STORM WATER MANAGEMENT GUIDEBOOK

Michigan Department of Environmental Quality
Land and Water Management Division

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Land and Water Management Division

Originally published March 1992
Revised August 1999

John Engler, Governor
Russell J. Harding, Director

DEQ Internet Home Page: www.michigan.gov/egle

DEQ, Land and Water Management Division
517-284-5567

Environmental Assistance Center
1-800-662-9278

Printed by the Authority of the
Clean Water Act, Section 319

Total Number of Copies Printed:
Total Cost:
Cost per Copy:

PRINTED ON RECYCLED PAPER

Michigan Department of Environmental Quality

ACKNOWLEDGEMENTS

This guidebook was funded by the United States Environmental Protection Agency. The substance and findings of this work are dedicated to the public. The Land and Water Management Division of the Michigan Department of Environmental Quality is solely responsible for the accuracy of the statements and interpretations contained in this publication. Such interpretations do not necessarily reflect the views of the federal government.

The preparation of this manual was made possible through the input and guidance of the following people:

Ms. Deborah Allen, Ms. Margaret Bostwick, Mr. James Boulton, Mr. Douglas Carter, Mr. Gerald Fulcher, Mr. David Hamilton, Mr. George Hosek, Mr. Kameron Jordan, Ms. Susan Klco, Ms. Ronda Phillips-Scales, Ms. Donna Richert, and Mr. Wallace Wilson of the Michigan Department of Environmental Quality.

Mr. Jeffrey Friedle of the Michigan Department of Agriculture.

Mr. Russell Bauerle and Tom Bourdon of the United States Department of Agriculture, Natural Resources Conservation Service.

Mr. Alan Boyer of Ledy Engineering.

Mr. Thomas Davenport of the United States Environmental Protection Agency.

Special thanks go to Ms. Carolyn Reed and Ms. Bonnie Weber her word-processing skills and patience shown in the preparation of this guidebook.

Special thanks go to Mr. Alan Wescoat for his word processing and graphics editing.

EXECUTIVE SUMMARY

Currently, the lack of standard stormwater-management practices within Michigan can result in facilities that do not function properly or are counterproductive. The purpose of this guidebook is to provide a reference for state and local officials and engineering consultants on stormwater management for both water-quantity and water-quality concerns. The primary focus will be on the design of stormwater retention/detention basins. In addition, the following topics are also included:

- Stormwater-management measures
- Hydraulics
- Hydrology
- Operation & maintenance
- Financing
- Laws & Ordinances relating to stormwater management

The guidebook also includes a reference listing sources of additional information on stormwater management.

It is not the intent of this guidebook to recommend design practices that will be used statewide, under all circumstances, or in all communities. Instead, the guidebook is intended to be used as a reference when considering solutions to specific problems, as it discusses what is being done in stormwater management throughout the country.

DEFINITIONS

Acre-foot - a volume of water 1 foot deep and 1 acre in area, or 43,560 cubic feet.

Aerator - a device that sprays water into the air, bubbles air through the water, or agitates the water, to incorporate oxygen into the water.

Antecedent Moisture Condition (AMC) - the quantity of moisture present in the soil at the beginning of a rainfall event. The Natural Resources Conservation Service has three classifications, AMC I, II, and III.

Backwater - the increased depth of water upstream of an obstruction, such as a dam or bridge, in the stream channel.

Base Flow - the part of the stream flow that is not due to direct runoff from precipitation; it is usually supported by water draining from natural storage in groundwater bodies, lakes, or wetlands.

Bedload - the sediment in a stream channel that moves by sliding, rolling, or skipping on or near the stream bottom.

Best Management Practice (BMP) - a practice or combination of practices that form an effective, practicable means of preventing or reducing the amount of pollution generated by non-point sources.

Bottomland - the land of a lake or stream which lies below the ordinary high-water mark of the lake or stream.

Culvert - a closed conduit used for the passage of surface water under a road or other embankment.

Curve Number - see runoff-curve number.

Detention Basin - temporarily stores water before discharging into a surface-water body. Primarily used to reduce flood peaks. Can be classified into three groups:

1. <u>Dry Detention Basin</u> - usually dry except for short periods following large rainstorms or snowmelt events. Not effective at removing pollutants. Pollutants that may settle in the basin will be "picked up" by future floods.

2. <u>Extended Dry Detention Basin</u> - is a dry detention basin that has been modified to increase the time which the stormwater will be detained in the basin. The typical detention time is 24 to 48 hours. Not effective at removing nutrients such as phosphorus and nitrogen, unless a shallow marsh at the outlet is incorporated into the design.

3. <u>Wet Detention Pond</u> - a detention basin that contains a permanent pool of water that will effectively remove nutrients in addition to other pollutants.

Detention Time - the amount of time that a volume of water will remain in the detention basin.

Discharge - the rate of flow (volume of water passing a point in a given period of time). Usually expressed as cubic feet per second.

Drainage Area - the area of a watershed usually expressed in square miles or acres.

Drainage Divide - the line which follows the ridges and high points of the ground surface that separate one drainage basin from another.

Emergency Spillway - a depression in the embankment of a pond or basin which is used to pass peak discharges in excess of the design storm.

Eutrophication - the process of enrichment of water bodies by plant nutrients which may lead to increased growth of algae or rooted plants.

First Flush - highly concentrated pollutant loading during the early portion of stormwater runoff due to the rapid runoff of accumulated pollutants.

Forebay - an extra storage area provided near the inlet to a detention basin to trap incoming sediments before they accumulate in the basin.

Hydraulic Radius - the area of the culvert or stream section divided by wetted perimeter (A/WP).

Hydrograph - a graph, usually of discharge or stage versus time, at a given point along a stream.

Hydrologic Cycle - the continuous process of the exchange of water between the earth and the atmosphere.

Impervious - a surface through which little or no water will move. Impervious areas include paved parking lots and roof tops.

Infiltration - the absorption of water into the ground.

Infiltration Capacity - the maximum rate at which the soil can absorb falling rain or melting snow. Usually expressed in inches/hour or centimeters/second.

In-line Detention - the detention is provided within the flow-carrying network (stream) .

Manning's Roughness Coefficient ("n") - a coefficient used in Manning's equation to describe the resistance to flow due to the roughness of a culvert or stream channel.

Mean Storm - over a long period of years, the average rainfall event, usually expressed in inches.

Mean Storm Volume - the runoff volume produced by the "mean storm."

Moisture Content - see antecedent moisture condition.

Non-Point Source Pollution - pollution that is not identifiable to one particular source, and is occurring at locations scattered throughout the drainage basin. Typical sources include erosion, agricultural activities, and urban runoff.

Off-Line Detention - detention placed outside of the natural watercourse or storm sewer system.

Off-Site Detention - detention is provided at a regional detention facility as opposed to storage on site.

One-Hundred-Year Flood (100-year flood) - the flood that has a 1-percent chance of occurring any given year.

On-Site Detention - stormwater is detained on the property as opposed to a regional site.

Ordinary High Water - marks the line between upland and bottomland which persists through successive changes in water level, below which the presence of water is so common or recurrent that the character of the soil and vegetation is markedly different from the upland.

Orifice - an opening in a wall or a plate.

Peak Discharge - the maximum instantaneous rate of flow during a storm.

Pervious - a surface that will allow water to infiltrate into the ground.

Pilot Channel - a channel that routes runoff through a detention basin to prevent erosion of the basin.

Point-Source Pollution - pollution that occurs at a specific location, such as an outlet pipe, and is usually continuous.

Precipitation - the supply of water received from the atmosphere, such as rain, snow, and hail.

Rating Curve - a curve that expresses a relationship between dependent quantities. Typically the graph will plot stage (elevation) versus discharge.

Regression Analysis - independent variables (such as drainage area and precipitation) are selected which relate to a dependent variable (discharge). Once an equation is developed, a discharge may be computed by knowing the independent variables. Such an analysis has been developed based on an evaluation of the stream gaging stations throughout Michigan.

Retention Pond - a stormwater management practice that captures stormwater runoff and does not discharge directly to a surface water body. The water is "discharged" by infiltration or evaporation.

Retrofit - to modify an existing structure to improve the pollutant-removal or flood-peak-reduction capability.

Riser - a vertical pipe attached to the outlet pipe of a detention basin that is used to control the discharge rate from the basin.

Routing - the derivation of an outflow hydrograph for a given reach of stream or detention pond from known inflow characteristics. The procedure uses storage and discharge relationships and/or wave velocity.

Runoff - the excess portion of precipitation that does not infiltrate into the ground, but "runs off" and reaches a stream, water body, or storm drain.

Runoff-Curve Number - indicates the runoff potential of a parcel and is based on soil group and land use. The higher the runoff-curve number, the higher the runoff potential.

Sediment - material that is being transported from its site of origin by water. May be in the form of bedload (along the bed), bouncing along the bed, suspended or dissolved.

Short Circuiting - the runoff does not spend enough time in a detention facility to remove the pollutants for which the facility was designed to remove.

Stormwater Utility - a source of funding the construction and maintenance of stormwater management facilities. User fees are typically charged based on the amount of runoff that may be anticipated from a property.

Swale - a slight depression or shallow ditch which can be used to convey, store, or filter runoff.

Time of Concentration - the time it takes for runoff to travel from the hydraulically farthest portion of the watershed to the design point.

Timing - the relationship in time of how runoff from sub-watersheds combines within a watershed.

Weir - a device that has a crest and some side containment, and is used to measure, regulate, or restrict flow. The amount of flow that may pass over the weir is a function of the weir geometry and upstream height of water above the crest.

Wetted Perimeter - the wetted surface of a stream (culvert) cross section which causes resistance to flow. The water-to-surface interface is a length, usually expressed in feet.

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INTRODUCTION

It was not very long ago that stormwater management meant increasing the size of the storm-sewer pipes or enlarging drains to allow stormwater to get away from an area as quickly as possible. However, in many instances this "solution" resulted in increased flooding, erosion, and water-quality problems in downstream areas.

Over the years, people have become more aware of the potential problems caused by increased runoff due to urbanization and increased flood peaks due to drain improvements. In an attempt to remedy the increased flooding and erosion problems, communities (and some states) began to implement stormwater detention.

Typically, stormwater detention involved the construction of dry detention basins that would reduce downstream discharges. The detention basins would "hold back" some of the runoff to be released at a later time. However, in some instances, detention basins were constructed that did not consider the hydrology of the entire watershed. As a result, the basins had little impact on flood discharges and, at times, actually increased flood peaks.

Stormwater management was originally concerned with the quantity of water and the downstream flooding potential. However, over the last 10 to 15 years, there has been a growing concern with the **quality** of the stormwater runoff and its impact on the environment. Stormwater runoff picks up pollutants that have accumulated on the land surface and washes them into receiving waters. The pollutants can include sediment, nutrients, and heavy metals to name a few. They enter the food chain, destroy aquatic habitat, and can essentially "kill" a lake or stream.

As a result of the water quantity and quality concerns, stormwater management has begun to evolve into a field that tries to integrate reducing future flood damages with water-quality improvements. The information presented in this guidebook will provide some background in stormwater management and will offer some approaches to addressing the urban runoff problem.

CHAPTER 1: BACKGROUND

HYDROLOGIC CYCLE

The first step in understanding stormwater management is to develop a feeling for the hydrologic cycle and how it is impacted by development. In simple terms, the hydrologic cycle involves the exchange of water between the earth and the atmosphere. Water is transported from the oceans to the atmosphere by evaporation, where it condenses and falls to the land in the form of precipitation. The water then makes its way back to the ocean, where the cycle is repeated. This is obviously a very simplistic explanation, as there are many sub-cycles (see figure 1.1) within the hydrologic cycle for the earth.

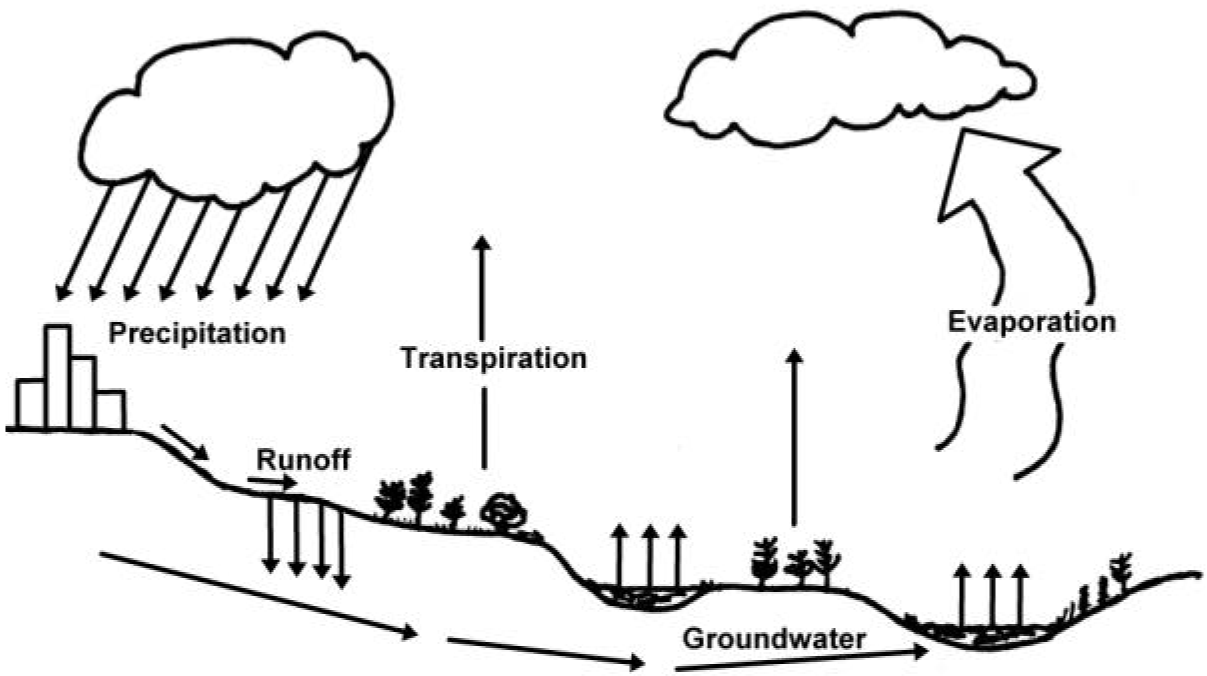


Figure 1.1 - Generalized Hydrologic Cycle

For stormwater management, we are primarily concerned about the portion of the hydrologic cycle that includes precipitation, infiltration, and runoff.

Whether in the form of rain or snow, **precipitation,** is the driving force behind the design of stormwater management facilities. The precipitation that occurs is either intercepted by vegetation (trees, plants, and etc.), evaporates, infiltrates into the ground, or results in runoff. Rainfall will be the primary focus of this guidebook.

Later on in the guidebook, we will discuss rainfall amounts and design suggestions for areas within Michigan. Obviously, snow can be a factor in estimating runoff volumes. Snow may not be immediately converted to runoff, as it is frozen and is "detained" until it can melt and runoff. However, when the snow does melt, it typically occurs in conjunction with a rainfall event which can compound the "runoff" problem.

Infiltration

The precipitation that infiltrates into the ground is either absorbed by the plants and soil or continues through the soil until it reaches the groundwater. The rate at which the water will infiltrate into the ground is dependent primarily upon three factors: soil type, soil moisture content, and land use.

One of the characteristics of soils is the ability to "absorb" moisture. A **soil type** such as sand has a high infiltration rate, while clay has a very low infiltration rate. Thus, all things being equal, a parcel with clay soil will produce higher runoff than if the soil is sand.

The **moisture content** of a soil also has considerable impact on the infiltration capacity of the soil. As an example, an area that has not received any rain in the last ten days will have a higher infiltration capacity than if it had received three inches of rain in the last two days. Thus, it is possible for an area that has not received any rain in the last couple of weeks to receive a "100-year rain" but not have a 100-year flood, since a large portion of the rainfall can be absorbed by the ground. Conversely, if a soil is saturated from recent rains, it may not take a 100-year rain to produce a "100-year flood".

Finally, the **land use** has a significant impact on the infiltration capacity of the soil. Residential, industrial, commercial developments, parking lots, and roads all result in the construction of **impervious** surfaces, such as pavement and rooftops. These impervious surfaces prevent water from infiltrating into the soil. If the water cannot infiltrate into the ground, runoff will result.

Even if impervious surfaces are not constructed, a change in land use can alter the runoff potential. Changing the land use from a meadow to straight row crops will increase the runoff potential. In this example, impervious surfaces were not added; however, changing the type and "density" of vegetation would impact the runoff volume.

Precipitation will become **runoff** when the infiltration capacity of the soil is exceeded by the intensity of the precipitation. In other words, "it comes down faster than it can soak in". As noted earlier, the amount of runoff will vary as the infiltration or land use is changed. It is this runoff that will be addressed in this guidebook.

IMPACTS OF URBANIZATION

The primary concerns in designing stormwater management facilities include the runoff volumes, the runoff peaks, and the pollutants carried by the runoff.

Runoff Volume

From figure 1.1, it can be seen that the runoff volume is a function of the amount of precipitation and infiltration. (During a precipitation event, evaporation and transpiration of water from plants to the atmosphere do not significantly affect the runoff.) It is apparent that the infiltration plays a key role in the quantity of runoff during a precipitation event.

As land is developed through the construction of buildings, roads, parking lots, and the like, infiltration capacity of a parcel of land is altered. The vegetation that allowed water to infiltrate into the soil is replaced by concrete and asphalt which are essentially impermeable. Instead of infiltrating into the soil, the water is forced to "runoff."

As an example, if a particular parcel is forested and has a sandy loam soil, the runoff from a 2-inch, 24-hour rain would be negligible. If that same parcel were a commercial area, such as a shopping mall or a central business district, a 2 –inch, 24 -hour rain would result in over 1.2 inches of runoff.

To get 1.2 inches of runoff from the parcel in a "natural" condition would require a 24 -hour rainfall of about 5.5 inches. In many areas of Michigan, a rainfall of 5.5 inches in 24 hours would have a frequency greater than a 100-year event.

Thus, if this parcel were to be developed from forest to a commercial area without regard to detention or retention, a rainfall that may occur 1 to 2 times in a year will have a runoff volume that is equal to a 100-year event prior to development. It is this increase in potential runoff volume that has raised the awareness of citizens in regard to stormwater management and the impact on downstream flooding.

Runoff Peaks

As runoff volumes are increased by urbanization and development, the potential for downstream flooding also increases. An increase of flooding problems will result in citizens demanding solutions to the flood problems. Typical solutions to flooding problems have been channelization, removing the "obstructions," and installation of larger-capacity storm drains. The primary goal has been to get the water out of the community and downstream as fast as possible. However, more often than not, improving the "systems" that transport the runoff, has simply passed the problem on to a downstream community. The channelization of streams and the installation of storm drains all result in the runoff reaching a location quicker. As figure 1.2 illustrates, the result of the "improvements" include:

- A flood peak that is larger than pre-development conditions.

- A flood peak that gets downstream quicker.

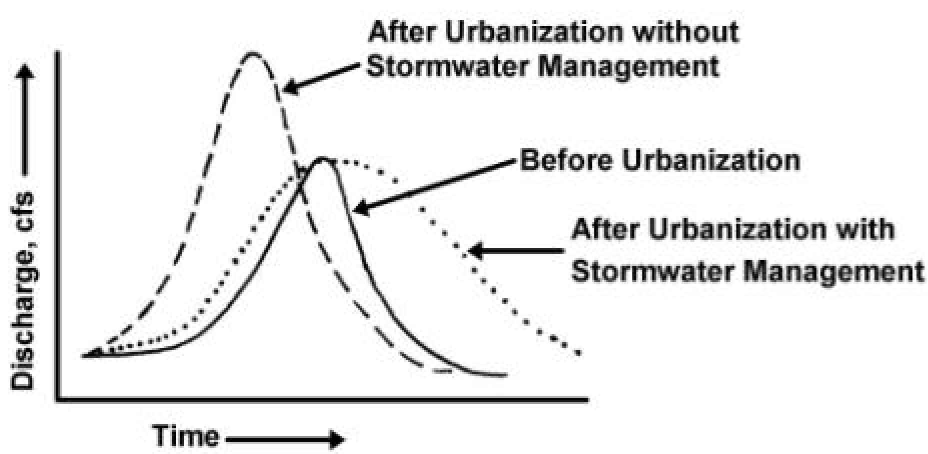


Figure 1.2 - Impact of Urbanization on Flood Peaks

Figure 1.2 also shows that proper stormwater management can limit the peak discharge on a stream to pre-development conditions by the controlling the time at which the runoff is allowed to travel downstream.

In addition to increasing the capacity of the transport system, urbanization will also tend to decrease the naturally occurring areas that provide storage of stormwater areas. Under natural conditions, a portion of the runoff will be captured in the natural storage areas, and will be **slowly** released back into the rivers and streams. The elimination of wetlands, depressions, or small ponds results in a greater runoff volume reaching the rivers and streams more quickly. As a result, flood peaks are increased, and the river levels will rise more rapidly. The elimination of natural storage areas can also lead to reduced base flow in streams during dry periods, which will degrade fish habitat.

Pollutants

As development takes place, there is an increase in the amount of materials that can be picked up by the stormwater runoff. Materials such as sediment, oils, toxic chemicals, fecal waste, and road salt all may be carried with the stormwater to a lake or stream. The construction of paved parking lots, streets with curbs and gutters, and storm sewers, result in little opportunity for the pollutants to settle out. The velocity of the transport systems keeps the pollutants in suspension until the runoff reaches a lake or a slower moving river. As a result, the water quality of the "receiving waters" will be diminished.

Thus, in addition to controlling the runoff volume and runoff peaks, an objective of stormwater management is also to improve the water **quality** of the stormwater runoff.

NON-POINT SOURCE POLLUTION

The sources that pollute a water-body can be classified into two groups: point source and non-point source pollution.

Point Source

As the name implies, point source pollution occurs at a specific location with a relatively consistent quality. The typical point source that is thought of as an example is an outlet pipe from an industrial complex, or a wastewater treatment plant.

Non-point Source

Non-point source pollution differs from point source in several ways:

1. It is not possible to identify one particular source. The pollution is occurring at locations scattered throughout the drainage basin.

2. The pollution is transported in a wide range of flows, with the majority of the pollutant transport occurring during runoff events due to a rainfall or snowmelt.

3. The quality of the runoff varies considerably during an event.

Non-point source pollution occurs in both urban **and** rural areas. In rural areas, non-point source pollution can result from construction-site erosion, agricultural activities (pesticides, herbicides, animal waste, and erosion), and natural erosion.

Best Management Practices

In 1972, the Federal Clean Water Act was amended to require permits for all point source discharges of pollutants to the waters of the United States. Throughout the 1970's, the primary focus of pollution control was the control of point source pollution. In the last 15 to 20 years there has been an increased awareness in non-point source pollution. Due to the nature of non-point source pollution, it became evident that it was not technically and economically feasible to eliminate **all** non-point source pollution. The term "best management practices" or **BMP** became popular. The U.S. Environmental Protection Agency defines BMPs as a practice or combination of practices that are effective, practicable means of preventing or reducing the amount of pollution generated by non-point sources.

BMPs can be divided into four categories, by identifying the methods which reduce the pollutant level of runoff discharging into a surface water body.

1. **Detention** - Water is temporarily stored before it discharges directly into a surface-water body. While the water is detained, the pollutant concentration can be reduced, as suspended solids and some pollutants settle out.

2. **Retention** (infiltration) - Water flows directly into the basin, and is not released. Water will leave the basin through infiltration and evaporation.

3. **Vegetated Swales & Strips** - The vegetation acts as a filter as it collects sedimentation and other pollutants. Water is also able to infiltrate as it is being transported by vegetated swales to a surface water body. The swales may be designed to "absorb" a given runoff condition, or it may be necessary to install a berm or "block" to detain the flow.

4. **Other Practices** - Reduce accumulated pollutants available to be picked-up by runoff. This may include sweeping parking lots and streets, catch-basin cleaning, erosion control enforcement, and infiltration of runoff from driveways and roofs. Regulate the amount of impervious area permitted through the use of zoning and ordinances. Eliminate inappropriate discharges to drains and storm sewers, such as sanitary or industrial sewage.

BMP methods are selected based on the water quality needs along with cost, drainage area, land use, soil, and topography. Using BMP, the stormwater management practices that are selected achieve the water quality needs in the most effective manner.

By incorporating several Best Management Practices, additional water quality benefits will be obtained, as opposed to relying on a single practice, such as the construction of a regional extended detention basin. A detention facility may be only a portion of the total BMP system, which may include:

a) Directing the runoff from downspouts and parking lots to vegetated swales or vegetated strips, instead of discharging directly to a stream.

b) Instituting and enforcing soil erosion control policies, including requiring a vegetated strip between cultivated land and a watercourse.

c) Instituting a policy of regular stormwater system maintenance, including street sweeping and cleaning catch basins; detecting and eliminating inappropriate hook-ups to storm drains.

d) Educating the public in the use of fertilizers, herbicides, and pesticides; in how to properly dispose of oils, paints, chemicals, and other waste/trash; and for the need for vegetated strips and wetland areas along lakes and streams.

Figure 1.3 indicates the "structural" type of BMP that would be feasible for given types of restrictions at a particular site. Later chapters will further discuss the restrictions and design guidelines.

	Soil Type								Drainage Area Acres		
BMP Type	Sandy-Loam	Loam	Silt-Loam	Silty Clay-Loam	High Water Table	High Sediment Input	Thermal Impacts	Limited Space	<5	5-20	20-100
Extended Detention	F	F	F	F	F	M	F	N	N	M	F
Wet Pond	M	F	F	F	F	M	N	N	N	M	F
Infiltration Basin	F	F	M	N	N	N	F	M	M	F	M
Grassed Swale	F	F	M	M	N	N	F	F	F	M	N
Filter Strip	F	F	F	M	M	N	F	F	F	M	N
Infiltration rate (in./hr.)	1.02	0.52	0.27	0.06							

Legend:
- F Feasible
- M Marginal - requires careful planning
- N Not Recommended

Figure 1.3 - Restrictions on BMPs*
*(Best Management Practices)
Source: References 10 & 38

Pollutants & Sources

There is large variety of pollutants that may be present in stormwater management, depending on the land use within the drainage basin. Following is a listing of some of the pollutants that are commonly found in stormwater runoff.

Sediment and Suspended solids: Sediments and other suspended solids account for the greatest amount of pollutants carried by stormwater runoff. Sediments can clog the gills of fish, cover spawning areas, harmfully affect other aquatic life, and reduce the flow-carrying capacity of the watercourse. The sediments may carry heavy metals and other contaminants.

The suspended solids can come from a wide range of sources including any activity that disturbs the land surface, such as clearing and grading activities, agricultural activities, and residential activities. Sediment will also occur from streets and road, and will occur naturally in the form of stream-bank erosion.

Heavy Metals: These pollutants include primarily copper, lead, zinc, and cadmium. Such metals can have a toxic impact on the aquatic life, and can contaminate the drinking water supply. The heavy metal pollutants can result from corrosion of metals, wood preservatives, algicides, paints, and electroplating. The metals are "picked up" by runoff from a variety of urban locations.

Oil and Grease: This category includes various hydrocarbon compounds, such as gasoline, oil, grease, and asphalt. The automobile is a major contributor of this pollutant. These pollutants will be picked up primarily from run-off from parking lots and streets.

Nutrients: The addition of phosphorus and nitrogen to the stormwater can result in increase growth of algae, odors, and decreased oxygen levels in the receiving waters. Such nutrient problems are particularly noticeable in detention ponds that have a detention time of greater than two weeks.

The nutrients typically come from sewage, and fertilizers used at homes, parks, golf courses, and in agriculture. In some areas, there are problems with sewers illegally connected to storm drains.

Fecal Coliform Bacteria: Bacteria are typically present in stormwater runoff. The bacteria may be a result of sanitary sewer overload, animal waste, or other sources that have not been identified. The introduction of bacteria into a receiving water body can make the water unfit for recreation and human use.

Oxygen Demand: Oxygen demand is a result of the decomposition of organic materials. If the depletion of dissolved oxygen is a concern to the receiving waters, it may be necessary to treat stormwater runoff with advanced wastewater treatment.

Other pollutants: There are many other pollutants, such as pesticides, chemical solvents, and phenols that may be found in stormwater runoff, but they are usually at very low concentrations.

"First Flush"

Most automobile drivers are aware that roads are the "slipperiest" after the first few minutes of a rainstorm. It is in those first few minutes that oil, grease, lead, and other pollutants that have accumulated on the pavement are picked up by the water on the roadway, and transported to storm drains or roadside ditches.

Stormwater runoff will result in concentrated pollutants being loaded into the storm -drains and receiving waters. As the rain continues, there are fewer pollutants available to be carried by the runoff, and thus the pollutant concentration becomes lower. Figure 1.4 shows a typical plot of pollutant concentration versus time. The sharp rise in the plot has been termed the "**first-flush**".

Some studies have yielded results which dispute the first flush theory. However, water quality measures that capture the first one-half inch of runoff would capture a high percentage of the runoff events that occur in Michigan. As a result, it is possible to capture a high percentage of the pollutants by retaining the first one-half inch of runoff.

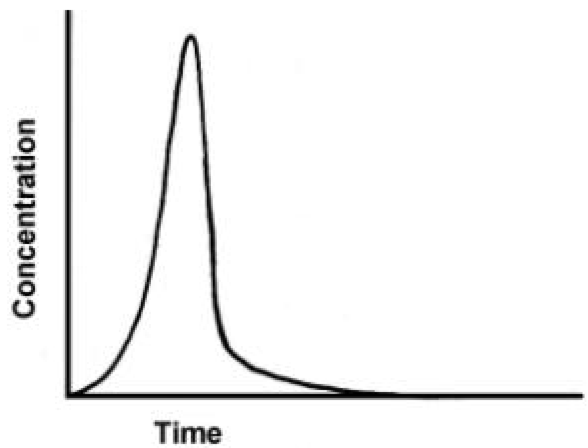


Figure 1.4 - Plot of Pollutant Concentration versus Time

In essence, most of the pollutants that have accumulated within the drainage basin since the last rain are "flushed" into the stormwater system in a very concentrated form. It is this initial pollutant loading that should be the prime concern of any stormwater management design. The design considerations will be discussed later on in the guidebook.

CHAPTER 2: LAWS & ORDINANCES

The following section will briefly describe the federal, state, and local laws currently in effect that have some impact on stormwater management. The purpose of this section is not to give a detailed analysis of each law, but to give a brief description of the law and how it may impact stormwater management. If additional information is needed about the law, it would be advisable to obtain a copy of the specific Act and the accompanying administrative rules.

FEDERAL LEGISLATION

One of the first national pieces of legislation that dealt with water quality was the Federal Pollution Control Act of 1948. In 1972, amendments to the Act (PL 92 -500) shifted responsibility from the state and local governments to the federal government. The amendment required National Pollutant Discharge Elimination System (NPDES) permits for all point source discharges in the United States. Due to limitations, the primary focus was on industrial and wastewater treatment plant discharges.

Section 404 of the Clean Water Act regulates the placement of fill in waters or wetlands of the United States. The Michigan Department of Environmental Quality (DEQ) administers a permit program under this section for interior portions of the state; in coastal areas, 404 permits must be obtained from the U.S. Army Corps of Engineers.

Section 405 of the Water Quality Act of 1987, amended section 402 of the Clean Water Act of 1972 by requiring the EPA to produce regulations requiring permit applications, no later than February 4, 1989, for stormwater discharges from industrial activity, and storm sewers from municipalities with populations of 250,000 people or more. February 1, 1992, the requirements included municipalities with populations of 100,000 people or more.

The industries that will be required to apply for permits under this Act cover a wide range. One notable industry is the construction industry, for activities that will disturb more than 5 acres of land. A notable exemption to the Act is the agriculture industry.

As a result of the amendments to the Water Quality Act, there will be an increased effort to eliminate non-storm water discharges into storm drains. There will also be an effort to reduce the discharge of pollutants through management, controls, and engineering methods. However, at this time, it is not known how the Act will be administered.

This legislation will increase awareness for stormwater management, non -point source pollution, and BMPs over the next few years. Specifics on the legislation can be obtained from the Surface Water Quality Division of the Michigan Department of Environmental Quality at 517-373-1949, or EPA Region V, 77 West Jackson Blvd., Chicago, Illinois 60604.

