizontal pipe culvert (referred to as a barrel) and then discharged at the downstream outlet.

A drop inlet spillway may operate under three conditions depending upon the height of flow over the inlet. When the water elevation is just slightly above the top elevation, the inlet operates as a weir. When the water increases and begins flowing into the inlet from all directions, the flow actually becomes unstable and the inlet operates as an orifice. For a circular pipe riser, this flow condition usually occurs when the height of flow (*H*) is 1.2 to 1.5 times the diameter (*D*). Eventually, the head can be high enough that the riser flows full and operates as culvert flow. Normally the spillway design prevents this level of flow from occurring.

There is a point in orifice flow where vortex flow occurs, resulting in severe turbulence and reduced inlet capacity. This condition is to be avoided by ensuring the riser is large enough to operate under the weir flow condition, the barrel is at full flow with as low a head over the crest as practical, and before orifice flow develops.

Hydraulically the ideal flow condition occurs when the outlet conduit and riser are operating at full flow. The riser should operate at weir flow with the lowest head over the top as practical. In the design of the riser, determine the elevation over the riser when the flow transitions from weir flow to orifice flow and then make sure that the barrel controls the flow at that elevation. Since this transition point is difficult to accurately determine, it is usually sufficient to assume that the transition occurs where the weir flow equation value is equal to the orifice flow equation value.

If the H/D ratio is less than 1.5, conduits smaller than 12 inches in diameter are usually analyzed as a submerged orifice. The barrel is usually designed as a pipe culvert flowing full with inlet control using the procedure outlined in HDS-5.

The drop inlet spillway is treated as a combination of weir, orifice and pipe flow. Various levels of flow are analyzed using the weir and orifice equations. If the head is high enough (normally avoided), the drop inlet acts as a pipe and the flow is determined by the equation:

$$Q = A\left[\frac{2gH}{1 + k_b + k_e + k_f L}\right]^{0.5} \qquad (6.76)$$

where

H = difference between headwater and tailwater elevations (ft)

k_b = bend loss coefficient, use 0.6

k_e = entrance loss coefficient, use 0.5

k_f = barrel friction loss coefficient

L = length of pipe (ft)

The other variables are as previously defined.

Since multi-stage risers have to be analyzed based on several design storms, various spillway combinations and flow conditions, computer software is used. A stage-discharge curve is developed plotting the discharge versus head for each of the flow conditions. The minimum flow for any given head is the discharge used in continuing the evaluation of the selected spillway, pond size etc.

Example 7, Attachment B, contains a simplified pond design example.

6.12 STORMWATER QUALITY MANAGEMENT

6.12.1 GENERAL

The preferred methods of protecting and improving water quality are referred to as Green Technology BMP’s. DNREC’s *Green Technology: The Delaware Urban Runoff Management Approach* provides detailed discussion, design guidance and construction details for designing nonstructural BMP’s. These include the biofiltration, bioretention and infiltration design concepts. BMP's are most effective and require the least maintenance if designed as a system involving several types of BMP’s working together.

6.12.2 BIOFILTRATION

Biofiltration techniques are intended for improving post-construction water quality. There are two basic types of biofiltration devices: swales and filter strips. Swales are shallow ditches that carry flow, whereas filter strips are vegetative patches that intercept sheet flow. The design should emphasize biofiltration, rather than transporting flow with the greatest possible hydraulic efficiency. The design is therefore based on criteria that promote sedimentation, filtration, and other pollutant removal mechanisms.

Biofiltration Swale — Grass

This BMP is designed to convey stormwater at a non-erosive velocity while improving water quality through infiltration, sedimentation and filtration. Check dams can be used to slow the flow and create small temporary ponding areas to promote infiltration. This option is one of the least expensive as it involves no engineered filter fabric, imported pervious materials or outlet structures. The design considerations are channel capacity; erosion; a runoff velocity of 1 fps for the design storm; and a total length providing the desirable residence time. These swales usually involve a typical section that has a wider bottom width, flatter side slopes and denser vegetation than a normal open channel.

Filter Strips

Filter strips are relatively flat graded areas that are heavily vegetated. They are designed to treat runoff and remove pollutants primarily through vegetative filtration. They are not intended as a stand-alone solution but as part of a treatment system and/or pre-treatment for another BMP. Filter strips are effective in providing a buffer between incompatible land-uses, can be landscaped and made aesthetically pleasing, and provide groundwater recharge in highly pervious soil areas. Filter strips may be constructed to allow treatment of sheet flow directly from a site or natural areas that are left around drainage channels. The flow characteristics for natural areas are similar to shallow concentrated flow.

Constructed filter strips are more effective if the entering flow is spread out maintaining sheet flow onto the filter and a shallow ponding area is created through the use of berms.

6.12.3 BIORETENTION

Bioretention BMP's are structural stormwater controls that accept and temporarily store the water quality volume using special soils and vegetation in shallow basins or landscaped areas. Their function is to remove pollutions, not control runoff. Some advantages are they can be easily integrated into a site's landscape plan, are aesthetically pleasing and need low maintenance.

Bioretention Areas

Bioretention areas are engineered, landscaped, shallow stormwater basins. Stormwater collected in the upper layer of the basin is filtered through surface vegetation, a mulch layer, and a pervious soil layer, and then temporarily stored in a stone aggregate base layer. The water quality volume is drained from the aggregate base by infiltration into the underlying soil or/or removed to an outlet through a perforated pipe subdrain. These systems work best if they are combined with a perimeter grass filter strip or grass swale to reduce the sediment and velocity prior to entering the biorentention area.

6.12.4 INFILTRATION

Most BMP's partially rely on infiltration for their effectiveness. Stand-alone infiltration BMP's are designed specifically to control a selected volume of the runoff, retain it, and infiltrate all or part of it into the ground.

Infiltration depends upon the underlying soils to provide significant capacity to absorb the proposed flow. In addition, there must not be a possibility of adversely affecting the existing groundwater or aquifer. This technique is the most site-sensitive, most maintenance-intensive and most likely to fail of the BMP's. Because of the potential of clogging infiltration, BMP's are usually constructed after a site has been stabilized and has an established vegetative cover with an upstream BMP for removal of sediment. The two types of infiltration techniques are infiltration trenches and basins.

Infiltration Trenches

Since they occupy the smallest area infiltration trenches are the BMP most commonly used to treat the water quality volume and recharge the groundwater. These trenches are long, narrow and filled with a freely draining stone aggregate. The trapped water is filtered into the underlying soil. Normally there is no engineered outlet. In order to prolong their effectiveness and reduce maintenance there must be a positive sediment trap system above the site. Although the trenches can process fine sediment, coarse sediment can quickly clog the trench.

Infiltration Basin

If more area is available, an alternative is to provide an infiltration basin. Essentially this is a dry pond to trap water and discharge it into groundwater flow through infiltration. This concept uses heavy vegetation to control velocities and increase the percolation.

Infiltration techniques are more maintenance intensive and site specific. There must be highly pervious underlying soils, no rock and no potential for contaminating the existing groundwater. In addition, construction scheduling must be adjusted to allow for project completion prior to their installation.

Figure 6-30
Design Steps for a Wet-Extended Detention Stormwater Pond

Step 1	Determine hydrologic data for the site and confirm pre-developed flow rates.
Step 2	Compute runoff water quality volumes and compute release rates.
Step 3	Estimate the permanent pool volume and extended detention volume.
Step 4	Determine forebay requirements and preliminary locations and size(s).
Step 5	Prepare a preliminary grading plan for the stage-storage curve.
Step 6	Field review the project. Locate a site area and confirm a stormwater pond is practical using: the estimated pond size; configuration; forebay locations; maintenance access; set aside areas; and safety features.
Step 7	Design the water quality orifice or weir.
Step 8	Set the permanent pool volume and elevation.
Step 9	Size the 2-yr control orifice or weir.
Step 10	Check for performance of the 2-yr opening.
Step 11	Size the 10-yr control opening.
Step 12	Check the performance of the 10-yr opening.
Step 13	Perform a hydraulic analysis of the facility including the forebay, the pond, the riser flow control, the barrel inlet flow and the barrel outlet.
Step 14	Size the 100-yr release opening or emergency spillway.
Step 15	Check total discharge and performance of the 100-yr control opening.
Step 16	Design any required outlet protection as per HEC-14.
Step 17	Perform any required buoyancy calculations.
Step 18	Determine the required seepage control.
Step 19	Check final design of inlets, sediment forebays, outlet structures, maintenance access, and safety features making sure all criteria has been met.
Step 20	Based on the final grading and design, determine the pond hazard classification.
Step 21	Develop a landscape plan.

Steps 7 to 12 are used to determine the appropriate outlet structure and stage-discharge curve for the storm events as mandated by DNREC.

Chapter Six

ATTACHMENT A
DRAINAGE DESIGN AIDES

GENERAL

DELDOT DESIGN CRITERIA

Design Frequency .... DelDOT Figure 6-1
Allowable Spread on Pavement Cross Section.. DelDOT Figure 6-2
Miscellaneous Design Criteria .... DelDOT Figure 6-3

RECOMMENDED PUBLICATIONS

- AASHTO - *Model Drainage Manual*
- DelDOT - *ES_2M Design Guide*
- DNREC
  - *Delaware Erosion & Sediment Control Handbook*
  - *Green Technology: The Delaware Urban Runoff Management Approach*
- FHWA
  - HDS-2 *Highway Hydrology* (FHWA-NHI-02-001)
  - HDS-3 *Design Charts for Open-Channel Flow* (FHWA-EPD-86-102)
  - HDS-4 *Introduction to Highway Hydraulics* (FHWA-NHI-01-019)
  - HDS-5 *Hydraulic Design of Highway Culverts* (FHWA-NHI-01-020)
  - HEC-2 *Water Surface Profiles Software and User's Manual*
  - HEC-11 *Design of Riprap Revetment* (FHWA-IP-89-016)
  - HEC-10 *Capacity Charts for the Hydraulic Design of Highway Culverts*, 1972
  - HEC-12 *Drainage of Highway Pavements*, 1984 (replaced by HEC-22)
  - HEC-14 *Hydraulic Design of Energy Dissipaters for Culverts and Channels*, 1975
  - HEC-15 *Design of Roadside Channels with Flexible Lining* (FHWA-IF-05-114)
  - HEC-19 *Hydrology*, 1985 (replaced by HDS-2)
  - HEC-22 *Urban Drainage Manual*, 2001 (FHWA-NHI-01-021)
- NRCS
  - *Urban Hydrology for Small Watersheds, Technical Release No. 55* (TR-55), 2003
  - *National Engineering Handbook*, 1997

RECOMMENDED INTERNET SITES

- DelDOT's Web Site (http://www.deldot.gov/) - Design Resource Center
- FHWA Hydraulics Engineering (http://www.fhwa.dot.gov/engineering/hydraulics)
- New Jersey Department of Transportation (http://www.state.nj.us/transportation/eng/)
- University of Wisconsin Cooperative Extension (http://learningstore.uwex.edu/) - Wisconsin Storm Water Manual: Grassed Swales (G3691-7)
- US Army Corps of Engineers Hydrologic Engineering Center (HEC) (http://www.hec.usace.army.mil)

- Virginia Department of Transportation (http://www.virginiadot.org) - Drainage Manual

RECOMMENDED SOFTWARE

- HY7 WSPRO (2001) - Gradually varied flow in open channels
- HY8 Culvert Analysis - Methodology and data found in HDS-5, HEC-14 and HEC-15
- HY22 Urban Drainage Design Program - Highway pavement drainage, open channel flow, critical depth solutions, storm stage-storage computations and reservoir routing
- TR-55 Urban Hydrology for Small Watersheds
- TR-20
- WSP-2

HYDROLOGY

REFERENCES

- FHWA
    - HDS-2 *Highway Hydrology* (FHWA-NHI-02-001)
    - HDS-4 *Introduction to Highway Hydraulics* (FHWA-NHI-01-019)
    - HEC-19 *Hydrology*, 1985 (replaced by HDS-2)
    - HEC-22 *Urban Drainage Manual* , 2001(FHWA-NHI-01-021)
- NRCS - TR-55 *Urban Hydrology for Small Watersheds, Technical Release No. 55*, 2003

DESIGN AIDES: FIGURES, NOMOGRAPHS AND TABLES

DelDOT's Criteria for Design Frequency .... DelDOT Figure 6-1
Frequency Factor, C_f .... Section 3.2.2.1, HEC-22
Delaware's Rainfall Intensity Estimates .... DelDOT Figures 6-5 thru 6-7
Runoff Coefficients .... DelDOT Figure 6-8; Table 5.7, HDS-2
Appendix B, Tables 11 and 12, HDS-4; Table 3-1, HEC-22
Velocities for Upland Method of Estimating T_{t1} and T_{t2} .... Figures 37 and 52, HEC-19
Figure 3-1, TR-55; USDA Hydrology Technical Note N4, Time of Concentration (1986)
Soil Group Descriptions for TR-55 Method .... DelDOT Figure 6-9
Hydrologic Soils Descriptions for TR-55 Method Common to Delaware .... DelDOT Figure 6-10
Delaware's 24-Hour Rainfall Depths .... DelDOT Figure 6-12
Runoff Curve Numbers for Urban Areas .... Table 5.4, HDS-2
Manning's Roughness Coefficients n (TR-55) for Sheet Flow .... Table 2.1, HDS-2
Table 3-4, HEC-22; Table 3-1, TR-55
Unit Peak Discharge (q_u) for NRCS Type II Rainfall Distribution .... Exhibit 4-II, TR-55
Adjustment Factor (F_p) - Pond & Swamp Areas Spread In Watershed .... Table 5.6, HDS-2
Table 3-9, HEC-22; Table 4-2, TR-55

OPEN CHANNEL FLOW AND ROADSIDE DITCHES

REFERENCES

- FHWA
    - HDS-3 *Design Charts for Open-Channel Flow* (FHWA-EPD-86-102)
    - HDS-4 *Introduction to Highway Hydraulics* (FHWA-NHI-01-019)

- HEC-11 *Design of Riprap Revetment* (FHWA-IP-89-016)
- HEC-14 *Hydraulic Design of Energy Dissipaters for Culverts and Channels*, 1975
- HEC-15 *Design of Roadside Channels with Flexible Lining* (FHWA-IF-05-114)
- HEC-22 *Urban Drainage Manual*, 2001 (FHWA-NHI-01-021)

- *Open-Channel Hydraulics*, Ven Te Chow, 1959

DESIGN AIDES: FIGURES, NOMOGRAPHS AND TABLES

Open Channel Flow Schematic .... Figures 6 and 7, HDS-4
Manning's Roughness Coefficients *n* for Open Channels .... Table 1, HDS-3
Tables 12 and 14, HDS-4; Tables 2.1 and 2.2, HEC-15
Formulas for Various Channel Geometrics .... Appendix B, HEC-15; Table 2-1, Chow
Nomograph for Solution of Manning's Equation .... Chart 83, HDS-3; Appendix C, Chow
Solution of Manning's Equation for Channels of Various Side Slopes .... Chart 16, HEC-12
Capacity of Trapezoidal Channels .... HDS-3 and HDS-4
Uniform Flow in Trapezoidal Channels by Manning’s Equation .... Table B.1, HEC-14
Geometric Design Chart for Trapezoidal Channels .... HDS-3; Appendix B, Chow
Trapezoidal Channel Geometry HDS-3
Maximum Nonscouring Ditch Grades with Grass Lining ...... DelDOT Figure 6-13; Tables 2-3, HDS-3
DelDOT’s Roadside Ditch Design Form .... DelDOT Attachment B
Permissible Velocities for Open Channels .... DelDOT Figures 6-14 and 6-15
K_b Factor for Maximum Stress on Channel Bends .... Chart 21, HEC-22
Correction Factor for Riprap Size .... Chart 2, HEC-11
Riprap Size Relationship .... Chart 1, HEC-11

PAVEMENT DRAINAGE AND STORM DRAINS

REFERENCES

- FHWA
  - HDS-3 *Design Charts for Open-Channel Flow* (FHWA-EPD-86-102)
  - HDS-4 *Introduction to Highway Hydraulics* (FHWA-NHI-01-019)
  - HEC-12 *Drainage of Highway Pavements*, 1984 (replaced by HEC-22)
  - HEC-22 *Urban Drainage Manual* , 2001(FHWA-NHI-01-021)

DESIGN AIDES: FIGURES, NOMOGRAPHS AND TABLES

Manning's Roughness Coefficients (n) for Pavements and Gutters HEC-22
Allowable Water Spread .... DelDOT Figure 6-2
Curb, Gutter and Inlet Selection .... DelDOT Standard Construction Details
Typical Gutter Sections .... Figure 4-1, HEC-22
Nomograph for Velocity in Triangular Gutter Sections .... Chart 4B, HEC-22
Average Water Spread Width in Triangular Gutter Section.... Chart 29, HDS-3; Table 4-4, HEC-22
Flow in Triangular Gutter Sections .... Chart 29, HDS-3; Chart 1B, HEC-22
Ratio of Frontal Flow to Total Gutter Flow .... Chart 2B, HEC-22
Flow in Composite Gutter Sections .... Chart 5, HEC-12
Gutter Flow Rate *Q* on Uniform Cross Slope (ft^3/s) .... DelDOT Web Site

Gutter Flow *Q* on Composite Section (ft^3/s) .... DelDOT Web Site
Average Gutter Flow Velocity for Uniform and Composite Sections .... DelDOT Web Site
Spread at Average Velocity in a Reach of Triangular Gutter .... Table 4-4, HEC-22
Frontal Flow Interception Factor, R_f .... Chart 5B, HEC-22
Side Flow Interception Factor, R_s .... Chart 6B, HEC-22
Curb-Opening and Slotted Drain Inlet Length for Total Interception .... Figure 35b, HDS-4
Chart 9, HEC-12
Curb-Opening and Slotted Drain Inlet Interception Efficiency .... Figure 36, HDS-4
Chart 10, HEC-12; Chart 8B, HEC-22
Grate Inlet Capacity in Sump Conditions .... Chart 11, HEC-12; Chart 9B, HEC-22
Distance to Flanking Inlets in Sag Vertical Curve .... Table 4-7, HEC-22
Depressed Curb Opening Inlet Capacity in Sump Conditions .... Chart 12, HEC-12
Chart 10B, HEC-22
Orifice Flow in Depressed Curb-Opening Inlet .... Chart 14, HEC-12; Chart 12B, HEC-22
Slotted Drain Inlet Capacity in Sump Locations .... Chart 15, HEC-12; Chart 13B, HEC-22
Ratio of Frontal Flow to Total Flow in Trapezoidal Channel .... Chart 17, HEC-12
Chart 15B, HEC-22
Inlet Clogging Factor of Safety .... DelDOT Figure 6-3
Minimum Slopes for Full Flow in Pipes at 3 ft/s .... DelDOT Figure 6-24; Table 7-7, HEC-22
Inlet Spacing Computation Form .... DelDOT Attachment B; Figure 4-19, HEC-22
Storm Drain Computation Table (Open Channel) .... DelDOT Attachment B; Appendix D, HEC-22
Hydraulic Gradeline Computation Table (Pressure Flow Design) .... DelDOT Attachment B
Appendix D, HEC-22

PIPES AND PIPE CULVERTS
REFERENCES

- FHWA
  - HDS-3 *Design Charts for Open-Channel Flow* (FHWA-EPD-86-102)
  - HDS-4 *Introduction to Highway Hydraulics* (FHWA-NHI-01-019)
  - HDS-5 *Hydraulic Design of Highway Culverts* (FHWA-NHI-01-020)
  - HEC-5 *Hydraulic Charts for the Design of Highway Culverts*, 1980
  - HEC-10 *Capacity Charts for the Hydraulic Design of Highway Culverts*, 1972
  - HEC-12 *Drainage of Highway Pavements*, 1984 (replaced by HEC-22)
  - HEC-13 *Hydraulic Design of Improved Inlets for Culverts*, 1972
  - HEC-14 *Hydraulic Design of Energy Dissipaters for Culverts and Channels*, 1975
  - HEC-22 *Urban Drainage Manual* , 2001(FHWA-NHI-01-021)

In addition to the above references there are manuals available from various pipe manufacturers, including the *Concrete Pipe Design Manual (CPDM)* (viewable online at http://www.concrete-pipe.org/techdata.htm) and the *Handbook of Concrete Culvert Pipe Hydraulics* by the American Concrete Pipe Association; and *Modern Storm Sewer Design* by the American Iron and Steel Institute. These give pipe size data, general hydrology and hydraulic design, structural design, durability, and recommended installation methodology. Much of this information is not available in other publications.

DESIGN AIDES: FIGURES, NOMOGRAPHS AND TABLES

Energy Concept in Pipe Flow............................Figure III-8 and III-9, HDS-5; Figure 7-8, HEC-22
Manning's Roughness Coefficients *n* for Pipes .... DelDOT Figure 6-27; Table 14, HDS-4
Table 4, HDS-5; Tables 3-4 and 7-1, HEC-22
Friction Slope for Circular Pipe, Full Flow..Charts 53 and 54, HDS-3
Circular Section Hydraulic Elements at Various Flow Depths .... Figure 40, HDS-4;
Charts 26 and 27, HEC-22
Circular Pipe Conveyance ....Table 3, *CPDM*
Flow for Circular Pipe Flowing Full ....Figures 2 thru 5, *CPDM*
Flow for Horizontal Elliptical Pipe Flowing Full...Figures 6 thru 9, *CPDM*
Uniform Flow in Circular Sections Flowing Partly Full .... Table B.2, HEC-14
Velocity in Pipe Conduits...Charts 35 thru 51, HDS-3
Velocity in Elliptical Pipes..Chart 74, HDS-3
Solution of Manning's Equation for Flow in Storm Drains .... Chart 25B, HEC-22
Flow for Circular Pipe Flowing Full ....*CPDM*
Flow for Horizontal Elliptical Pipe Flowing Full...*CPDM*
Flow Coefficients for Circular Pipe..*CPDM*
Uniform Flow for Pipe Culverts.. Del DOT Web Site
Uniform Flow for Concrete Elliptical Pipes.. Del DOT Web Site
Critical Depth of Flow for Circular Conduits.........................Chart 4B, HDS-5; Chart 27B, HEC-22
Critical Depth for Elliptical Pipes .... Chart 31B, HDS-5
Energy Loss Coefficients .... Table 12, HDS-5; Table 7-5b, HEC-22
Culvert Design Form .... Figure 7-7, HEC-22
Headwater Depth for Concrete Pipe Culverts with Inlet Control............................ Chart 1B, HDS-5
Chart 28B, HEC-22
Head for Concrete Pipe Culverts Flowing Full .... Chart 5B, HDS-5
Headwater Depth for C.M. Pipe Culverts with Inlet Control................................. Chart 2B, HDS-5
Chart 29B, HEC-22
Head for Standard C.M. Pipe Culverts Flowing Full .... Chart 6B, HDS-5
Headwater Depth for C.M. Pipe-Arch Culverts with Inlet Control....................... Chart 34B, HDS-5
Head for Standard C.M. Pipe-Arch Culverts Flowing Full.................................. Chart 39B, HDS-5
Headwater Depth for Oval Concrete Pipe Culverts with Inlet Control .... Chart 29B, HDS-5
Head for Elliptical Concrete Pipe Culverts Flowing Full..................................... Chart 33B, HDS-5
Allowable Fill Heights for RCP Pipe .... DelDOT Web Site
Allowable Fill Heights for Round Corrugated Steel Pipe .... DelDOT Web Site
Allowable Fill Heights for Pipe Arch Corrugated Steel Pipe................................ DelDOT Web Site
Allowable Fill Heights for Round Corrugated Aluminum Pipe............................ DelDOT Web Site
Allowable Fill Heights for Pipe Arch Corrugated Aluminum Pipe .... DelDOT Web Site
Typical Cross-Sections Elliptical Concrete Pipe.............................CPDM, Illustrations 5.1 and 5.3
Elliptical Concrete Pipe Characteristics .... Figure 7-2, HEC-22

STORMWATER QUANTITY MANAGEMENT - DETENTION PONDS

REFERENCES

- DNREC
  - *Delaware Erosion & Sediment Control Handbook*
  - *DURMM: The Delaware Urban Runoff Management Model*
    - *Technical Manual - Green Technology: The Delaware Urban Runoff Management Approach*
    - *User's Manual*
    - *Computer Program*
- DelDOT - *ES$_2$M Design Guide*
- FHWA
  - HDS-2 *Highway Hydrology* (FHWA-NHI-02-001)
  - HEC-11 *Design of Riprap Revetment* (FHWA-IP-89-016)
  - HEC-14 *Hydraulic Design of Energy Dissipaters for Culverts and Channels*, 1975
  - HEC-15 *Design of Roadside Channels with Flexible Lining* (FHWA-IF-05-114)
  - HEC-19 *Hydrology*, 1985 (replaced by HDS-2)
  - HEC-22 *Urban Drainage Manual* , 2001(FHWA-NHI-01-021)
- NRCS
  - *Engineering Field Handbook*, 1984
  - *National Engineering Handbook*, 1997
  - Pond Code 378
  - *Urban Hydrology for Small Watersheds, Technical Release No. 55* (TR-55), 2003
- *Open-Channel Hydraulics,* Ven Te Chow (1959)
- *City of Tacoma Surface Water Management Manual*
- *Low Impact Development——Technical Guidance Manual for Puget Sound*
- Various Internet sites
- *TP-61 Handbook of Channel Design for Soil & Water Conservation*
- City of Tacoma Surface Water Management Manual Volume V Runoff Treatment BMP's
- "Sand Filter Design for Water Quality Treatment" Shaver, E., and Baldwin, R., 1991, Stormwater Conference Proceedings, Mt. Crested Butte, CO, ASCE, Washington, D.C. (Also see "A Catalog of Ultra Urban Best Management Practices Using Sand Filters" Bell, W., 1992, City of Alexandria, Alexandria, VA.)
- Grass Swales: "Biofiltration Systems for Storm Runoff Water Quality Control" Horner, R., 1988, Washington State Department of Ecology, Seattle, WA.
- Website link - http://lowimpactdevelopment.org

DESIGN AIDES: FIGURES, NOMOGRAPHS AND TABLES

Chapter Six

ATTACHMENT B
EXAMPLE PROBLEMS

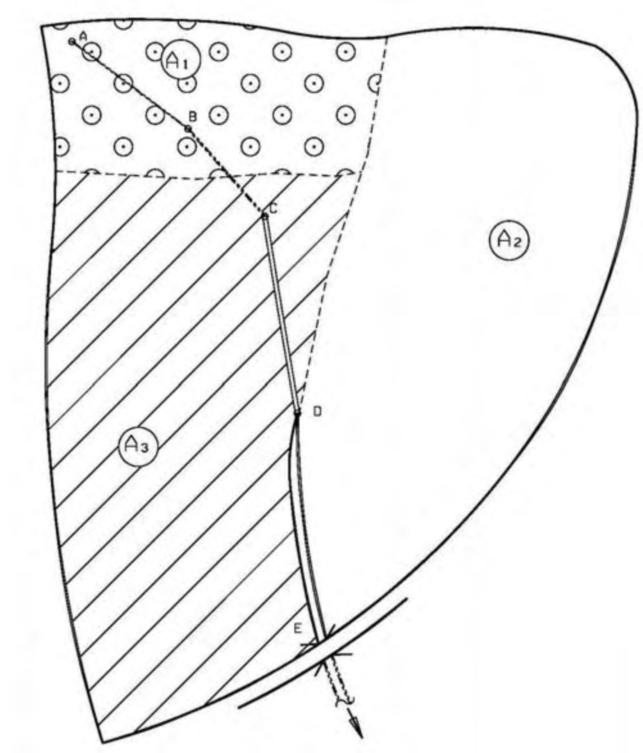
Introduction: These example problems are being presented to show the basic steps for drainage design. It is expected that the designer will use current available software and spreadsheets as much as possible to facilitate drainage calculations. To ensure compliance with the criteria and expedite review, the designer should ensure that all final calculations, size selections, water spread, etc., are compared with the Department's approved drainage criteria and are highlighted in the summary and/or within the drainage report itself.