STATE LEGISLATION

Part 31, Water Resources Protection, of the Natural Resources and Environmental Protection Act, 1994 PA 451, as amended (NREPA). The Act was created to protect and conserve the water resources of the state. This includes the prohibition of pollution of the state's waters, and to prohibit the obstruction and occupation of the floodways, and prohibit activity that would harmfully interfere with the stage discharge characteristics of the rivers and streams of the state.

The "quantity" portion of Part 31 focuses on the floodway occupation and the harmful interference aspects, and does not specifically regulate stormwater runoff. The "harmful interference" portion of Part 31 may be a factor in a stormwater management design that involves in-line detention. The construction of structures which restrict or detain water must not increase the flooding potential onto another person's property without compensation. There are currently no state regulations that specifically address stormwater runoff. To date stormwater regulation has remained at the local level.

The administration of the "quality" portion of Part 31 has primarily focused on point source pollution. In the past, it had been considered not economically practical to monitor and regulate the sources of non-point pollution, since it can come from such a wide range of areas and sources. However, in recent years, Part 31 has been increasingly used for the regulation of non-point source pollution.

Part 91, Soil Erosion and Sedimentation Control, of NREPA. Part 91 provides for the control of soil erosion, and protects the waters of the state from sedimentation. Part 91 is applicable to earth changes of one acre or greater and all earth changes within 500 feet of a lake or stream, regardless of the size of the area disturbed.

Part 91 is enforced at three different levels of government: local (city, village, or charter township), county, or state. In some instances some public agencies, such as road commissions and drain commissions are self-enforcing. The primary responsibility for administering Part 91 is with the county. The State's primary role is to oversee the overall operations of the State and local agencies.

Part 91 may not specifically address stormwater runoff; however, the methods for minimizing erosion have a significant impact on the amount of runoff as well as controlling sediments. Since sedimentation is estimated to be a pollutant in about 95% of the watersheds in Michigan, Part 91 is very important in controlling a high percentage of the non-point source pollution problem.

Part 301, Inland Lakes and Streams, of NREPA. Part 301 was created to regulate inland lakes and streams; and to protect riparian rights and the public trust in inland lakes and streams. The numerous public trust values include fisheries and wildlife habitat, public recreation, and water quality.

A permit must be obtained under Part 301 whenever bottomlands are dredged or filled and adequate soil erosion control measures are a condition of the permit. As noted above, the control of erosion and sedimentation is essential to begin to solve non -point source pollution.

Part 303, Wetland Protection, of NREPA. Part 303 provides for the preservation, management, protection, and use of wetlands. A permit is required for alteration or use of a wetland. Part 303 applies to wetlands that are contiguous (connected) to a lake, pond, river, or stream; or isolated wetlands that are greater than five acres in size in counties with a population of 200,000 or greater; or is determined to be essential to the preservation of the natural resources of the state from pollution, impairment, or destruction.

Part 303 indicates that the following benefits may be derived from a wetland:

1. Flood and storm control by the absorption of water and storage capacity.

2. Pollution treatment by serving as a biological and chemical oxidation basin.

3. Erosion control by serving as a sedimentation area and filtering basin, absorbing silt and organic matter.

When a wetland will be used as a part of a stormwater management project, it is imperative that the project be closely coordinated with the District Office of the Land & Water Management Division, Michigan Department of Environmental Quality. (Appendix A gives the address and telephone number of the District Offices throughout the state).

Part 305, Natural Rivers, of NREPA, includes zoning ordinances and rules that can limit construction of stormwater management facilities through restrictions such as building setbacks, limitations on land alteration in areas of high ground water, maintenance of natural vegetation strips, and similar controls.

Michigan Environmental Protection Act (MEPA), 1970 P.A. 127, is an extremely important piece of legislation, as it provides protection of the air, water, and other natural resources, and the public trust associated with these resources. The Act also gives the right to any person in the State to bring action against another person, agency, corporation, or political subdivision for conduct that may pollute, impair, or destroy the air, water, or natural resources.

In regard to stormwater management, MEPA could be used as a means to require detention/retention, to reduce the amount of runoff or the amount of pollutants being added to a waterbody.

Land Division Act, 1967 PA 288, as amended. The Act was passed to regulate the subdivision of land; and to promote the public health, safety, and general welfare. Among the provisions of the Act (Section 192) is the review by the county drain commissioner, or the governing municipality for adequate storm water facilities within the proposed subdivision. At this time, there is no statewide standard that is being used in regard to quality and quantity issues. As a result, a standard, if it exists, will vary among communities and counties.

Michigan Drain Code, 1956 P.A. 40, as amended. The Act was passed with the primary objective of improving the drainage of agricultural lands. Over the years as these areas have become developed, the flooding problems faced by the county drain commissioner have increased.

The establishment of drains or improvements on existing drains is initiated by petition from either a percentage of landowners in the drainage district or two or more public bodies. Under Chapter 8 of the Drain Code, one municipality may petition the drain commissioner. Once drainage districts are established, assessments may be levied to finance drain improvements. In the past, county drain projects have typically consisted of drain enclosures and clean-outs. However, in recent years stormwater management has become a primary focus in various counties around the state.

Part 315, Dam Safety, of NREPA. Part 315 requires a dam construction permit for the construction of a structure that will be six feet or more in height <u>and</u> will impound five surface acres or more at the design flood elevation. Depending on size, some detention ponds may fall under the authority of Part 315.

Part 315 requires dams to have a specified spillway capacity, based on the hazard rating of the dam. As an example, “low -hazard potential” dams must have a spillway capacity that is capable of passing the 100-year flood, or the flood of record whichever is greater. “Low-hazard potential” dams are located in areas where failure would pose little to no danger to individuals, and damage would be limited to agriculture, uninhabited buildings, structures, or township or county roads and where environmental degredation would be minimal. Other dam classifications with a height of less than 40 feet would require a spillway that is capable of passing the 200-year flood, or the flood of record whichever is greater.

For additional information on the classification of dams or Part 315, please contact the Dam Safety Unit of the Land & Water Management Division, MDEQ, at 517-284-5570.

LOCAL ORDINANCES

As of July 1999, a comprehensive stormwater management law does not exist in Michigan, and stormwater management regulations have been left up to local government. The following are elements that would typically be included in local storm water management zoning ordinances (reference 9):

1. **Statement of Authority to Regulate** (What statute gives the community the authority to enact the ordinance.)

2. **Goals and Objectives** of the Stormwater Management Program.

3. **Definitions of terms** used in the ordinance.

4. **Relationship between current and existing legislation** should be included to avoid conflict.

5. **Stormwater Management Plan Review**

   a) Specifications (Descriptions, standard format and certifications that are required.)
   b) Evaluation of Plans (The agency that will evaluate the plans, and the criteria that will use for the evaluation.)
   c) Zoning Approval (The proposal must meet current zoning requirements.)
   d) Review Fees (The fee schedule for review and evaluation.)

6. **Permits**

   a) When State and Local Permits are Required (The situations that will require permits should be specifically spelled out.)
   b) Waivers (Circumstances in which permit requirements are waived.)
   c) Appeals (An appeal procedure must be present to handle denials of a permit or waiver.)
   d) Expiration and Renewal (The permit should be given an expiration date. There should also be a method to apply for an extension or renewal.)
   e) Suspension or Revocation of Permit (To ensure that the construction and implementation of the stormwater management plan is completed.)

f) Fees (Any permit fees should be listed.)
g) Performance Bonds (To ensure the completion of the project.)
h) Compliance (The responsibility of completing the project should be clearly designated to the owner.)
i) Liability Insurance (An alternative to a performance bond, the liability insurance would allow the project to be completed even if the developer is not financially able.)

7. Design Criteria

a) Acceptable Methods of Stormwater Management
b) Performance Standards (List the amount of protection or control that is expected. Such as no increase in 100-year runoff.)
c) Acceptable Methods of Evaluating Stormwater Management Facilities
d) Reference List (Stormwater management technical references.)
e) Safety and Aesthetics (When the use of fencing is required.)
f) Emergency Spillways (When design conditions are exceeded, how the emergency spillway will function.)

8. Maintenance and Inspection

a) Access to Site (Access to the site must be guaranteed during and after construction.)
b) Inspection During and After Construction
c) Responsibility of Maintenance (The responsibility should be noted in the ordinance. If given to landowner, the property title must indicate that the responsibility will transfer if the land is sold.)
d) How Funds for Maintenance will be Collected

9. **Severability** (If one portion of the ordinance is found to be unenforceable, the other provisions will remain in effect.)

CHAPTER 3: STORMWATER MANAGEMENT MEASURES

DETENTION BASINS

As noted earlier, detention basins temporarily store stormwater before discharging it directly into a surface-water body. Until recently, the primary function of a detention pond was to try to reduce the flood peak. Little if any consideration was given to the pollutants carried by the stormwater.

For the purposes of this guidebook, three types of detention basins will be considered: dry, extended dry, and wet. Each basin can be designed to reduce flood peaks; however, their impact on stormwater quality varies for each design.

Dry Detention Basins

The dry detention basin is probably the most popular design that has been used throughout the United States. The basins are usually dry, except for short periods following large rainstorms, or snowmelt events. The basins can be in the form of excavated basins, athletic fields, parking lots, or most any storage area that has the outlet restricted in some way. If the basin can be used for something in addition to detention, the dual use allows for the recovery of the land cost.

The primary function of the dry detention basin has been as a flood-control device to reduce flood peaks, reduce downstream flood elevations, and to some degree reduce downstream erosion. The National Urban Runoff Program (NURP, Reference 51) monitored dry detention basins and found them to have very little impact on water quality. Sedimentation may occur in the basins; however, later runoff events will scour the bottom and move the sediments downstream. If water quality improvement is an objective in a watershed, a dry detention basin is **not** a recommended best management practice.

Extended Detention Basins (See figure 3.1)

The U.S. Environmenatal Protection Agency found that by modifying the outlets of dry detention basins, it was possible to achieve water quality benefits. The outlet modification results in the basins containing water for most storms. About 1 to 2 days after a storm, the basin will be drained. The purpose of the extended detention basin is to increase the time the stormwater will remain in the detention basin, which will result in more pollutants settling out. However, the sediment must be removed regularly, to prevent the re-suspension of pollutants by future runoff events. These basins are **not** effective at removing nutrients that are soluble, such as phosphorus and nitrogen. Even though extended detention basins empty following a storm, they may have a particulate pollutant removal rate of up to 90%, yet the basin will likely cost only about 10% more than a "conventional" dry detention basin. Thus for a little extra money, there can be a great potential for improving the downstream water quality.

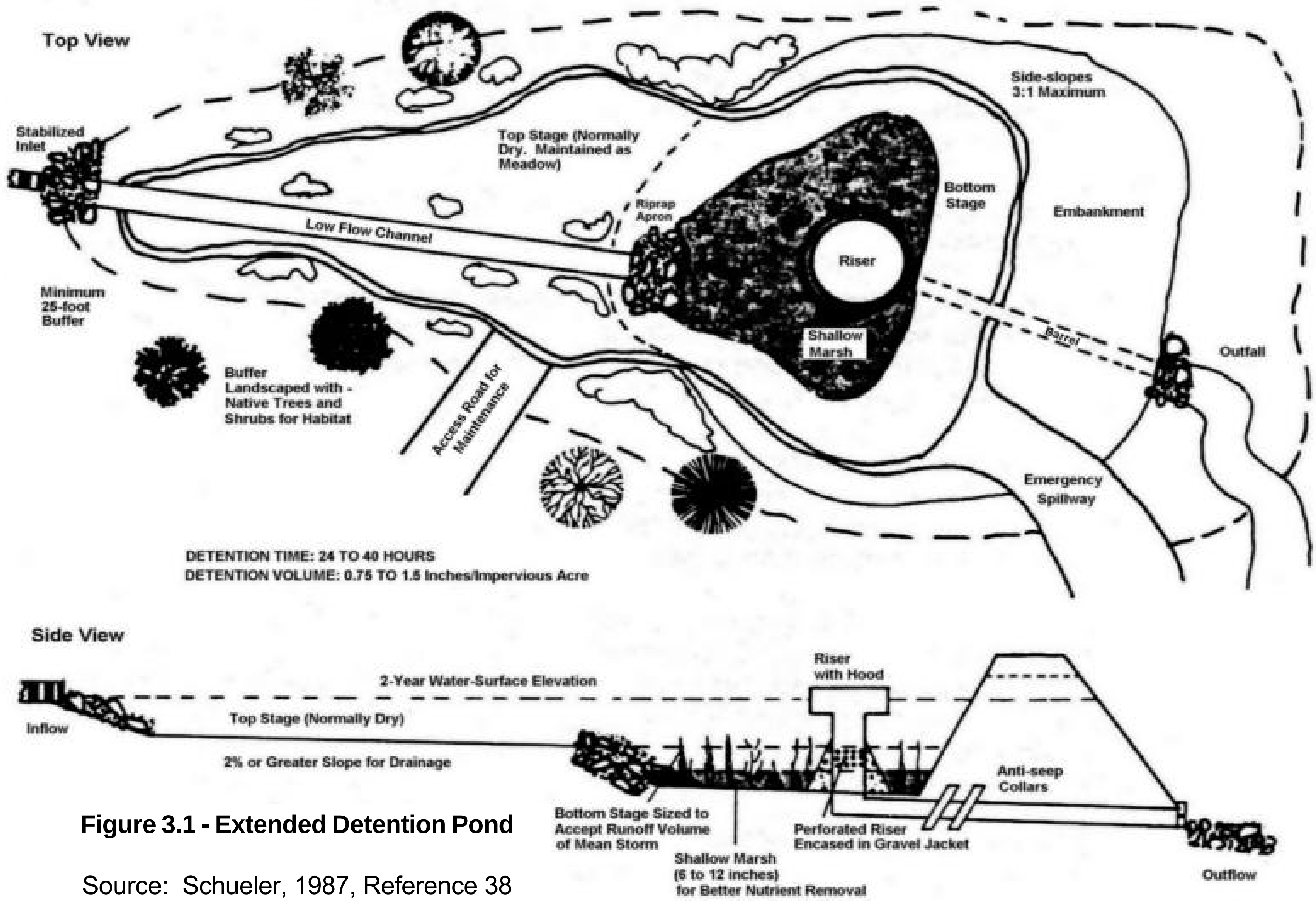


Figure 3.1 - Extended Detention Pond

Source: Schueler, 1987, Reference 38

Following are some guidelines for the design of extended detention basins:

1. **Basin Volume**

The volume of storage required within a basin is dependant upon the function that the basin will be expected to perform. If water quality is the primary concern, there are various methods that are utilized in other regions. A straightforward method requires a storage volume that is equal to one-half inch of runoff from the contributing watershed. (For residential areas, 1/2 inch of runoff would be about a 1 -year rainfall event in Michigan). For the high percentage of particulate pollutant removal, the detention basin should be designed so that it will take at least 24 hours to drain the entire volume stored (Reference 38).

Extra volume should also be provided to account for sediment build-up over a 5 to 10 -year period.

If water quantity is also a concern, it will be necessary to determine what flood protection is desired. The volume of storage to provide 2 -year protection will be significantly less than for a 100-year storm. Later in the guidebook stormwater quantity volumes and flow rates will be reviewed.

2. **Basin Configuration**

The basin shape is about three to five times as long as it is wide. It is also advisable to be narrow at the inlet and wide near the outlet (See figure 3.1).

When both water quantity and quality concerns are to be considered in the design of the extended detention basin, the basin can be designed using a two-stage concept. The lower stage would be designed to be wet frequently and would function as a wetland or shallow pond. This lower stage is designed to contain the water "quality" volume noted above. The upper stage of the basin would be designed to contain the water "quantity" volume. Figure 3.1 shows a typical configuration of a "two -stage" extended detention basin.

The upper stage of the basin should be sloped at a grade of about 2% or more, so it drains well and can be maintained as a meadow-type land -use. Since the lower stage will be wet frequently, it could be maintained as a wetland.

3. **Side slopes**

The side slopes leading to the detention basin should be no steeper than 3:1 (horizontal:vertical) and no less than 20:1 to provide for easy maintenance and to insure proper drainage to the pond. Slopes flatter than 20:1 may result in wet areas that will make maintenance difficult. The slope within the lower stage of the basin should be relatively steep, about 3:1, to minimize the frequently wetted land surface.

4. **Buffer Area**

Surrounding the pond there should be at least a 25 -foot buffer area that is planted with shrubs, trees and low maintenance grasses. The buffer area may improve the "appearance" of the basin, and may also provide a potential habitat for wildlife.

5. **Low-flow channel**

If the basin is to be dry the majority of the time, it will be necessary to provide a low -flow channel through the basin. The channel should be lined with rip -rap to prevent scour. The basin storage area should drain toward the low flow channel so the area may be used and maintained when not flooded.

6. **Outlet Control**

The most common outlet control device for extended detention basins typically consists of a vertical corrugated metal pipe (cmp) that has been perforated with holes that are generally 1 to 2 inches in diameter. The tube is surrounded by wire mesh and gravel larger than the size of the perforations to prevent clogging (see figure 3.2). The riser will overflow only when the design volume has been exceeded. (As an example, set the top of the riser equal to the elevation of the pond needed to store 1/2 inch of runoff from the watershed draining into the detention basin.)

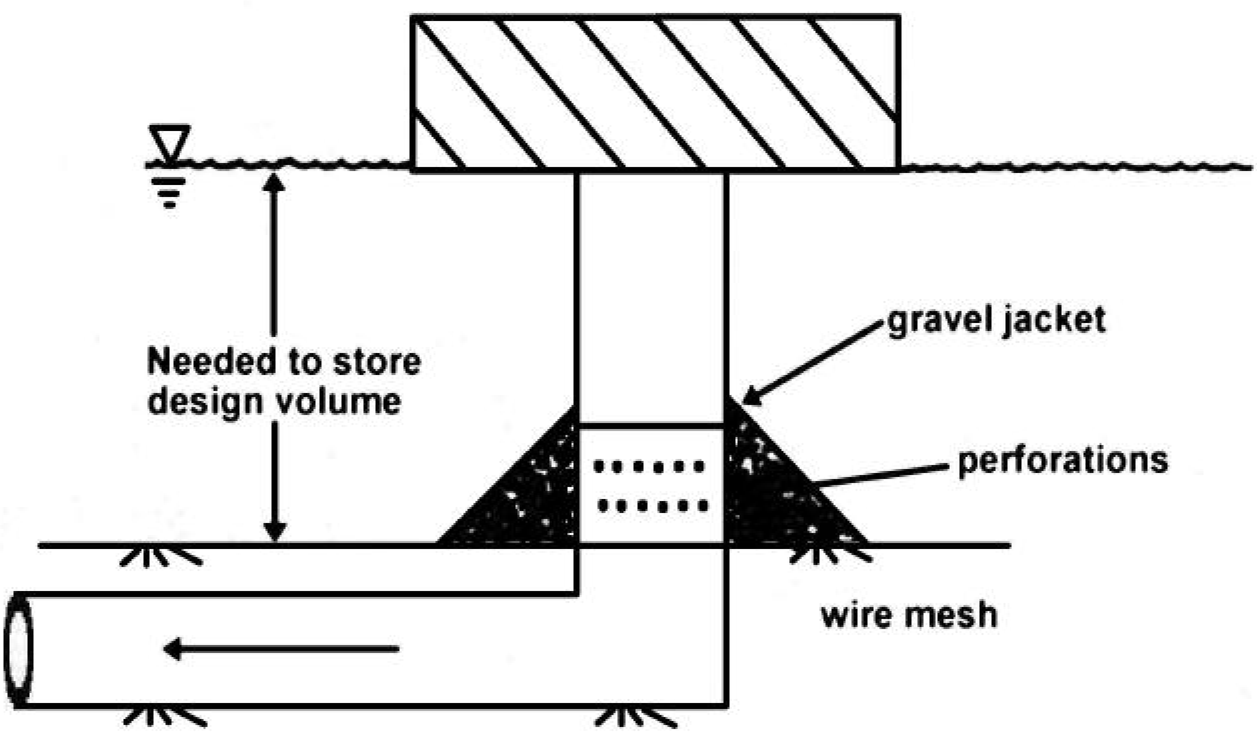


Figure 3.2 - Perforated Riser

Later in the guidebook the design of outlet structures will be discussed in greater detail. However, the rate of outflow can be estimated using the following equation:

$$Q_{out} = [(V)43560]/\ 3600\ (T) \qquad (1)$$

where: **Q_{out}** - outflow in cubic feet per second, cfs
V - design runoff volume to basin, Acre-feet
43560 - square feet per acre
T - detention time, hours; a minimum of 24 hours is suggested
3600 - conversion from hours to seconds

Using a 24-hour detention time, the equation is reduced to:

$$Q_{out} = 0.504\ V \qquad (2)$$

To estimate the amount of outlet area required to carry the design outflow, it is possible to use the following equation for orifice flow:

$$Q_{out} = CA\ (2gh)^{.5} \qquad (3)$$

where: **Q_{out}** - outflow, cfs, as estimated above
C - discharge coefficient, for circular perforations, a C of 0.6 is a reasonable value. (See reference 27)
A - Area of openings (perforations)
g - acceleration due to gravity, 32.2 feet/sec
h - average height of water above the openings (see figure 3.3a)

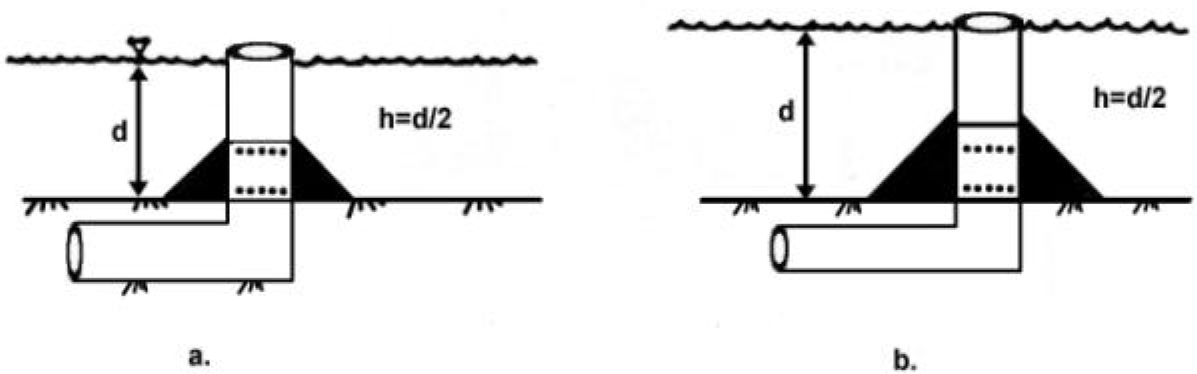


Figure 3.3 - Average height (h) for a perforated pipe outlet structure

Since the water elevation within the basin will be constantly changing as the water flows out of the basin, "h" will not be constant. As an estimate "h" can be taken to be equal to 1/2 the depth of water above the opening. (It must be noted that using this average "h" will result in an "average" discharge; this will **not** determine the **peak** discharge. If peak outflow is a concern, the design elevation for the basin must be used to determine the "h".)

Rearranging equation (3), and inserting the constants, the area of the perforations can be estimated to be:

$$Q_{out}/A = (.6)(64.4\ h)^{.5} \qquad (4)$$

Example 3.1 - Given a 100-acre parcel with a water quality design criteria of storing 1/2 inch of runoff, determine:

(a) volume of runoff required to store
(b) the design outflow, using a 24-hour detention time
(c) an estimate of the total number of 2-inch circular perforations required (water surface for design storage is 4 feet above the center of the outlet, in figure 3.3a, d= 4 feet).

a. Volume of runoff = 100 acres x 1/2 in. x 1 ft/12 in. = **4.2 acre-feet**

b. Using equation (2), Q_{out} = (.504 cfs/acre-foot)V
Design outflow (Q_{out}) = .504 (4.2 acre-feet) = **2.1 cfs**

c. Using equation (4), $Q_{out}/A = (.6)64.4h)^{.5}$

(note: H is one-half of the distance from the center of the perforation to the design water surface, as shown on figure 3.3a. For this example h = d/2 = 4/2 = 2)

Total area required = 2.1/A $=(.6)\{(64.4)(2)\}^{.5}$

= 2.1/A = 6.81

A = 2.1/6.81 = **.308 sq. feet**

The perforations will be 2-inch circular holes, thus the total number of perforations can be estimated to be:

For a circle: $A = \frac{Pi(D)^2}{4}$; $\mathbf{A_{total}} = \frac{(n)Pi(D)^2}{4}$; (5)

where: **A_{total}** - total area of all of the perforations
Pi - a constant of 3.14156
D - the diameter of the perforation, ft
n - number of perforations

by rearranging:(5) $n = \frac{4\,A_{total}}{Pi\,D^2}$

Using equation (5) for the example above:

$n = \frac{(4)\ .308 \text{ square feet}}{3.14(.167)^2}$ = 14.1, **use 15 perforations**

There are no specific design guidelines associated with the spacing of the perforations in the riser pipe. It is suggested that the spacing between the perforations be at least one and one-half to three times the diameter of the perforation.

If all of the perforations cannot be made on one row of the riser pipe, the "h" in equation (4) will not be the same for each row of perforations (See figure 3.3b). It may be necessary to re-estimate the area needed to achieve the computed outflow, using an "h" that is centered on the rows of perforations.

For flows exceeding the water quality design flow, principal and emergency spillways must be provided to prevent overtopping of the embankments.

7. **Cost**

A cost study in Washington, D.C. (Wiegand et al, 1986) derived the following rough cost estimate for the construction of a dry extended detention basin, greater than 10,000 cubic feet:

$$C = 10.71\ V^{.75}$$

where: **C** = construction cost in 1985 dollars
V = volume of storage (cubic feet), including the permanent pool, up to the crest of the emergency spillway

As an example, if 30,000 cubic feet of storage is to be provided, the estimated cost in 1985 dollars is about:

$$C = 10.71\ (30{,}000)^{.75} = \$\ 13{,}200$$

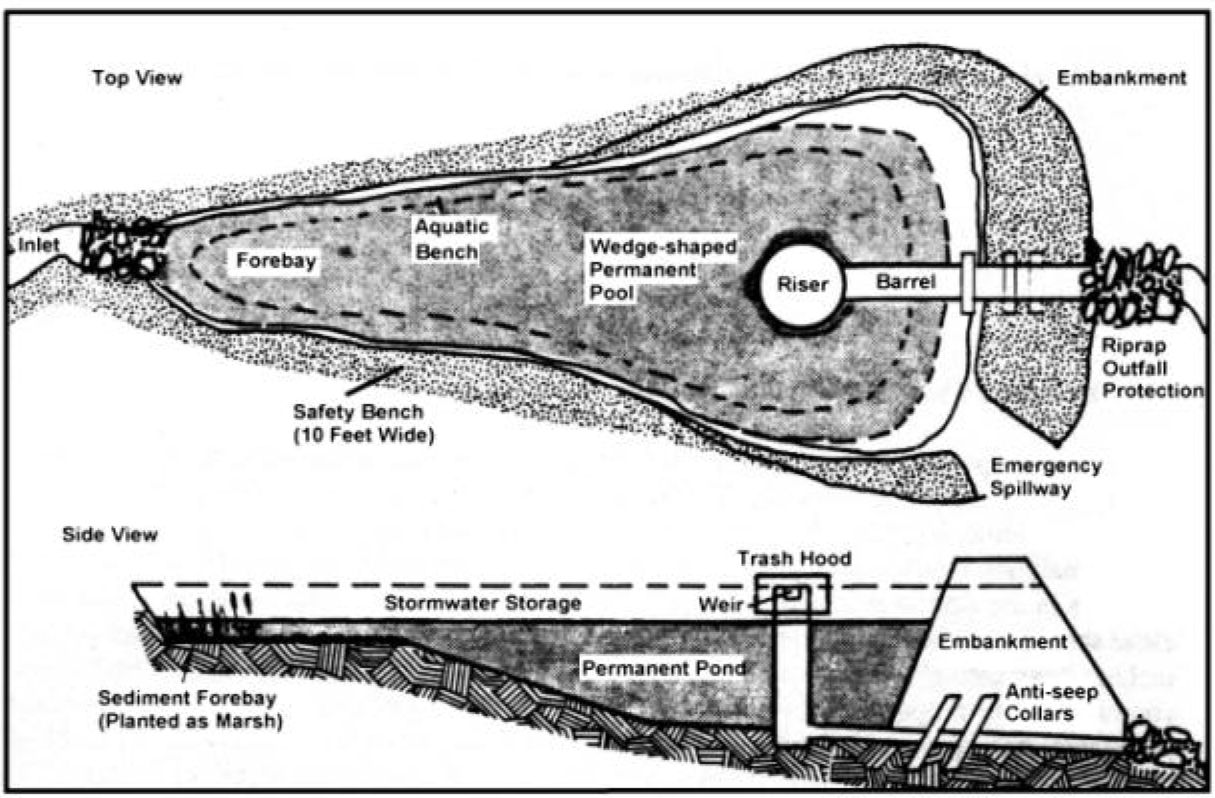


Figure 3.4 - Wet Detention Pond

(Source: Schueler, 1987, reference 38)

Wet Detention Ponds (See figure 3.4)

Of the three types of detention basins, the wet pond is the most effective at removing sediment and pollutants, including nutrients. The biological processes (algae and plant life) make the pond effective at removing nutrients, unlike dry and extended detention basins. Since wet detention basins maintain a permanent pool of water, there is a possibility of algae forming due to the nutrients in the stormwater. For the wet pond to remain effective at removing the nutrients, the algae should be removed regularly. Typical methods of controlling algae and other aquatic plants include "harvesting," dewatering, or herbicides. The use of herbicides is contrary to the purpose of the wet detention pond. The intention of the pond is to remove pollutants, not introduce additional pollutants. If no other alternative is available, herbicides must be applied with extreme caution to prevent contamination of receiving waters. The application of herbicides in surface waters will require a permit from the Michigan Department of Environmental Quality, Land and Water Management Division, Inland Lakes and Wetlands Unit (telephone # 517-284-5531).

If a watershed is experiencing problems with nutrients within the stormwater runoff, a wet detention pond is really the only detention design that will provide some removal of the nutrients.

Following are some of the design guidelines for wet detention basins, for water quality purposes:

1. **Basin Surface Area**

The surface area of the basin is critical in allowing particles to settle out. The following table gives a rough estimate of the permanent pool's surface area expressed as a percentage of the area draining into the pond, the land use in the watershed, and the size of particles that will be settled out. As a point of reference for particle size, fine sand is about 40 to 100 microns, silt is about 10 microns, and clay is about 1 micron.

The 5-micron control listed will capture all particles greater than 5 microns in size, or about 90% of the particulates in urban runoff. It should be noted that some studies indicate that 10 microns is about the smallest size portion that could be expected to settle out in the "field". A 20-micron control will capture about 65% of the particulates.

Table 3.1 - Basin Size Expressed in Percent of Drainage Area

	Particle Control Size	
Land Use	**5 micron**	**20 micron**
Freeways	2.8%	1.0%
Industrial	2.0%	0.8%
Commercial	1.7%	0.6%
Institutional	1.7%	0.6%
Residential	0.8%	0.3%
Open Space	0.6%	0.2%

Source: Reference 33

As an example, a 100-acre residential subdivision would require a surface area of about 0.8 acres of wet detention to capture particles larger than 5 microns. (From table 3.1, residential land use for 5-micron control shows a basin area that is 0.8% of the total drainage area. Hence, 100 Acres x 0.008= 0.8 Acres). If the same parcel were industrial, and the same 5-micron control were desired, the basin surface area would have to be about 2.0 acres.

Of course, Table 3.1 is just an initial sizing estimate for water quality purposes. Additional information on runoff volume and outflow rates will have to be considered.

2. **Basin Volume**

There are various methods used in estimating the volume required in a wet detention basin, designed for **water quality** purposes. Each of the methods provides moderate levels of sediment removal. The design of a wet pond will require a water-quality volume to be computed. The water-quality volume is stored above the permanent pool (see figure 3.4). To achieve pollutant removal, the permanent volume of the wet pond should also be equal to or greater than the water quality volume. If flood control is also a prime concern, a water-**quantity** volume must be computed. The storage required for water quality concerns will be discussed later. For the purposes of this guidebook, the four following methods of computing the **water-quality volume** are discussed.

a) **First-flush method**. Probably the most common method used to estimate the size of a detention basin is the "first flush" method. With this criterion, the basin volume required is determined using 1/2 inch of runoff per impervious acre of the land draining to the basin.

   If a 100-acre site has 38 acres that are impervious, a detention basin would require 1.6 acre-feet of storage (38 acres x .5 inch/acre x 1-foot/12 inches).