EXAMPLE 1─RATIONAL METHOD

PROBLEM STATEMENT

The watershed area in Figure 6B-1 has a composite watershed of 36.5 acres and drains to a culvert at E on a local road in Kent County. The surface characteristics of the watershed and the stream flow path are as shown in figures 6B-2 and 6B-3. The design problem is to determine the peak discharge at point E for a 25-yr storm frequency using the Rational Method.

Figure 6B-1
Watershed for Rational Method Example



Solution

Step 1. Determine t_c

Figure 6B-2
Flow Path of Watershed

Segment	Description	Slope %	Length (ft)
A-B	Residential lawn	1.0	50
B-C	Forest and meadow	2.0	50
C-D	Shallow gully	0.8	250
D-E	Open channel	0.8	475

Compute the sheet flow time, T_{t1}*, for segments A-B and B-C.

Use equation 6.4:

$$T_{t1} = \frac{0.42(nL_1)^{0.8}}{P_{24}{}^{0.5} s^{0.4}}$$

where:

Segment A-B	Segment B-C
n = 0.15 from Table 3-2, HEC-22	n = 0.30 from Table 3-2, HEC-22
L_1 = 50 ft	L_1 = 50 ft
P_{24} = 3.3 from Figure 6-11	P_{24} = 3.3 from Figure 6-11
s = 0.01	s = 0.02

$$T_{t1} = \frac{0.42[(0.15)(50)]^{0.8}}{(3.3)^{0.5}(0.01)^{0.4}} + \frac{0.42[(0.30)(50)]^{0.8}}{(3.3)^{0.5}(0.02)^{0.4}} = 7.31 + 9.65 = 16.96 \min$$

*Note: A great deal of the literature indicates that sheet flow should be limited to 100 ft.

Compute the shallow concentrated flow time, T_{t2} for segment C-D. Use the Shallow (Gutter) Gully Flow Line of Figure 6B-2 and 0.8% watercourse slope. From Figure 37, HEC-19, V_2 = 1.35 f/s. Using equation 6.5 and V_2 = 1.35 ft/s

$$T_{t2} = \frac{L_2}{60 V_2} = \frac{250}{60 \ x \ 1.35} = 3.08 \ minutes$$

Compute the open channel flow time, T_{t3}, for segment D-E. From survey data, the small stream has a base width (*B*) of 3 ft with 4:1 side slopes (*z* = 4), banks approximately 2 ft high (d), and a streambed slope of 0.008 ft/ft. This meandering channel has some weeds with light brush. Therefore, *n* = 0.045 (Table 1, HDS-3, or Table 3-4, HEC-22).

The streambed cross sectional area (*A*) and wetted perimeter (*P*) from Figure 5-6, HEC-22:

$$A = 3 \ x \ 2 + (4 \ x \ 2^2) = 22 \ ft^2$$

$$P = 3 + 2 \ x \ 2\sqrt{4^2 + 1} = 19.49 \ ft$$

Therefore using equation 6.7a:

$$R = \frac{A}{Pw} = \frac{22}{19.49} = 1.13 \, ft$$

With s = 0.008, *n* = 0.045 and *R* = 1.13, then V_3 = 3.21 ft/s from Chart 83, HDS 3, or equation 6.7:

$$V = \frac{1.49}{n} R^{0.67} s^{0.5}$$

$$V_3 = \frac{1.49}{0.045}(1.13)^{0.67}(0.008)^{0.5} = 3.21$$

$$T_{t3} = \frac{L_3}{60V_3} = \frac{475}{60x3.21} = 2.47 \min$$

Using equation 6.8:

$$t_c = T_1 + T_2 = T_3 = 16.96 + 2.31 + 2.47 =$$

21.74 rounded to 22 min

Step 2. Determine I

Using Figure 6-6, for t_c=22 minutes (duration) and 25-yr frequency, I =4.06 in/hr.

<u>Step 3.</u> Determine C_f

From HEC-22, C_f=1.10 for a 25-yr frequency.

Step 4. Determine the weighted average runoff coefficient using Figure 6B-2, Equation 6.2 and runoff coefficients for the respective ground characteristics (Figure 6-8).

$$C = \frac{\sum C_x A_x}{A_{total}}$$

$$C = \frac{0.35\ x\ 6.0 + 0.40\ x\ 16.9 + 0.25\ x\ 13.6}{6.0 + 16.9 + 13.6}$$

C = 0.34

Figure 6B-3
Surface Characteristics of Watershed*

Area (ac)	Description	Runoff Coefficient
A_1 = 6.0	Residential, single family, large lots	0.35
A_2 = 16.9	Residential, single family, small lots	0.40
A_3 = 13.6	Woods, light vegetation, loam soil	0.25

*Assumed for this example.

Step 5 Calculate Q_{25}, peak discharge, by using Equation 6.1.

$$Q_{25} = C_f\,(CIA) = 1.10x0.34\ x\ 4.46x36.5$$

= 60.9, rounded to 61 ft^3/s

Useful formulas for various channel shapes are as follows. (Also see Appendix B, HEC-15 or Figure 5-6, HEC-22). z is dimensionless and represents the horizontal ditch slope rate, for a 2:1 slope z = 2; 3:1 slope z=3.

<u>V-Shape</u>

Symmetrical

$$A = zd^2$$

$$P = 2d\sqrt{z^2+1}$$

Unsymmetrical

$$A = \frac{(z_1+z_2)d^2}{2}$$

$$P = \left(\sqrt{z_1^2+1}+\sqrt{z_1^2+1}\right)d$$

<u>Trapezoidal</u>

Symmetrical

$$A = Bd + zd^2$$

$$P = B + 2d\sqrt{z^2+1}$$

Unsymmetrical

$$A = Bd + \frac{(z_1+z_2)d^2}{2}$$

$$P = B + d\left(\sqrt{z_1^2+1}+\sqrt{z_2^2+1}\right)$$

<u>Parabolic</u>

$$A = \frac{2}{3}Td$$

$$P = T + \frac{8d^2}{3T}$$

Approximation 0 $< d/T \le 0.25$

For $d/T \ge 0.25$ use:

$$P = \frac{1}{2}\sqrt{16^2+T^2} + \frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$$

$$T = 1.5\frac{A}{d}$$

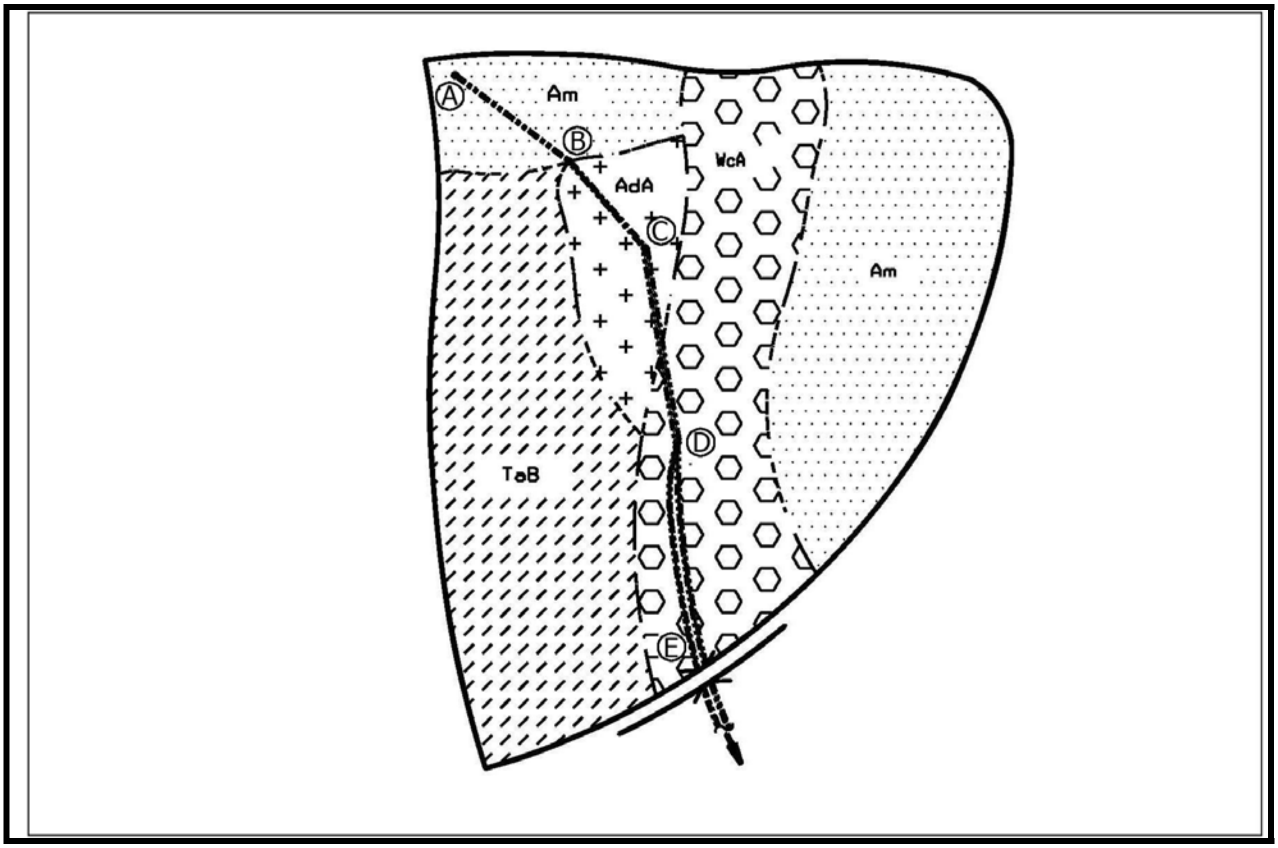
EXAMPLE 2─NRCS TR-55 METHOD

PROBLEM STATEMENT

The watershed area shown in Figure 6B-4 has a composite watershed area of 36.5 acres draining into a culvert at E on a local road in New Castle County. The design frequency is 25-yr. Estimate the peak discharge at point E using the NRCS TR-55 Graphical Peak Discharge method.

The designer should be aware that when using the TR-55 Method the methodology, variables and approach to solving this problem is exclusive to this method. Mixing of variables, runoff coefficients etc. from the Rational Method will lead to inaccurate results.

Figure 6B-4
Watershed for NCRS Method Example



SOLUTION

For this problem since the watershed has homogenous areas and has no stormwater management facilities requiring analysis of several storm frequencies and routing hydrographs, the Graphical Peak Discharge Method is applicable.

The peak discharge equation 6.14 is:

$$Q = q_u A_m q_d F_p$$

First, the runoff depth (q_d) is calculated, second, the time of concentration is calculated and used to obtain the unit peak discharge,(q_u), a pond and then a swamp factor (F_p) is selected and then Q determined.

Step 1. Obtain the 24-hr rainfall for the 25-yr frequency from Figure 6-11. $P_{24} = 6.0$.

Step 2. Establish hydrologic soil groups for the composite watershed and obtain CN_T.

The respective hydrologic soil groups were obtained from the Soils Maps for New Castle County, the curve numbers were obtained from Figure 6B-5 based on the land use, and the weighted average curve number is computed using the data from Figure 6B-5 and the equation:

$$\overline{CN} = \frac{\sum(CN_m \times A_m)}{\sum A_m}$$

The results are:

$$\overline{CN} = \frac{2696.75}{36.50} = 73.88\text{, Rounded to 74}$$

Since impervious areas exist only on residential areas, and the CN values for residential areas, depending on their sizes, have accounted for these impervious areas. Therefore, for this example $CN_T = 74$

Figure 6B-5
Watershed Data

Soil Type	Hydrologic Soil Group	Land Use	CN	Area (ac)
Am	C	Residential - 1 Ac	79	2.50
Am	C	Residential - 1/2 Ac	80	12.60
AdA	C	Residential - 1/4 Ac	83	0.75
AdA	C	Woods (Good hydrological Conditions)	70	1.10
TaB2	B	Residential - 1/4 Ac	75	1.00
TaB2	B	Woods (Good hydrological Conditions)	55	9.00
WcA	D	Residential - 1 Ac	84	1.75
WcA	D	Residential - 1/2 Ac	85	4.30
WcA	D	Woods (Good hydrological Conditions)	77	3.50

Other than the given information all the other data is the same as in Example 1.

Step 3. Calculate Runoff Depth q_d from Equations 6.9 and 6.10.

$$q_d = \frac{(P_{24} - 0.2S))^2}{P_{24} + 0.8S}$$

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

$$q_d = \frac{\left[6.0 - 0.2\left(\frac{1000}{74} - 10\right)\right]^2}{6.0 + 0.8\left(\frac{1000}{74} - 10\right)} = 3.2 \text{ in}$$

Step 4. Calculate t_c Note that TR-55 uses hours when calculating and using t_c.

Overland (Sheet) Flow (Two segments: A-B & B-C) Refer to the topographic data on Figure 6B-6 for T_{t1}, using equation 6.11:

$$T_{t1} = \frac{0.007(nL_1)^{0.8}}{P_2^{\;0.5} s^{0.4}}$$

Manning's runoff coefficient n in this equation is selected from Table 3-1, TR-55. These coefficients have been developed exclusively for use when using the TR-55 method for sheet flow time of concentration.

For segment A-B, n = 0.24 for lawn from Table 3-1, TR-55

For segment B-C, n = 0.40 for woods (light underbrush) from Table 3-1, TR-55.

Figure 6B-6
Flow Path of Watershed

Segment	Description	Slope %	Length (ft)
A-B	Residential lawn	1.0	50
B-C	Forest and meadow	2.0	50
C-D	Shallow gully	0.8	250
D-E	Open channel	0.8	475

Therefore:

$$T_{t1} = \frac{0.007\left(0.24\ x\ 50\right)^{0.8}}{(3.2)^{0.5}(0.01)^{0.4}}$$

$$+\frac{0.007\left(0.40\ x\ 50\right)^{0.8}}{(3.2)^{0.5}(0.02)^{0.4}} = 0.34hr$$

Shallow Concentrated Flow (C-D)

TR-55 has three equations that are acceptable for determining T_{t2}. The designer, after field reviewing the watershed, may decide that using the equation, $V_2 = 32.8ks^{0.5}$ and assigning a k factor from Figure 6-4 will give the most realistic velocity value. In this case:

$V_2 = 32.8(0.457)\ (0.008)^{0.5} = 1.34$ ft/s

The other two equations develop an average k for unpaved areas resulting in equation 6.12:

Unpaved $\quad V_2 = 16.10(s)^{0.5}$

$$V_2 = 16.10(0.008)^{0.5} = 1.44\ ft/s$$

Select the higher of the velocities, since it will result in a more conservative peak discharge, to find T_{t2} and use equation 6.5 modified to calculate hours:

$$T_{t2} = \frac{L_2}{3600V_2} = \frac{250}{3600\ x\ 1.44} = 0.05\ min$$

Open Channel Flow

The travel time for the open channel segment calculation involves the same procedure as in the Rational Method. From Example 1, T_{t3} = 2.47 minute or 0.04 hr.

Therefore, from equation 6.8:

$$t_c = T_{t1} + T_{t2} + T_{t3}$$
$$t_c = 0.34 + 0.05 + 0.04 = 0.43hr$$

<u>Step 5.</u> Determine Unit Peak Discharge q_u from equation 6.15 First, determine I_a/P using equation 6.10 and that I_a is assumed to be 20% of S:

$$I_a/P = \left(\frac{200}{CN_T} - 2\right)\Big/P_{24} = \left(\frac{200}{74} - 2\right)\Big/6.0$$
$$= 0.12$$

I_a/P can also be obtained from Table 4-I, TR-55.

This value and t_c are used to obtain the value of q_u from Exhibit 4-II, TR-55, by interpolating between the curves for $I_a / P = 0.12$

$$q_u = 575\left(ft^3 s / mi^2 / in\right)$$

Step 6. Obtain Pond Adjustment Factor, F_p.

Since the percent of pond and swamp area is zero for the watershed, F_p = 1.0 from Table 5.6, HDS-2, Table 3-9 HEC-22, or Table 4-2, TR-55.

Step 7. Calculate Q from equation 6.14:

$$Q_i = q_u A_m q_d F_p$$

A_m = Drainage Area = 36.5/640 = 0.057 square miles. Therefore, the peak discharge is:

Q = 575 x 0.057 x 3.20 x 1.00

= 105 ft^3/s

Exhibit 4-II, which is not easily or accurately read, is based on the equation:

$$\log(q_u) = C_o + C_1 \log(t_c) + C_2 \left[\log(t_c)\right]^2$$

C_o, C_1, C_2 are coefficients derived from interpolation of Table F-1, TR-55.

Figure 6B-7
Coefficients for Unit Peak Discharge

I_a/P	C_o	C_1	C_2
0.10	2.55323	-0.61512	-0.16403
0.30	2.46532	-0.62257	-0.11657
0.35	2.41896	-0.61594	-0.08820
0.40	2.36409	-0.59857	-0.05621
0.45	2.29238	-0.57005	-0.02281
0.50	3.20282	-0.51599	-0.01259

The designer may find it simpler to use the equation and coefficients.

EXAMPLE 3—OPEN CHANNEL HYDRAULICS

PROBLEM STATEMENT

A grass-lined, unsymmetrical trapezoidal channel with a base width of 4 feet, a 4:1 foreslope, a 2:1 backslope, a stream bed slope of 0.012 ft/ft (1.2%), and *n* = 0.05, carries a design discharge of 115 ft^3/s. Determine the depth of flow, velocity, and if the flow is supercritical. There are charts and nomographs available in HDS-3, HEC-15 (1986 or 1988 version) and HEC-22 that can be used to quickly solve for these two values.

SOLUTION

Using the channel equations given at the end of Example 1, the depth of flow (*d*) will be obtained by trial and error.

Trial 1.

Try *d* = 2 ft

$$A = Bd + \frac{(z_1 + z_2)d^2}{2}$$

$$A = (4x2) + \frac{(2+4)2^2}{2} = 20\,ft^2$$

$$P = B + d\left(\sqrt{z_1^2 + 1} + \sqrt{z_2^2 + 1}\right)$$

$$P = 4 + 2\left(\sqrt{2^2 + 1} + \sqrt{4^2 + 1}\right)$$

$$P = 4 + 2(6.359) = 16.718\,ft$$

$$R = A/P = \frac{20}{16.718} = 1.196\,ft$$

(equation 6.7a)

$$V = \frac{1.49}{n} R^{2/3} s^{1/2}$$

$$= \frac{1.49}{0.05}(1.196)^{2/3}(0.012)^{1/2} = 3.58\,ft/s$$

(equation 6.7)

$$Q = AV = 20x3.58 = 73.6\,ft^3/s$$

(too low for 115 ft^3/s)

Trial 2.

Try *d* = 2.5 feet

$$A = (4x2.5) + \frac{(4+2)x2.5^2}{2} = 28.75\,ft^2$$

$$P = 4 + (6.359x2.5) = 19.898\,ft$$

$$R = A/P = 1.445\,ft$$

$$V = \frac{1.49}{0.05}(1.445)^{2/3}(0.012)^{1/2} = 4.17\,ft/s$$

$$Q = 28.75x4.16 = 119.60\,ft^3/s$$

(greater than 115 ft^3/s)

Trial 3.

Try d = 2.45 feet

$$A = (4x2.45) + \frac{(4+2)2.45^2}{2} = 27.81\,ft^2$$

$$P = 4 + (6.359x2.45) = 19.58\,ft$$

$$R = A/P = 1.42\,ft$$

$$V = \frac{1.49}{0.05}(1.42)^{2/3}(0.012)^{1/2} = 4.125\,ft/s$$

$$Q = 27.81x4.125 = 114.71\,ft^3/s$$

≈115 ft^3/s)

Therefore:

Depth of flow *d* = 2.45 ft and

V = 4.12 ft/s

Instead of performing the calculations, Chart 1, HEC-15, or Chart 22, HEC-22, could have been used. From these charts and using:

$\frac{d}{B} = \frac{2.45}{4} = 0.6125$ and, $z_1 + z_2 = 6.0$

R/d is estimated at 0.58.

Therefore:

R = 0.58 x 2.45 = 1.421ft.

A/Bd = 27.81/ (4) (2.45) = 2.84, and

A = 2.84 x 2.45 x 4 = 27.83 ft^2.

The designer can also use Chart 83, HDS-3, or Chart 14B, HEC-22, to solve Manning's equation, connect the slope-scale at 0.012 with the *n*-scale of 0.05 then draw a line passing through R = 1.42 and the point of intersection of s and n at the turning line, and extend it to the velocity scale which is read at 4.15 ft/s. Therefore, Q = 27.44 x 4.15 = 113.88 ft^3/s which is close to 115 ft^3/s, and the assumed d = 2.45 ft is O.K.

For example, assume $z_1 + z_2 = 3$. On Chart 3, HEC-15 (May 1986), join s = 0.05 with Qn = 115 x 0.05 = 5.75 and extend to the turning line. Connect that point of intersection with B = 4 and let the line intersect z = 0 from there proceed horizontally until z = 3. That point of intersection corresponds to d/B = 0.615 from which d = 4 x 0.615 = 2.46ft.

Froude Number:

$$F_r = \frac{V}{\sqrt{gD}}$$

$$D = \frac{A}{T} = \frac{27.81}{6x2.45+4} = 1.49\,ft$$

$$F_r = \frac{4.12}{\sqrt{32.2x1.49}} = 0.60 < 1$$

Therefore, the flow is subcritical.

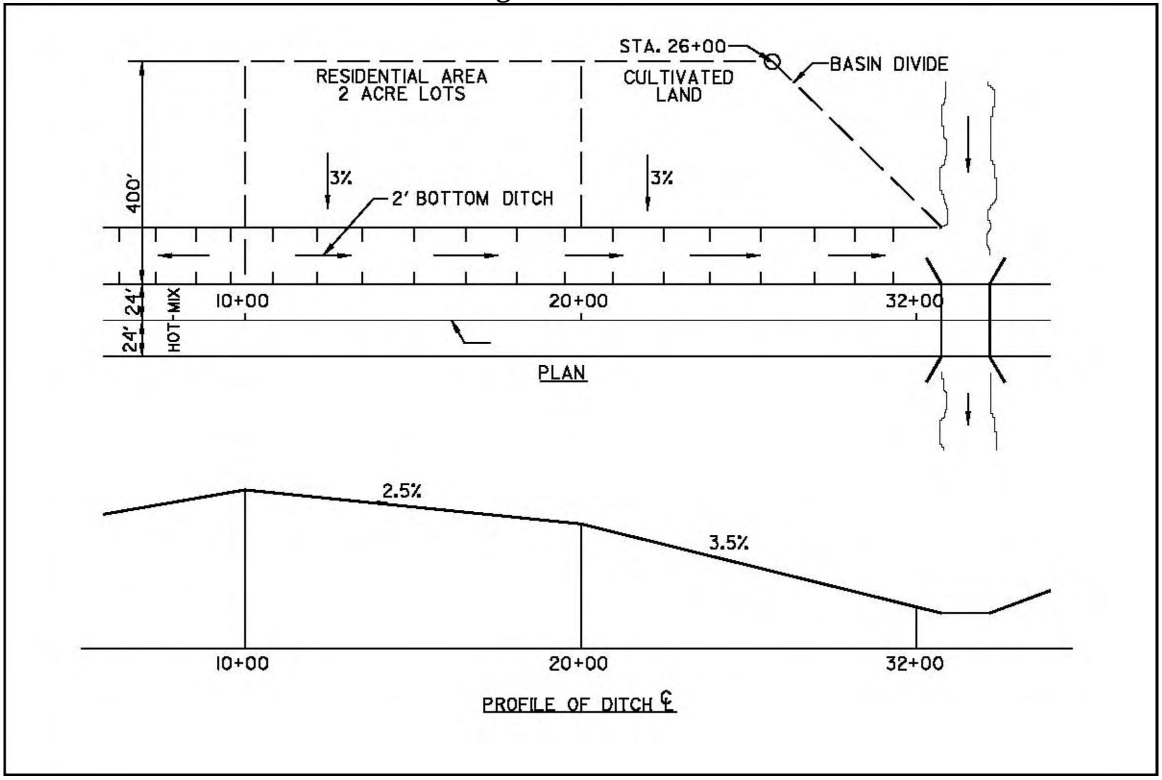
EXAMPLE 4—ROADSIDE DITCH

PROBLEM STATEMENT

Figure 6B-8 shows a roadside ditch along a Sussex County rural collector with trapezoidal cross section, 2-foot bottom width, and 4:1 side slopes. The ditch lining is average grass in sandy loam.

Compute the velocities of flow along the ditch, and determine if additional erosion protection is necessary. From Figure 6-1, the design frequency shall be for a 10-yr period.

Figure 6B-8
Schematic Diagram for Roadside Ditch



SOLUTION

Enter the following critical stations on a Roadside Ditch Design computation form:

Station 20 + 00 (Back)

Station 20 + 00 (Ahead)

Station 32 + 00 (Outfall Point)

Leave blank spaces for intermediate stations at this time. The calculations proceed as follows. The sequence numbers for each station pertain to the respective columns on the computation sheet.

Station 20+00 (Back)

Step 1. From Sta. 10+00 to Sta. 20+00,

- Pervious Area = (400)(1000)/43560 = 9.18 ac.
- Impervious Area = (24)(1000)/43560 = 0.55 ac.
- Total Area = 9.18 + 0.55 = 9.73 ac

Step 2. Select C values from Figure 6-8.

- Pervious: For residential areas with 2-acre lots, use C = 0.40.

- Impervious: For asphalt pavement (hot-mix roadway & shoulder), $C = 0.95$.

Step 3. Calculate total *CA*

Use $CA = C_pA_p + C_IA_I$

$CA = (0.40)(9.18) + (0.95)(0.55) = 4.19$

Step 4. Determine t_c

(a) Since the watershed is homogenous and not complex, the Rational Method can be used to find to the find the discharge flow. Therefore, use equation 6.4:

$$T_{t1} = \frac{0.42(nL_1)^{0.8}}{P_{24}^{\ 0.5} s^{0.4}}$$

where:

n = 0.40 from Figure 6-16
L_1 = 100 ft
P_{24} = 3.4 in/hr from figure 6-11
s = 0.03

Therefore:

$$T_{t1} = \frac{0.42\left[(0.40)(100)\right]^{0.8}}{(3.4)^{0.5}(0.03)^{0.4}}$$
$$= 17.71 \text{ min}$$

The remaining 300 ft is to treated as shallow concentrated flow using equations 6.5 and 6.6:

$$T_{t2} = \frac{L_2}{60v_2}$$

$$V_2 = 32.8ks^{0.5}$$

The latter equation has been modified into the condition for unpaved areas in equation 6.12:

Unpaved $V_2 = 16.10(s)^{0.5}$

Therefore:

$$V_2 = 16.10(0.03)^{0.5} = 2.79\ ft/s$$

$$T_{t2} = \frac{300}{60\ x\ 2.79} = 1.79 min$$

(b) For 1000 feet open channel flow, T_{t3}, assume maximum permissible velocity of 4.0 ft/s for grass on sandy loam from Figure 6-15.

Therefore:

$$T_{t3} = \frac{1000}{60\ x\ 4} = 4.17\ min$$

and, using equation 6.8:

$$t_c = T_{t1} + T_{t2} + T_{t3}$$
$$= 17.71+1.79+4.17 = 23.67 \text{ min}$$

Step 5. Find *I* from Figure 6-7 for the 10-year frequency and t_c = 23.67 minutes

I = 3.83 in/hr (interpolating between 15 and 30 min)

Step 6. Calculate *Q* using equation 6.1:

$Q = C_f CIA = C_f (CA) I$

Q = 1.00 x 4.19 x 3.83 = 16.05, rounded to 16 ft^3/s

Step 7. Enter ditch cross sectional data

$z_1 = 4$, $B = 2$ feet, $z_2 = 4$, and $S = 0.025$

Step 8. Select Manning's *n*

n = 0.05 for grass from Figure 6-16

Step 9. Calculate *d*

Use Chart 3, HEC-15 (1986), or Chart 14B, HEC-22. For s = 0.025, Qn = 16 x 0.05 = 0.8, and B = 2, d/B reads 0.45 when z = 4. Thus d = 0.45 x 2 = 0.9 feet.