   A variation of this method involves using 1 inch of runoff per impervious acre. In essence, this variation doubles the volume requirement of the detention basin. In the example above, the storage requirement would have been 3.2 acre-feet instead of 1.6 acre-feet.

b) **Runoff method**. A simple method to apply involves using one -half inch of runoff for the entire drainage basin. As an example, a 100-acre site would require 4.2 acre -feet of storage. (100 acres x 0.5 inch/acre x 1 foot/12 inches)

   This method does not give credit for low runoff (pervious) surfaces within the watershed. A watershed that is heavily industrialized would have the same water-quality volume requirements as a residential development.

   The other methods discussed in the guidebook are dependent on land-use. Thus as the land use changes, the volume requirements will also change. The "runoff method" would remain at 1/2-inch runoff regardless of land-use.

c) **Design-storm method**. Basin volume is equal to the runoff produced by a selected design storm. One possibility is the use of a 1 -year, 24-hour duration storm. (Appendix B lists various storm frequencies for the counties of Michigan). This method will require that the land use and soil types be determined for the watershed, in addition to the rainfall amount.

For residential developments, a 1-year, 24-hour duration storm method would be similar to assuming between 0.5 and 1 inch of runoff from the entire drainage area. For industrial and commercial areas, a 1-year storm could produce over 1.5 inches of runoff.

d) **Mean Storm volume**. The basin volume is determined to be a multiple of the mean storm runoff volume, when only the impervious acres are considered. Mean storm volume is defined as the volume runoff produced by the mean rainfall event. Studies have indicated basin volumes that exceed 3 times the mean storm runoff volume yield, diminishing returns. The mean storm volume is determined by a statistical analysis of the rainfall data for the area.

For Lansing, the mean storm volume is approximately 0.3 inches, (reference 54). This value varies across the state; however, 0.3 inches is a reasonable estimate if rain gage information is not available. If a 100-acre parcel has 38 impervious acres, and the runoff coefficient for the impervious area is 0.95, the mean runoff volume for the parcel is estimated to be:

(38 acres x 0.95 x 0.3 in. x 1 ft/12 in.) = 0.90 acre -ft

The basin volume requirement is estimated to be three times the mean runoff volume from the impervious area:

Basin volume = 0.90 acre-feet x 3 = **2.7 acre-feet**

In general, the larger the pond the more efficient the pond will be at removing the pollutants. Since there is a cost factor involved, at some point, the extra cost associated with a larger basin does not significantly increase the efficiency of the basin. Studies have indicated that basins which have a volume more than 3 times the mean runoff volume have diminishing returns on the money invested.

Each design method will provide different results to be used to size the detention basin. The table on the next page provides a comparison of the four methods for a 100-acre parcel in Lansing, Michigan. For each runoff method, four different land use types have been considered. Following are the four land use types and the corresponding percentage of the total drainage basin that is impervious: commercial/business districts, 85% impervious; industrial areas, 72% impervious; 1/4 acre residential, 38% impervious; and 1/2 acre residential, 25% impervious.

From table 3.2, it can be seen that a wide range of storage volumes can be computed depending on the runoff criteria used. As noted earlier, storage volumes exceeding three times the mean runoff volume have a diminishing return on the cost of the basin. Thus, from a water-quality standpoint, three times the mean runoff volume could be thought of as the upper limit, and the first flush method would represent the lower limit of the volume requirements.

It is not the purpose of this guidebook to provide a method that should be used in all communities, but to present methods that are currently in use throughout the United States.

Actual criteria should be established at the local level. The 0.5 inch of runoff for the entire watershed is the simplest method to administer. However, from a water- quality aspect, this method will be very conservative in residential areas.

Table 3.2 - Detention Basin Storage Volume (acre-feet)
Comparison of different runoff methods
For a 100-acre site in Lansing, Michigan

Percent Impervious	First-Flush Method	Runoff Method	Mean Runoff Volume x 3 Method	One-Year Design Storm Method
85	3.5 A.ft.	4.2 A.ft.	6.1 A.ft.	13.2 A.ft.
72	3.0	4.2	5.1	11.2
38	1.6	4.2	2.7	4.5
25	1.0	4.2	1.8	2.1

3. **Basin Depth**

To prevent scouring and resuspension of sediments, the basin pond should be permanently 4 to 6 feet deep over most of the basin. The depth will also minimize the growth of aquatic plants and may allow the planting of small fish and minnows that eat algae and mosquitoes. Depths less than 3 feet may result in scour, while depths greater than 6 to 8 feet may result in thermal stratification and water-quality problems.

Near the basin inlets, extra depth may be constructed to provide sediment storage capacity. It is much cheaper to initially provide extra storage than it is to dredge out accumulated sediment.

4. **Basin Shape**

The basin shape should allow for good circulation and easy maintenance. If the shape is not adequately considered, "short circuiting" may occur. When short circuiting occurs, the incoming water does not displace the "old water" already in the basin. Instead, the incoming water passes right through the basin with minimal pollutant removal, as a result, water quality is not improved. It is recommended that the flow length from inlet to outlet be about three to five times the width of the pond. If it is not feasible to construct a basin with such dimensions, baffles should be used to achieve the flow path length. (Figure 3.5 provides some examples of short circuiting, baffles to increase flow path, and a recommended shape.)

The most common pond configuration is wedge shaped, narrow at the inlet and wide near the outlet. Such a shape allows for good circulation. The pond shape should also be irregular to achieve a "natural" look that will fit in with the surroundings. However, in achieving the "irregularity", care should be taken to not create areas that will prohibit the circulation of water.

If the basin is functioning properly, it will be necessary to provide some maintenance dredging to remove accumulated sediments. Thus, the pond shape should also consider future maintenance needs. As an example, a long narrow pond may be easier to dredge, than a circular pond.

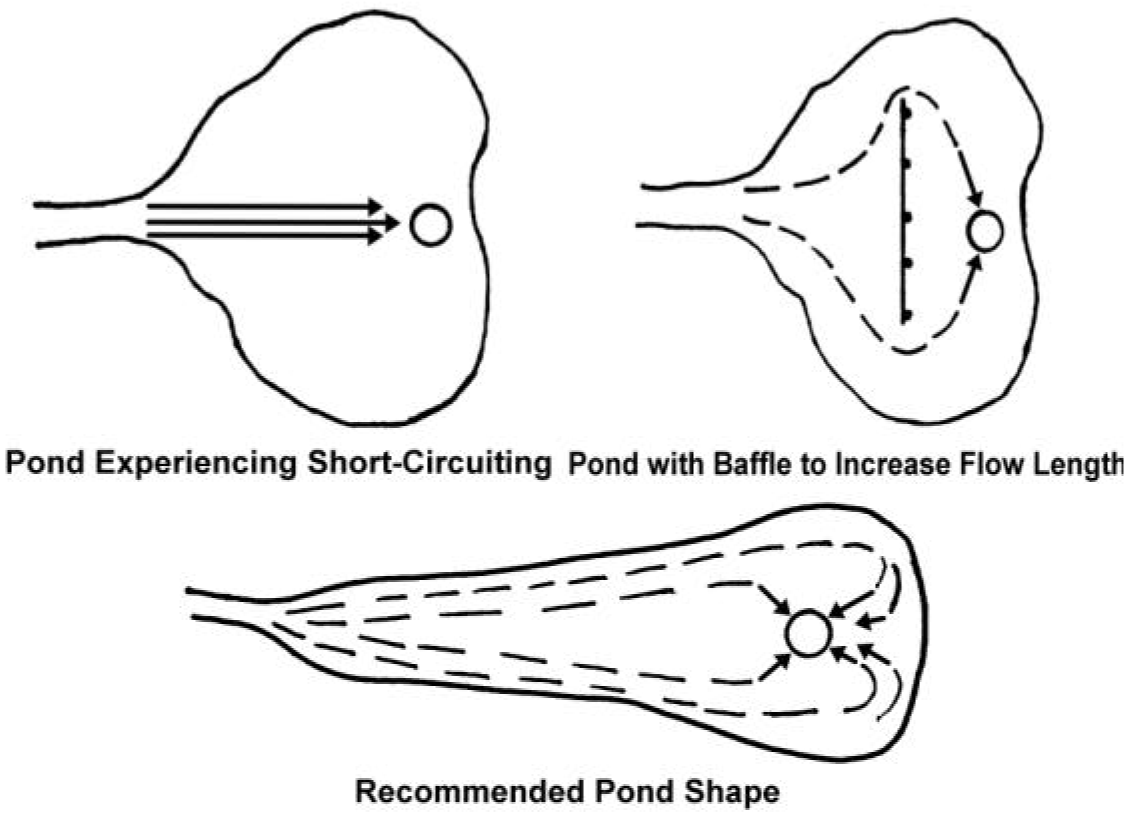


Figure 3.5 - Examples of Water Circulation within a Detention Basin

5. **Side-slopes**

Figure 3.6 shows a suggested configuration for the side slopes of a wet detention basin. The perimeter of the pond should be surrounded by a relatively flat shelf that is at least 10 feet wide. A permanent pool of water, about 1/2 foot to 1-1/2 feet deep, should cover the perimeter shelf. The shelf should be planted with rooted aquatic plants. The primary purpose of the plants is to act as a vegetative barrier to prevent easy access to deeper water to discourage swimming. However, the plants also provide a "natural" appearance to the basin. The side slope leading to the pond shelf should be at a relatively steep slope of about 3:1 (h:v). Such a slope will result in less land being frequently inundated, and thus will reduce the mosquito problems. The 3:1 slope should be continued up to the water level elevation anticipated for the water quality design storm (such as 0.5 inches of runoff). The side slope from the basin shelf to deep water should be 3:1 maximum.

The side slope up to the elevation that will contain the design flood (as an example a 10-year flood) is suggested to range from 4:1 to 20:1 depending on the area available. A side slope of no less than 20:1 will provide an area that is easy to maintain and will drain well. These side slopes should be planted with water-tolerant grasses, shrubs, and trees and should be maintained as a meadow. It is important to note that trees should not be planted on any filled embankments that were created to impound water. The roots of the trees will provide seepage paths for water during impoundment, which may lead to a failure of the embankment.

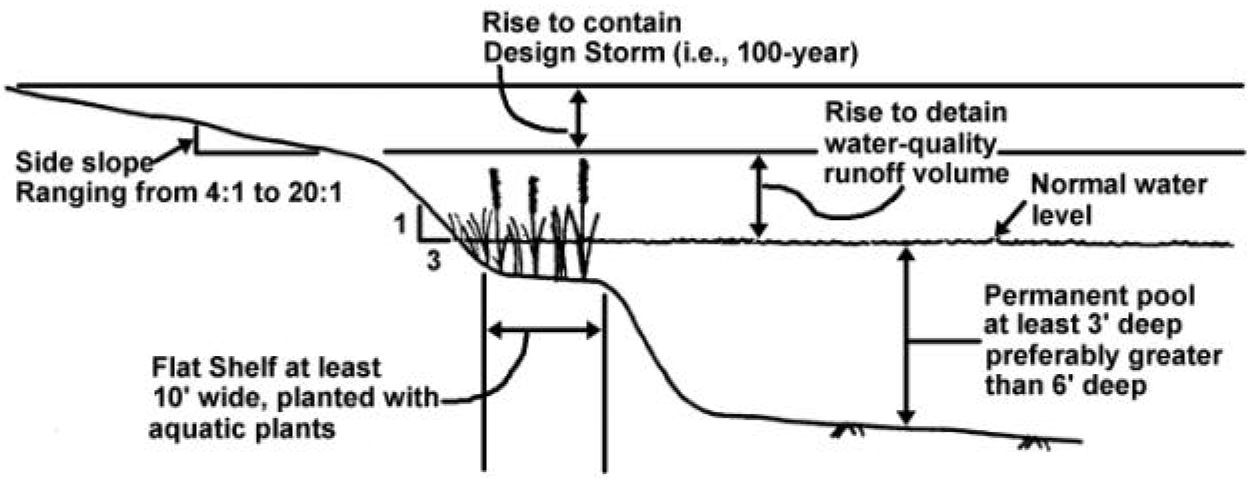


Figure 3.6 - Typical Wet Detention Pond Cross Section

6. Cost

A cost study in Washington, D.C. (Reference 56) derived the following rough cost estimate for the construction of a wet detention pond, less than 100,000 cubic feet:

$$C = 6.1\ V^{.75}$$

where: **C** = construction cost in 1985 dollars
V = volume of storage (cubic feet), including the permanent pool, up to the crest of the emergency spillway

For ponds greater than 100,000 cubic feet, a rough cost estimate would be:

$$C = 34\ V^{.64}$$

The estimate does not include land costs, only construction costs are included. An additional 25% may be added to the estimated cost to try to account for contingencies, inspections, and costs of securing permits.

It must be remembered that these cost estimates are only for initial planning purposes, and are **not** to be considered final estimates.

7. Outlet Rate For Water-Quality Purposes

The outflow from a detention pond will be restricted to achieve the necessary water-quality and quantity benefits. The outlet structure must be designed to achieve the desired results.

Later in this guidebook, flood-control considerations will be reviewed. The following discussion is in regard to the water- **quality** requirements.

Two critical factors in determining the effectiveness of the removal of particulates in wet detention basins are the settling velocity of the particulate and the velocity within the basin.

For a particulate to be removed from suspension in a detention basin, the settling velocity must be great enough for the particulate to fall below the outlet elevation before it reaches the outlet (See figure 3.7). Particulates that do not settle fast enough are kept in suspension and will flow from the outlet. It can be generalized that the slower the velocity within the basin, the smaller the particulates that will settle out. For higher velocities, only the large particulates will settle before they reach the outlet.

From figure 3.7, a particulate will travel a distance of L at a horizontal velocity of V, in a time of t (L = vt). The same particulate will settle a vertical distance of D at a vertical (settling) velocity of v in a time of t (D = vt). For a particulate to be retained in the basin, the time it takes to travel a distance of L must be greater than or equal to the time it takes to settle the distance D.

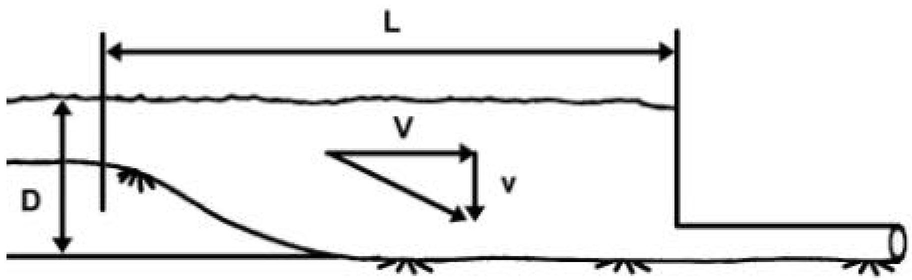


Figure 3.7 - Settling Velocity and Pond Dimension

(adopted from references 25 and 33)

In other words, the particulate must settle below the outlet elevation before it reaches the outlet. The largest particulate that will be captured by the basin, will be the particulate that travels the distance L in the same time that it takes to settle distance D. It can be shown that:

$$t = L / V, \text{ and } t = D/v; \text{ or}$$
$$L / V = D/v \qquad (6)$$

where: **t** - is the time it takes for the particle to settle
L - length of the basin
D - depth of the basin
V - horizontal velocity component
v - critical settling velocity

rearranging equation (6) :

$$v = VD/L;$$

multiplying by basin width **W**:

$$v = VDW/LW$$

DW represents the cross-sectional area of the basin. Area (DW) times velocity (V) is equal to discharge out (Q_{out}):

$$v = Q_{out}/LW \tag{7}$$

The surface area of the basin (A) is defined by length (L) times width (W), thus:

$$v = Q_{out}/A \tag{8}$$

where: **v** - critical settling velocity in feet per second
Q_{out} - Outflow from the basin in cubic feet per second
A - is the surface area of the detention basin, in square feet.

Linsley and Franzini (reference 25) define Q_{out}/A as the overflow rate.

From equation (8), it can be seen that the critical settling velocity is a function of the outflow rate and detention basin surface area. It is also interesting to note that increasing the depth of a basin does **not** increase the efficiency of the basin. (However, increasing the basin depth does reduce the possibility of scour, provides additional volume to accumulate sediment, limits winter fish kill, and reduces the amount of attached aquatic plants). To remove smaller size particulates, it would be necessary to either decrease the outflow rate or increase the basin surface area.

If equation (8) is rearranged:

$$Q_{out} = Av \tag{9}$$

The settling velocity of a particulate is a function of particle density, size, and shape as well as the density of the liquid (water). Studies have shown that the densities of particulates in stormwater runoff vary considerably, from 2650 kilograms/cubic meter (kg/m^3) to 1100 kg/m^3 (reference 41). (Note: Water has a density of about 1000 kg/m^3.) Since the density can vary so much, the settling velocity of particulates can vary significantly. The settling velocity of particulates can be site specific. The most appropriate method would require the sampling of particulates contained in runoff from a specific site.

If sampling information is not available, figure 3.8 provides an estimate of settling velocity, based on particle size. The figure has been developed assuming a particle density of 1500 kg/m^3, and a water temperature of 68°F.

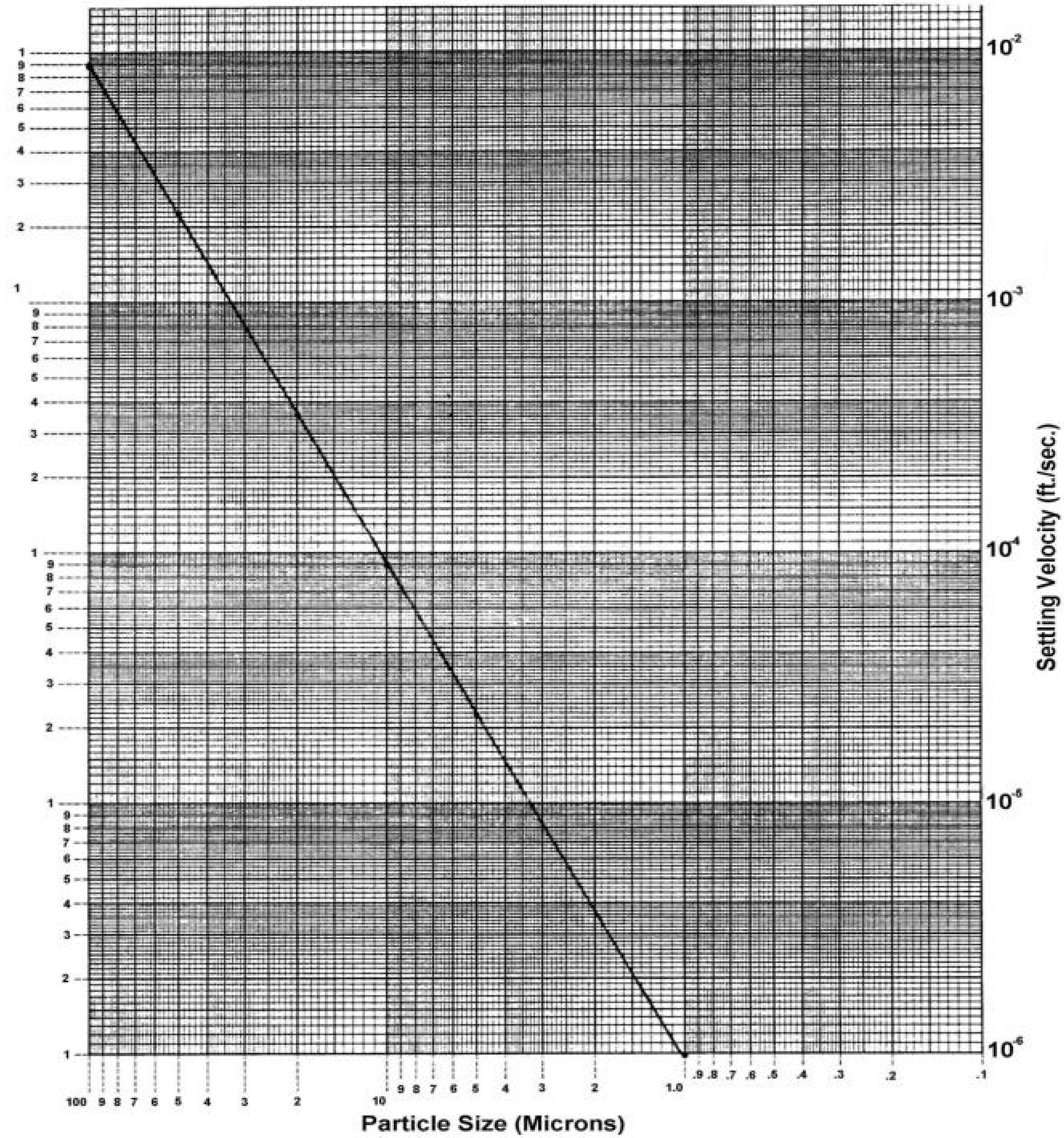


Figure 3.8 - Settling Velocity for Stormwater Runoff Particles

The following example illustrates how table 3.1 and figure 3.8 may be used in the design of a wet pond for water-quality purposes.

Example 3.2. Given a 100-acre industrial site for which 5-micron control is desired, find (a) required surface area; and (b) the maximum outflow rate.

a) From **Table 3.1**, to achieve 5-micron control for an industrial site, the **surface area** required is:

   2% of the drainage area, or; .02 x 100 acres = **2 acres**

b) From **Figure 3.8** the **settling velocity** of a 5-micron particle is:

   $v = 2.3 \times 10^{-5}$ ft/sec

   Using equation (9), the **maximum** outflow needed to achieve 5-micron control is:

   $Q_{out} = Av$

   Q_{out} = 2 acres x 43560 sq.ft./ acre x 2.3×10^{-5} ft/sec

   $\mathbf{Q_{out}}$ = 2.1 cubic feet/sec (cfs)

With the **maximum** outflow known, it is possible to design an outlet structure that will restrict the outflow to less than 2.1 cfs, at the water elevation needed to store the water **quality** portion of the runoff. The typical wet pond cross section shown in figure 3.6, shows the rise in the pond needed to store the water-quality runoff volume.

TYPICAL OUTLET STRUCTURE CONFIGURATION

The outlet for the wet detention basin typically consists of an outlet tube with a riser (See figure 3.4) or a weir configuration. In addition to the outlet pipe, it will also be necessary to include an emergency spillway to safely handle flows that will exceed the capacity of the outlet structure.

If an increase in downstream water temperature is a concern, it may be necessary to consider a subsurface outlet structure (See figure 3.9). The inlet to this pipe must still be at least three feet above the bottom of the pond to prevent bottom materials from being re-suspended due to scour (The basin must also be at least 6 to 8 feet deep so the water on the bottom is cooler). A negatively sloped outlet pipe with an inlet that is below the water surface of the pond is one method of discharging from the bottom of the pond. This type of outlet will not be affected significantly by floating debris. As a result, the amount of maintenance that will be required will be reduced.

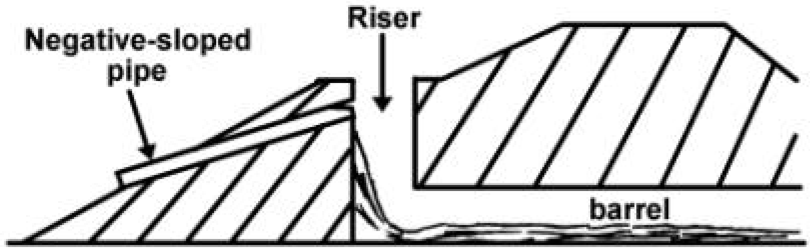


Figure 3.9 - Sub-surface Draw Outlet Structure

There are several potential problems with the design of a wet detention basin:

1. Excessive algae must be controlled to prevent odors, and to maintain nutrient removal capacity. If the aquatic plants are not harvested, the pollutants that have been removed during the growing season will be released when they die in the fall.

2. If the basin is functioning properly, it will be necessary to periodically (about 5 to 10 years) dredge the accumulated sediment. The configuration of the pond should allow easy access to the pond to allow dredging.

3. The water quality within the ponds will be poor. As a result, water contact recreation (such as swimming) should be discouraged.

4. Since the pond will have a permanent pool of water, there may be a local concern about safety. **Except in the vicinity of the outlet structure, the use of fences should be avoided.** The use of fences to try to deny access to a pond will result in the pond not being maintained properly and will likely result in the pond becoming a dumping ground for various types of refuse. It is recommended that the pond be designed and landscaped in such a manner as to discourage easy access to the pond by little children.

5. Using natural wetlands for treating stormwater runoff can modify the hydrologic characteristics of the wetland. It is highly recommended that natural wetlands **not** be used for stormwater treatment. When alternatives are available, the stormwater should be treated before discharging to a natural wetland. It is strongly urged that the District Office of the Land and Water Management Division, Michigan Department of Environmental Quality (MDEQ) be involved early in the planning stage (see Appendix A).

6. If the runoff will contain a high concentration of toxic contaminants, it may be necessary to "pre-treat" the runoff before discharging to the wet pond. (One alternative would involve retaining the runoff on-site). The Surface Water Quality Division of the DEQ may be able to provide some guidance in pre-treating stormwater runoff for toxic contaminants.

CHAPTER 4: RETENTION BASINS

The terms **detention** and **retention** many times are considered to have the same meaning. However, in this guidebook, a **retention** basin will be defined as a stormwater management practice that captures stormwater runoff, and does not directly discharge to a surface water body. Water that is "retained" is "discharged" from the basin either by infiltration or evaporation. Retention basins will typically have minimal impact on 100-year flood peaks, since they are usually not designed to retain the 100-year runoff.

The two driving forces in the design of a retention (infiltration) basin is the amount of runoff that will be retained, and the infiltration capacity of the soil. Since infiltration capacity is critical, soils that contain a high percentage of silt of clay cannot be used for infiltration basins.

The use of retention (infiltration) basins can result in a high percentage of pollutant removal. Table 4.1 indicates estimated removal rates for a retention (infiltration) basin for two types of sizing requirements.

Table 4.1 - Estimated Long-Term Pollutant Removal Rates (%) For Infiltration Basins

POLLUTANT	SIZING RULE 0.5 in/imper acre	SIZING RULE 2-yr runoff volume
SEDIMENT	75%	99%
TOTAL PHOSPHOR	50-55%	65-75%
TOTAL NITROGEN	45-55%	60-70%
TRACE METALS	75-80%	95-99%
BOD	70%	90%
BACTERIA	75%	98%

(Source: Schueler 1987, reference 38)

The larger the basin, the more efficient the basin will be at removing pollutants. However, since larger basins cost more, there will be a point at which the additional cost of a larger basin will not translate into a significant increase in the efficiency of the basin.

Following is a list of guidelines for the design of a retention (infiltration) basin:

1. **Volume Requirements -** for water quality purposes.

   The most widely applied runoff methods include:

   a) Storage of 0.5 inches of runoff per impervious acre.
   b) Storage of 0.5 inches of runoff from the entire drainage basin.
   c) Storage of the volume of runoff from a 2 -year storm.

If the basin is to provide water- **quantity** benefits, the retention volume has to be significantly higher, which may not be feasible. It would be more appropriate to use a retention basin to capture the "first flush" and use a detention basin for water quantity control.

2. **Infiltration Capacity**

The other design consideration for a retention basin is the infiltration capacity of the soil. For a site to be considered feasible to use a retention basin design, the infiltration capacity of the soil should be greater than 0.52 inches per hour (reference 38).

To insure that an accurate evaluation of the soil type is made at the basin, soil borings are needed at least 5 feet below the bottom of the proposed basin. Adequate soil information is essential to have before the basin is designed. Without such information, there is a high probability that the basin will fail.

Table 4.2 - Infiltration Rates for Soil Groups

Soil Class	Infiltration Rate (inches/hour)	National Resource Conservation Service Hydrologic Soil Group
Sand	8.0	A
Loamy Sand	2.0	A
Sandy Loam	1.0	B
Loam	0.5	B
Silt Loam	0.3	C
Sandy Clay Loam	0.2	C

From table 4.2, only soil groups A & B would be feasible for the use of an infiltration retention basin. If the soils are C or D, the basin would likely remain wet and eventually lose its capacity of retaining stormwater runoff. In addition, if the basin remains wet, the basin may be considered an eye sore, and adjacent property owners will likely want the basin filled in.

A 1987 survey in Maryland by Pensyl and Clement (reference 32) found about one -third of the infiltration basins contained standing water. The reasons given for the standing water include, low infiltration rates due to compaction during construction, sedimentation, and poor preliminary soil investigation.

Ferguson (reference 14) offered additional views, in which he indicated that the design of retention basins is typically based only on a design runoff; the everyday rainfall and runoff events are not considered. Ferguson also concluded that "a basin sized only for a 0.5 inch first flush is not likely to be capable of capturing the first flush; a basin sized only for a design storm is not likely to be capable of capturing the design storm..." This conclusion is a result of basin designs that ignore the "everyday" flows, which can accumulate in the basin and reduce the capacity of the basin.

The "everyday" flows accumulate in a retention basin when the "flow" into the basin exceeds the infiltration capacity of the basin. As a factor of safety, it is suggested that the infiltration capacity of the basin floor be multiplied by at least 0.5 when designing the basin. The factor of safety is to try to account for the compaction of the basin floor and the accumulation of sediments on the basin floor. If the retention basin happens to be an area that will be used as a recreation area, such as a playground, it would be advisable to apply an additional factor of safety. Heavy foot traffic will tend to compact the basin floor, and reduce the infiltration capacity.

The factor of safety in combination with a minimum infiltration rate of 0.5 inches/hour should minimize the potential for standing water occurring in the retention basin.

3. **Basin-bottom elevation**

To ensure that the basin will be able to function properly, the basin bottom should be at least 4 feet above the seasonal high-water table and/or bedrock.

4. **Maximum ponding time of 72 hours**

If the ponding time exceeds 72 hours, it is possible that the basin will be continually wet. An infiltration basin that is continually wet cannot be maintained properly, and may turn into an eye sore.

5. **10 feet from the nearest basement wall**

The retention basin should be placed at least 10 feet from the nearest basement wall.

6. **100 feet from nearest well**

To limit the possibility of contamination, the basin should be located at least 100 feet from the nearest water supply well.

7. **Not placed in filled areas**

The basin should <u>not</u> be constructed in "filled" areas.

8. **Use water-resistant grasses**

The side slopes and bottom should be vegetated using grasses that can withstand being covered by water for up to 72 hours.

9. **Avoid compaction of basin bottom.**

In many instances, the retention basin is only a portion of a large project. The basin area should be staked out and avoided by heavy equipment during construction to prevent compaction of the soil. Care must also be taken during the actual construction of the retention basin to prevent compaction of the bottom of the basin by construction equipment. To prevent compaction, it may be necessary to excavate from the sides of the basins, rather than placing the equipment on the basin bottom.

10. **Provide overflow area**

Provide an area which may overflow should the design criteria be exceeded. The area should be stabilized to prevent erosion. When overflow occurs, a drainageway must be available to carry the water.

11. **Reduce amount of sedimentation that gets into the basin**

It is essential to remove as much sedimentation as possible before the flow gets to the basin. The use of erosion-control measures, sedimentation basins, and grass filter strips before and during basin construction is very effective. The retention basin should **not** be used as a sedimentation basin during the construction phase. The sediment will tend to seal the basin bottom, which will significantly reduce the infiltration capacity of the basin. If there are no other alternatives, **all** the sediment that has accumulated during construction should be removed down to "natural" soil.

12. **Removal of sediment**

Even with erosion control measures in place, sedimentation may accumulate in the basin. If the sedimentation is not removed, the basin floor will "seal" and the basin will turn into a "mud hole". The sedimentation should be allowed to dry before **light** equipment is used to remove the sedimentation. Once the sedimentation is removed down to the basin floor, the floor should be tilled and revegetated to restore infiltration rates.

EXAMPLE 4.1: Retention Basin Design

The runoff from a 10-acre site is to be retained. Estimate the basin size given the following criteria:

a) The basin is commercially developed (85% impervious)
b) Retain 0.5 inches of runoff/impervious acre
c) Drain pond in at least 48 hours
d) Infiltration capacity of the soil is 1.0 inch/hour
e) Multiply the infiltration capacity of the basin floor by 0.5 as a factor of safety.