Step 10. Calculate Average Velocity

Use $V = Q/A$

$$A = dB + zd^2 = 0.9\ x\ 2.0 + 4\ x\ 0.9^2 = 5.04\ ft^2$$

$$V = \frac{16}{5.04} = 3.18\ ft/s$$

NOTE: Check the computed velocity against the assumed velocity used to determine Step 4. If the velocities differ by more than 20%, repeat Steps 4 through 9 using the velocity calculated in Step 10. At this location, the velocities differ by ≈14%.

Step 11. Check the computed velocity in Step 10 against the permissible velocity from Figures 6-13 through 6-15. If the computed velocity is greater than the maximum permissible velocity, a stronger protective lining is required.

Step 12. In this case, V = 3.18 ft/s and V_{max} = 4 ft/s. Since $V < V_{max}$ for grass, specify grass.

Step 13. Grass must cover the entire ditch. From a practical consideration, grass should cover the entire area within the limits of consideration.

Station 20+00 (Ahead)

Based on the allowable velocity for a grassed lined channel, the calculated velocity at Station 20+00 and the increase in ditch slope after Station 20+00, it is likely that after Station 20+00 and before the outfall that; (1) some change in the type of ditch lining may need to be made; (2) an increase in the ditch cross section or; (3) some type of velocity dissipation structure(s) will be needed.

To find the point at which a lining change may be advisable and since excessive outfall velocities have to be considered in the design at all locations, a good point of reference would be to next calculate the maximum expected velocity at the outfall based on the conditions as presented in the problem. Knowing this value will give an indication of the maximum erosion potential.

After determining this value iterations of the process would begin at Station 20+00 and proceed toward the outfall and determine the location for making a decision on addressing erosion. Using a form similar to that included in this problem will allow the designer to keep the data in a systematic manner.

Station 32+00 (Outfall Point)

Step 1. From Sta. 20+00 to Sta. 32+00,

Pervious Area

$$A_p = \frac{(600)(400) + 1/2(600)(400)}{43560}$$

= 8.26 ac

$$A_{Ptotal} = 9.18 + 8.26 = 17.44ac$$

Impervious Area

$$A_{I\ totsl} = \frac{(24)(2200)}{43560} = 1.21ac$$

Step 2. Select C values from Figure 6-8.

Pervious C from Sta. 10+00 to Sta. 20+00 = 0.40 (For residential lots)

Pervious C from Sta. 20+00 to Sta. 32+00 = 0.30 (For cultivated land from Figure 6-8)

Weighted Average Pervious C from equation 6.2:

$$C_p = \frac{(0.4)(9.18) + (0.30)(8.26)}{17.44} = 0.35$$

Impervious C from Sta. 10+00 to Sta. 32+00

C_I = 0.95

Step 3. Find total CA

Use the equation $CA = Cp\,Ap + C_I A_I$

CA = (0.35)(17.44) + (0.95)(1.21) = 7.25

Step 4. Determine t_c

Find T_{t1} (equation 6.4) and T_{t2} (equation 6.5) for the cultivated area.

$$T_{t1} = \frac{0.42(nL_1)^{0.8}}{P_{24}^{\ 0.5} s^{0.4}}$$

where:

n = 0.17 from Table 3-2, HEC-22
L_1 = 100 ft
P_{24} = 3.4 in/hr from Figure 6-11
s = 0.03

Therefore:

$$T_{t1} = \frac{0.42[(0.17)(100)]^{0.8}}{(3.4)^{0.5}(0.03)^{0.4}}$$

= 8.93 min

The remaining 300 ft is to treated as shallow concentrated flow using the following equations 6.5 and 6.6:

$$T_{t2} = \frac{L_2}{60v_2} \text{ and,}$$

$$V_2 = 32.8ks^{0.5}$$

This equation has been modified for unpaved areas in equation 6.12:

Unpaved $V_2 = 16.10(s)^{0.5}$

Therefore:

$$V_2 = 16.10(0.03)^{0.5} = 2.79\, ft/s$$

$$T_{t2} = \frac{300}{60 \, x\, 2.79} = 1.79\, min$$

Assuming a grass lining from Sta. 20+00 to Sta. 32+00 (1200 feet open channel flow) for which the maximum permissible velocity is 4 ft/s (Figure 6-15), the travel time is:

1200/(60 x 4) = 5 min

Add this to the travel time at Sta. 20+00, which is 23.67 min.

Thus using equation 6.8,

t_c = 23.67+5+8.93+1.79 = 39.39 min.

Step 5. From Figure 6-7, I = 2.94 in/hr.

Step 6. Calculate Q as before from equation 6.1

Use $Q = (CA)\, I$

Q = (7.25) (2.94) = 21.32, rounded to 21 ft^3/s

Step 7. Enter ditch cross sectional data

$z_1 = 4$, $B = 2$ feet, $z_2 = 4$, and $s = 0.025$

Step 8. Select Manning's n

n = 0.05 for grass from Figure 6-16

Step 9. Calculate d

Use Chart 3, HEC-15 (1986), or Chart 14B, HEC-22.

For s = 0.025, Qn = 21 x 0.05 = 1.05, and B = 2, d/B reads 0.51 when z = 4

Thus d = 0.51 x 2 = 1.02 feet

Step 10. Calculate Average Velocity

Use $V = Q/A$

$$A = dB + zd^2 = 1.02x\, 2.0 + 4\, x\, 1.02^2 = 6.20\, ft^2$$

$$V = \frac{21}{6.20} = 3.39\ ft/s$$

NOTE: Check the computed velocity against assumed velocity used to determine Step 4. If the velocities differ by more than 20%, repeat Steps 4 through 9 using the velocity calculated in Step 10. At this location, the velocities differ by about 22%. The second iteration yields the results in Figure 6B-9.

If the velocity had been greater than 4.0 ft/s, it would be necessary to begin at an intermediate location after Station 20+00 and determine the station at which a protective lining, ditch cross section change or energy dissipater should be introduced. This is a trial and error process and could be worked from the outfall back or from an upstream station. Since the drainage procedure has several assumptions, it is a matter of engineering judgment in defining a point at which an alternative ditch lining and/or riprap would be considered. In this problem, based on the data given, it appears that the grass lining would be sufficient. However, it would be prudent from an environmental and erosion control perspective to place riprap for at least the last 25 ft.

Figure 6B-9
Roadside Ditch Design Form – Completed for Example 4

ROADSIDE DITCH DESIGN

Prepared by:__________ SHEET NO. 1 of 1
Checked by:__________ LOCATION: Andrews Lake RD. CONTRACT NO:__________
Storm Frequency: 10-yr

	1			2		Ditch Section							8	9	10	11	12	13	14
	Area (ac)			C		3	4	5	6	7									
Station/Location	Total	Pervious (ac)	Impervious (ac)	Pervious (n)	Impervious (n)	Total C x A	t_c (min)	I (in/hr)	Q (ft^3/s)	z_1	B	z_2	S (ft/ft)	n	d(ft)	V (ft/s)	Permissible V (ft/s)	Erosion Control Type	Width (ft)
20 (Bk)	9.73	9.18	0.55	0.4	0.95	4.19	23.67	3.83	16	4	2	4	0.025	0.05	0.9	3.17	4.0	grass	total
32	17.44	9.18	0.55	0.4	0.95														
		8.26	0.66	0.55	0.95														
						7.25	39.39	2.94	21	4	2	4	0.025	0.05	1.02	3.39	4	grass	total

Figure 6B-10
Roadside Ditch Design Form

ROADSIDE DITCH DESIGN

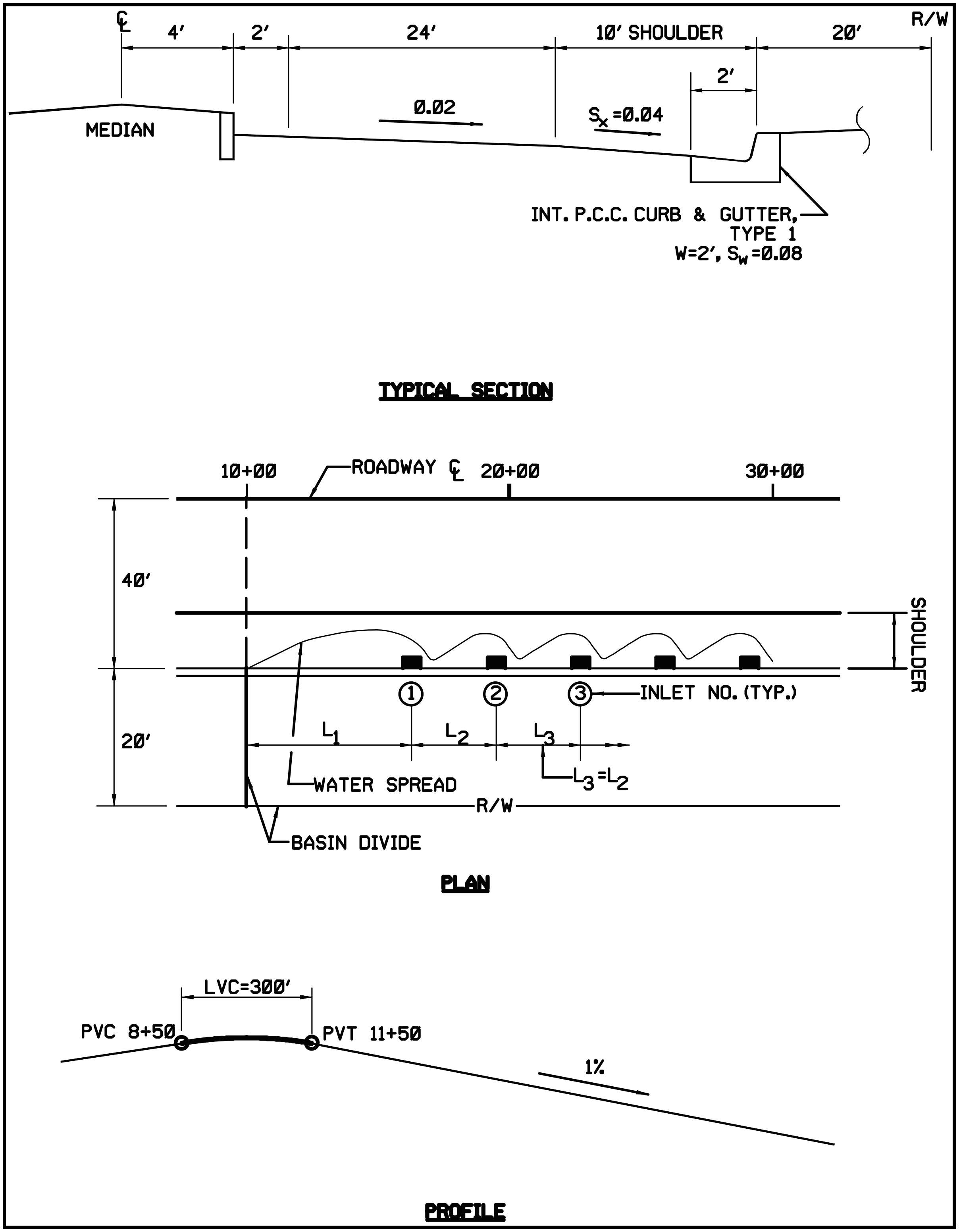
Prepared by:__________________ **SHEET NO.**____**of** _____
Chk'd. by:____________________ **LOCATION:**____________________ **CONTRACT NO:**_________________
Storm Frequency:______**-yr**

	1			2		Ditch Section							8	9	10	11	12	13	14
	Area (ac)			C															
						3	4	5	6	7									
Station/Location	Pervious	Impervious	Total	Pervious	Impervious	Total C x A	t_c (min)	I (in/hr)	Q (ft^3/s)	z_1	B	z_2	S (ft/ft)	n	d(ft)	V (ft/s)	Permissible V (ft/s)	Erosion Control Type	Width (ft)

EXAMPLE 5 – INLET SPACING
PROBLEM STATEMENT

Figure 6B-11 shows data for a 4-lane divided New Castle County urban arterial with a *C* of 0.88.

Figure 6B-11 – Typical Section, Plan and Profile



The following describes the procedure for determining inlet spacing based on the equations in Section 6.8.1, the hydraulic data given, use a C = 0.88, and DelDOT's standard 20"x36" Type 1 grate. This example has been simplified and does not include site-specific factors, such as maximum spacing based upon access for maintenance, location of commercial and residential entrances, curb ramps, superelevation transitions, curb height, etc. HEC-22 provides a comprehensive description of the procedure for determining inlet capacity and placement. HEC-12 (1984) presents a more basic understanding of the concept of gutter flow and for determining inlet spacing using charts and nomographs.

Solution:

Step 1. Select the design frequency.

From Figure 6-1, the design frequency is 10-yr return period for an urban arterial.

Step 2. Select the allowable spread.

From Figure 6-2, T = 10 ft.

Note: Tables for values of Q_t and Q_i with variables of $S_{L,}$ S_X, and S_w can be found on DelDOT's web site. These values can be used to check the calculated values but are based on untested performance data

The given composite cross section has an S_x = 0.04, S_w =- 0.08, W = 2 ft, S_L = 0.01, W/T = 0.2 and S_w/S_x = 2

Step 3. From Charts 1B and 2B, HEC-22:

E_o = 0.485

Q_s = 4.2

$Q_t = Q_s / (1-E_o)$

Q_t = 4.2 / (1- 0.485) = 8.2 ft^3/s

Chart 1B represents solutions to the equation: $Q = (K_u/n)\ S_x^{1.67}\ S_L^{0.5}\ T^{2.67}$ where K_u= 0.56. This is used in conjunction with Chart 2B together they solve the equation for composite gutter flow. Q_t = 8.25 ft^3/s using the equation. Using the design aid chart Q_t = 8.26 ft^3/s.

E_o for composite gutter sections can be derived from the equation:

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

Step 4. Find the grate interception capacity using the following equations 6.45 and from Figure 6-22:

$$Q_i = Q\left[R_f E_o + R_s\left(1-E_o\right)\right]$$

$$R_s = \frac{1}{\left[1+0.15\left(V^{1.8}\right)/S_x L^{2.3}\right]}$$

R_f = 1.0 if $V \le 6.0$

From DelDOT's web site:

Q_i = 4.99 ft^3/s (

Step 5. Therefore $E = Q_i / Q_T$ = 4.99/8.2

= 0.61 ≤ 0.70

Try T = 8 ft for which: W/T = 0.25

E_o = 0.6

Q_s = 1.94

$Q_t = Q_s / (1-E_o)$

Q_t = 4.85 ft^3/s

Q_i = 3.32 ft^3/ from DelDOT's web site

E = 3.32 / 4.85 =0.68 < 0.70

Try T = 6 for which W/T = 0.333

E_o = 0.73

Q_s = 0.66

$Q_t = Q_s / (1-E_o)$

Q_t = 2.44 ft^3/s

Q_i = 1.96 ft^3/ from DelDOT's web site

E = 1.962 / 2.44 =0.80 > 0.70

Based on the various assumptions used in drainage calculation, using T = 6 is probably too conservative.

Therefore, use T = 8 ft and Q_t =4.85

$Q_B = Q_T – Qi$

$Q_B = 4.85 - 3.32 = 1.53$ ft^3/s

Spacing of Inlet 1

Assume $t_c = 5$ minutes

From Figure 6-5, $I = 6.42$ in/hr

$$L_1 = \frac{Q_{iT} x 43560}{C_1 I_1 W_1} = \frac{4.85 x 43560}{0.88 x 6.42 x 60}$$

L_1 = 623.25 ft (round to a more convenient 625 ft

The location of Inlet 1 equals station:

(10 + 00) + (6 + 25) = 16 + 25

Check t_c: $V = 2$ ft/s for shallow gutter flow on a 1% slope, so using equation 6.5:

$$T_t = \frac{L}{60V} = \frac{625}{(60)(2)} = 5.21 \text{min}$$

Therefore, t_c = 5.21 min as this is the minimum recommended

I = 6.36 in/hr

Spacing of Inlet 2

Using equation 6.61,

$$L_2 = \frac{Q_{2T} - Q_{B1}}{C_2 I_2 \overline{W}} x 43560$$
$$= \frac{4.85 - 1.53}{0.88 x 6.36 x 60} x 43560 = 430.66$$

The location of Inlet 2 equals station:

(16+25) + (4 +30) = 20+55

Subsequent inlets shall be spaced at 430 feet as long as the geometric controls remain unchanged and there is clean-out access at a minimum distance of 300 ft (Figure 6-3). Stationing of inlets can be rounded to the nearest foot for construction convenience and can be adjusted slightly to accommodate other project requirements. An exaggerated degree of precision in calculations is rarely justified.

All inlets of the drainage system shall be numbered and the computed results shall be presented in a tabular format using the Inlet Spacing Computation sheet.

<u>Inlet Spacing Design Form</u>

S_x, S_w, and W are the cross sectional elements of the gutter section from the constructions plans and Standard Construction Details. For concrete gutter sections the recommended $n = 0.016$. Refer to Section 6.8 for details on the procedure, applicable equations and additional references.

The drainage design begins by using a set of preliminary construction plans and marking preliminary drainage areas and known inlet locations such as curb ramps, entrances, sags etc. as specified in Section 6.8.1.8. The initial area is 300 to 500 feet long below the high point.

The primary control is the spread. The upstream inlet will define the initial spread and the design continues by locating inlets as necessary to control the spread as defined by the design criteria. To locate these inlets incremental areas of flow are selected and calculations are performed. The flow for the area is determined and including any flow that has bypassed previous inlets and the next inlet placed to intercept this flow before the design spread is exceeded.

The form and the following description allow for an orderly determination and recording of this process. For a continuous longitudinal slope the process is relatively simple if the contributing subsequent areas have uniform runoff characteristics. After determining the spacing between the first two inlets all downstream inlets are similar spaced until there is a change in the defining drainage data.

Procedure:

1. Enter in Col. 1 the inlet number. All inlets in a storm drain shall be numbered in sequence starting with the first inlet from the crest down to the inlet at the low point of the sag curve where the interception is 100%.
2. Enter in Col. 2 the inlet's location by its baseline station and whether it is left or right of the baseline.
3. Enter in Col. 3 the partial drainage area, ΔA (ac), which discharges into the inlet. It

is the uppermost part of the entire drainage area for the first inlet, or the partial area above the inlet up to its upstream inlet for subsequent inlets. Note that in some cases areas outside the pavement may have to be included in ΔA.

4. Enter in Col. 4 the runoff coefficient of the drainage area ΔA of the inlet. If the drainage area is composite, then a weighted value of C using Equation 6.2 should be used. C values are to be selected from Figure 6-8.

5. Enter in Col. 5 the travel time T_t (min) for the surface runoff to travel distance, L, as in Section 6.6.

6. Enter in Col. 6 the intensity of rainfall I (in/hr) for the inlet corresponding to t_t for the 10-yr frequency, the values of I are obtained from Figures 6-6, 6-7 or 6-8.

   If drainage inlets are located on a depressed roadway, then a 50-yr frequency must be used for most types of highways as indicated in the design criteria.

7. Enter in Col.7, the gutter flow, Q, the product of Columns 4, 5and 6.

8. Enter in Col.8 the longitudinal slope of the gutter at the inlet, S_L (ft/ft). For vertical curves determine S_L from the slope equation for a parabolic curve. This value of S_L is used to determine the gutter flow rate and the water spread on the pavement.

9. Enter in Col. 9 the cross slope of the gutter/pavement section.

10. Enter in Col. 10 the bypass flow rate from the up stream inlet as shown in column 18 of the previous row.

11. Enter in Col. 11 the total gutter flow, Q_T, approaching the inlet. Q_T is equal to ΔQ in Column 7 plus the bypass flow rate of the upstream inlet Q_B of Column 18 of the previous row of the table.

12. Enter in Col. 12 the depth of flow at curb, d (ft). It must be 0.5 ft or less. Follow the procedure in 14 if d controls T so that T may be decreased.

13. Enter in Col. 13 the width of the gutter or grate.

14. Enter in Col. 13 the width of water spread, T, on the gutter section, due to Q_T (ft). T can be calculated for the gutter section with either a uniform cross slope or composite cross slope as explained in Sections 6.8.1.5.1 and 6.8.1.5.2. T must not exceed the permissible water spread specified in the design criteria. If T is excessive, reduce Q_T to the amount that can be accommodated within the permissible T. Use a trial and error procedure by decreasing the distance between inlets resulting in a smaller ΔA to find the matching Q_T

15. Enter in Col. 15 the ratio of gutter/grate width divided by the spread. This ratio is needed when using the charts in HEC-22 to determine the interception efficiency and solving for other design values.

16. Enter in Col. 16 the inlet type, be sure to indicate if using other than the standard grate types

17. Enter in Col. 17 the intercepted flow, Q_i. Compute Q_i as explained in Section 6.8.1.6.2.1. For standard grates on grade, Q_i can be obtained from DelDOT'S web site**.**

18. Enter in Col. 18 the by-pass flow, Q_B, by subtracting column 17 from column 11.

19. Enter in Col. 18 remarks such as sump condition, 50-yr frequency, etc

One of the final checks is to determine the interception efficiency, E, of the inlet. E is equal to Q_i in column 17 divided by Q_T in column 11. If E is less than 0.7, use a trial and error procedure by decreasing T (see 14) and repeating the computations until $E \approx 0.7$.

Figure 6B-12
Inlet Spacing Computation Form

INLET SPACING COMPUTATION SHEET									Project No.______ Sheet No.______ of _____ Computed By: ________ Chk. By: _______								
INLET		GUTTER DISCHARGE Design Frequency________-yr						GUTTER DISCHARGE Allowable Spread ______ ft							INLET DISCHARGE		
No.	Sta.	Drain Area ΔA (ac)	Runoff Coefficient C	Time of Concentration t_t (min)	Rainfall Intensity I (in/hr)	$\Delta Q = \Delta CI\Delta A$ (ft^3/s)	Longitudinal Slope S_L (%)	Cross Slope S_x or S_w (%)	Previous Bypass Flow (ft^3.s)	Total Gutter Flow (ft^3/s)	Depth d (ft)	Grate or Gutter Width W (ft)	Spread T (ft)	W/T	Inlet Type	Interception Q_i (ft^3/s)	Bypass Flow Q_b (ft^3/s)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
REMARKS:																	

EXAMPLE 6 ─ STORM DRAINS

Detailed examples, acceptable forms, and available software for the design of storm drains are found in HDS-3, HDS-4, HDS-5, HEC-22 and other references in Sections 6.8 and 6.9. Below is an acceptable process for using the forms.

PROCEDURE FOR COMPLETING STORM DRAIN COMPUTATION FORM

Col. 1 ─ Pipe Unit Number. Enter the designated number of the pipe unit. The computations begin with the first pipe unit in the uppermost location of the drainage area, gradually proceeding downstream in sequence.

Col. 2 ─ Upper Inlet Number. Enter the inlet (junction) number at the upper end of the pipe unit under consideration. Below this inlet number and within the space, indicate its baseline station and whether to the left or right of the baseline.

Col. 3 ─ Lower Inlet Number. Enter the inlet (junction) number at the lower end of the pipe being designed. As in Column 2, indicate its location.

Col. 4 ─ Drainage Area. Enter the incremental area, ΔA, of the (portion of) watershed draining into the inlet in Column 2. For the first pipe, ΔA is the uppermost area of the watershed where the storm water flows on the surface into the inlet. For subsequent pipes, it is the drainage area bounded between the inlet in Column 2 and the previous upstream inlet.

Col. 5 ─ Runoff Coefficient. Enter the runoff coefficient C for the drainage area in Column 4. It is the composite runoff coefficient for the drainage area ΔA as in Equation 6.2:

$$C = \frac{\sum C_x A_x}{A_{Total}}$$

The incremental drainage area may be assumed to be composed of only pervious and impervious runoff coefficients. Select the appropriate runoff coefficients from Figure 6-8 and compute the weighted average for the composite runoff coefficient, C.

Col. 6 ─ Product. Enter the product of Column, 4 and 5, which is $C \; x \; \Delta A$.

Col. 7 ─ Cumulative Product. Enter the sum of entries of Column 6 applicable to the pipe unit. Determine CA for the n'th pipe unit as follows:

$$(CA)_n = (CA)_{n-1} + (C \; x \; \Delta A)_n$$

Col. 8 ─ Time of Concentration. Enter the time of concentration for the pipe, t_c (min), which is the travel time of stormwater along the hydraulic length to reach the inlet in Column 2. Analyze all possible routes. Use the route with the longest travel time.

For the first pipe, the hydraulic length includes only surface flow from the uppermost areas of the watershed that drains into the inlet in Column 2. The travel time for surface flow is computed by using Equation 6.3, 6.4 and/or 6.5.

For subsequent pipe segments, the hydraulic length (surface flow) is underground along the storm drain. Thus the time of concentration for a pipe is the time of concentration for stormwater to travel from the upstream pipe plus the time required by storm water to travel the length of the pipe. For the n'th pipe, it is represented by the equation:

$$(t_c)_n = (T_c)_{n-1} + (T_t)_{n-1}$$

On the form, t_c is entered by adding Column 8 and Column 16 of the previous row. The minimum period for t_c is 5 minutes.

Col. 9 ─ Rainfall Intensity. Enter the intensity of rainfall corresponding to the time of concentration in Column 8, I (in/hr). Use Figures 6-6, 6-7 or 6-8 for the design frequency.

Col. 10 ─ Design Discharge. Enter the design discharge for the pipe unit, Q. It is the product of the entries in Columns 7 and 9.

Col. 11 ─ Pipe Size. Enter the pipe diameter in inches. Use Figure 6-29 to size concrete pipes for the design discharge at friction slope or slightly steeper. Otherwise the equations and referenced resources can be used to find a suitable pipe size and an acceptable slope. See equations, charts and tables referenced in Section 6.9.

Col. 12 ─ Length of Pipe. Enter the length of the pipe, *L* (ft).

Col. 13 ─ Friction Slope. Enter the friction slope of the pipe unit, S_f (ft/ft). Use Equations 6.64 and 6.65.

Col. 14 ─ Actual Slope. Enter the actual slope of the pipe, S (ft/ft). Note that $S \geq S_f$.

Col. 15 ─ Flow Velocity. Enter the velocity of flow in the pipe unit, *V*. See Section 6.9.

Col. 16 ─ Travel Time. Enter the travel time for the flow to travel over distance *L* in the pipe, T_t (equation 6.5):

$$T_t = \frac{L}{60V} = \frac{(Col.12)}{(60)(Col.15)}$$

Col. 17 ─ Upper Invert Elevation. Enter the invert elevation of the pipe at the inlet in Column 2. The elevation for the first pipe is normally based on the minimum cover requirement for the selected type of pipe.

Col. 18 ─ Lower Invert Elevation. Enter the invert elevation at the lower end of the pipe. Col. 18 = Col. 17 – (Col 12 x Col. 14)

Figure 6B-13
Storm Drain Computation Form

STORM DRAIN COMPUTATION SHEET									Project No.______ Sheet No.____ of ____								
									Computed By: ______			Chk. By: ______					
Pipe Unit			ΔA	C	C x ΔA	CA	t_c (min)	I (in/hr)	Q = CIA (ft^3/s)	Diameter (in)	Length (ft)	S_f (ft/ft)	S (ft/ft)	V (ft/s)	t_t (min)	Invert Elevation	
No.	From	To														Upper	Lower
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)

REMARKS:

PROCEDURE FOR COMPLETING HYDRAULIC GRADELINE COMPUTATION FORM

Col. 1─Junction Number. Enter the junction number that represents the catch basin, manhole or junction box immediately upstream of the outflow pipe. HGL computations begin at the outfall and are calculated for each junction in the storm drain.

Col. 2─Baseline Station. Enter the baseline station for the junction indicating whether it is to the left or right of the baseline.

Col. 3─Outlet Water Surface Elevation. Enter the outlet water surface elevation. For the first junction it is the outfall design tailwater elevation; for subsequent junctions it is the inlet water surface elevation of the previous downstream junction.

Col. 4─Outfall Pipe Diameter. Enter the diameter of the outflow pipe unit, D_o (in).

Col. 5─Design Discharge. Enter the design discharge for the outflow pipe, Q_o.

Col. 6─Outflow Pipe Length. Enter the length of the outflow pipe, L_o.