I) **Compute runoff volume**:
(Total Area) x (% impervious) x retention requirement
10 acres x 0.85 x 0.5 in/acre = 4.25 acre -in (0.35 acre-ft)

II) **Compute the basin depth needed**:
(infiltr. time) x (infiltr. capacity) x factor of safety
48 hrs. x 1.0 in./hr x 0.5 = 24 in.; or 2 feet in 48 hours

III) **Compute the surface area of the basin**:
volume of runoff / infiltration available
0.35 acre-feet / 2 feet = 0.175 acres = 7623 square ft

suggest **8000 square feet**

It should be noted that this sizing estimate has excluded the infiltration that may be occurring through the sides of the retention basin. For shallow retention basins such as this one, the infiltration through the sides will be much less than will be occurring through the basin floor.

PROBLEMS AND CONCERNS WITH RETENTION BASINS

Potential Groundwater Contamination

Under current State of Michigan regulations, a ground-water discharge permit is not required for the discharge of stormwater via an infiltration basin.

Studies done on infiltration basins in Long Island, New York and Fresno, California (reference 51) indicated metals and other pollutants accumulated in the upper few inches of the soil in the basin and did not reach the groundwater. Pitt (reference 33) noted that these studies did not thoroughly investigate the impact of soluble organics on the groundwater.

If soluble organics are present and may be picked up by stormwater runoff, from areas such as industrial facilities, it is best to identify the source of the pollutants and eliminate the source.

If source elimination is not possible, the distance between the basin bottom and the seasonal high ground-water table should be kep t as large as possible. The four-foot distance, mentioned above is a minimum for all retention basins, if organics are present the distance should be greater. At this time there is no "rule of thumb". The State of Wisconsin considers sites that have a 20-foot depth to ground water as being minimally susceptible to ground water contamination.

Additional study is needed into the potential groundwater problems from soluble organics that may result from infiltration of stormwater runoff.

Sedimentation

If sedimentation is a problem in the drainage basin, it is essential to provide some method of capturing or reducing the sedimentation before it reaches the retention basin. Excessive sedimentation will "seal" the bottom of the basin, which will result in a continually wet basin. Maintenance may be necessary to remove the excess sedimentation that may accumulate in the bottom of the basin, loosen the bottom soil, and revegetate.

Property Owners

If on-site retention basins are used, the property owner ma y view it as a drainage problem, especially if the infiltration capacity has been reduced. Since the basin is on site, there may be problems keeping the basin maintained. In addition, it is possible that the property owner will become upset with having the water accumulating on the property and may try to fill in or regrade the basin. To minimize problems from property owners, it would be advisable to place the basins in "common" areas where they can be maintained.

Oil and Grease

If the runoff from the drainage basin will contain oil or grease, it will be necessary to use an oil/grit separator to remove these pollutants before they reach the retention basin (see figure 4.1). The oil and grease will tend to seal the basin bottom, which will result in standing water. A typical oil/grit separator consists of three chambers, and provides 400 cubic feet of wet storage per acre of contributing drainage area. The first chamber captures sediment, while the second chamber captures the oil and gas films which are eventually absorbed by particles and settle. The pool of water in the first two chambers should be at least four feet deep and is controlled by an inverted elbow. Between the first two chambers are two six-inch orifices protected from clogging by a trash rack. The separator will have to be cleaned out regularly for it to remain functional.

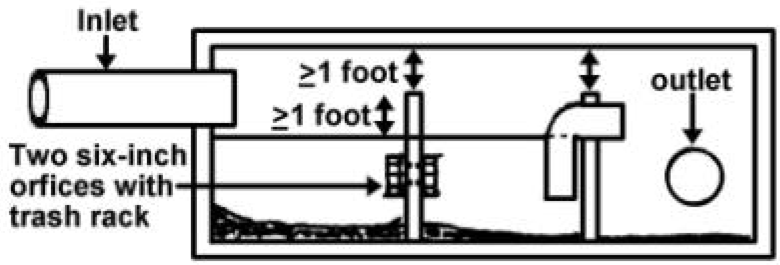


Figure 4.1 Schematic of Oil/Grit Separator (Reference 38)

Winter Freeze-up

When the ground is frozen there will be very little infiltration capacity available to the basin. As a result, winter and early spring runoff may not infiltrate immediately, but will pond in the basin. It is very likely that the capacity of the basin will be exceeded during early spring. If extra storage capacity is not provided for in the pond, a stabilized overflow area should be provided.

Slope Stability

Whenever water is introduced into the ground, there is a potential that the stability of the soil will be impacted as a result of the infiltration. It is suggested that a geotechnical engineer be consulted to determine if water from the infiltration basin will result in stability problems in the vicinity of the basin.

CHAPTER 5: OTHER INFILTRATION DEVICES

The following infiltration practices may be implemented for very small drainage areas, such as a single residence, a parking lot, or a commercial building. As with retention basins, infiltration capacity and runoff volume are the two primary components in the design. These practices can be implemented in the "upland " areas to reduce stormwater runoff quantity and improve quality, by removing stormwater from the surface water regime and putting it into the sub-surface or groundwater regime.

INFILTRATION TRENCHES AND DRY WELLS

These two devices are very similar in that they consist of a hole in the ground that is filled with coarse aggregate, and then covered with a pervious layer of soil. The purpose of these methods, is to direct the runoff to the infiltration area, where it will "soak into" the ground.

The dry well is used primarily to retain runoff from residential and commercial rooftops (Fig. 5.1). Infiltration trenches are used to capture runoff from streets and parking lots (Fig. 5.2).

Two primary criteria for determining if a particular site is suitable for an infiltration trench or dry well, is the same as they are for retention basins.

1. **Seasonal high groundwater and bedrock are at least 4 feet below the bottom of the trench/dry well.**

2. **Infiltration capacity of the soil is at least 0.52 inches/hour, 4 x 10 $^{-4}$ cm/sec** (U.S. Soil Conservation Service soil classification group A or B).

If either of these two criteria is not met, an infiltration method should not be used at the site.

There are various in-depth methods that have been developed to determine the size of an infiltration trench (reference 41). However, instead of going into a detailed analysis, the following estimate is provided.

As a minimum, provide storage volume equal to 0.5 inches of runoff per acre of impervious surface.

EXAMPLE 5.1: An infiltration trench is to capture 0.5 inches of runoff from a 1 acre parking lot, determine the trench dimensions.

1. The volume of runoff from the 1-acre parking lot is determined by:

   1 acre x 43560 sq. ft./acres x 0.5 inch x 1 ft./12 inch = **1815 cubic feet**

2. The storage volume available in the trench does not include the aggregate backfill. The volume of the trench can be estimated by:

$$V_{tr} = V_{ro} /0.4 \qquad (10)$$

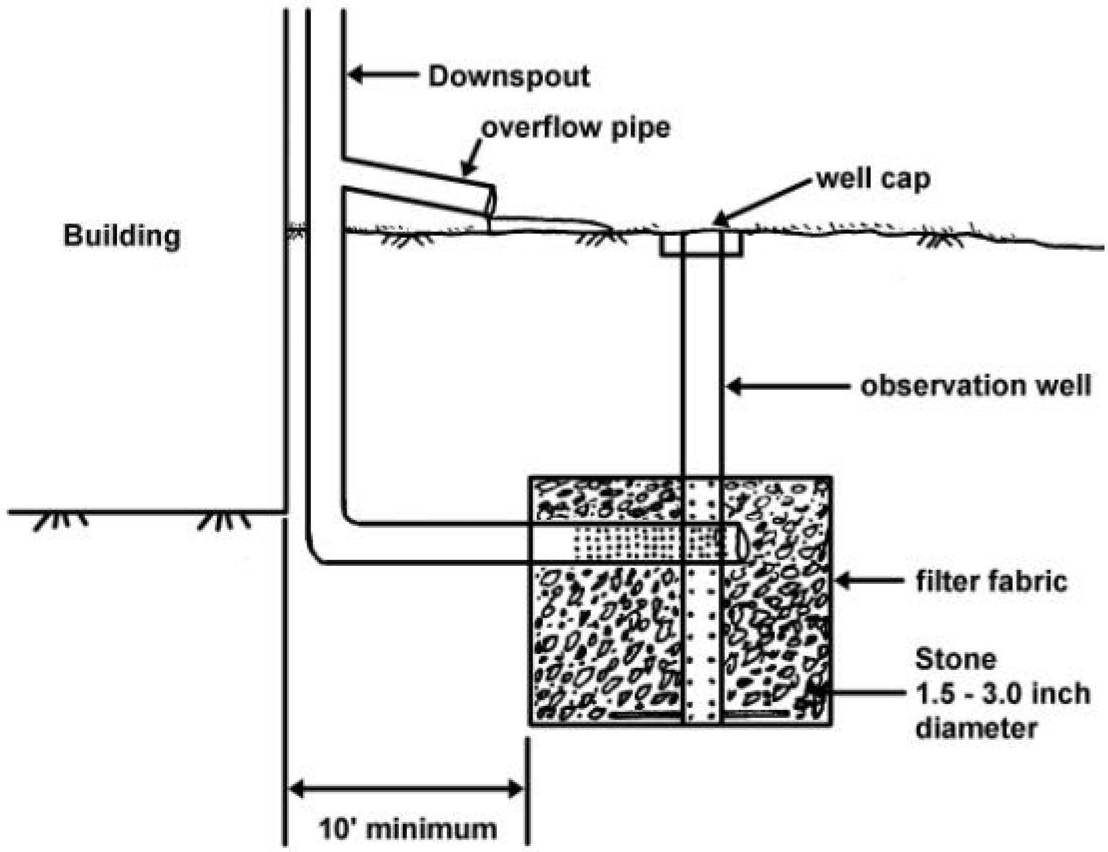


Figure 5.1 - Typical Drywell

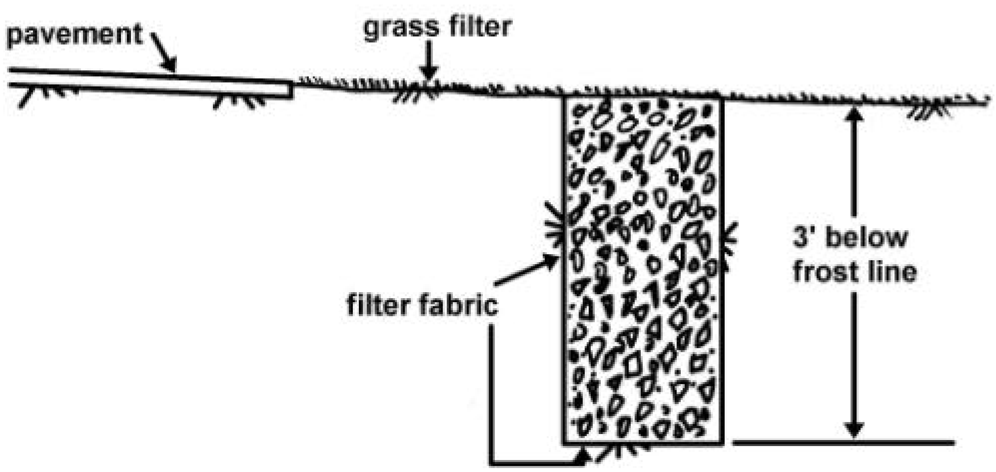


Figure 5.2 - Infiltration Trench

(adopted from reference 38)

where: $\mathbf{V}_{tr}$ - total volume of trench
$\mathbf{V}_{ro}$ - total volume of runoff
0.4 - is the effective porosity, to account for the volume occupied by the aggregate

The total volume of the trench would be estimated using equation (10):

V_{tr} = 1815 cubic feet / 0.4 = **<u>4540 cubic feet</u>**

3. If the trench is 6 feet wide and 4 feet deep, the required length of the trench should be:

$$L_{tr} = V_{tr} / w \times d \qquad (11)$$

where: $\mathbf{L}_{tr}$ - length of trench
$\mathbf{V}_{tr}$ - volume of trench
w - width of trench
d - depth of trench

L_{tr} = 4540 cu.ft. / (4 ft. x 6 ft.) = **<u>190 ft</u>**

Following are some guidelines for designing infiltration trenches:

1. Infiltration rate of the soil should exceed .52 inches per hour.
2. The bottom of the trench should be at least four feet above the seasonal high groundwater.
3. The trench should be backfilled with washed aggregate, 1-1/2 to 3 inch in diameter. If fine material is used, the voids in the aggregate will be reduced, which will reduce the storage capacity in the trench. Better pollutant removal can be achieved using a trench configuration that is broad and shallow, as opposed to being narrow and deep.)
4. Clogging of infiltration trenches by sediment is a primary mode of failure. Thus, **it is essential that either the sediment be controlled before it is picked up by runoff, or it is captured before it reaches the trench.** There should be a vegetative filter strip at least 20 feet wide between the runoff source and the trench.
5. Filter fabric (non-woven is recommended) must surround the backfill material. Without the filter fabric, the trench will become clogged with sediment, and it will be necessary to dig up the entire the trench. Filter fabric will make maintenance somewhat easier.
6. To accommodate flows that exceed the capacity of the infiltration trench, provide a non-erosive channel leading to a watercourse.
7. For infiltration trenches to work during freezing weather, it is suggested that the bottom of the trench be placed about 3 feet below the frost line. (Thus, in Michigan, such trenches would have to be extremely deep to be effective.)
8. Install an observation well in the trench to determine if the trench is functioning.
9. The bottom of the trenches should have a flat bottom (0% slope).

GRASSED (VEGETATED) SWALE

The most common practice of drainage is through the use of curb and gutter, or "drain enclosures," which allow the water to be carried away quickly, solving the drainage problem. However, as has been pointed out earlier, getting the water away quickly simply moves the problem to a downstream property owner or community and may not actually solve the problem. In addition, conveying runoff through drain enclosures has virtually no positive impact on water quality.

A grassed swale, to many, would be referred to as a "ditch." Ditches are something that property owners and drainage engineers have been trying to eliminate for years. However, in the past few years, it has been realized that there are water quality benefits to using swales in lieu of pipes or gutters. Grassed swales allow pollutants to be filtered out by the grasses while also allowing infiltration into the ground. As a result, pollutant loading can be reduced significantly through the use of grassed swales.

Various studies throughout the United States and Canada indicate significant reduction in runoff rates and pollutant loadings when grassed swales are used as opposed to pipes or gutters. However, the biggest obstacle to overcome when proposing a grassed swale is the general public's perception that grassed swales are "drainage problems," and "eye-sores."

The swale may require periodic maintenance to remove trapped sediments. The primary concern with swale maintenance is keeping good cover of grass, which may require periodic reseeding or sodding. Property owners adjacent to the swale should be educated in the function of the swale, as their actions may impact negatively on the swale's performance by keeping the grass too short or applying fertilizers and herbicides.

Figure 5.3 provides a sketch of a grassed swale, which has incorporated swale blocks. It would be desirable to configure the check dam in a "V" shape, to try to minimize the erosion at the ends of the check dam. The purpose of the swale block is to provide a "mini" in -line retention basin. The storage capacity behind the swale blocks is designed equal to the volume of runoff that is desired to be retained.

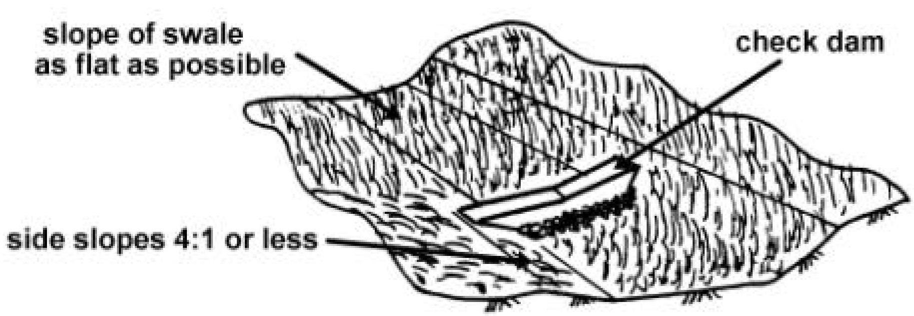
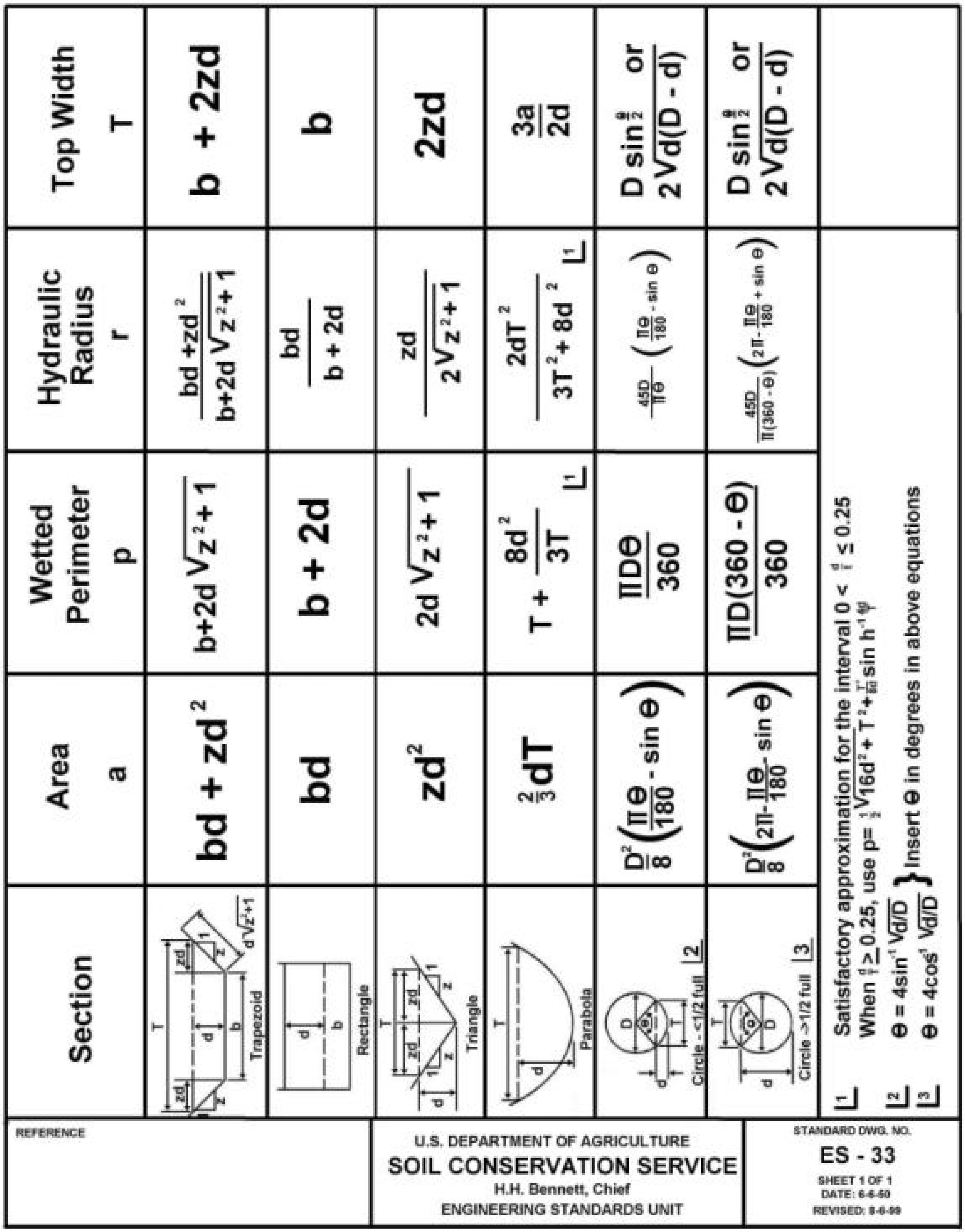


Figure 5.3 - Grassed Swale with Check Dam

Figure 5.4 provides a listing of area, wetted perimeter, and hydraulic radius for various swale shapes. Figure 5.5 provides an approximate method for computing the volume of storage behind swale blocks.



Section	Area a	Wetted Perimeter p	Hydraulic Radius r	Top Width T
Trapezoid	$bd + zd^2$	$b + 2d\sqrt{z^2+1}$	$\frac{bd + zd^2}{b + 2d\sqrt{z^2+1}}$	$b + 2zd$
Rectangle	bd	$b + 2d$	$\frac{bd}{b + 2d}$	b
Triangle	zd^2	$2d\sqrt{z^2+1}$	$\frac{zd}{2\sqrt{z^2+1}}$	$2zd$
Parabola	$\frac{2}{3}dT$	$T + \frac{8d^2}{3T}$ [1]	$\frac{2dT^2}{3T^2 + 8d^2}$ [1]	$\frac{3a}{2d}$
Circle - <1/2 full [2]	$\frac{D^2}{8}\left(\frac{\pi\Theta}{180} - \sin\Theta\right)$	$\frac{\pi D\Theta}{360}$	$\frac{45D}{\pi\Theta}\left(\frac{\pi\Theta}{180} - \sin\Theta\right)$	$D \sin\frac{\Theta}{2}$ or $2\sqrt{d(D-d)}$
Circle - >1/2 full [3]	$\frac{D^2}{8}\left(2\pi - \frac{\pi\Theta}{180} + \sin\Theta\right)$	$\frac{\pi D(360-\Theta)}{360}$	$\frac{45D}{\pi(360-\Theta)}\left(2\pi - \frac{\pi\Theta}{180} + \sin\Theta\right)$	$D \sin\frac{\Theta}{2}$ or $2\sqrt{d(D-d)}$

[1] Satisfactory approximation for the interval $0 < \frac{d}{T} \leq 0.25$
When $\frac{d}{T} \geq 0.25$, use $p = \frac{1}{2}\sqrt{16d^2 + T^2} + \frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

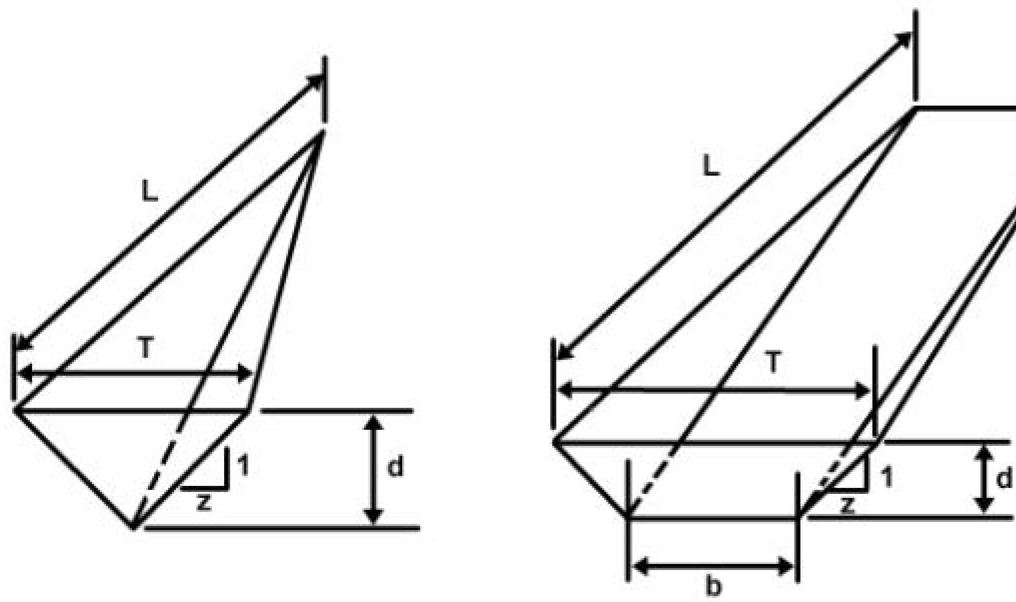
[2] $\Theta = 4\sin^{-1}\sqrt{d/D}$
[3] $\Theta = 4\cos^{-1}\sqrt{d/D}$
} Insert Θ in degrees in above equations

REFERENCE

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
H.H. Bennett, Chief
ENGINEERING STANDARDS UNIT

STANDARD DWG. NO.
ES - 33
SHEET 1 OF 1
DATE: 6-6-50
REVISED: 8-6-99

Figure 5.4 - Hydraulic Elements of Channel Sections



Triangular-Shaped Swale

Top width (T) = 2zd

Volume = $\frac{d^2zL}{6}$

Trapezoidal-Shaped Swale

Top width (T) = b + 2zd

Volume = $(d^2 zL)/3 + (dbL)/2$

Figure 5.5 - Estimated Volume of Storage Behind Swale Blocks

Example 5.2: Given: One-acre parcel that is 80% impervious. Design swale block spacing to retain 0.5 inch of runoff per acre of impervious surface from the parcel within the swale. The swale has a 4 -foot bottom width, and a side slope of 4:1(h:v), and a bottom sl ope of 0.005 feet/feet (.5%). In addition, the swale should be designed to carry the 2 -year flow which is estimated to be 30 cfs at this location. The roughness coefficient is taken to be 0.07.

Design the swale, and the swale block spacing.

1. **Estimate volume of runoff** from the parcel that is to be stored within the swale behind the swale block:

   Volume = 1 Acre x 43560 sq.ft/acre x 0.8 imp x 0.5/12 ft runoff = 1452 cubic feet

2. **Estimate volume available** behind the swale block, assuming a trapezoidal shape:

   Volume = (dzL)/3 + (dbL)/2 (see Figure 5.5) (12)

   where:
   - b - bottom width
   - d - depth
   - z - side slopes (h:1v)
   - L - length between swale blocks

Using equation (12), and assuming a depth of 1.5 feet, the length of the swale is estimated to be:

$$1452 = [(1.5)^2 4(L)]/3 + [(1.5)4L]/2$$
$$1452 = 3L + 3L$$
$$L = 242 \text{ feet}$$

(The required length could be reduced to 156 feet if the depth were increased to 2 feet.)

3. **Estimate channel depth** to carry the 30 cfs design flow using Manning's equation:

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2} \qquad (13)$$

where: **A** - area (for trapezoid) - $(bd + zd^2)$ (ft.^{2})
R - hydraulic radius - (see figure 5.4) (ft.)
n – Manning's roughness coefficient
S - slope (feet/feet)

Note: When computing the area and hydraulic radius, b (width) is the channel "bottom" width at the top of the swale block and d(depth) is the distance between the water surface and the top of the swale block. From figure 5.5, the channel width at the top of the swale block in this example is equal to:

$$T = b + 2zd = 4 + 2(4)(1.5) = 16 \text{ ft.}$$

Using equation (13):

$$30\text{cfs} = \frac{1.486}{0.07} A R^{2/3} (.005)^{1/2}$$

Thus: $ar^{2/3} = 20$

d	a*	p**	r	$ar^{2/3}$
0.1 1.1	20.0 22.4	24.3 25.1	0.82 0.90	17.6 20.8

The area (a) can be computed using:

$$* \; a = bd + zd^2 = 16d + 4d^2 \qquad \text{(from figure 5.4)}$$

By trial and error it is possible to determine the depth needed to obtain $AR^{2/3} = 20$. By assuming a depth, the wetted perimeter (p) can be computed using:

$$** \; p = b + 2d\sqrt{z^2 + 1} = 16 + 2d\sqrt{17} = 16 + 8.25d$$

For an assumed depth of 1.1 feet, the computed $AR^{2/3}$ is very close to the $AR^{2/3}$ value that is required. It would be advisable to include a freeboard elevation try to account for any uncertainties. Figure 5.6 shows the swale configuration. Note that the depth (d) is the distance above the swale blocks. The area below the top of the swale blocks is storage area, and does not convey floodwaters.

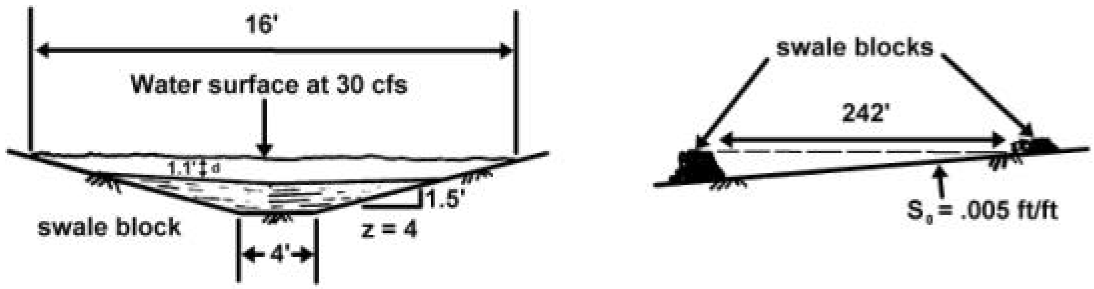


Figure 5.6 - Swale Configuration For Example

Following are some guidelines for grassed swales:

1. The side slopes should be 4:1 (h:v) or flatter.
2. Underlying soil should have a permeability that is .5 inches/hour or greater (an A or B type soil).
3. Dense vegetation that is water tolerant and resistant to erosion should be planted.
4. Slope should be less than 2% (2 feet per 100 feet). Slopes that exceed 2% should include check dams to limit the velocity and potential erosion.
5. Velocities should be less than 5 feet per second.
6. Set the top of the swale at least .5 feet above the design flow water surface elevation.

GRASS FILTER STRIPS

The use of grass filter strips can be quite effective in removing particulate pollutants from overland flow. Some of the uses include directing runoff from parking lots or rooftops across a filter strip before discharging into a drainage course, or infiltration basin. The object of a filter strip is for the grass to act as an obstruction to flow and result in the particulates settling out. For a filter strip to work, it is necessary for the depth of flow to be less than the grass height.

Research has been done relating to the effectiveness of filter strips. Such research (references 6 & 31) indicated that the effectiveness of the filter strip is a function of several variables, such as rainfall intensity, total rainfall, slope of the filter strip, depth of flow on the filter strip, length of contributing area, particle size, and filter-strip length.

In addition to these variables, there are unknowns, such as spacing of the plants and sediment accumulation. Instead of trying to include design charts for all of the possible variables, it is suggested that a filter strip width of about 20 feet at a slope of about 1% be used where possible. This criteria would capture more than 90% of particles that are 10µ (10 micron) or larger for most conditions. (Note: As a reminder, fine sand is about 40µ to 100µ, silt is about 10µ, and clay is about a 1µ-size particle.)

In many instances, more than 90% of particles that are less than 10µ will be captured by a 10-foot filter strip. To capture particles that are 1µ, filter strip widths in excess of 400 feet may be required. Research has indicated that filter strip widths to capture 1µ particles would have to be up to 100 times longer than required for 10µ particles.

Filter strips are typically used in conjunction with other stormwater-management practices to reduce the sediment being introduced into the drainage system. Because the filter strips are very effective at capturing particulates, there will be considerable amount of maintenance that will be required to keep the filter strips functioning. The grass should be cut only when absolutely necessary to ensure that the filtering capacity of the strip is maintained. In addition, it will be necessary to frequently vacuum near the point at which the flow will enter the filter strip. Without adequate maintenance, the effectiveness of the filter strip will be greatly reduced, and there is a possibility that the sediments will be picked up by future runoff events.

At times it may be necessary to incorporate the sediments into the soil by plowing up the grass strip and replanting the area, preferably with sod.

Following are some guidelines that may be used for the construction of filter strips:

1. It is suggested that, at a minimum, the filter strip be about 20 feet wide with a slope of about 1%. This width and slope does contain a factor of safety. Thus, if site conditions require some modifications to the filter strip, the modifications can be done, and the strip can still achieve significant sediment reduction.
2. Grasses that are used in the filter strip should be resistant to water inundation and salt. Grasses such as perennial rye grass, tall fescue, and creeping red fescue have shown a resistance to salt and can grow in a Michigan climate. It would be advisable to plant a mixture of grasses to minimize the possibility of a disease or fungus killing the ground cover composed of a single species.
3. Care should be taken in the final grading so that flow is not channelized on the filter strip. The runoff from the contributing area should be as wide as possible to allow the flow to spread out, which will facilitate the deposit of particles.
4. Filter strips are most applicable for small watershed areas, typically less than 5 acres.
5. Soils most suitable for filter strips include types A, B, and C. D soils may be used, but they are less desirable.
6. If the contributing area has a high output of sediment, the filter strip may require an excessive amount of maintenance to keep it functioning. Thus, to keep the filter strip effective, erosion-control techniques may have to be incorporated into the contributing area to reduce the sediment runoff.

CHAPTER 6: WATER-QUANTITY CONSIDERATIONS

Thus far, the primary focus of this guidebook has been on the water quality aspects of stormwater management. Before the actual design is discussed several design considerations should be investigated.

DESIGN STORM

Before a detention or retention facility can be designed, it is necessary to determine what type of protection is desired. More communities around the United States are beginning to use the 100-year, 24-hour storm as the design standard. Such a standard is consistent with the National Flood Insurance Program and current floodplain mapping for the State of Michigan.