Col. 7─Friction Slope. Enter the friction slope of the outflow pipe flowing full, S_{fo} (ft/ft). Refer to Figure 6-29 for concrete pipes, or use equations 6.67 and 6.69 to derive:

$$S_{fo} = \frac{H_f}{L_o} = \left(\frac{Qn}{0.46D^{2.67}}\right)^2$$

Col. 8─Friction Head Loss. Enter the head loss due to friction for the outflow pipe, H_f by using equation 6.71:

$$H_f = L_o S_{fo}$$

On curved alignment, the additional head loss due to friction, H_c is given by:

$$H_c = \frac{0.002 \varnothing V_o^2}{2g}$$

Where,

$\varnothing$ = Angle of curvature in degrees,

V_o = Velocity in the outflow pipe (ft/s)

$$V_o = \frac{Q_o}{A}$$

A = Area of outflow pipe (ft^2)

g = 32.2 ft/s^2.

Add H_c to H_f and enter the sum as total H_f in Column 8.

Col. 9─Outflow Velocity. Enter the velocity of flow in the outflow pipe, V_o (ft/s), as determined in Step 8.

Col. 10─Contraction Head Loss. Enter the contraction head loss, H_o, by using the formula:

$$H_o = \frac{0.25 V_o^2}{2g}$$

Col. 11─Inflow Pipe Velocity. Enter the velocity of flow in the inflow pipe that flows into the junction, V_i (ft/s). V_i equals the design discharge for the inflow pipe (ft^3/s) divided by its area (ft^2).

When more than one pipe is flowing into the junction, V_i is determined as follows. For each pipe flowing into the junction, compute the product of its velocity of flow and the discharge. The pipe that has the greatest product will be considered; its V_i will be entered in Column 11. In this situation, the time of concentration that influences the design discharge may have to be adjusted depending on the controlling pipe whose V_i is entered in Column 11.

Column 12─Expansion Loss. Enter the controlling expansion loss, H_i using the equation:

$$H_i = \frac{0.35 V_i^2}{2g}$$

Col. 13─Skew Angle. Enter the angle of skew between the controlling inflow pipe Δ (degrees).

Col. 14─Bend Loss. Enter the bend loss, H_Δ, which is obtained from the equation:

$$H_\Delta = \frac{K V_i^2}{2g}$$

Where:

K is a loss coefficient,

The values of *K* are listed at the bottom of form for various skew angles.

Col. 15─Total Head Loss. Enter the total head loss at junction, H_t, which is equal to the sum of H_o in Column 10, H_I in Column 12, and H_Δ in Column 14. That is: $H_t = H_o + H_i + H_\Delta$

Col. 16─Adjusted Head Loss. Enter the adjusted total head loss at junction, $\overline{H_t}$, determined by multiplying H_t with a coefficient as follows:

- If the junction incorporates a drainage inlet that intercepts at least 10% of mainline flow, then the coefficient is 1.3.
- When the interception is under 10%, the coefficient is 1.0.
- If the junction incorporates a manhole or a junction box, then the coefficient is 0.5.

Column 17─Gross Total Head Loss. Enter the gross total head loss, H, which is the sum of friction loss, H_f, in Column 8 and adjusted junction loss $\overline{H_t}$, in Column 16.

Col. 18─Potential Water Surface Elevation. Enter the sum of the elevation in Column 3 and *H* in Column 17. This elevation is the potential water surface elevation for the junction under consideration.

Col. 19─Top Elevation of Structure. Enter the top of manhole elevation or the inlet elevation on the gutter flow line (elevation at the top of catch basin). If this elevation is not at least one foot higher than the elevation in Column 18, then the hydraulic gradeline must be lowered to satisfy the design criteria.

The HGL can be lowered by increasing slopes and sizes of pipe, or by reducing the design discharge that can be achieved through proper stormwater management.

Figure 6B-14
Hydraulic Gradeline Computation Form

Project No._____________________ **Sheet No.**______ **of** _____

Computed By: ________ **Chk. By:** _______

Junction No.	Sta.	Outlet Water Surface El.	Pipe Loss					Junction Loss									Inlet Water Surface El.	Top EL of MH or CB.
			D_o	Q_o	L_o	S_{fo}	H_f	V_o	H_o	V_i	H_i	Δ	H_Δ	H_t	$\overline{H_t}$	H		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)

$H_i = 0.35\frac{V_i^2}{2g}$ $H_o = 0.25\frac{V_o^2}{2g}$ $H_\Delta = K\frac{V_i^2}{2g}$ $H_t = H_o + H_i + H_\Delta$ **Final** $H = H_i + H_t$

K Value

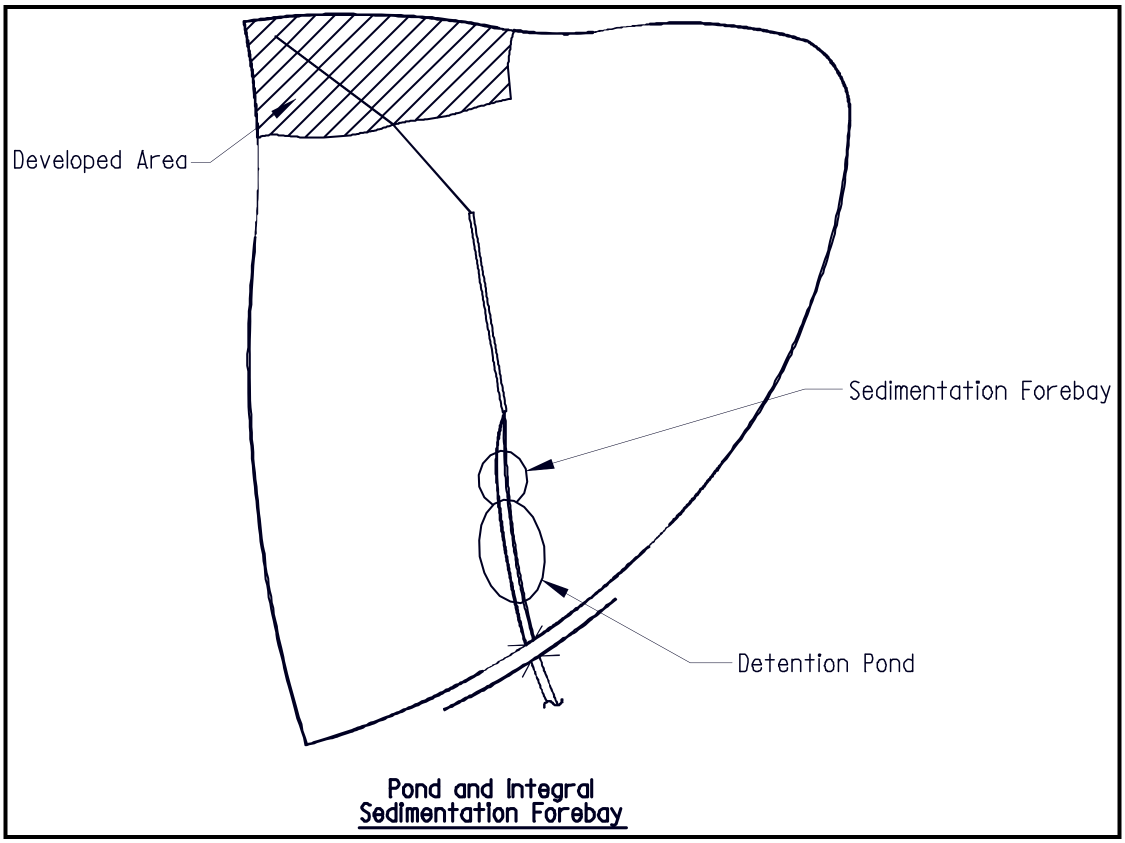
90°	0.70	70°	0.61	50°	0.47	30°	0.28	20°	0.16
80°	0.66	60°	0.55	40°	0.38	25°	0.22	15°	0.10

EXAMPLE 7 ─ POND DESIGN

PROBLEM STATEMENT:

Design a stormwater detention pond for Example 2 using the TR-55 method. The area to be commercially developed is classified as Am and a detention pond with a release structure will control stormwater runoff increases back into the existing channel due to the construction of the commercial site. Since the project is located above the C&D Canal, the design will be based on the TR-55 Tabular Hydrograph Method for developing inflow and out flow routing and stage-discharge relationships that allow for determining the pond size and most efficient control structure.

Figure 6B-15
Location of Pond



This example is simplified to illustrate the basic concept of controlling stormwater runoff through the use of a retention basin. In reality, the regulations would require the new commercial site to provide onsite stormwater quantity and quality features to limit the site discharge to the pre-developed peak flow and reduce the total suspended solids to 80%. The current State regulations should be consulted for design criteria. A detention pond permitted at the location in this example would have to be designed combining each subarea's inflow and outflow hydrographs and routings to the outfall. The analysis would be made for the 2-, 10- and 100-yr Type II 24-hr storms. Bridge Design would be requested to analyze the downstream culvert and channel for possible flooding, as channel stability and wetland encroachment.

Figure 6B-16
Watershed for Example 6

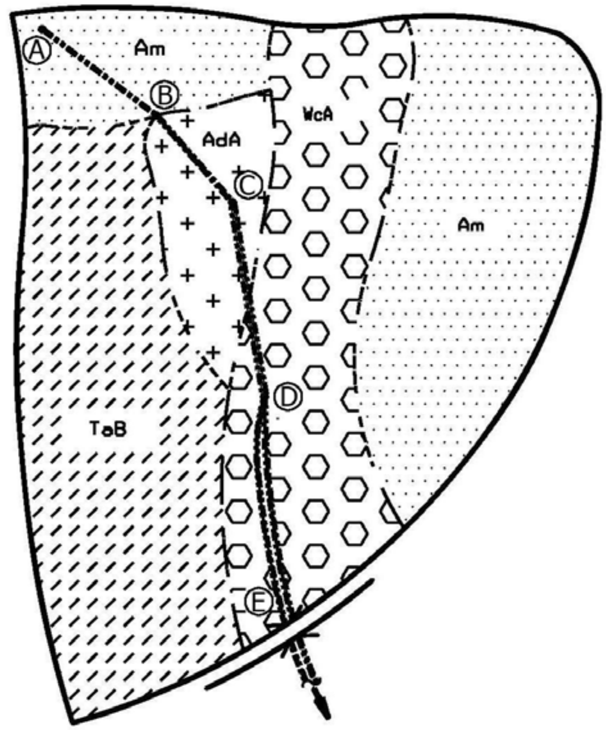


Figure 6B-17
Flow Path of Watershed

Segment	Description	Slope %	Length (ft)
A-B	Commercial and Business	1.0	50
B-C	Forest and meadow	2.0	50
C-D	Shallow gully	0.8	250
D-E	Open channel	0.8	475

The first step is to establish all of the hydrologic parameters and regulatory design controls. The design data for this problem is based on Example 2. The area to be developed is the upper watershed, segment A-B, designated as Am that is to be developed from "1 ac Residential" to "Commercial and Business". A 25-yr frequency is to be used. The data developed in Example 2 applies.

The pre-development design data from Example 2 is:

A_{total} = 36.5 ac or 0.057 mi^2

$\overline{CN} = 74$ q_d = 3.2 in

t_c = 0.43 hr I_a/P = 0.12

q_u = 575 ft^3s/mi/in Q = 105 ft^3/s

Determine the post-development data.

The weighted average curve number due to the proposed construction is computed as:

$$\overline{CN} = \frac{\Sigma\,(CN\ x\ A)}{\Sigma\, A} = \frac{2734.25}{36.50} = 74.9 \cong 75$$

Calculate Runoff Depth q_d from equations 6.9 and 6.10:

$$q_d = \frac{(P_{24} - 0.2S))^2}{P_{24} + 0.8S}$$

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

$$q_d = \frac{\left[6.0 - 0.2\left(\frac{1000}{75} - 10\right)\right]^2}{6.0 + 0.8\left(\frac{1000}{75} - 10\right)} = 3.28in$$

CALCULATE t_C

Use the topographic data from Figures 6B-16 and 17.

Sheet (Overland) Flow (segments A-B & B-C) this type of flow is assumed to be no greater than 3 inches in depth.

Find T_{t1} using equation 6.11:

$$T_{t1} = \frac{0.007\,(nL_1)^{0.8}}{P_2^{0.5} s^{0.4}}$$

From Table 3-1, TR-55, $n_{A\text{-}B}$ = 0.011 (asphalt) and $n_{B\text{-}C}$ = 0.40 (woods, light underbrush). So,

$$T_{t1} = \frac{0.007\,(0.011\ x\ 50)^{0.8}}{(3.2)^{0.5}(0.01)^{0.4}} + \frac{0.007\,(0.40\ x\ 50)^{0.8}}{(3.2)^{0.5}(0.02)^{0.4}} = 0.22hr$$

Shallow Concentrated Flow (segment C-D) assumed to remain the same. Flow depths for this type of flow are assumed to be in the 4 to 6 inch range.

$$T_{t2} = \frac{250}{3600\ x\ 1.44} = 0.048hr$$

Open Channel Flow (segment D-E)

The travel time is calculated the same as for the Rational Method, so T_{t3} = 0.041 hr.

Therefore, from equation 6.8:

t_c = 0.22 + 0.048 + 0.041 = 0.31 hr

DETERMINE UNIT PEAK DISCHARGE q_U

The unit discharge, q_U is determined from Exhibit-II, TR-55.

Find the ratio I_a/P using Table 4-1, TR-55, or using equation 6.10 and that I_a is assumed to be 20% of S:

$$\frac{I_a}{P} = \frac{\frac{200}{CN} - 2}{P_{24}} = \frac{\frac{200}{75} - 2}{6.0} = 0.11$$

Therefore, from Exhibit 4-II:

q_u = 660 ft^3/s /sq mi/in

Pond Adjustment Factor F_p remains F_p = 1.0

CALCULATE *Q*

Drainage Area A_m = 36.5/640 = 0.057 sq mi

From equation 6.14:

$Q_{25\text{-}developed} = q_u\, A_m\, q_d\, F_p$

= 660 x 0.057 x 3.28 x 1.00 = 123 ft^3/s

Therefore, the developed condition peak discharge Q_{25} equals 123 ft^3/s.