A primary goal of stormwater management is to maintain flood discharges at current levels, even after development has taken place. Without adequate stormwater management, flood discharges, flood damages, and erosion may take place at downstream locations, as a drainage basin changes from undeveloped to developed.

Before selecting a design storm, it is advisable to look at downstream properties to see what is or may be impacted by flooding. If flood damages are occurring frequently, it will not be enough to look at only one design storm, such as the 100 -year flood. It will be necessary to look at a range of storms to be sure that the proposal is not increasing flooding potential for downstream properties. There may be instances in which a detention pond that reduces or maintains the existing 100 -year discharge may increase the impact of flooding caused by the more frequent floods.

Table 6.1 is an example of frequency and rainfall amounts. Appendix B shows the plots of the remainder of the state.

Table 6.1 - Frequency and Rainfall Amounts for Eaton County

Frequency	1-yr	2-yr	5-yr	10-yr	50-yr	100-yr
24-hr. rainfall (inches)	2.2	2.6	3.1	3.5	4.6	5.1

LOCATION OF DETENTION STORAGE

In the past, communities have passed ordinances that require peak runoff rates after development to be less than or equal to runoff rates before development. The criteria may change from community to community; however, the goal is to maintain the current runoff rates through the use of on -site storage. While the concept may be honorable, in many instances, the result of the ordinance is the construction of a number of detention basins throughout the community for which the combined effects actually increase downstream flooding.

The size and location of detention storage impacts the peak flood flows (reference 20). Basin wide planning is essential to result in properly sized basins and to prevent flood discharges from being increased.

In 1986, the DEQ studied the Sargent Creek watershed in Oakland County to determine the impact that detention has had on the flood flows of this urbanized basin (reference 29). As the watershed was urbanizing, on -site stormwater detention was required. The study looked at the impact that the on -site detention basins had on the flood flows as compared to a regional detention basin or a series of detention basins. It was found that an in -line detention basin would need about one-half of the amount of land that th e on-site detention basins needed to accomplish the same impact on flood discharges. The study also indicates that in some instances regulated on -site detention ponds have increased peak flows downstream by delaying outlet peaks to the extent that all of the flood peaks combine simultaneously.

At the extreme upper and lower ends of the watershed, detention ponds will have little beneficial impact on peak flows. Since the runoff from the extreme upper end of the watershed will reach the downstream areas after the flood peaks have already occurred, detention in the upper watershed area would virtually have no impact on peak flows. At the extreme lower end, detention would delay runoff that normally would have been gone and release it when the peak flow from upstream reaches the site. This would result in an increased flood peak. Figure 6.1 illustrates the most effective locations for detention ponds. It should be noted that this figure applies only for water- **quantity** purposes. Treatment for water **quality** should be addressed **throughout** the watershed.

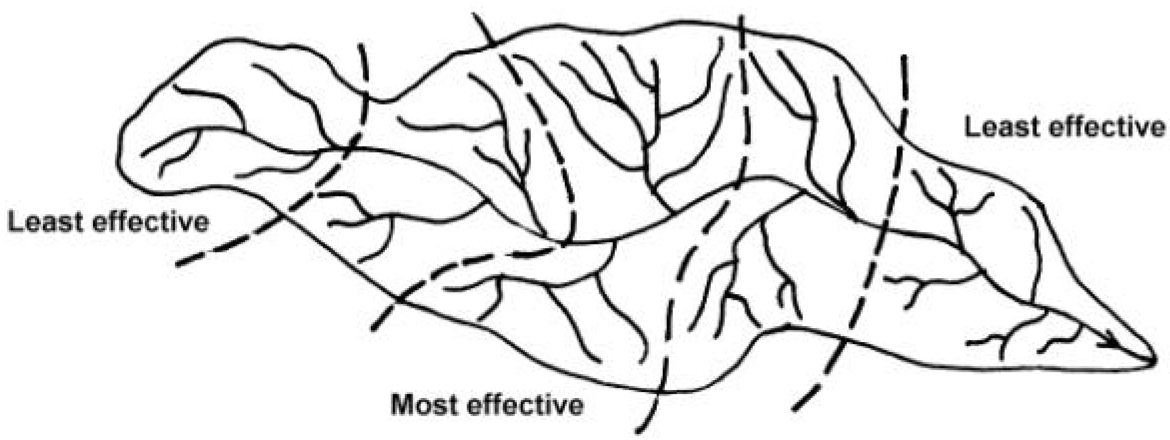


Figure 6.1 - Effectiveness of Detention Location within a Watershed

The installation of detention facilities at the lower end of a subwatershed may hold water that would have normally been gone. The release of the water may occur at the same time that the flood peak on the main channel reaches the site. As a result, the detention basin at such a location may actually increase the flood peak. For this reason, it is essential that the entire watershed be considered when the stormwater management plan is being developed. An effective detention pond design must look at the timing of the flood hydrographs, in addition to the volume of runoff.

ON-SITE DETENTION

There is quite a bit of information available for the design of individual detention ponds, which primarily deal with volume of runoff and rate of runoff. However, there is not much information on the impacts of on -site detention ponds as opposed to a regional detention ponds. Some studies have indicated that randomly placed on-site detention can actually increase peak flows and that a regional approach would be more effective.

Another concern with on -site detention is the long -term maintenance requirements. Since the detention pond will be placed on private property, it will be necessary to have a maintenance agreement or easement to ensure that the ponds are maintained. If they are not maintained, the basins will not be effective, and will likely turn into "eyesores." Because of the maintenance requirements and the potential problems, the public may not readily accept a pond being placed on their properties.

The use of on-site detention/retention is most appropriate for water-quality benefits. The use of grassed swales, filter strips, and infiltration basins can have a very beneficial effect on the quality of stormwater runoff. If on -site detention is required to control the volume of runoff, it is essential that the entire watershed be considered. A detailed hydrologic analysis must be prepared to determine the effect that the detention requirements will have on the flow characteristics of a watercourse.

REGIONAL FACILITIES

As noted earlier, in many instances, it is neither feasible nor advisable to require on-site detention. In such instances, a **regional** facility can be used to achieve the required detention.

A regional facility will usually require less land than would be required to achieve the same effects from numerous on -site facilities. There will also be a savings on construction and maintenance costs associated with a regional facility, as opposed to many on -site facilities.

As with on-site detention, the placement of a regional detention facility will require a hydrologic analysis of the watershed. Since a regional facility is normally placed on public land, the problem with easement and responsibility of maintenance will be minimized. However, there will still be the problem of providing adequate maintenance.

Regional facilities can be more readily accepted by the public if designed and maintained properly. Since regional facilities will be larger than on -site facilities, it is possible to incorporate multi-purpose uses into the design (such as soccer fields, football fields, fishing ponds, and parks).

Regional facilities are typically located in areas which provide natural storage. However, in most instances, wetlands are the "natural storage areas." Early in the planning process it is critical to consider the impacts that the detention facility will have on the wetland complex. The district office of the Land and Water Management Division should be consulted early in this process. See Appendix A for office locations.

OFF-LINE DETENTION (OFF STREAM)

Off-line detention is placed outside of the natural watercourse or storm sewer system (See Figure 6.2). The detention is achieved by diverting flows into a storage facility, when a certain flow rate is exceeded. Low flows will bypass the facility, thereby minimizing the warming of the water which may happen if the water passes through a detention facility.

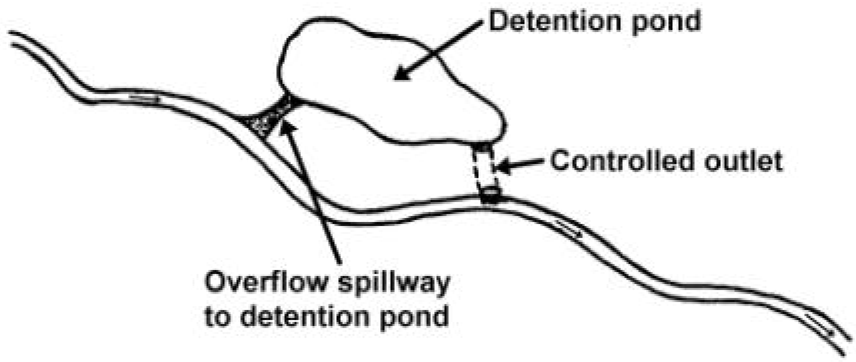


Figure 6.2 - Off-line Detention

Since the storage is not within the conveyance system, water may be stored as long as desired to achieve the necessary improvement in water quality or peak-flow reduction. However, care must be taken to ensure that the detention does not result in objectionable odors or health problems.

If not adequately designed, the inlet control devices may be overloaded, and the peak flow will not be attenuated as had been desired.

Off-line detention will require storage that may be considered to be "developable," and thus it may be difficult to obtain a detention site. Along the same line, if the area is already developed, a site may not be available that is off line.

IN-LINE (ON STREAM)

In-line detention is placed within the flow-carrying network (See Figure 6.3). If designed with adequate storage capacity, the in -line detention facility can provide attenuation to flood peaks. However, to achieve the required storage, it may be necessary to construct embankments and control structures, which will increase flood stages within the influence of the basin. If upstream property owners are affected by the increased flood stages, it will be necessary to obtain flooding easements. In some instances, it may be difficult and very expensive to obtain the flooding easements.

There is potential that water detained by an in -line detention basin may be warmed. If the basin discharges into a coldwater stream, it may be necessary to include a design that minimizes the warming, such as drawing water from near the bottom of the basin.

In-line detention can have a significant effect on the impoundment area upstream of the outlet structure. If wetlands are present, the impoundment may change the character of the wetlands. Also, the water quality within the basin may be degraded and important natural stream values lost. Thus, it is critical to work with the district office of the Land and Water Management Division to identify the wetland areas and if possible design a detention facility without significant degradation of natural resources.

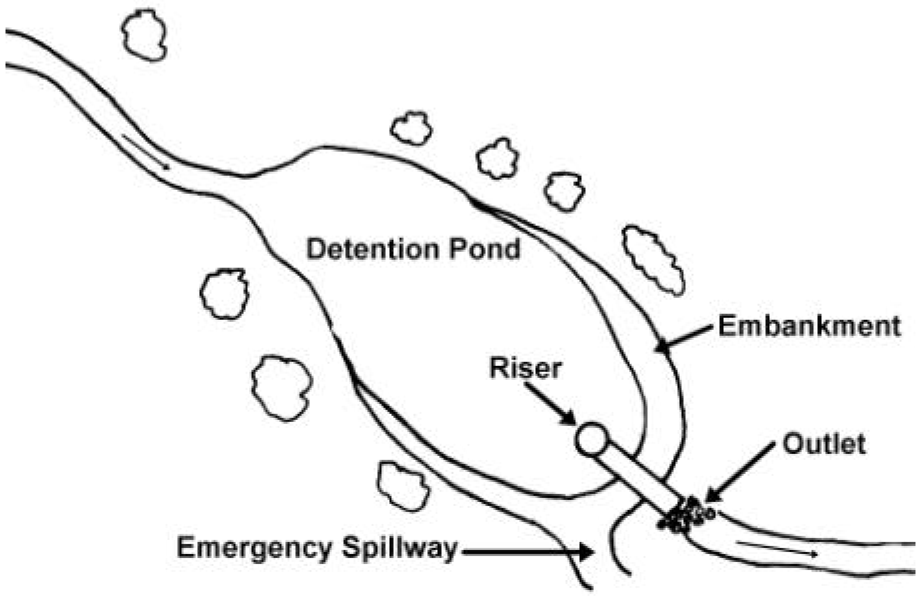


Figure 6.3 - In-line Detention

CHAPTER 7: HYDROLOGY

For a stormwater-management design to be effective, it is necessary to develop an understanding of how much water will be running off a watershed and the rate at which the runoff will occur. This section of the guidebook will discuss various methods that are available to estimate the runoff and the peak flows. It is important to note that hydrology is a combination of "art" and science. Since many methods are available that will produce a discharge, there is no one method that is always "correct." This guidebook will focus on the methodologies that are used by the Michigan Department of Environmental Quality, Land and Water Management Division, Hydrologic Studies Unit (telephone # 517-284-5570. Other methods will be mentioned; however, they will not be discussed in detail.

There are numerous variables that are required to compute flood flows using the various hydrologic methods. Two variables that would be required for the majority of the methods include **drainage area** and **precipitation.**

WATERSHED DELINEATION

A very important component of any hydrologic study or analysis is determining the amount of area that will be contributing runoff to the design point. The boundary of a watershed for surface water runoff will follow ridges or high points that separate one drainage basin from another (See figure 7.1).

An inaccurate boundary will result in inaccurate runoff volumes and peaks. Thus, it is critical to spend the time necessary to get the delineation as accurate as possible. The watershed boundary is drawn using the following considerations:

1. Obtain the most up-to-date topographic information, such as USGS quadrangle, aerial photographs, county or community topographic mapping, or storm -drain maps. In urban areas, the use of only a USGS quadrangle may result in significant errors in the delineation. Storm-drain maps and better topographic information is critical in urban areas.
2. Identify the main watercourse and all of its tributaries. Identify ridges and high points that outline the boundary of the watershed.
3. Water will flow perpendicularly to the contours on the topographic map.
4. The topographic contours point upstream when it crosses a watercourse.
5. In urban areas, street grades may define the drainage boundary.
6. Delineate areas that will be draining into depressions from which flow will not escape (natural retention areas). These depressions will have to be deep enough so water will be retained. Such drainage areas will be subtracted from the total drainage area as they will not contribute to the runoff. Care should be taken to make sure depressions are not filled or drained during development.
7. It is necessary to field check the delineation to determine if it is appropriate. It is possible that urbanization or drain projects may have changed the drainage patterns since the topographic map has been prepared. In some instances, the topographic map may not be adequate to accurately determine drainage boundaries due to the contour intervals on the map.
   In reference 1, the watershed delineations for 12 watersheds in the Denver area based solely on a topographic map were compared to field-verified delineations. It was found that the watershed area determined by a delineation based solely on a topographic map ranged from 5 times greater to 8 times less than the field-verified delineation.

These are obviously extreme examples; however, they point out the need for field checking watershed boundaries.

8. It is also recommended that the delineation consider the effects that future development may have on the watershed divide. Future development may alter the runoff patterns which may change the size and shape of the watershed.

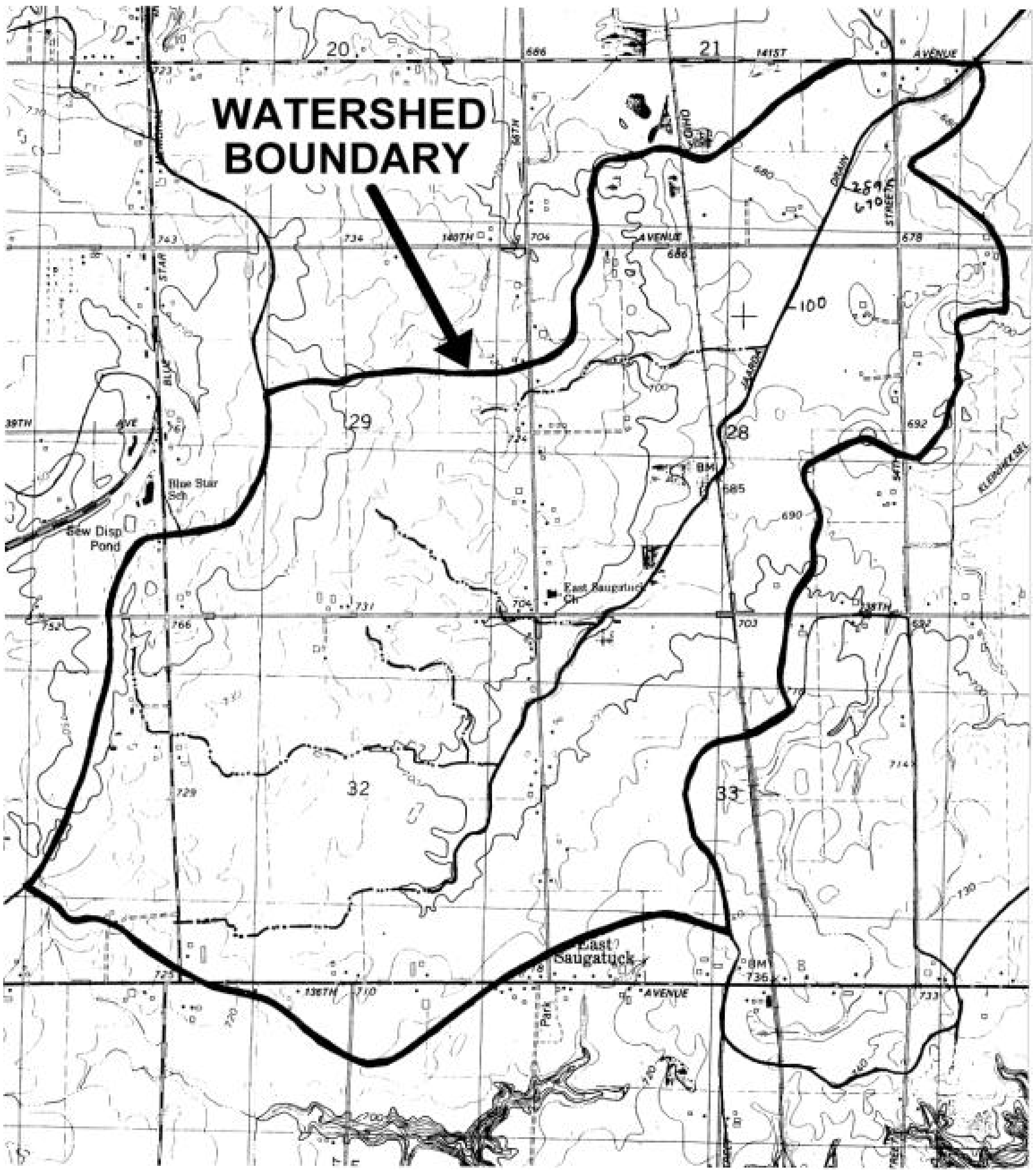


Figure 7.1 - Typical Watershed Delineation

Precipitation

The amount of precipitation that will be occurring in a watershed will be essential in determining the volume of runoff and flood peaks. The National Weather Service is the primary source of rainfall data. Technical Paper 40, “Rainfall Frequency Atlas of the United States" published by the National Weather Service (reference 50) contains expected rainfall durations ranging from 30 minutes to 24 hours, and for frequencies ranging from 1 to 100-year events. In 1990, the DEQ updated the rainfall frequencies for Michigan (reference 40).

Figure 7.3 gives the updated 100-year, 24-hour rainfall amounts for the state of Michigan. Appendix B lists other rainfall amounts for the State. It is suggested that future designs in Michigan utilize the updated rainfall data.

For large drainage areas (greater than 10 square miles), it may be necessary to make an adjustment to the rainfall as it is unlikely that the rainfall will be spread uniformly over a large drainage area. This guidebook is mostly concerned with smaller drainage areas, which will not require an adjustment to the rainfall. However, figure 7.2, does provide the adjustment to the rainfall that would be required for larger drainage areas.

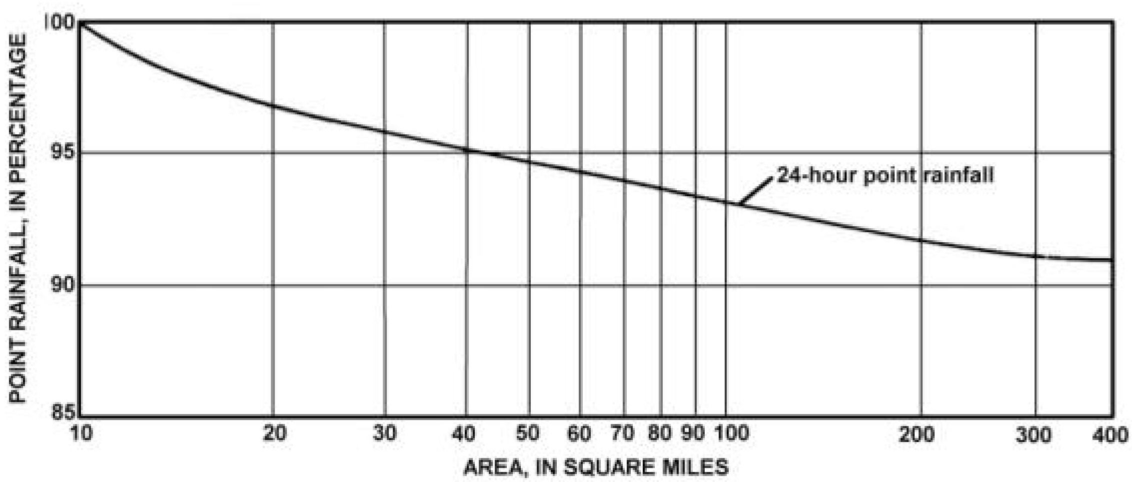


Figure 7.2 - Area-Depth Curve for Adjustment of Point Rainfall

(Adapted from U.S. Weather Bureau, 1961)

OVERVIEW OF HYDROLOGIC METHODS

It is not possible to discuss all of the hydrologic methods that are available to estimate runoff volumes and flood peaks. The methods can range from the rational method to computer models which provide a continuous hydrologic simulation. A survey by the American Public Works Association indicated that over 40 various hydrologic methods are currently in use around the Country. Listed below are some of the methods used by the Michigan Department of Environmental Quality:

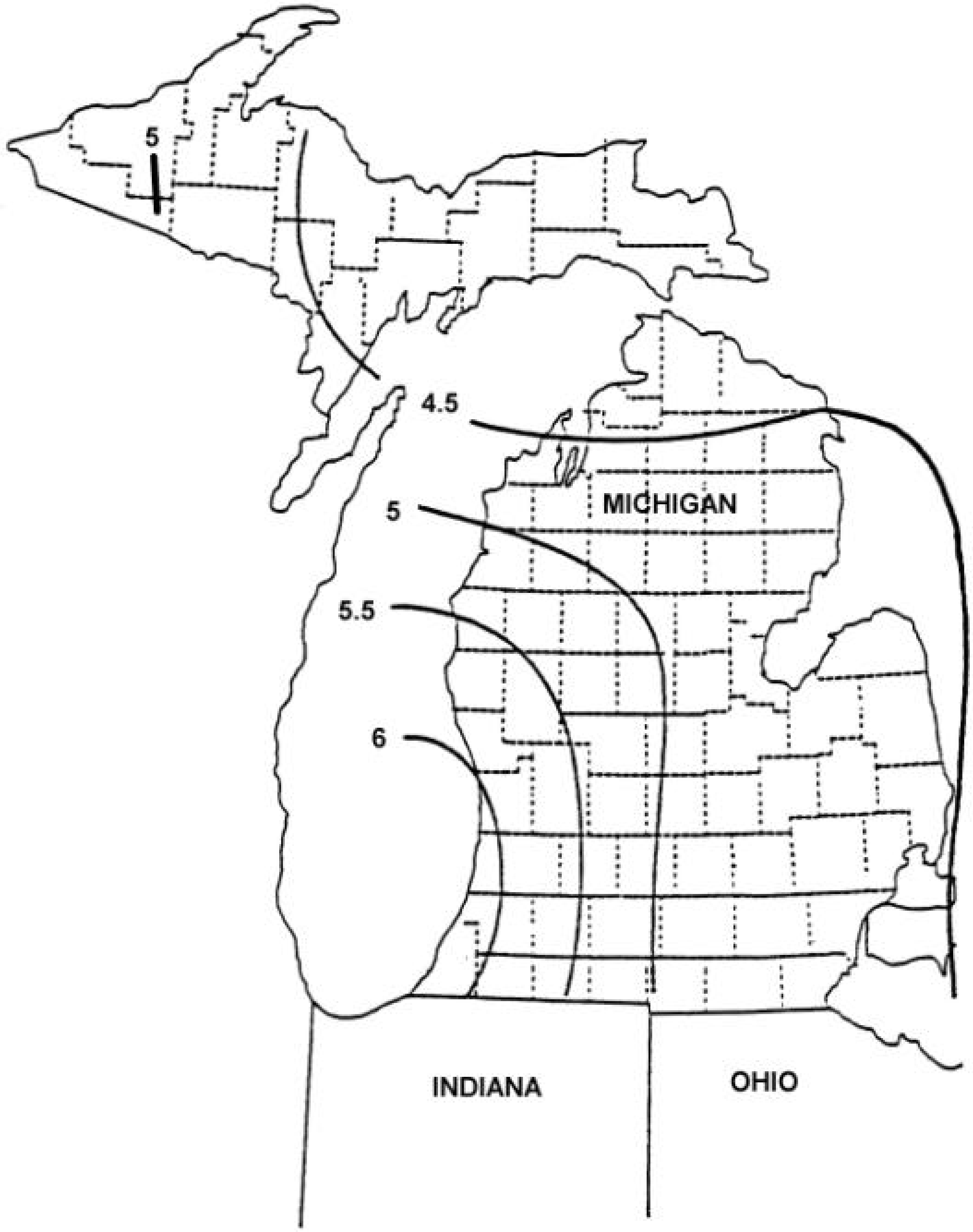


Figure 7.3 - 100-year, 24-hour Point Rainfall Depths, State of Michigan

(From MDNR 1991, Reference 40)

1. **Gaged Locations - Statistical Analysis**

The U.S. Geological Survey in cooperation with State agencies maintains about 145 continuous stream-flow gaging stations, and 48 crest-stage partial-record stations in Michigan. No matter how good the theoretical methods may be, there is no substitute for having information from actual flood events.

A statistical analysis of the gaging-station record provides a discharge-probability relationship for the watercourse at the gaging-station site. Such information can be obtained from either the U.S. Geological Survey (USGS), or from the Michigan Department of Environmental Quality, Land & Water Management Division, Hydrologic Studies Unit (telephone # 517-284-5570).

In addition to obtaining discharge-probability relationships for gage sites, continuous recording gages will also provide a hydrograph at the site. The crest-stage recorders would only provide a peak stage.

Several of the drawbacks of using the gaging stations include:

1. Very few of the rivers and streams have gaging station information. Usually the gaging stations are located on watercourses that have a relatively large drainage area. The majority of the stormwater management designs will likely be on "ungaged" small watersheds.
2. Many of the gages have a relatively short record. Trying to estimate the 100-year flood flows may require extrapolation of the data, which may lead to potential error.
3. Funding reductions for the stream-gaging program has resulted in over 100 gaging-station sites being eliminated in the past 20 years. Thus, some records may be incomplete, and have missed substantial flood events.
4. Some of the watersheds that have gaging stations may have under-gone changes over the years. These changes may include urbanization and channel improvements. As a result, the records may not be homogeneous. In other words, the flows produced by the watershed for similar rainstorms may change over the years due to changes in the watershed or the river system. A statistical analysis may not produce reliable results.

If a stormwater-management study is to be prepared for a watershed, it is advisable to set up a gaging station and a rain-gage network to determine flow and runoff characteristics of the basin. The gage should be in for at least a year, and preferably more. The characteristics of the watershed, and how it responds to various runoff events, will be better understood the longer the gage is in place.

2. **Transfer Methods**

The transfer method uses the peak-flow information computed at a location and extrapolates the information upstream, downstream, or to a different watershed.

The transfer method is limited, as there is an assumption that the flows are a function of the size of the drainage area. If the basin characteristics change from the gage site to the design location, a transfer may not be appropriate. Of particular concern would include changes in land-use, soil type, channel slope, or storage (such as lakes, reservoir or valley storage). There is also the limitation that this method only computes peak flows.

3. **Regression Analysis (Regional Method)**

A regression analysis was developed by the USGS and the DEQ for Michigan (reference 18). The regression analysis is a regional method that allows the designer to compute a peak flow (100-year, 10-year, and etc.) when several physical variables are known. The regression is based upon an evaluation of gage sites throughout Michigan. The variables in the Michigan regression equation include:

basin area
precipitation
channel slope
slenderness ratio (stream length squared divided by the contributing drainage area)
forested area
mean snowfall depth
temperature
geological characteristics (such as clay, glacial till, moraines, glacial outwash, muck, etc.)

There are a couple of limitations on using the regression equation.

1. The equation is not applicable for areas that are either urbanized or where flow is regulated.
2. Caution should be used when the drainage area of a basin is less than 10 square miles.
3. The equation will only compute a peak flow.

4. **SCS Methodology as Adapted to Michigan by the DEQ**

The runoff curve number methodology developed by the Soil Conservation Service (SCS) was adapted to Michigan in a publication prepared by the DEQ (reference 39). The method has been subsequently updated in October 1991 (reference 40). If additional information is needed, it is suggested that references 40 and 49 be reviewed. The method is very straight forward, as it considers drainage area, rainfall data, land use, soil type, time of concentration, antecedent moisture content, and adjustments for swamps and ponding.

5. **SCS Technical Release 55 (TR-55)**

TR-55, <u>Urban Hydrology for Small Watersheds</u> (reference 46), provides simplified procedures to calculate runoff volumes, peak flows, hydrographs, and storage-volume requirements for detention ponds. The methods contained in TR-55 are primarily applicable for small urban/urbanizing watersheds and are also available in a computer-program format.

The methods used in TR-55 to compute the volume of runoff are the same as used in the UD-21 Methodology. The runoff is computed based on soil type, precipitation, and land use.

Under urbanized conditions the terms impervious and "connected" become much more important in regard to runoff. As noted earlier, impervious conditions would include roof-tops, parking lots, roadways, and etc. An impervious surface is "connected" to a drainage course if it drains directly into a drainage course. Table 7.1 shows the percent impervious for given urban land uses, and it also assumes that impervious surfaces are connected directly to the drainage courses. If the land use has a different percentage of impervious area, and/or has less than 100% of the impervious area connected to the drainage course, the TR-55 method includes a method to adjust the runoff curve number.

Table 7.1 - Percent Impervious Areas for Urban Land Uses

Land Use	Average Percent Impervious Area
streets and roads	98
Commercial and business	85 / 85
industrial	72
Residential lot 1/8 acre or less 1/4 acre 1/3 acre 1/2 acre 1 acre	65 38 30 25 20

6. **Computer Models**

Using one of the above methods, basin characteristics are developed and can be input into a computer model such as the Corps of Engineers HEC-1 (reference 44) and HEC-HMS (reference 52), or SCS TR-20 (reference 47). The computer models can generate, combine, and route the flood hydrographs. These computer programs that were once limited to "main-frame" computers are now run on personal computers. Once the basic watershed model is set up, it is possible to consider numerous scenarios and design configurations with minimal additional effort.

OTHER METHODS

In addition to the methods that are used by the DEQ, there are numerous other methods that are being used around the nation. Following is a listing of three other methods that are available:

1. **Rational Method**

Probably the most widely used (and sometimes misused) method for computing runoff volumes and peaks is the rational method (references 2, 4, & 36). The rational method was developed in 1889 as a method of sewer design for urban areas. The rational equation is defined by:

$$Q = CiA \qquad (14)$$

where: **Q** - peak runoff in cfs
C - runoff coefficient
i - average intensity in inches/hour
A - Drainage area in acres

At first glance, the method looks very simple and straightforward. It is true that it is easy to get an "answer" from the equation, but how appropriate is the "answer"? There is a considerable amount of judgement involved in selecting the C coefficient which considers infiltration, land use, rainfall intensity, and depression storage. The average intensity, i, is a function of local precipitation, frequency-duration, and time of concentration. If the designer has considerable experience and is well aware of the methodology and its limitations the rational method can be applied to small drainage areas. The limitations on the size of the drainage area can range from 20 acres to 200 acres, depending on the complexity of the watershed (reference 2). For additional information on the rational method, it is suggested that reference 4 be used.

2. **Continuous Simulation**

Of the various methodologies available for analyzing the hydrology of a basin, a continuous simulation is by far the most complex. The analysis requires a continuous accounting of the soil moisture, evaporation, precipitation and runoff. This methodology requires extensive input data and computation time.

3. **Colorado Urban Hydrograph Procedure**

This method was originally developed in 1969 for the Denver Regional Council of Governments. The method will allow the designer to develop a unit hydrograph, and the design-storm hydrograph for the basin. The procedure requires the following information:

a) Rainfall data
b) Basin information: basin size, slope, soils, land use (pervious and impervious areas), detention storage, and depression storage
c) Data to correlate the model, such as past flooding or gaging station information

The method has been used in some Midwest areas with reasonable results. For more information on the method, it is suggested that reference 9 be obtained.