Figure 6B-18
Curve Number Computations

Soil Type	Hydrologic Soil Group	Land Use (Good Hydrological Conditions)	*CN*	Area, *A* (Acres)	*CN x A*
Am	C	Business	94	2.50	235.00
Am	C	Residential, ½ Acre	80	12.60	1008.00
AdA	C	Residential, 1/4 Acre	83	0.75	62.25
AdA	C	Woods	70	1.10	77.00
TaB_2	B	Residential, 1/4 Acre	75	1.00	75.00
TaB_2	B	Woods	55	9.00	495.00
WcA	D	Residential, 1 Acre	84	1.75	147.00
WcA	D	Residential, 1/2 Acre	85	4.30	365.50
WcA	D	Woods	77	3.50	269.50
			Total	36.50	2734.25

Figure 6B-19
Impervious Area Computations

Land Use	Acres	Percent Impervious	Impervious Acres
Woods	13.60	12*	1.63
Residential 1 acres	1.75	20	0.35
Residential ½ acres	16.90	25	4.23
Residential ¼ acres	1.75	38	0.67
Business	2.50	85	2.13
		Total	9.01
		Percent	25

*Assumes at least a minimum future change in land use

RETENTION/DETENTION DESIGN

The problem statement is to design a storage facility and an outfall structure that discharges at the pre-development rate. Storage basins are frequently designed to function as a wet pond (retention facility for quantity and quality control) and a detention facility releasing runoff volume at an allowable discharge rate. The design is based on hydrologic routing of inflow and outflow based on several design storms and various types of outfall structures. For the example the pond is being sized for the 25-yr storm that may or may not be in conformance with the current requirement.

Computer models are readily available to perform all the design steps. Even though the example has limited the problem to a single storm event, this is still a very time consuming and substantial undertaking if done manually. The example will describe the process in designing a stormwater management pond.

The following subsection will allow the designer to manually size a trial pond to use in assessing whether a proposed location is even physically feasible and should be further evaluated through the software. The method is from TR-55. As pointed out in the document this method is only about 25% accurate and is to be confirmed by using complete hydrograph routing and volume analysis through a software program.

For generating runoff hydrographs, the State has been divided into two parts with the Chesapeake and Delaware Canal as the dividing line. The reasoning is that the general hydrological soil conditions below the canal provide better initial absorption rate for runoff than above the canal and that the average watershed slope is <5% thus increasing the time to reach peak discharge. Therefore, the NRCS Type II Unit Dimensionless Hydrograph is used above the Canal and the Delmarva Unit Hydrograph is used below the Canal.

POND VOLUME DETERMINATION

The TR-55 approximation method consists of four steps:

Step 1. Based on the design frequency, determine the peak discharges Q_1 and Q_2 for the pre-and post development conditions. Based on the existing criteria, Q_1 is used as the allowable downstream peak discharge.

Step 2. Calculate the ratio Q_1/ Q_2 and obtain the ratio V_s/V_r from Figure 6-1, TR-55, using the Type II curve. For areas below the canal, this value would be somewhat less for ratios above 0.3 and would give a greater storage volume than is probably required. V_s is the required storage volume in acre-feet and V_r is the total runoff volume in acre-feet.

Step 3. The runoff volume is calculated using the equation:

$$V_r = 53.33\, q_d\, A_m \text{ ac-ft}$$

Where:

q_d = Runoff depth in inches for the post-development condition

Am = Drainage area in mi^2

Therefore for this example:

Q_1 = 105 ft3/s $\quad$ Q_2 = 123 ft3/s

Q_1/Q_2 = 0.85

Previously calculated:

q_d = 3.28 in $\quad$ A_m = 0.057 mi^2

From TR-55 Figure 6-1, V_s/Vr = 0.2

The equation for determining this value is:

$$\frac{V_s}{V_r} = 0.683 - 1.43(Q_1/Q_2) + 1.64(Q_1/Q_2)^2 - 0.804(Q_1/Q_2)^3$$

Using this equation the value is 0.16 (use 0.2)

Using the equation for finding V_r:

V_r = (53.33)(3.28)(0.057) = 9.97 ac-ft

Step 4. Determine the storage volume, V_s in acre-feet by multiplying the known V_s/V_r ratio of Step 2 with V_r obtained from Step 3.

Therefore,

V_s = (9.97)(0.2) = 2.0 ac-ft or

V_s = 86,859 ft^3

Using this value an approximation of the average release rate can be determined as follows:

V_s is to be released over a 24-hr period, therefore:

86,859 ft^3/ (24 hrs x 3600 sec/hr)=1.0 ft^3s

Step 5. Determine an initial pond size using Equation 8-8 and 8-9 in TR-55. Assume that the side slopes are 3 to 1, therefore Z = 3. Because, we have selected this slope ratio a safety will be incorporated in the final pond layout. Slopes greater than 4:1 do no currently require a dry safety bench. However, ponds with side slopes 4:1 or less require a 10-foot wide aquatic bench one foot below the permanent pond level and a dry 10-foot safety bench 1 foot above the permanent pond level

Based on the site survey data there is 7 ft of topographic relief available at the proposed pond location and the water table level allows for the following:

Bottom elevation	93.0
Aquatic bench elevation	95.0
Permanent pool elevation	96.0
Outlet structure elevation	96.0
Safety bench elevation	97.0
Design water surface level	98.0
Emergency Outfall	100.0

It should be pointed out that because the designer would like this pond to be a wet pond and to ensure there is no rock or other problems in constructing the pond at this location, soil borings locating the normal water table and type of soils were conducted. If there had been a possibility that the natural aquifer would be contaminated, it would have been necessary to provide an impervious clay liner up to the desired pond depth depending upon the potential adverse affect to the underlying aquifer.

The estimating procedure for sizing a pond is (1) assume a trapezoidal shape with a length (in the direction of flow) to width ratio of 2:1 or greater, (2) side slopes not to exceed 3:1, preferably greater, and (3) provide at least 1-ft of freeboard above the highest design water elevation. It is generally preferred that ponds be irregular in shape with the longest dimension being parallel to the flow.

The formula for determining the volume of a trapezoidal basin is:

$$V = LWD + (L+W)ZD^2 + \frac{4Z^2D^3}{3}$$

where:

V= Volume (ft^3)

L = Length of bottom (ft)

W = Width of bottom (ft)

D = Depth of basin (ft)

Z = Side slope factor (ratio horizontal to vertical)

An approximate method to estimate a trial basin knowing the storage volume is to use the equation:

$$L = \frac{-ZD(r+1) + \left[(ZD)^2(r+1)^2 - 5.33(ZD)^2 r + \frac{4rV}{D}\right]^{0.5}}{2r}$$

r = Ratio of width to length of basin

Using this equation and the available data find the approximate size of the basin. The data is:

Z = 3   D = 2 ft   r = 0.5 V_s = 86,569 ft^3

Results: L = 290 ft   W = 145 ft

The depth of 2 ft is based on the depth available between the permanent pool level and the proposed outfall structure. Because of the inherent assumptions and approximations used in determining stormwater values, the additional storage created by the use of a bench(s) is frequently ignored and the simple trapezoidal shape is used in the calculations.

In this example the benches have been used for determining available storage volumes etc. At elevation 95.0 L and W increase by 32 ft or L= 322 ft and W = 177 ft. at elevation 97.0 L and W increase by another 32 ft or L= 357 ft and W = 209 ft.

Using these dimensions, calculate the available volumes at various elevations.

Elev. 95.0

Data:
L = 290 ft W = 145 ft D = 2 ft Z = 3

V_{95} = 89,416 or 2.05 ac-ft

Elev. 96.0 Permanent Level

Data:
L = 290 ft W = 145 ft D = 2 ft Z = 3

V_{96} = 89,416 or 2.05 ac-ft plus the volume between elev.95.0 and 96.0 using L = 322 ft W = 177 ft D= 1 ft and Z= 3 the additional volume is 58,503 for a total volume at the permanent pool elevation of 147,919 ft^3 or 3.4 ac-ft.

For the storage calculations using the wet pond concept the available storage volume to be released at elevation 96.0 is 0.0. The storage volume available from Elevation 96.0 to 97.0 is:

Elev. 97.0

Data: L = 322 ft W = 177 ft D= 1 ft and Z= 3
V_{97} = 58,503 ft^3 or 1.27 ac-ft.

Next, find the volume at Elevation 98.0

Elev. 98.0

Data:
The volume at 98.0 includes the volume available at 97.0 (58,503 ft^3) plus the volume between these two elevations.
Data: L = 354 ft W = 209 ft D = 1 ft Z = 3

V_{98} = 58,503 + 75,687 = 134,190 ft^3
or 3.1 ac-ft.

The design has set elevation 96.0 as the elevation for any outfall structures with storage between elevation 96.0 and 98.0. The allowable storage must be at least equal to the calculated volume of 86,859 ft^3.

Elev. 99

Data: L = 354 ft W = 209 ft D = 2 ft Z = 3

V_{99} = 58,503 + 154,824 = 213,329 ft^3
or 4.9 ac-ft.

Elev. 100

Data: L = 354 ft W = 209 ft D = 3 ft Z = 3

V_{100} = 58,503 + 237,483 = 295,986 ft^3
or 6.8 ac-ft.

At this point a review of the initial basin configuration would indicate that the storage volume between elevations 96.0 and 98.0 is 3.15 ac-ft and is considerably larger than the 2.0 ac-ft required. Therefore iterations of several dimensions would probably be necessary. One of the easiest solutions would be to use the initial V_s divide by the allowable depth to find the required surface area at the design storage elevation. Than using the desired between length to width of at least 2:1 as selected in the example, L = 2W in the equation L x W = Surface area and making the substitution, $2W^2$ = Area. Therefore:

$V_s/2$ =A or 86,569/2 = 43285 ft^2

$2W^2$ = 43285 or W = 147 and L = 294.

These two dimensions apply at elevation 98.0 on a trapezoidal shape having a bottom elevation of 93.0. At this elevation, the two dimensions would be reduced by 30 feet would result in a trial size of L = 260 and W = 130. The volume equation for a trapezoid allows for a quick check of available storage between 93.0 and 96.0 V = 2.36 and between 93.0 and 98.0 V = 4.49. The available storage would be the difference between these two values or 2.13 ac-ft. This value is closer to the required volume. The proposed two benches will provide the additional storage to meet the inherent margin of error recognized in drainage design. Figure 6B-21 shows the tabulated values using the second trial size.

Figure 6B-20
Trial 1 (L=290 ft & W=145ft) Stage Storage

Elevation	Surface Area (ac)	Cumulative Storage (ac-ft)*	Usable Storage (ac-ft)**
93	0.97	0.00	0.00
94	1.03	1.0	0.00
95	1.09	2.05	0.00
95.1***	1.31	2.19	0.00
96	1.38	3.39	0.00
97	1.45	4.8	1.41
97.1	1.70	5.05	1.66
98	1.78	6.54	3.15
99	1.86	8.35	4.96
100	1.94	10.25	6.86

*Assumes pond is completely empty and is used for calculating quality control storage later.
**Assumes pond is full to Elevation 96.0 (permanent pool).
***Often on the first trial the bench elevations are not known. In this problem, assume there is a requirement for a permanent pool at Elevation 96.0, so the benches go at 95.0 and 97.0.

Figure 6B-21
Stage-Storage
Trial 2 (L=260 ft & W=130 ft)

Elevation	Surface Area (ac)	Cumulative Storage Volume (ac-ft)*	Usable Storage Volume (ac-ft)**
93	0. 78	0.00	0.00
94	0.83	0.80	0.00
95	0.88	1.66	0.00
95.1***	1.08	1.77	0.00
96	1.15	2.78	0.00
97	1.21	3.96	1.18
97.1	1.44	4.11	1.33
98	1.52	5.44	2.66
99	1.59	7.0	4.22
100	1.66	8.62	5.84

*Assumes pond is completely empty and is used for calculating quality control storage later.
**Assumes pond is full to Elevation 96.0 (permanent pool).
***Often on the first trial the bench elevations are not known. In this problem, assume there is a requirement for a permanent pool at Elevation 96.0, so the benches go at 95.0 and 97.0.

If the designer is confidant that the initial trial size is suitable for the designated site, then the design can be refined and verified through the appropriate computer software. The necessary input data for the software has been generated and the program will develop and route an inflow hydrographs through the pond, generate an outflow hydrograph based on the selected principal and emergency spillway configuration. The designer selects and inputs spillway types and sizes considered appropriate for the site and downstream conditions. This structure could be; a concrete wall built into the embankment with one or series of broad crested or v-notch weirs; or a riser pipe with an outfall pipe (barrel); or a rectangular riser structure with one or more weir and orifice openings at different elevations to control a combination of storm frequencies and an outfall pipe. Risers with a pipe outlet are frequently referred to as drop inlets.

There are several types of flow calculations that have to be made. The pipe discharge flow capacity of any outfall pipe, the riser flow based on weir or orifice flow conditions, and weir flow when using either rectangular or trapezoidal (V-notch) shaped openings in the riser structure. In undeveloped areas with a natural outfall stream or channel, the broad crested weir or the V-notch weir without a riser structure is more economical and can be constructed as part of the pond embankment. Some of these flow analysis have to include structure losses, i.e. friction and various tailwater conditions. The software will include an analysis of the emergency spillway based on the designer's input data. The final product is the stage-discharge relationship shown as a curve or tabulation that sums all flows through both spillways based on the water depth in the pond. Using this data, the designer can determine if all design parameters have been met. If not adjustment in input data are made and the program rerun.

Now that the first goal of sizing the pond and determining the spillway design has been met, the other required post-development storm flows have to be routed through the pond to verify that the design also meets the criteria set for them. Normally, the analysis will include the 2-, 10-, and 100- year storms are evaluated. Current regulations set which one of these will control the pond size. The stage-discharge data for all the storms are used to confirm any predetermined pond elevations such as benches. Any requirement for providing a permanent pool volume also enters into this evaluation. Usually the 100-yr storm flow sets the emergency spillway capacity and required total depth of the pond.

As initially stated using computer software is almost a necessary in order to select the most feasible and economical design. However, the designer's input data and choices in alternative design selections are crucial in making this happen. HEC-22 and the other resources listed in Section 6-10, 11 and 12 have detailed examples for pond design.