SCS METHODOLOGY

In the preceding pages, we have mentioned the various methods that are available to compute runoff volumes, peak flows, and in some instances hydrographs. In this section, the SCS methodology will be discussed in greater detail. In addition, an example problem will be worked. The methodology will require the following information:

Drainage Area
As discussed earlier, it is extremely important to have an accurate delineation of the watershed boundary.

Rainfall Data
Rainfall information is available from various sources, as noted earlier.

Land Use
The type of land use is critical in determining the amount of runoff that would be anticipated from a watershed. It was also discussed earlier that different land uses produce different runoff amounts. In an attempt to quantify the runoff potential for various land uses and soil

Table 7.2 - Runoff Curve Numbers for Selected Agricultural, Suburban, and Urban Land Use.
(antecedent moisture condition II, and $I_a = 0.2S$) (Reference 49)

Land Use	**Treatment or practice**	**Hydrologic condition**	**Hydrologic Soil Groups A**	**B**	**C**	**D**
Fallow	Straight row		77	81	88	91
Row crops	Straight row	Poor	72	81	88	91
	Straight row	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	Contoured	Good	65	75	82	86
	…and terraced	Poor	66	74	80	82
	…and terraced	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
	Straight row	Good	63	75	83	87
	Contoured	Poor	63	74	82	85
	Contoured	Good	61	73	81	84
	…and terraced	Poor	61	72	79	82
	…and terraced	Good	59	70	78	81
Close-seeded	Straight row	Poor	66	77	85	89
legumes or	Straight row	Good	58	72	81	85
rotation	Contoured	Poor	64	75	83	85
meadow	Contoured	Good	55	69	78	83
	…and terraced	Poor	63	73	80	83
	…and terraced	Good	51	67	76	80
Pasture or		Poor	68	79	86	89
range		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	Contoured	Fair	25	59	75	83
	Contoured	Good	6	35	70	79
Meadow			30	58	71	78
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Residential	1/8 acre or less lot size		77	85	90	92
	1/4 acre		61	75	83	87
	1/3 acre		57	72	81	86
	1/2 acre		54	70	80	85
	1 acre		51	68	79	84
Open spaces (parks, golf courses, cemeteries, etc.)						
Good condition: Grass cover > 75% of area			39	61	74	80
Fair condition: Grass cover 50-75% of area			49	69	79	84
Commercial or business area (85% impervious)			89	92	94	95
Industrial district (72% impervious)			81	88	91	93
Farmsteads			59	74	82	86
Paved areas (roads, driveways, parkind lots, roofs)			98	98	98	98
Water Surface (lakes, ponds, reservoirs, etc.)			100	100	100	100
Swamp At least 1/3 is open water			85	85	85	85
Swamp Vegetated			78	78	78	78

types, a value termed "runoff curve number" (RCN) was developed. Table 7.2 provides a listing of land use, soil type, and RCN.

Soil Type

As discussed earlier, different soil types have different infiltration capacities. The soils are broken down into four hydrologic categories: A, B, C, and D. Appendix C lists the hydrologic categories for the various soil types.

Soils that are classified as A soils have a high infiltration rate and a low runoff potential. These soils consist of well -drained sand and gravel.

B soils have a moderate infiltration rate. These soils are fine to moderately coarse in texture, including sandy loam, loam, and silt.

C soils have a slow infiltration rate. These soils are fine or finely textured, and include clay loam.

D soils have the slowest infiltration rate, and the highest runoff rate. The soils are mostly clay and have a high water table.

In some instances, a soil type may have more than one hydrologic classification. As an example, Kinross has a soil classification of D/A. This designation indicates that the soil would exhibit D tendencies if in its natural state. However, if the soil has been artificially drained, such as by tiling, the soil will act as an A soil.

Time of Concentration

Time of concentration (t_c), is the time it takes for a drop of water to travel from the farthest point of the watershed to the design point. The farthest point is based on travel time and not necessarily the longest distance. As an example, for a given distance, it will take longer for water to flow overland than it will to travel along a channel.

The smaller the time of concentration, the quicker flood flows can get to the design point and the higher the peak discharge. For a given watershed, if the time of concentration is reduced, the peak discharge will be increased. On large drainage basins, such as the Grand River, it may take days for the peak flows to reach the design point. While on a small, urbanized watershed, the time of concentration may be less than an hour.

An empirical formula has been developed to estimate the velocity of the flood flow which in turn can be used to determine the travel time (time of concentration).

$$V = KS^{0.5} \quad \text{(Reference 39)} \qquad (15)$$

where: **K** - coefficient depending on the type of channel
S - Slope expressed in percent
V - velocity in feet per second

The K coefficient has been determined for the three types of channels:

1. Small tributaries and swamps with channels. These channels are typically shown on topographic maps as solid or dashed blue lines. (K = 2.1)

2. Overland waterways which are well defined by elevation contours and do not have blue lines indicating a channel. This classification would also include swamps with channels (K = 1.2).
3. Sheet flow that is not well defined by el evation contours (K = 0.48).

By substituting the K values into equation 15, the following equations are obtained:

$$V = 2.1\ S^{0.5} \quad \text{(small tributaries and swamps w/channels)} \tag{16}$$

$$V = 1.2\ S^{0.5} \quad \text{(waterways and swamps without channels)} \tag{17}$$

$$V = .48\ S^{0.5} \quad \text{(sheet flow)} \tag{18}$$

Once velocities are known, it possible to determine the travel time to a design point.

$$t_c = \text{Length} / V \times 3600 \tag{19}$$

where: **t_c** - time of concentration
Length- distance, in feet, from the most dist ant point in the watershed
V - velocity, in feet per second
3600 - converts seconds to hours

In most situations, different flow types will be occurring as the water flows from the headwaters to the design point. As a result, it will be necessary to compute the t_c for each of the flow types, then add all of the t_c's together. In addition, if there is a significant change in slope, it would be necessary to break a flow type down further to reflect the slope change.

Example 7.1: Time of Concentration

Compute the time of concentration given the following information:

1. Small tributary length of 4000 feet of which, 3000 feet is at a slope of 0.2%, and 1000 feet is at a slope of 1%.
2. The waterway length is 800 feet at a slope of 2%.
3. The sheet flow length is 500 feet at a slope of 0.5%.

Flow Type	Length	Slope(%)	V(fps)*	tc(hrs.)**
Small tributary	3000	0.2	0.94	0.89
Small tributary	1000	1.0	2.10	0.13
Waterway	800	2.0	1.70	0.13
Sheet	500	0.5	0.34	0.41

The total t_c = 0.89 + 0.13 + 0.13 + 0.41 = **1.56 hrs**

* Note: V computed using equations 16, 17, and 18
** t_c = length / V x 3600, (3000/.94 x 3600) = .89 hrs

SURFACE RUNOFF

The major component of any stormwater management design is the amount of surface runoff (SRO). As discussed earlier, runoff will occur when the infiltration capacity of a soil is exceeded by the rainfall intensity. The runoff curve number (RCN) is used in the following equation to estimate the SRO:

$$SRO = \frac{(P-200/RCN+2)^2}{(P+800/RCN-8)} \qquad (20)$$

where:
- **SRO** - runoff, inches
- **P** - rainfall, inches
- **RCN** - runoff curve number, Table 7.2

Figure 7.4 provides a graphical solution of the above equation.

As an example, if a basin has a RCN of 74, and a total rainfall of 4.3 inches, the SRO is computed to be:

$$SRO = \frac{(4.3-200/74+2)^2}{(4.3+800/74-8)} = \frac{(3.60)^2}{(7.11)} = \quad 1.82 \text{ inches}$$

It is rare for a basin to have a single hydrologic soil type and land use. When a basin does contain more than one hydrologic soil type or land-use type, it will be necessary to break the basin up into soil types and land uses. Following is a suggested procedure determining the runoff from a basin with multiple land uses and soil types (figure 7.8 may be useful in this computation):

1. Determine the percentage of the hydrologic soil types throughout the basin.
2. Determine the different land uses that are present for each soil type. Then determine the percentage of each land use within each soil type.
3. Assign RCN and compute the runoff from each land use within each soil type. Add the runoff from each land use/soil type to determine the runoff from entire basin.
4. If an average RCN is needed for the entire basin, the total runoff volume is divided by the total drainage area to obtain an average runoff. Using the average runoff, and the precipitation, the graph in figure 7.6 can be used to determine an average RCN.

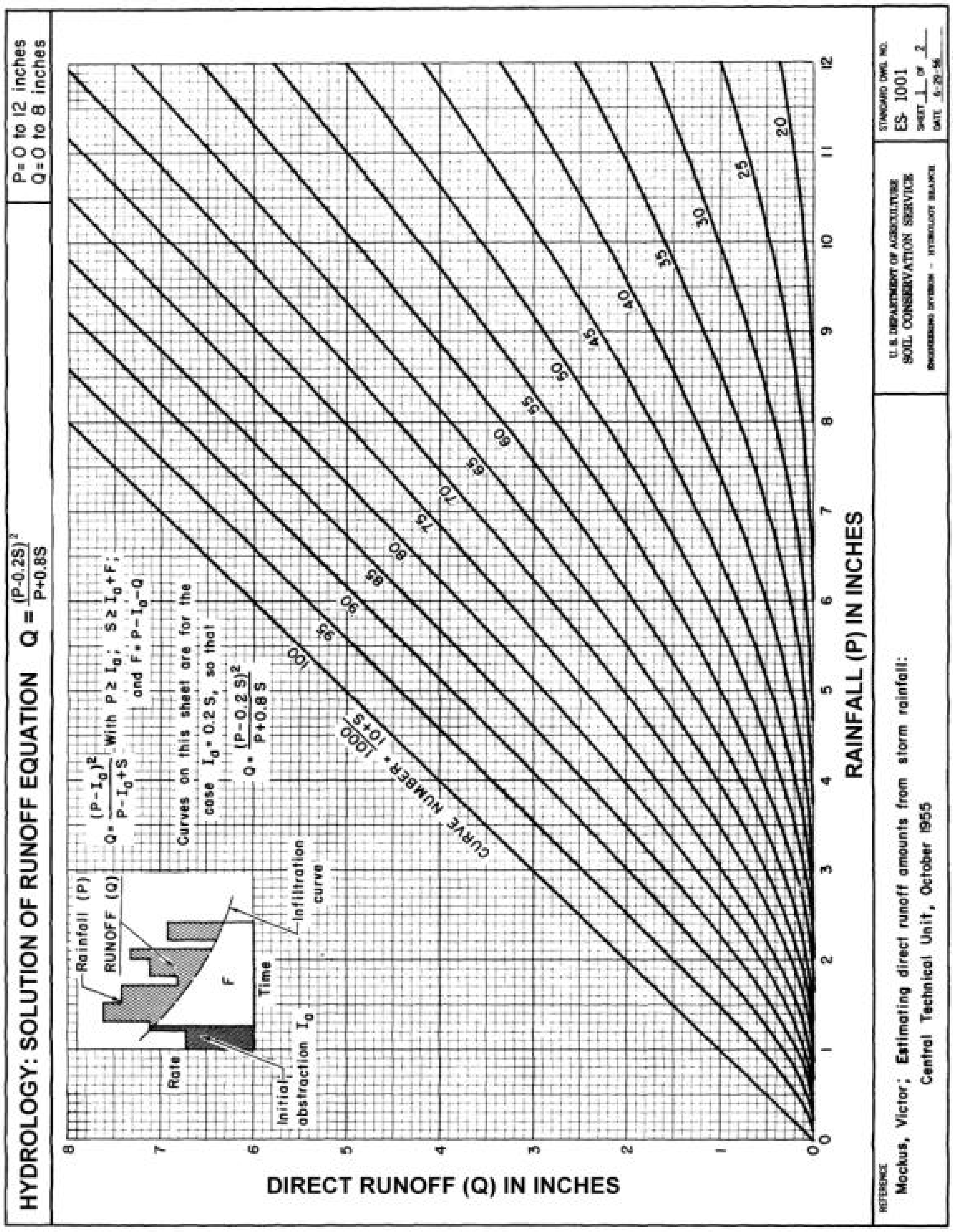


Figure 7.4 - Graphical Solution of Runoff Equation

(Source: Soil Conservation Service, reference 49)

Example 7.2: # Runoff Computations

Compute the runoff from a 1.1-square-mile basin which includes the following hydrologic soil types and land uses. The 100-year 24-hour rainfall is 5.1 inches.

1. B soils - 30%, of which 40% is forest, 40% is 1/2 acre residential, and 20% is parks.
2. C soils - 60%, of which 20% open space, 80% is 1/2 acre residential.
3. D soils - 10%, of which 100 % is meadow.

Soils			**Land Use**			**Runoff**		
Group	%	sq. mi.	Type	%	sq. mi.	RCN	r.o.	Sq. mi.-in.
B	30	0.33	forest res. park	40 40 20	0.132 0.132 0.066	70 70 61	2.11 2.11 1.43	0.279 0.279 0.094
C	60	0.66	open res.	20 80	0.132 0.528	74 80	2.44 2.98	0.322 1.573
D	10	0.11	meadow	100	0.11	71	2.19	0.241

The **total volume of runoff** is:

$$= 0.279 + 0.279 + 0.094 + 0.322 + 1.573 + 0.241$$

$$= \mathbf{2.79\ sq.mi\text{-}in}$$

The average runoff $= \dfrac{2.79\ \text{sq.mi.-in.}}{1.1\ \text{sq. mi.}} =$ **2.54 inches**

Using a precipitation of 5.1 inches and a runoff of 2.54 inches, an average RCN of 75 for the basin may be determined from Figure 7.4.

ANTECEDENT MOISTURE CONDITION

The antecedent moisture condition (AMC) of a soil is an index of the "wetness" of the soil. For the SCS methodology there are three levels of AMC:

1. **AMC-I** has the lowest runoff potential. The soils are relatively dry.
2. **AMC-II** is an average condition.
3. **AMC-III** occurs when the watershed is saturated, thus the runoff potential is the highest.

Table 7.3 lists the AMC groups based on the total 5 -day previous rainfall:

Table 7.3 - Total 5-day antecedent rainfall, inches

AMC Group	Dormant Season	Growing Season
I	less than 0.5	less than 1.4
II	0.5 to 1.11	1.4 to 2.1
III	over 1.1	over 2.1

(From reference 49)

A soil that is dry will produce less runoff than the same soil that is saturated. Most everyone knows of instances in which there has been a "100 -year" rainfall but not a 100-year flood. The moisture content plays a major role in affecting the amount of runoff that will occur.

Table 7.2 lists the RCN values for various land uses and soil types. These values are based on an AMC condition II. If a moisture content other than condition II exists, table 7.4 lists a method of modifying the RCN values to a condition I or III. For most design conditions, a type II condition would be used. However, if the hydrologic analysis is trying to match a past flood, it will be necessary to use the correct moisture condition that was present at the time of the flood event.

UNIT HYDROGRAPH PEAK

A unit hydrograph results when a 24-hour rainfall produces a 1-inch depth of runoff over the given drainage area (reference 4). The unit hydrograph will show the rates at which the runoff will occur from the watershed, for the 1-inch runoff. In theory, the unit hydrograph will be constant for a given duration storm. For runoff amounts other than 1 inch, the ordinates of the hydrograph are multiplied by the runoff amount.

Once the total runoff volume is computed using the procedure above, it is possible to compute the peak flow. The first step is to compute the unit hydrograph peak, $\mathbf{Q_p}$, in cfs/sq.mi.-inches. Figure 7.5 plots $\mathbf{Q_p}$ versus $\mathbf{t_c}$. The value of $\mathbf{Q_p}$ may also be computed using the following equation:

$$Q_p = 270.9\ (t_c)^{-0.81} \qquad (21)$$

This equation is applicable for 24-hour rainfall events, and for drainage areas of less than twenty square miles. For additional information on this procedure, it is suggested that the designer obtain reference 40 from the DEQ, Land and Water Management Division, Hydrologic Studies Unit (telephone # 517-284-5570).

Once Q_p is obtained, it is possible to determine the design peak discharge from the following equation:

$$Q = Q_p \times SRO\ (\text{sq.-mi.-in.}) \qquad (22)$$

Table 7.4 - Curve Numbers for Different AMC Conditions

(Source: Soil Conservation Service, Reference 49)

Curve Number for:			Curve Number for:			Curve Number for:		
AMC Condition II	AMC Condition I	AMC Condition III	AMC Condition II	AMC Condition I	AMC Condition III	AMC Condition II	AMC Condition I	AMC Condition III
100	100	100	76	58	89	52	32	71
99	97	100	75	57	88	51	31	70
98	94	99	74	55	88	50	31	70
97	91	99	73	54	87	49	30	69
96	89	99	72	53	86	48	29	68
95	87	98	71	52	86	47	28	67
94	85	98	70	51	85	46	27	66
93	83	98	69	50	84	45	26	65
92	81	97	68	48	84	44	25	64
91	80	97	67	47	83	43	25	63
90	78	96	66	46	82	42	24	62
89	76	96	65	45	82	41	23	61
88	75	95	64	44	81	40	22	60
87	73	95	63	43	80	39	21	59
86	72	94	62	42	79	38	21	58
85	70	94	61	41	78	37	20	57
84	68	93	60	40	78	36	19	56
83	67	93	59	39	77	35	18	55
82	66	92	58	38	76	34	18	54
81	64	92	57	37	75	33	17	53
80	63	91	56	36	75	32	16	52
79	62	91	55	35	74	31	16	51
78	60	90	54	34	73			
77	59	89	53	33	72			

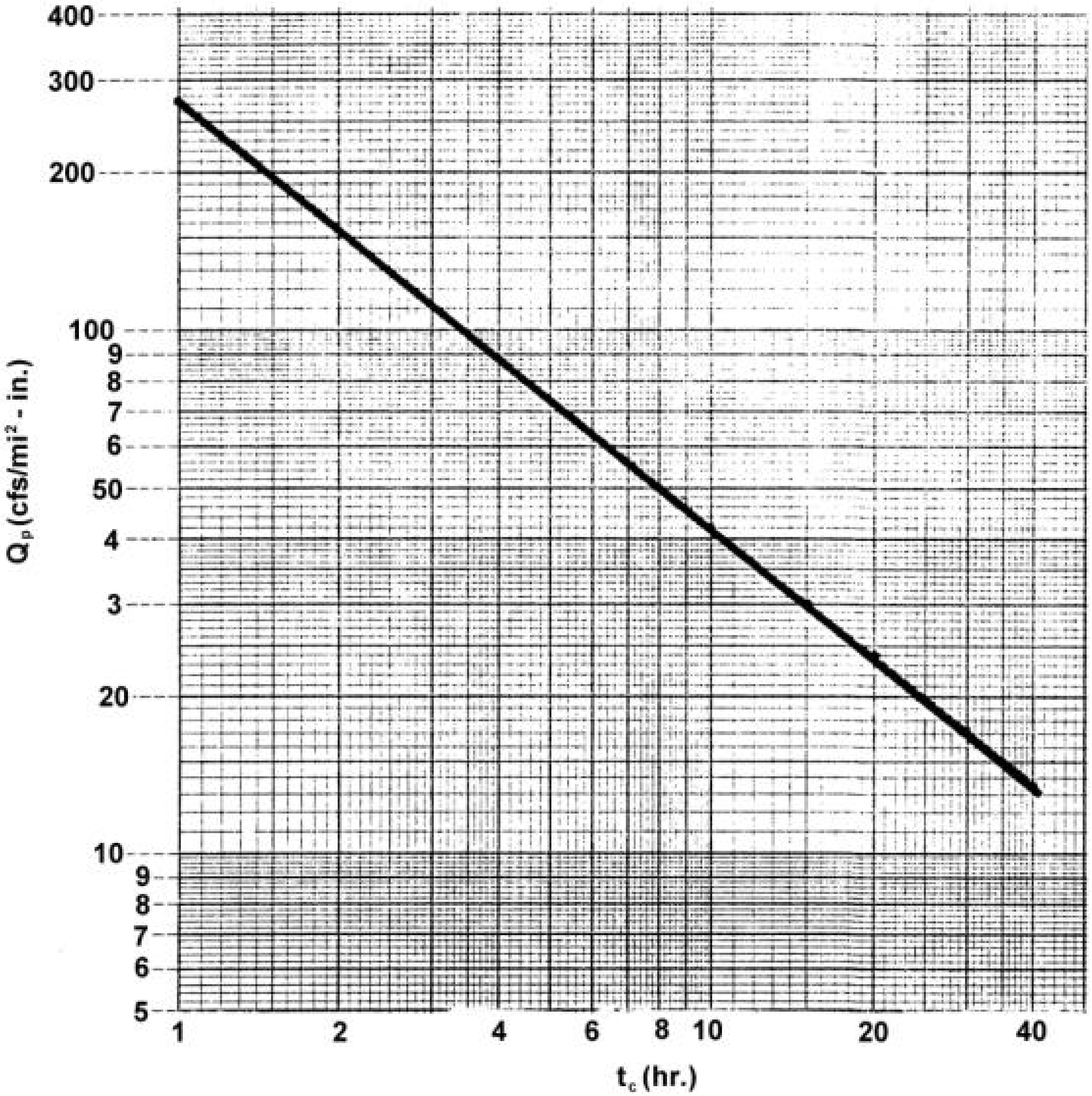


Figure 7.5 - Unit Hydrograph Peak, (Q_p) Versus Time of Concentration (t_c).

Example 7.3: Peak-Discharge Computation

Compute the peak discharge at the given design point, using the following information that had been computed in Example 7.1 and 7.2:

t_c = 1.56 hours (from example 7.1)

SRO = 2.79 sq.-mi.-in. (from example 7.2)

1. From equation 21:

   $Q_p = 270.9\ (1.56)^{-0.81} = 189$ cfs / sq.-mi.-in.

2. From equation 22:

   Q = 189 cfs / sq.mi.-inch x 2.79 sq.-mi.-in. = **527 cfs**

ADJUSTMENTS FOR SWAMPS AND PONDS

The methodology discussed so far has assumed that flow will continue downstream at a uniform rate and will not be stored. In basins where there is ponding or swampy areas, there is potential for temporary storage which will reduce flood peaks.

Table 7.5 provides swamp adjustment factors to be applied to the computed peak flow. The factors are a function of storm frequency, ratio of drainage area to storage area, and location of the storage area.

Example 7.4: Swamp Adjustment

If a watershed has a drainage area that contains 2% of swamp or ponding, which is located near the design point, the adjustment factor for a 100 -year storm would be 0.86 (from table 7.5). Thus, for the example above, the adjusted 100 -year peak flow would be:

Qpeak = 527 cfs x 0.86 = 453 cfs

Figure 7.6 is a form that may be used to determine time of concentration, volume of runoff, and peak flow.

Table 7.5 - Swamp Adjustment Factors

A. -- Ponding and swampy areas are at the design point

Ratio of drainage area to ponding and swampy area	Percentage of ponding and swampy area	Storm frequency (years) 2	5	10	25	50	100
500	00.2	0.92	0.94	0.95	0.96	0.97	0.98
200	00.5	0.86	0.87	0.88	0.90	0.92	0.93
100	01.0	0.80	0.81	0.83	0.85	0.87	0.89
050	02.0	0.74	0.75	0.76	0.79	0.82	0.86
040	02.5	0.69	0.70	0.72	0.75	0.78	0.82
030	03.3	0.64	0.65	0.67	0.71	0.75	0.78
020	05.0	0.59	0.61	0.63	0.67	0.71	0.75
015	06.7	0.57	0.58	0.60	0.64	0.67	0.71
010	10.0	0.53	0.54	0.56	0.60	0.63	0.68
005	20.0	0.48	0.49	0.51	0.55	0.59	0.64

B. -- Ponding and swampy areas are spread throughout the watershed or occur in central parts of the watershed.

Ratio of drainage area to ponding and swampy area	Percentage of ponding and swampy area	Storm frequency (years) 2	5	10	25	50	100
500	00.2	0.94	0.95	0.96	0.97	0.98	0.99
200	00.5	0.88	0.89	0.90	0.91	0.92	0.94
100	01.0	0.83	0.84	0.86	0.87	0.88	0.90
050	02.0	0.78	0.79	0.81	0.83	0.85	0.87
040	02.5	0.73	0.74	0.76	0.78	0.81	0.84
030	03.3	0.69	0.70	0.71	0.74	0.77	0.81
020	05.0	0.65	0.66	0.68	0.72	0.75	0.78
015	06.7	0.62	0.63	0.65	0.69	0.72	0.75
010	10.0	0.58	0.59	0.61	0.65	0.68	0.71
005	20.0	0.53	0.54	0.56	0.60	0.63	0.68
004	25.0	0.50	0.51	0.53	0.57	0.61	0.66

C. -- Ponding and swampy areas are located only in the upper reaches of the watershed.

Ratio of drainage area to ponding and swampy area	Percentage of ponding and swampy area	Storm frequency (years) 2	5	10	25	50	100
500	00.2	0.96	0.97	0.98	0.98	0.99	0.99
200	00.5	0.93	0.94	0.94	0.95	0.96	0.97
100	01.0	0.90	0.91	0.92	0.93	0.94	0.95
050	02.0	0.87	0.88	0.88	0.90	0.91	0.93
040	02.5	0.85	0.85	0.86	0.88	0.89	0.91
030	03.3	0.82	0.83	0.84	0.86	0.88	0.89
020	05.0	0.80	0.81	0.82	0.84	0.86	0.88
015	06.7	0.78	0.79	0.80	0.82	0.84	0.86
010	10.0	0.76	0.77	0.78	0.80	0.82	0.84
005	20.0	0.74	0.75	0.76	0.78	0.80	0.82

BY ____________________________ DATE ___________________ FILE NO. __________

WATERCOURSE ____________________________ COUNTY ______________________

DRAINAGE AREA _____________________ RECURRENCE INTERVAL ______________

FLOW TYPE | LENGTH | ELE (FT) | SLOPE(%) | VEL(fps) | tc(hrs)

Total tc, hrs. = _________________

SOILS **GROUP % sq. mi.**	LAND USE **Type % sq. mi. CN**	Hr. RF = Adj. RF **R.O.**	Hr. RF = Adj. RF **Sq. mi.-in.**	24 Hr. RF = Adj. RF **R.O.**	24 Hr. RF = Adj. RF **Sq. mi.-in.**
A					
B					
C					
D					
	Total sq. mi.-in.				
	Avg. R.O., in.				
	Comp. CN				
	Qp, cfs/sq. mi.-in				
	Q (R.O. x Qp)				
	Adj. factor				
	Q _________________				

Figure 7.6 - SCS Methodology Form

SCS METHODOLOGY TR-55

As noted in the overview, TR-55 was developed primarily for urban/urbanizing watersheds. Rather than try to duplicate all of the information that is contained in the TR-55 manual, it is suggested that the manual be obtained as a reference. Request "Urban Hydrology for Small Watersheds", Technical Release No. 55. from:

National Technical Information Service (NTIS)
U.S. Dept. of Commerce
5285 Port Royal Road
Springport, Virginia 22162
(703) 487-4600

There are some differences between the UD-21 method, discussed above, and TR-55 which include:

1. **RCN adjustment**

If the land use has a different percentage of impervious area or has less than 100% of the impervious area connected to the drainage course, then figures 7.7 & 7.8 may be used to adjust the RCN. These figures were taken directly from the TR-55 manual.

The following equations were used to develop the figures:

For **composite CN** with **connected impervious** areas.

$$CN_c = CN_p + (P_{imp}/100)(98-CN_p) \qquad (23)$$

where: CN_c - composite runoff curve number
CN_p - runoff curve number for the pervious area
P_{imp} - percent imperviousness

Example 7.5: RCN Adjustment

Given: A 1/2-acre residential lot, with B soils. The typical CN value for a 1/2-acre residential lot on B soils is 70. However, this assumes that the lot has 25% impervious area connected to the drainage course. If only 20% of the impervious area is connected, it is necessary to adjust the CN. The CN for the pervious portion of the lot is 61. (B soils, open space, good condition). The composite or adjusted CN is computed as follows:

$$CN_c = 61 + (20/100)(98-61) = \mathbf{68}$$

For **composite CN** with **unconnected impervious areas** and total impervious area less than 30%

$$CN_c = CN_p + (P_{imp}/100)(98-CN_p)(1 - 0.5R) \qquad (24)$$

where: **R** - unconnected impervious area/total impervious area

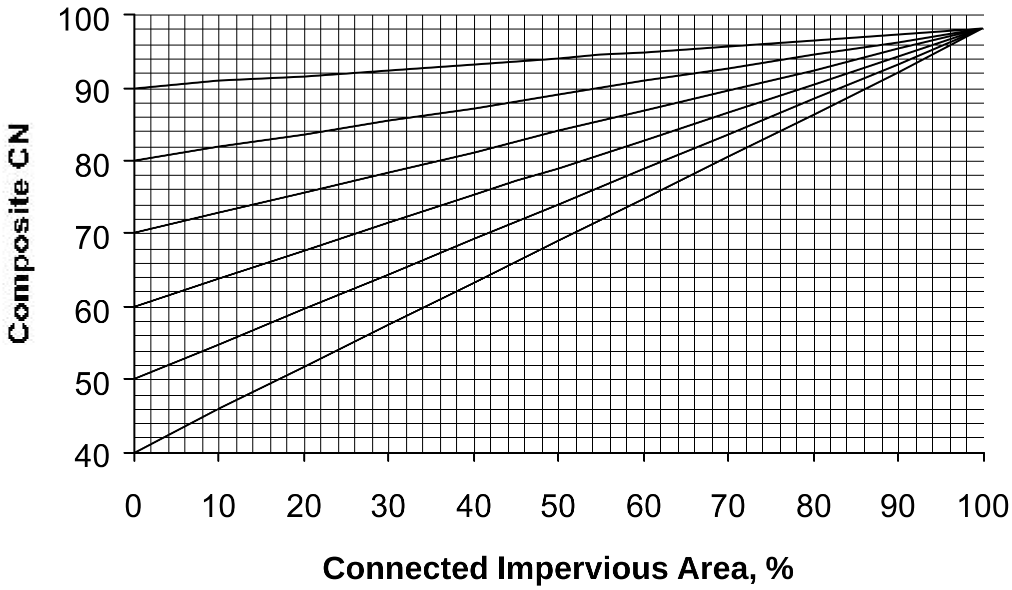


Figure 7.7 - Composite CN with connected impervious area

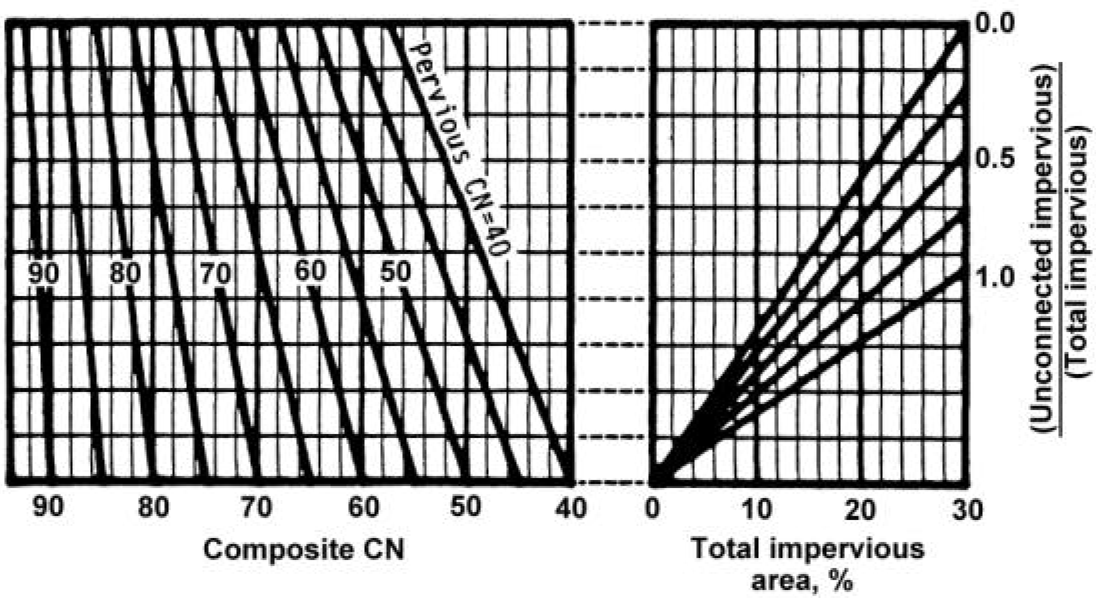


Figure 7.8 - Composite CN with unconnected impervious areas and total impervious area less than 30%.

(Source: Reference 46)

Example 7.6:

Given: A ½-acre residential lot on C soils, with 25% impervious area, of which 30% is connected to the drainage course (or 70% of impervious area is unconnected, which means R = 70/100 = 0.70). The CN value for the pervious portion of the lot is 74 (open space on C soils, good condition).

CN_c = 74 + (25/100)(98-74)(1 - 0.5(0.70)) = **78**

The average CN value for a 1/2-acre residential lot on C soil is 80. This example computed a CN of 78 when only 30% of the impervious area is connected to the drainage course.

It is interesting to note from these two examples how the curve number (and therefore the runoff) can be reduced by an on-site stormwater management technique such as **not** connecting the impervious areas with the drainage course. In other words, directing downspouts onto lawns and directing runoff from parking lots across grassed areas.

2. Time of Concentration

The method for computing the time of concentration in TR- 55 is somewhat different than was shown for the UD-21 method. The UD-21 method uses three simple formulas to determine velocity for channel, waterway and sheet flow. The UD-21 method does not have a good method for including the impact on the time of concentration as channel improvements (such as a drain improvements) are made.

The following equation taken from TR-55 provides an estimate of travel time for sheet flow:

$$T_t = \frac{.007(nL)^{0.8}}{(P_2)^{0.5}\, S^{0.4}} \qquad (25)$$

where:
- **T_t** - travel time, hours
- **n** - Manning's roughness coefficient
- **L** - flow length, feet,
- **P_2** - 2-year, 24-hour rainfall, inches
- **s** - slope of channel, ft/ft

Note: If flow length exceeds 300 feet, use figure 7.9.

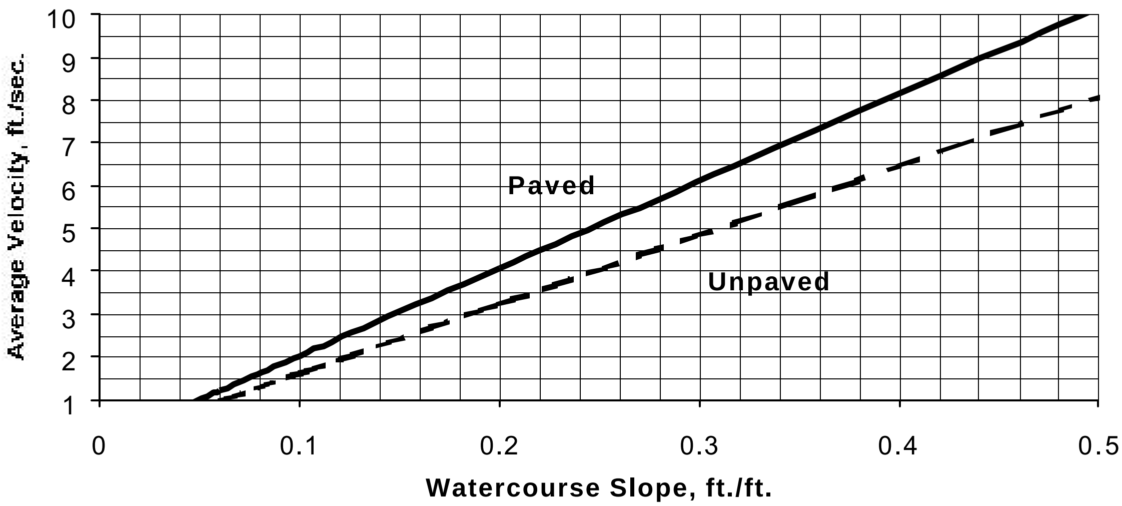


Figure 7.9 - Average velocities for estimating travel time for shallow concentrated flow.
(Source: Reference 46)

Figure 7.9 is from the TR-55 manual and provides an estimated velocity as a function of watercourse slope for shallow concentrated flow. Equations may also be used to determine the velocity of flow:

Unpaved watercourse: $V = 16.1345\ (s)^{0.5}$ (26)

Paved watercourse: $V = 20.3282\ (s)^{0.5}$ (27)

where: **V** - velocity in feet/sec
s - slope of watercourse in ft/ft

For open channels, the velocity of flow for bank-full conditions can be estimated using Manning's equation:

$$V = \frac{1.49\ r^{2/3} s^{1/2}}{n} \qquad (28)$$

where: **V** - velocity, ft/sec
r - hydraulic radius (area/wetted perimeter; wetted perimeter is the wetted surface of the channel)
s - hydraulic gradient (slope of channel) feet/feet
n – Manning's roughness coefficient

Example 7.7: Time of Concentration

Compute the time of concentration given the following information:

– Small tributary length of 4000 feet of which, 3000 feet is at a slope of 0.2%, & 1000 feet is at a slope of 1%. Use a hydraulic radius of 1.5 & a Manning's n value of 0.06.

– The waterway length is 800 feet, unpaved at a slope of 2%.

– The sheet flow length is 250 feet at a slope of 0.5%. Use P_2= 2.5 inches, and n = .15.

a) Compute velocity for the small tributary using equation (28):

$$V_1 = \frac{1.49\,(1.5)^{2/3}\,(0.002)^{1/2}}{0.06} = 1.46 \text{ ft/sec}$$

$$V_2 = \frac{1.49\,(1.5)^{2/3}\,(0.01)^{1/2}}{0.06} = 3.25 \text{ ft/sec}$$

b) Compute velocity for **shallow** concentrated flow using (26):

$$V = 16.1345\,(s)^{0.5} = 16.1345\,(0.02)^{0.5} = 2.28 \text{ ft/sec}$$

c) Compute **travel time** for **sheet** flow using equation (25):

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}\,S^{0.4}} = \frac{.007\,(.15 \times 250)^{0.8}}{(2.5)^{0.5}\,(.005)^{0.4}} = 0.67 \text{ hrs.}$$

d) Determine the total time of concentration

Flow Type	Length	Slope (%)	V(fps)	tc (hours)
Small Trib	3000	0.2	1.46	0.57
Small Trib	1000	1.0	3.25	0.09
Waterway	800	2.0	2.28	0.10
Sheet	250	0.5	--	0.67

$*t_c$ = length / V x 3600 = (3000/1.46 x 3600) = .57 hrs

Total t_c = 0.57 + 0.09 + 0.10 + 0.67 = **1.43 hrs** .

3. **Unit Peak**

Once the time of concentration is determined, it is possible to compute the peak discharge using figure 7.10. From the figure, it can be seen that the peak discharge is a function of t_c and a ratio of Ia/P.

Ia - is the initial abstraction, or all the losses before runoff begins. (Such as infiltration, interception, and evaporation)
P - is the rainfall in inches

Table 7.6 gives **Ia** values for different runoff curve numbers. Once Ia is known, it is very straight forward to compute Ia/P, and then using figure 7.10, to compute the unit peak discharge:

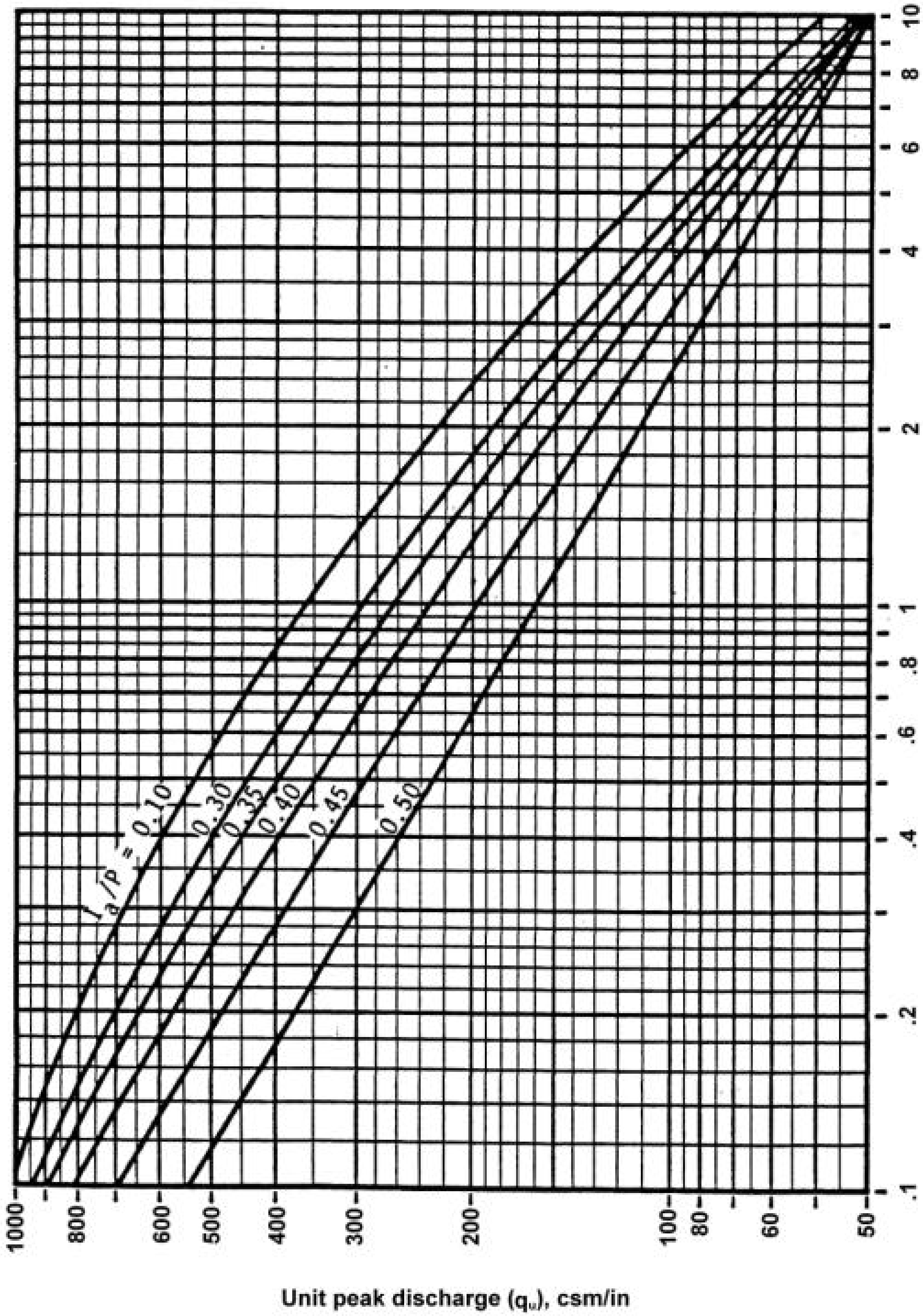


Figure 7.10 – Unit Peak Discharge (q_u) SCS Type II Rainfall Distribution

(Source: reference 46)

Figure 7.10 - Unit Peak Discharge

Table 7.6 - Ia values for runoff curve numbers

Curve Number	Ia (in.)	Curve Number	Ia (in.)	Curve Number	Ia (in.)	Curve Number	Ia (in.)
40	3.000	55	1.636	70	0.857	85	0.353
41	2.878	56	1.571	71	0.817	86	0.326
42	2.762	57	1.509	72	0.778	87	0.299
43	2.651	58	1.448	73	0.740	88	0.273
44	2.545	59	1.390	74	0.703	89	0.247
45	2.444	60	1.333	75	0.667	90	0.222
46	2.348	61	1.279	76	0.632	91	0.198
47	2.255	62	1.226	77	0.597	92	0.174
48	2.167	63	1.175	78	0.564	93	0.151
49	2.082	64	1.125	79	0.532	94	0.128
50	2.000	65	1.077	80	0.500	95	0.105
51	1.922	66	1.030	81	0.469	96	0.083
52	1.846	67	0.985	82	0.439	97	0.062
53	1.774	68	0.941	83	0.410	98	0.041
54	1.704	69	0.899	84	0.381		

Source: Reference 46, TR-55, Second Edition, June 1986

Example 7.7: The basin has a RCN of 75, a precipitation of 5.1 inches, Type II rainfall distribution, and 2.79 sq.mi-inches of runoff. The t_c is 1.43 hours, compute the unit peak discharge.

For a RCN = 75, from Table 7.6, the initial abstraction (Ia) is **.667 inches.**

Ia/P = 0.667/5.1 = 0.13

From figure 7.10, interpolating between Ia/P= 0.1 and 0.3, to Ia/P = 0.13, the unit peak discharge is **280** cfs/square mile-inch.

Just like the UD-21 method, the peak flow can be determined by using equation 22:

Q = Qp x surface runoff
= 280 cfs/sq.mi.-inch x 2.79 sq.mi.-inch
= **780 cfs**

4. Swamp and Pond Adjustment Factor

As in the UD-21 methodology, it is necessary to adjust the peak flow if there is ponding or swampy areas within the drainage basin. Table 7.5 that was used in the UD-21 method is also applicable to TR-55.

A sample work sheet for using the TR-55 graphical peak method is given in Figure 7.11.

Worksheet - Graphical Peak Method

Project ________________ By ____________________ Date _______

Location ______________ Checked _______________ Date _______

1. Pertinent Data:

Drainage area	A =	___________	$mi.^2$
Runoff Curve Number	RCN =	___________	
Time of Concentration	Tc =	___________	hr.
Pond and Swamp Areas	=	___________	percent

2. Frequency	yr.	___________	___________
3. Rainfall, P (24-hour), Appendix Cin.	in.	___________	___________
4. Initial Abstraction, Ia, Table 7.6	in.	___________	___________
5. Compute Ia/P			
6. Unit peak discharge, qu, Figure 7.12	csm/in	___________	___________
7. Runoff, Ro	in.	___________	___________
8. Pond and swamp adjustment factor, FP (Use table 7.5)		___________	___________
9. **Peak discharge**, qp (Where qp = qu A Ro Fp)	cfs	___________	___________

Figure 7.11 - Graphical Peak Discharge Method Worksheet

(Source: Reference 46, TR-55, Second Edition, June 1986)

5. Development of Hydrographs

Once the basin characteristics, runoff and peak flow have been determined, it is possible to develop a flood hydrograph for the basin. TR-55 contains a straightforward method of developing a portion of the hydrograph for a single sub-basin. For multiple sub-basins, it will be necessary to develop hydrographs for each of the sub- basins, route the hydrographs, and combine hydrographs. Thus, if more than one sub-basin is to be considered, it is suggested that references 44-, 46, 47 or 49 be used for guidance.

Figures 7.12 a-f contain tabulations of unit discharges as a function of time of concentration and travel time. For this guidebook, it will be assumed that only one sub-basin is being considered. The hydrograph that will be developed will be at the design point, as a result, the travel will be equal to 0. The travel time is considered when more than one sub-basin is being considered, and it is necessary to route the hydrographs to the design point. In addition, if the time of concentration exceeds 2 hours, one of the above references would have to be used.

Figure 7.12a - Tabular Hydrograph Unit Discharge (cfs/sq.mi-inch)
Type II Rainfall Distribution

TRAVEL TIME (HRs.)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	24.0
				IA/P = 0.10											TC = 0.5 HR.												IA/P = 0.10					
0.00	17	23	32	57	94	170	308	467	529	507	402	297	226	140	96	74	61	53	47	41	36	32	29	26	23	21	20	19	16	14	12	0
0.10	16	22	30	51	80	140	252	395	484	499	434	343	265	162	108	80	65	55	49	42	36	33	29	26	23	21	20	19	16	14	12	0
0.20	14	19	25	38	47	69	116	207	332	434	477	449	378	238	149	101	77	62	53	45	39	34	30	27	24	22	20	19	17	14	12	0
0.30	13	18	24	35	43	60	97	170	378	382	446	448	401	270	171	114	83	66	56	46	40	34	31	27	24	22	20	19	17	15	12	0
0.40	12	15	21	29	33	40	53	83	141	233	332	408	434	361	243	157	107	79	64	51	43	36	32	28	25	22	21	20	17	15	12	0
0.50	11	15	20	28	31	37	48	71	118	194	286	367	412	378	271	178	119	86	68	53	44	37	32	29	25	23	21	20	17	15	12	0
0.75	9	11	14	19	21	24	27	31	37	49	74	118	182	319	374	328	244	169	117	76	56	43	35	31	28	25	22	21	18	16	12	1
1.00	7	9	12	16	17	19	21	24	27	32	40	55	83	188	309	359	322	245	172	102	68	49	38	32	29	26	23	21	19	16	12	1
1.50	5	7	8	11	12	13	14	15	17	19	21	23	27	43	89	175	269	322	309	225	140	77	49	38	32	29	25	23	20	17	13	5
2.00	3	4	6	7	8	8	9	10	10	11	12	14	15	18	23	35	65	123	202	297	280	181	88	52	39	33	29	26	21	19	14	10
2.50	2	3	4	5	5	6	6	7	8	8	9	9	10	12	15	18	24	36	66	150	244	278	171	87	52	39	33	29	23	20	15	11
3.00	1	1	2	3	3	4	4	4	5	5	6	6	7	8	9	11	13	16	20	37	86	198	263	182	96	56	40	33	26	21	16	11
				IA/P = 0.30											TC = 0.5 HR.												IA/P = 0.30					
0.00	0	0	0	1	9	53	157	314	433	439	379	299	237	159	115	95	81	71	65	56	50	46	42	38	34	31	30	28	25	22	19	0
0.10	0	0	0	0	1	6	37	117	248	372	416	391	330	218	150	113	92	79	70	60	53	47	43	39	35	32	30	29	26	22	19	0
0.20	0	0	0	0	1	4	26	87	194	313	382	388	349	244	167	122	97	82	72	62	54	48	43	39	35	32	30	29	26	22	19	0
0.30	0	0	0	0	0	0	3	19	64	151	259	341	372	316	223	156	117	94	80	67	58	50	45	41	36	33	31	29	26	23	19	0
0.40	0	0	0	0	0	0	2	13	47	116	211	298	354	328	245	172	127	100	83	69	59	51	45	41	37	33	31	29	26	23	19	0
0.50	0	0	0	0	0	0	0	1	9	34	89	170	255	341	303	225	161	120	96	76	64	54	47	42	38	34	31	30	27	24	19	0
0.75	0	0	0	0	0	0	0	1	4	14	41	89	152	270	305	268	207	155	118	87	70	57	48	44	39	35	32	30	27	24	19	0
1.00	0	0	0	0	0	0	0	0	0	0	2	7	22	98	212	295	285	237	181	120	88	67	53	46	42	38	34	31	28	25	19	2
1.50	0	0	0	0	0	0	0	0	0	0	0	0	0	5	30	95	183	249	265	217	152	96	66	53	46	41	37	34	30	26	20	8
2.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	18	59	125	221	245	182	105	69	54	47	42	38	32	28	22	16
2.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	21	84	174	230	172	103	69	54	46	42	34	30	23	18
3.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	13	56	157	217	163	101	68	53	46	37	31	25	18
				IA/P = 0.50											TC = 0.5 HR.												IA/P = 0.50					
0.00	0	0	0	0	0	2	26	89	170	217	229	200	179	144	119	104	93	85	78	70	64	59	55	51	46	43	41	40	36	32	28	0
0.10	0	0	0	0	0	0	1	18	65	135	190	216	205	170	137	115	101	91	83	74	67	61	56	52	47	44	42	40	36	32	28	0
0.20	0	0	0	0	0	0	1	12	47	106	162	198	203	178	145	121	105	94	85	76	68	61	57	52	48	44	42	40	37	32	28	0
0.30	0	0	0	0	0	0	0	1	8	34	82	135	177	194	168	139	117	102	92	80	71	63	58	54	49	45	43	41	37	33	28	0
0.40	0	0	0	0	0	0	0	0	6	25	63	111	155	189	174	146	122	106	94	82	73	64	58	54	50	45	43	41	37	33	28	0
0.50	0	0	0	0	0	0	0	0	4	18	48	90	133	184	177	152	128	110	97	84	74	65	59	55	50	45	43	41	38	33	28	0
0.75	0	0	0	0	0	0	0	0	1	7	22	47	80	142	169	164	144	124	108	91	79	68	61	56	51	47	44	42	38	34	28	0
1.00	0	0	0	0	0	0	0	0	0	0	1	3	11	51	112	155	166	154	134	109	91	76	65	59	54	49	45	43	39	35	28	2
1.50	0	0	0	0	0	0	0	0	0	0	0	0	0	2	16	50	97	136	154	145	121	89	75	64	58	54	49	45	41	37	29	10
2.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	18	47	86	134	146	125	94	75	64	58	53	49	42	39	31	21
2.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	11	44	95	140	127	97	77	65	58	54	45	41	33	26
3.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	29	86	135	122	95	76	65	58	49	43	35	27

Figure 7-12b - Tabular Hydrograph Unit Discharge (cfs/sq.mi-inch)
Type II Rainfall Distribution

TRAVEL TIME (HRs.)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	24.0
				IA/P = 0.10											TC = 0.75 HR.												IA/P = 0.10					
0.00	13	18	24	36	46	68	115	194	294	380	424	410	369	252	172	123	93	74	61	49	41	35	31	27	24	22	20	19	17	15	12	0
0.10	13	17	23	34	42	59	97	162	250	337	395	405	381	279	191	135	100	79	65	51	42	36	31	28	25	22	21	19	17	15	12	0
0.20	11	15	20	28	32	39	52	82	135	211	295	362	391	351	255	178	127	95	75	57	46	38	32	29	26	23	21	20	17	15	12	0
0.30	11	14	19	26	30	36	47	70	113	179	256	326	379	360	277	196	140	103	80	60	48	38	33	29	26	23	21	20	18	15	12	0
0.40	10	12	16	22	25	28	33	42	61	96	151	221	291	367	336	255	182	131	98	69	54	42	34	30	27	24	22	20	18	16	12	0
0.50	9	12	16	21	24	27	31	39	53	82	128	190	258	358	343	274	200	144	106	74	56	43	35	30	27	24	22	20	18	16	12	0
0.75	8	10	13	17	18	21	23	26	31	39	55	82	122	230	314	329	281	217	161	104	72	51	38	33	29	26	23	21	19	16	12	1
1.00	6	8	10	13	14	15	17	19	21	23	27	32	42	89	177	272	319	303	249	163	105	66	45	36	31	27	24	22	19	17	13	3
1.50	4	6	7	9	10	10	11	12	14	15	16	18	20	27	46	90	163	241	295	275	204	119	66	45	35	31	27	24	20	18	13	7
2.00	3	4	5	6	7	7	8	9	9	10	11	12	13	16	20	28	48	89	151	245	274	213	115	65	44	35	30	27	22	19	14	10
2.50	1	2	3	4	4	5	5	6	6	7	7	8	5	10	12	14	17	24	37	86	170	260	219	127	71	47	36	31	24	20	16	11
3.00	1	1	2	3	3	3	4	4	4	5	5	5	6	7	8	10	11	14	17	30	64	157	247	205	122	70	46	36	27	22	17	12
				IA/P = 0.30											TC = 0.75 HR.												IA/P = 0.30					
0.00	0	0	0	0	1	6	30	86	174	266	326	348	328	246	181	138	110	92	79	66	57	49	44	40	36	32	31	29	26	23	19	0
0.10	0	0	0	0	0	1	4	22	65	137	223	292	329	303	228	170	131	106	89	73	61	52	46	41	37	33	31	29	26	23	19	0
0.20	0	0	0	0	0	0	3	15	48	108	185	256	305	321	245	184	141	112	93	75	63	53	46	42	37	34	31	30	27	23	19	0
0.30	0	0	0	0	0	0	2	11	36	84	151	221	277	308	260	199	152	120	98	78	65	54	47	42	38	34	31	30	27	23	19	0
0.40	0	0	0	0	0	0	0	1	8	27	65	122	188	286	301	243	187	144	114	87	71	57	48	43	39	35	32	30	27	24	19	1
0.50	0	0	0	0	0	0	0	1	6	20	50	98	158	263	292	254	200	155	122	91	74	59	49	44	40	35	32	30	27	24	19	1
0.75	0	0	0	0	0	0	0	0	0	2	8	23	51	140	231	269	253	211	167	119	90	68	53	46	42	37	34	31	28	25	19	2
1.00	0	0	0	0	0	0	0	0	0	0	0	1	4	29	96	156	249	261	231	169	120	84	61	50	44	40	36	33	29	26	20	5
1.50	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	34	91	163	220	241	197	131	83	61	50	44	40	35	31	27	21	12
2.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	36	85	174	226	200	127	82	60	49	44	39	32	29	22	17
2.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	37	105	196	214	135	87	62	51	44	36	31	24	18
3.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	24	96	205	189	130	85	62	50	39	32	26	18
				IA/P = 0.50											TC = 0.75 HR.												IA/P = 0.50					
0.00	0	0	0	0	0	0	2	16	45	92	137	166	185	170	146	125	110	98	89	79	70	63	58	53	48	44	42	41	37	33	28	0
0.10	0	0	0	0	0	0	0	1	11	34	73	115	149	180	163	141	122	107	96	84	74	65	59	54	50	45	43	41	38	33	28	0
0.20	0	0	0	0	0	0	0	1	8	25	57	96	131	173	166	146	126	111	99	86	76	66	59	55	50	46	43	41	38	34	28	0
0.30	0	0	0	0	0	0	0	0	1	5	18	44	79	143	170	160	141	122	108	92	81	69	61	56	52	47	44	42	38	34	28	1
0.40	0	0	0	0	0	0	0	0	0	4	14	34	64	127	166	162	145	127	111	95	82	70	62	57	52	47	44	42	38	34	28	1
0.50	0	0	0	0	0	0	0	0	0	0	2	10	26	82	138	162	157	140	123	103	88	75	64	58	53	49	45	43	39	35	28	2
0.75	0	0	0	0	0	0	0	0	0	0	1	4	12	47	98	139	154	148	135	113	96	80	67	60	55	50	46	43	39	36	29	3
1.00	0	0	0	0	0	0	0	0	0	0	0	0	0	6	30	73	119	146	151	134	113	91	74	63	58	53	48	45	41	37	29	7
1.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	30	66	105	143	143	117	90	73	63	57	52	48	42	39	30	18
2.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	30	77	121	137	114	88	72	63	57	52	44	40	32	25
2.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	19	55	111	132	111	87	71	62	56	47	42	34	27
3.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	12	51	112	128	108	86	71	62	51	44	36	27

Figure 7-12c - Tabular Hydrograph Unit Discharge (cfs/sq.mi-inch)
Type II Rainfall Distribution

IA/P = 0.10 TC = 1.0 HR. IA/P = 0.10

TRAVEL TIME (HRs.)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	24.0
0.00	11	15	20	29	35	47	72	112	168	231	289	329	357	313	239	175	133	103	83	63	50	40	33	29	26	23	21	20	17	15	12	0
0.10	10	13	17	24	27	33	42	62	95	144	202	260	306	340	293	222	165	126	98	72	56	43	35	30	27	24	22	20	18	15	12	0
0.20	10	13	17	23	26	30	38	54	82	123	176	232	281	332	303	238	179	136	105	76	59	45	35	30	27	24	22	20	18	16	12	1
0.30	9	12	16	22	24	28	35	48	70	105	152	205	256	323	310	254	193	146	113	81	61	46	36	31	27	24	22	20	18	16	12	1
0.40	8	11	14	19	21	23	27	32	42	61	91	132	181	276	318	294	237	181	138	95	70	51	39	32	28	25	23	21	18	16	12	1
0.50	8	10	13	18	20	22	25	30	38	53	78	114	159	253	311	300	251	195	149	102	74	53	40	33	29	25	23	21	18	16	12	1
0.75	7	8	11	14	16	17	19	21	25	30	38	53	76	146	228	284	293	256	208	143	99	66	46	36	31	27	24	22	19	17	13	2
1.00	5	7	8	11	12	13	14	16	17	19	22	25	31	57	111	188	256	286	272	208	144	90	56	41	33	29	26	23	20	17	13	4
1.50	4	5	6	8	8	9	10	11	12	13	14	15	17	22	33	59	107	171	231	268	235	157	88	56	41	33	29	25	21	18	14	8
2.00	2	3	4	5	5	6	6	7	7	8	9	9	10	12	15	19	27	44	78	157	231	252	167	96	59	42	34	29	23	20	15	11
2.50	1	2	2	3	4	4	4	5	5	6	6	7	7	8	10	12	15	19	27	58	120	214	241	159	94	59	42	34	26	21	16	11
3.00	0	1	1	2	2	3	3	3	4	4	4	5	5	6	7	8	10	12	14	22	44	113	214	231	152	91	58	42	29	23	17	12

IA/P = 0.30 TC = 1.0 HR. IA/P = 0.30

TRAVEL TIME (HRs.)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	24.0
0.00	0	0	0	0	1	4	16	42	83	137	195	243	271	292	227	178	143	117	98	79	66	55	47	42	38	34	31	30	27	23	19	0
0.10	0	0	0	0	0	0	3	12	32	66	113	168	218	279	260	213	169	136	113	88	72	59	49	43	39	35	32	30	27	24	19	1
0.20	0	0	0	0	0	0	2	9	24	52	93	143	193	271	271	225	180	145	119	92	75	60	50	44	39	35	32	30	27	24	19	1
0.30	0	0	0	0	0	0	1	6	18	41	75	120	169	246	264	234	191	153	125	96	78	62	51	44	40	36	33	31	27	24	19	1
0.40	0	0	0	0	0	0	0	1	4	14	32	61	100	190	251	259	222	181	146	109	86	67	53	46	41	37	33	31	28	25	19	2
0.50	0	0	0	0	0	0	0	1	3	10	24	49	83	168	237	254	230	191	155	115	90	69	54	47	42	37	34	31	28	25	19	2
0.75	0	0	0	0	0	0	0	0	0	1	4	12	25	76	150	213	239	228	198	149	112	82	61	50	44	39	35	32	29	26	20	4
1.00	0	0	0	0	0	0	0	0	0	0	0	1	2	15	51	113	182	226	234	197	150	104	72	56	47	42	38	34	30	27	20	7
1.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	18	51	104	162	220	210	158	102	71	56	47	42	37	31	28	22	13
2.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	20	49	121	187	209	152	100	70	55	47	41	34	29	23	17
2.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	32	87	171	199	146	98	69	54	46	37	31	24	18
3.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	13	62	158	192	151	103	73	56	41	34	26	18

IA/P = 0.50 TC = 1.0 HR. IA/P = 0.50

TRAVEL TIME (HRs.)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	24.0
0.00	0	0	0	0	0	0	1	7	21	42	71	101	126	160	154	138	123	110	100	87	77	67	60	55	50	46	43	41	38	34	28	1
0.10	0	0	0	0	0	0	0	1	5	15	33	58	87	134	156	149	134	120	108	93	82	71	62	57	52	47	44	42	38	34	28	1
0.20	0	0	0	0	0	0	0	1	4	12	26	48	74	123	153	153	137	123	111	95	84	72	63	57	52	47	44	42	38	34	28	1
0.30	0	0	0	0	0	0	0	0	3	9	20	38	62	111	143	150	140	127	114	98	86	73	63	58	53	48	45	42	39	35	28	1
0.40	0	0	0	0	0	0	0	0	0	2	6	16	31	75	120	145	148	137	123	106	91	77	66	59	54	49	45	43	39	35	29	2
0.50	0	0	0	0	0	0	0	0	0	1	5	12	25	64	109	139	146	139	127	108	94	79	67	60	55	50	46	43	39	36	29	3
0.75	0	0	0	0	0	0	0	0	0	0	2	5	12	39	78	115	136	140	134	117	101	84	70	62	56	51	47	44	40	36	29	4
1.00	0	0	0	0	0	0	0	0	0	0	0	0	1	7	26	59	96	125	139	133	117	97	78	66	59	54	49	46	41	37	29	8
1.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	26	54	86	123	133	119	95	77	66	59	54	49	43	39	31	17
2.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	25	64	104	129	116	93	76	65	58	53	45	41	33	24
2.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	10	34	84	125	117	96	78	66	59	49	43	35	27
3.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	32	89	122	114	94	77	66	53	45	37	27

Figure 7-12d - Tabular Hydrograph Unit Discharge (cfs/sq.mi-inch)
Type II Rainfall Distribution

TRAVEL TIME (HRs.)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	24.0
				IA/P = 0.10											TC = 1.25 HR.												IA/P = 0.10					
0.00	10	13	18	25	29	38	54	81	118	163	213	256	284	311	266	212	163	129	104	78	61	47	37	31	27	24	22	20	18	16	12	1
0.10	10	13	17	23	28	34	47	69	102	143	189	234	267	297	274	226	175	138	111	82	64	48	38	31	27	24	22	20	18	16	12	1
0.20	9	11	15	20	22	26	31	42	60	88	124	168	212	280	292	261	212	166	131	95	72	53	40	33	28	25	23	21	18	16	12	1
0.30	8	11	14	19	21	24	29	38	53	76	108	148	190	263	288	268	224	177	140	101	76	55	41	34	29	25	23	21	18	16	12	2
0.40	8	10	13	18	20	23	27	34	46	66	94	130	170	245	282	273	235	188	149	107	80	58	42	34	29	26	23	21	19	16	12	2
0.50	7	9	12	16	17	19	22	25	31	41	58	82	114	190	256	279	262	222	178	127	93	65	46	36	31	27	24	22	19	17	13	2
0.75	6	8	10	14	15	17	19	21	25	31	41	56	78	139	207	254	265	245	208	152	110	75	51	39	32	28	25	22	19	17	13	3
1.00	5	6	8	10	11	13	14	15	17	19	22	26	33	60	109	173	230	261	255	208	153	100	64	46	36	30	26	24	20	18	13	5
1.50	3	4	5	7	7	8	9	9	10	11	12	13	15	19	27	45	79	130	186	247	239	180	108	68	48	37	31	27	22	19	14	10
2.00	2	3	4	5	6	6	7	7	8	8	9	10	11	13	16	22	35	59	98	171	236	236	156	95	62	44	35	30	23	20	15	11
2.50	1	2	2	3	4	4	4	5	5	5	6	6	7	8	10	12	14	19	28	58	114	197	226	163	102	65	46	36	26	21	16	11
3.00	0	1	1	2	2	2	2	3	3	3	4	4	4	5	6	7	9	10	13	19	35	88	184	218	169	109	70	49	31	24	18	12
				IA/P = 0.30											TC = 1.25 HR.												IA/P = 0.30					
0.00	0	0	0	0	0	2	9	25	50	86	130	174	208	253	235	201	164	136	115	92	76	61	51	44	39	35	32	30	27	24	19	1
0.10	0	0	0	0	0	0	1	6	19	40	71	110	153	217	247	227	191	157	131	103	84	66	53	46	41	36	33	31	28	24	19	2
0.20	0	0	0	0	0	0	1	4	14	31	58	93	133	202	239	231	199	165	138	108	87	68	55	47	41	37	33	31	28	25	19	2
0.30	0	0	0	0	0	0	0	1	3	10	24	46	77	152	210	236	222	190	158	122	97	74	58	49	43	38	34	32	28	25	20	3
0.40	0	0	0	0	0	0	0	0	2	8	19	37	64	134	196	232	225	198	166	127	101	77	59	50	43	38	35	32	28	25	20	3
0.50	0	0	0	0	0	0	0	0	0	2	6	14	30	82	151	206	228	217	189	146	113	85	64	52	45	40	36	33	29	26	20	5
0.75	0	0	0	0	0	0	0	0	0	1	2	7	15	49	105	164	205	218	205	166	129	95	69	55	47	41	37	33	29	26	20	6
1.00	0	0	0	0	0	0	0	0	0	0	0	0	1	9	32	77	134	185	214	203	166	120	83	63	52	45	39	35	30	27	21	10
1.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	33	72	121	184	203	171	117	82	62	51	44	39	32	29	22	15
2.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	21	67	132	194	174	123	86	64	52	45	35	31	24	18
2.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	13	46	121	187	166	119	84	63	52	39	32	25	18
3.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	44	129	180	160	116	83	63	44	35	27	18
				IA/P = 0.50											TC = 1.25 HR.												IA/P = 0.50					
0.00	0	0	0	0	0	0	1	5	13	26	44	68	91	125	142	142	128	117	107	94	83	72	63	57	52	47	44	42	38	34	28	2
0.10	0	0	0	0	0	0	0	0	3	10	20	36	57	100	129	140	136	125	114	100	88	76	65	59	54	49	45	43	39	35	29	3
0.20	0	0	0	0	0	0	0	0	2	7	16	30	48	90	122	139	139	127	117	102	90	77	66	60	54	49	45	43	39	35	29	3
0.30	0	0	0	0	0	0	0	0	0	2	5	12	24	59	98	126	137	134	125	109	96	82	69	61	56	51	46	44	40	36	29	4
0.40	0	0	0	0	0	0	0	0	0	1	4	10	19	51	89	119	134	136	127	112	98	83	70	62	56	51	47	44	40	36	29	5
0.50	0	0	0	0	0	0	0	0	0	1	3	7	15	43	79	112	131	135	129	114	100	85	71	63	57	52	47	44	40	36	29	6
0.75	0	0	0	0	0	0	0	0	0	0	0	1	3	15	39	71	102	123	130	125	112	94	78	67	60	54	49	46	41	37	29	9
1.00	0	0	0	0	0	0	0	0	0	0	0	0	1	4	17	40	71	101	121	129	121	103	84	71	62	56	51	47	42	38	30	13
1.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	26	51	92	119	125	105	86	72	63	57	52	44	40	32	23
2.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	11	35	72	112	122	103	85	71	63	56	47	42	34	26
2.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	24	66	111	119	101	83	71	62	51	44	36	27
3.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	27	71	110	116	99	82	70	55	46	37	27

Figure 7.12 e - Tabular Hydrograph Unit Discharge (cfs/sq.mi-inch)
Type II Rainfall Distribution

TRAVEL TIME (HRs.)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	24.0
				IA/P = 0.10											TC = 1.5 HR.												IA/P = 0.10					
0.00	9	11	15	21	25	31	41	58	82	112	147	184	216	255	275	236	198	159	129	98	76	57	43	35	30	25	23	21	18	16	12	1
0.10	8	10	13	18	20	23	28	37	51	72	98	131	166	226	265	254	226	187	151	113	86	63	46	37	31	26	23	21	19	16	13	2
0.20	8	10	13	17	19	22	26	33	45	63	87	116	149	212	259	259	233	197	160	119	90	66	48	38	32	27	24	22	19	16	13	2
0.30	7	9	12	16	18	21	24	30	40	55	76	103	134	197	244	255	238	206	169	125	95	68	49	38	32	27	24	22	19	17	13	2
0.40	7	8	11	14	15	17	19	23	28	36	49	67	91	151	208	247	252	230	196	146	109	77	54	41	34	29	25	22	19	17	13	3
0.50	6	8	10	13	15	16	18	21	26	33	43	59	80	136	194	238	249	235	204	154	115	81	56	42	34	29	25	23	20	17	13	3
0.75	5	7	8	11	12	13	14	16	18	21	25	32	42	76	125	179	222	240	233	193	148	102	67	48	38	32	27	24	20	18	13	5
1.00	4	5	7	8	9	10	11	12	13	14	16	18	22	34	59	101	152	201	236	230	193	135	86	59	44	35	30	26	21	18	14	7
1.50	3	4	5	6	6	7	8	8	9	10	11	12	13	16	22	34	58	95	141	203	226	197	131	84	58	43	35	29	23	20	15	10
2.00	1	2	3	4	4	5	5	6	6	7	7	8	9	10	12	16	22	34	56	110	172	218	187	126	82	57	43	34	25	21	16	11
2.50	1	1	2	2	3	3	3	4	4	4	5	5	6	7	8	9	11	14	18	34	69	141	210	190	133	57	60	44	30	23	17	12
3.00	0	0	1	1	2	2	2	2	3	3	3	3	4	5	5	6	8	9	11	16	27	66	149	204	181	128	85	58	35	25	18	12
				IA/P = 0.30											TC = 1.5 HR.												IA/P = 0.30					
0.00	0	0	0	0	0	1	6	15	31	53	80	112	144	193	225	208	156	157	134	108	89	70	56	48	42	37	34	31	28	25	20	2
0.10	0	0	0	0	0	0	1	4	12	25	43	68	97	157	198	219	203	178	151	120	98	77	60	50	44	38	35	32	28	25	20	3
0.20	0	0	0	0	0	0	0	1	3	9	19	35	57	114	168	201	213	198	171	135	108	84	64	53	46	40	36	33	29	26	20	4
0.30	0	0	0	0	0	0	0	1	2	7	45	29	48	100	155	193	210	200	177	140	113	87	66	54	46	41	36	33	29	26	20	5
0.40	0	0	0	0	0	0	0	0	2	5	12	23	39	87	141	184	207	202	182	146	117	89	68	55	47	41	36	33	29	26	20	5
0.50	0	0	0	0	0	0	0	0	0	1	4	9	18	51	101	153	190	205	197	164	131	99	73	58	49	43	38	34	30	26	20	7
0.75	0	0	0	0	0	0	0	0	0	0	2	4	9	30	68	116	160	189	197	179	147	110	80	62	52	45	39	35	30	27	21	8
1.00	0	0	0	0	0	0	0	0	0	0	0	0	1	5	20	49	92	138	175	195	178	137	97	72	57	48	42	37	31	28	21	12
1.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	21	47	85	145	187	178	133	95	71	57	48	42	34	29	23	16
2.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	13	45	97	162	180	138	99	74	58	49	38	32	25	18
2.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	31	89	161	174	133	97	72	58	42	34	26	18
3.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	29	98	160	169	129	95	71	48	37	28	19
				IA/P = 0.50											TC = 1.5 HR.												IA/P = 0.50					
0.00	0	0	0	0	0	0	0	3	8	16	27	42	59	92	115	128	130	121	112	100	90	78	67	60	55	50	46	43	39	35	29	4
0.10	0	0	0	0	0	0	0	2	6	12	22	35	51	84	110	125	128	123	114	102	91	79	68	61	55	50	46	43	39	35	29	4
0.20	0	0	0	0	0	0	0	0	1	4	10	18	29	60	91	114	126	128	120	108	97	83	71	63	57	52	47	44	40	36	29	5
0.30	0	0	0	0	0	0	0	0	1	3	8	14	24	52	83	108	123	126	122	110	98	85	72	63	57	52	48	44	40	36	29	6
0.40	0	0	0	0	0	0	0	0	0	1	2	6	12	31	60	90	112	124	126	116	104	90	75	66	59	54	49	45	41	37	29	8
0.50	0	0	0	0	0	0	0	0	0	0	2	4	9	26	53	83	106	121	125	118	106	91	77	67	60	54	49	46	41	37	29	8
0.75	0	0	0	0	0	0	0	0	0	0	1	2	5	16	36	62	88	108	119	122	112	97	81	69	62	56	51	47	42	38	30	11
1.00	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	26	49	75	98	118	121	108	90	76	66	59	54	49	43	39	31	16
1.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	11	25	45	80	107	118	106	89	75	65	59	53	45	41	32	23
2.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	11	32	63	100	115	104	87	74	65	58	48	42	34	26
2.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	16	48	94	113	105	89	76	66	53	45	36	27
3.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	15	54	96	111	103	88	75	58	48	38	28

Figure 7.12f - Tabular Hydrograph Unit Discharge (cfs/sq.mi-inch)
Type II Rainfall Distribution

TRAVEL TIME (HRs.)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	24.0
				IA/P = 0.10											TC = 2.0 HR.											IA/P = 0.10						
0.00	7	9	12	16	18	21	27	36	49	64	82	104	127	171	201	226	208	193	171	132	105	79	58	45	36	30	26	23	20	17	13	3
0.10	6	8	10	14	15	17	20	25	33	43	57	74	94	139	179	204	218	205	188	150	118	88	63	48	38	32	27	24	20	17	13	4
0.20	6	8	10	13	14	16	19	23	29	39	51	66	84	128	169	198	213	207	192	157	123	91	65	49	39	33	28	24	20	17	13	4
0.30	6	7	9	12	14	15	18	21	27	35	45	59	76	117	159	191	211	208	196	163	128	95	68	51	40	33	28	25	20	18	13	4
0.40	5	6	8	11	12	13	15	17	20	24	31	41	53	87	128	167	197	209	205	180	145	106	75	55	43	35	30	26	21	18	14	5
0.50	5	6	8	10	11	13	14	16	18	22	28	37	48	78	118	158	190	208	208	185	151	111	77	57	44	36	30	26	21	18	14	5
0.75	4	6	7	9	10	11	12	13	15	18	22	27	35	58	91	129	164	191	202	194	167	125	87	63	48	38	32	27	22	18	14	6
1.00	3	4	6	7	8	8	9	10	11	12	14	16	18	28	46	74	110	147	178	201	193	156	108	76	56	43	35	30	23	19	14	8
1.50	2	3	3	5	5	5	6	6	7	8	8	9	10	12	16	23	36	57	86	137	178	195	160	113	79	58	45	36	26	21	16	11
2.00	1	2	2	3	3	4	4	4	5	5	6	6	7	8	10	12	16	23	35	67	112	169	190	154	110	78	57	44	30	23	17	11
2.50	0	1	1	2	2	2	3	3	3	4	4	4	5	6	7	8	9	12	16	28	52	105	170	185	149	107	76	56	35	26	18	12
3.00	0	0	1	1	1	1	1	2	2	2	2	3	3	3	4	5	6	7	8	12	18	41	99	161	180	152	112	80	45	30	19	12
				IA/P = 0.30											TC = 2.0 HR.											IA/P = 0.30						
0.00	0	0	0	0	0	1	3	8	15	25	38	54	74	115	148	168	185	170	159	131	110	89	70	57	49	42	38	34	29	26	20	5
0.10	0	0	0	0	0	0	0	2	6	12	21	32	47	85	124	153	169	180	168	145	120	96	75	60	51	44	39	35	30	26	20	6
0.20	0	0	0	0	0	0	0	2	4	10	17	27	41	75	114	146	165	175	170	149	124	99	76	62	52	45	39	35	30	27	21	6
0.30	0	0	0	0	0	0	0	0	1	3	7	14	23	49	86	122	151	170	174	160	136	107	82	66	54	47	41	37	31	27	21	8
0.40	0	0	0	0	0	0	0	0	1	2	6	11	19	43	77	113	144	165	173	163	140	111	85	67	55	47	41	37	31	27	21	8
0.50	0	0	0	0	0	0	0	0	1	2	4	9	16	37	68	104	136	160	171	165	144	114	87	69	56	48	42	37	31	27	21	9
0.75	0	0	0	0	0	0	0	0	0	0	1	2	5	15	34	62	96	127	152	167	160	132	100	77	62	52	45	40	32	28	22	11
1.00	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	24	48	79	111	150	166	153	118	90	71	58	49	43	34	29	23	14
1.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	10	24	45	88	130	161	148	115	88	70	57	48	37	31	24	17
2.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	10	32	68	122	157	143	113	87	68	56	42	34	26	18
2.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	16	51	114	153	144	116	89	70	49	38	27	19
3.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	15	59	118	150	140	113	88	57	42	29	19
				IA/P = 0.50											TC = 2.0 HR.											IA/P = 0.50						
0.00	0	0	0	0	0	0	0	1	4	8	13	20	28	51	73	92	104	111	112	106	97	86	75	66	60	54	49	46	41	37	30	7
0.10	0	0	0	0	0	0	0	1	3	6	11	17	24	45	68	87	101	109	112	107	98	88	76	67	60	55	50	46	41	37	30	8
0.20	0	0	0	0	0	0	0	1	2	5	9	14	21	40	62	82	98	107	111	108	100	89	77	68	61	55	50	47	41	37	30	8
0.30	0	0	0	0	0	0	0	0	0	2	4	7	12	26	46	67	86	100	108	111	104	93	80	70	63	57	52	48	42	38	30	10
0.40	0	0	0	0	0	0	0	0	0	1	3	6	10	22	41	62	81	96	106	110	105	94	81	71	63	57	52	48	42	38	30	11
0.50	0	0	0	0	0	0	0	0	0	0	1	2	4	13	27	46	67	85	99	110	108	98	85	74	66	59	54	49	43	39	31	13
0.75	0	0	0	0	0	0	0	0	0	0	0	1	2	7	18	33	52	71	88	104	108	102	89	77	68	61	55	50	44	39	31	15
1.00	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	13	25	43	62	87	103	108	97	84	73	65	59	53	45	41	32	20
1.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	12	24	48	74	99	106	95	83	72	64	58	48	43	34	25
2.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	17	37	69	99	104	94	82	72	64	52	45	36	27
2.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	27	65	95	102	95	83	73	58	49	38	28
3.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	32	68	95	101	93	82	64	52	40	28

Example 7.8: Develop a partial hydrograph using the following information that was computed above:

1. Time of concentration = 1.43 hours
2. Type II rainfall distribution
3. Runoff = 2.79 sq. mi.-inches
4. Ia/P = .13

The t_c of 1.43 is rounded to the nearest value, which is 1.5 hours. The Ia/P of 0.13 is between the values of 0.1 and 0.3 shown on Figure 7.12e. The following information is a result of interpolating between the values using the Ia/P of .13.

Tabular Unit Hydrograph, Type II Rainfall, Ia/P = 0.13

Hours	**q_u**	**Hours**	**q_u**	**Hours**	**q_u**	**Hours**	**q_u**
11.0	8	12.4	74	13.6	196	16.5	32
11.3	9	12.5	103	13.8	159	17.0	27
11.6	13	12.6	137	14.0	130	17.5	25
11.9	18	12.7	173	14.3	100	18.0	23
12.0	21	12.8	205	14.6	78	19.0	20
12.1	27	13.0	246	15.0	59	20.0	17
12.2	36	13.2	268	15.5	45	22.0	13
12.3	52	13.4	232	16.0	37	26.0	1

Note: **q_u** is in cfs/sq.mi-in
Q on hydrograph = (qu) x (vol. of runoff)

Figure 7.13 shows a plot of the hydrograph that would result from a runoff of 2.79 sq.mi.-in. As noted above, the Q on the hydrograph results from multiplying q_u by 2.79 sq. mi.-in. (total runoff). The hydrograph has been extrapolated for the time less than 11 hours. If this portion of the hydrograph were critical, it would have been necessary to use more comprehensive methods (references 46, 47 and 49).

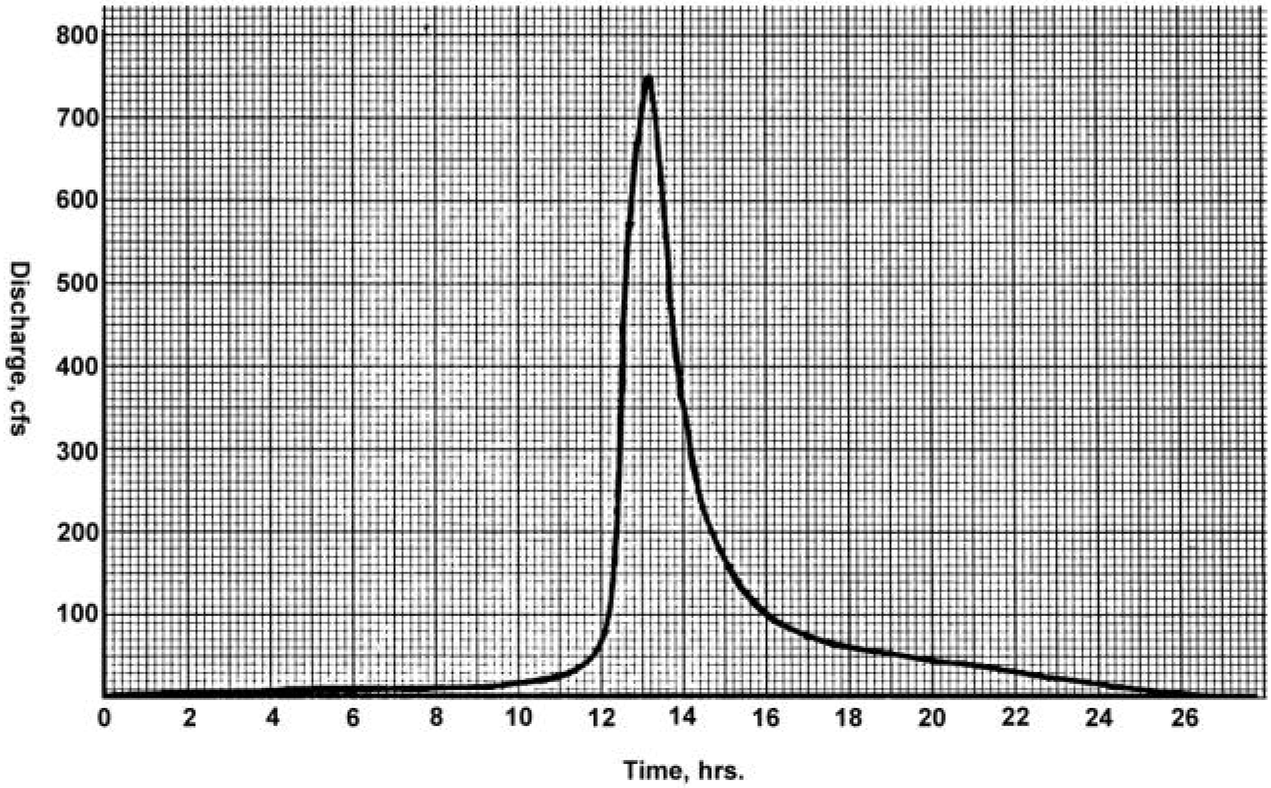


Figure 7.13 - Plot of Hydrograph for Example 7.8

CHAPTER 8: HYDRAULICS OF OUTLET STRUCTURES

The purpose of the outlet structure of a detention pond is to control the rate of outflow from the pond. The success or failure of a detention pond is dependent on the outlet structure. There is a wide variety of types of structures that can be used to regulate the outflow, including culverts, drop structures, and spillways. If an outlet structure is properly designed, the detention pond will provide the needed attenuation of flood flows for a range of flood frequencies, while also achieving water-quality benefits.

Outlet structures can be divided into two "hydraulic" groups: orifices/culverts and weirs. This section of the guidebook will cover the basic equations that are used in designing the outlet structure.

TAILWATER ELEVATION

Before an outlet structure can be designed, it is essential to determine the water surface elevation downstream of the structure. Ignoring the tailwater elevation is a common error that is made in determining the flow-carrying capacity of a structure. If the tailwater is not considered, the flow carrying capacity of the structure may be over -estimated (the structure would be under-sized). Under-sized structures will be subject to frequent over -topping, and may require costly revisions to the structure to prevent its failure.

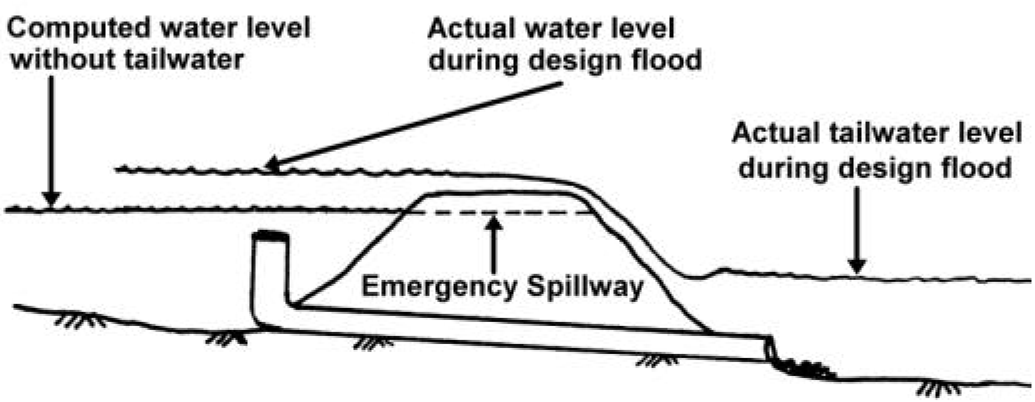


Figure 8.1 - Impacts of Not Considering Tailwater Elevation

There are various methods that can be used to determine the tailwater elevation (TWEL), including:

1. **Existing Information**

In many areas throughout Michigan, detailed hydraulic analyses currently exist. These analyses may contain water-surface elevation information for the location of interest. The primary sources of information include:

a) <u>Flood-Insurance Studies</u>. Prepared by the Federal Emergency Management Agency (FEMA). The studies will typically include profiles for the 10-, 50-, 100-, and 500-year floods. In addition, the hydraulic support data that was used to prepare a study is on file with the DEQ, Land and Water Management Division and with

FEMA. There are currently Flood-Insurance Studies available for about 370 communities in Michigan.

b) Flood Hazard Analyses. Prepared by the Natural Resources Conservation Service. These studies also include 10-, 50-, 100-, and 500-year flood profiles. In most instances, the hydraulic support data is on file with the DEQ Land and Water Management Division.

c) Corps of Engineers Floodplain Information. The reports will usually include 50- and 100-year profiles and in some instances a past flood event profile. The hydraulic data may not be available.

d) DEQ, Land & Water Management Division. Over the years, the DEQ has compiled a significant amount of information on various watercourses throughout the State. In most instances, the information would be limited to 100 -year elevations and possibly some stream -valley cross-section information.

e) Other sources. County or local government agencies may have information relating to hydraulic capacities of various watercourses. Some agencies that may have information could include the county drain commissioner, public works department, or the community engineering department.

2. **Normal-Depth Solution**

In many areas it is possible to approximate the tailwater elevation by using a "normal-depth solution." The normal-depth solution is a method of determining the water surface elevation for a given discharge at a particular location using Manning's equation referenced earlier:

$$Q = \frac{1.486}{n} A R^{2/3} S_f^{1/2} \qquad (15)$$

where: **Q** - discharge, cfs
n - Manning's roughness coefficient
A - area of the cross section, at the given water surface elevation
R - hydraulic radius (area/wetted perimeter)
S_f - slope of the energy gradient

a) Discharge - in the earlier section, the different met hods of computing discharges were discussed.

b) n - The selection of an appropriate "n" value requires engineering judgement and experience. There are several excellent references available which provide some guideline for selection of an "n." Probably the most widely referenced book is Chow's Open Channel Hydraulics (Reference 8). Chow indicates that the roughness coefficient is a function of several factors: material, degree of irregularity, variation of channel cross-section, obstructions, vegetation, and degree of meandering. Appendix D provides a method for estimating the "n" value. Following are some typical roughness coefficients:

~~channel condition~~	~~n~~
Straight, clean channel	0.03
w/ some weeds & stones	0.04
winding channel, some weeds or stones	0.05
same as above, with some obstructions	0.06
w/ significant obstructions	0.08
overbank areas with moderate brush	0.10
w/ dense brush & grass	0.15

It is recommended that the method contained in Appendix D be used to estimate the "n" values for stream channels.

c) Area & Hydraulic Radius - The area and hydraulic radius can be obtained from the stream-valley cross section. The **area**, or waterway area is the portion of the cross section that will be carrying flow. The **hydraulic radius** is defined as the area divided by the wetted perimeter.

The **wetted perimeter** is the wetted surface of the cross section which causes resistance to flow. When the cross section is broken up into sub-sections to define different roughness coefficients, the water-to-water interface is not included when computing the wetted perimeter. (See figure 8.2).

d) Energy Gradient (S_f) - The **energy gradient** (friction slope) can be thought of as the slope of the water surface. For most normal depth solutions, the slope must be determined. The slope may be estimated from:

1) U.S.G.S. quadrangle. . The contour angle on a quadrangle limit the accuracy of determining the friction slope. As a result, this method should be used when no other method is available. However, topographic maps that have 1-, 2-, or 4-foot contour intervals can provide a reasonable estimate of the friction slope.
2) Slope of the Water Surface at the Time of Survey. When survey information is being obtained at the site, it should include water surface elevations at several downstream locations. Judgement is needed to determine how far downstream to take elevations. As a rough guide, the elevations should be taken about every 100 feet for about 300 feet downstream. Once the distance and change in water surface elevation is known, an estimated slope may be used in the normal depth solution. It is very likely that the measured slope will not be uniform. Some judgement must be used to get a reasonable average of the slope.
3) Slope of High-Water (Flood) Profile. The best method of estimating the friction slope of a watercourse is to obtain high-water marks along the stream. A considerable amount of information can be obtained from talking with long-time residents. However, actually being on-site during a flooding event can provide a significant amount of insight into how a watercourse will function under flooding conditions. During high water, the water-surface elevation slope may be different than would be occurring during low flow. During these high stages, the flow may begin to encounter obstructions that will increase the resistance to flow.

Once the slope of the water-surface elevation is estimated, Manning's formula (equation 15) can be used to calculate the water-surface elevation for a given discharge. This water-surface elevation will be the "tailwater elevation" at the outlet structure.

e) Normal-Depth-Solution Computations. In most cases, the cross section that has been taken downstream of the outlet structure will consist of a channel, a left overbank, and a right overbank (see figure 8.2). In addition, the "n" value will not likely be constant across the entire cross section. To compute a normal depth solution, it would be necessary to break the cross section up into subsections based on "n" values.

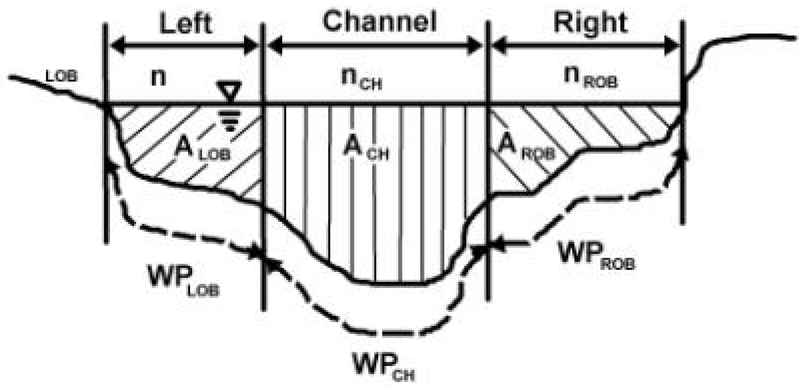


Figure 8.2 - Cross Section with More than One "n" value

The procedure for determining the water surface elevation for a given discharge is a trial-and-error process:

1. Assume a water surface elevation and compute the hydraulic properties for each sub-section (area, wetted perimeter, and hydraulic radius).

2. Compute the discharge (Q) for each subsection of the cross section at the assumed water surface elevation using Manning's formula:

   $$Q = \frac{1.486}{n} A R^{2/3} S_f^{1/2}$$

3. Combine the discharges for each of the sub -sections to obtain the total d ischarge for the cross section at the assumed water-surface elevation.

   $$Q_t = Q_1 + Q_2 + Q_3$$

4. Plot the water surface elevation (stage) versus discharge on a graph, such as shown on figure 8.3.

5. The process is repeated until the graph defines the "rating curve" for the cross section.

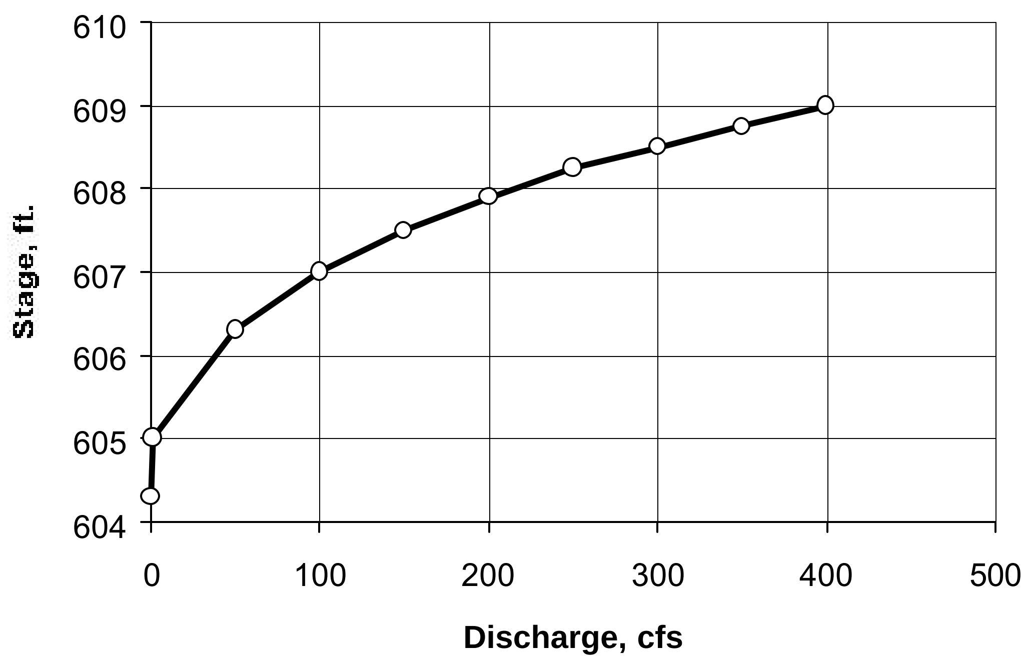


Figure 8.3 - Typical Rating Curve

Once the rating curve is developed downstream of the outlet structure, it is possible to determine the water-surface elevation for a given discharge. The tailwater-elevation information is essential in defining the outlet-structure rating curve.

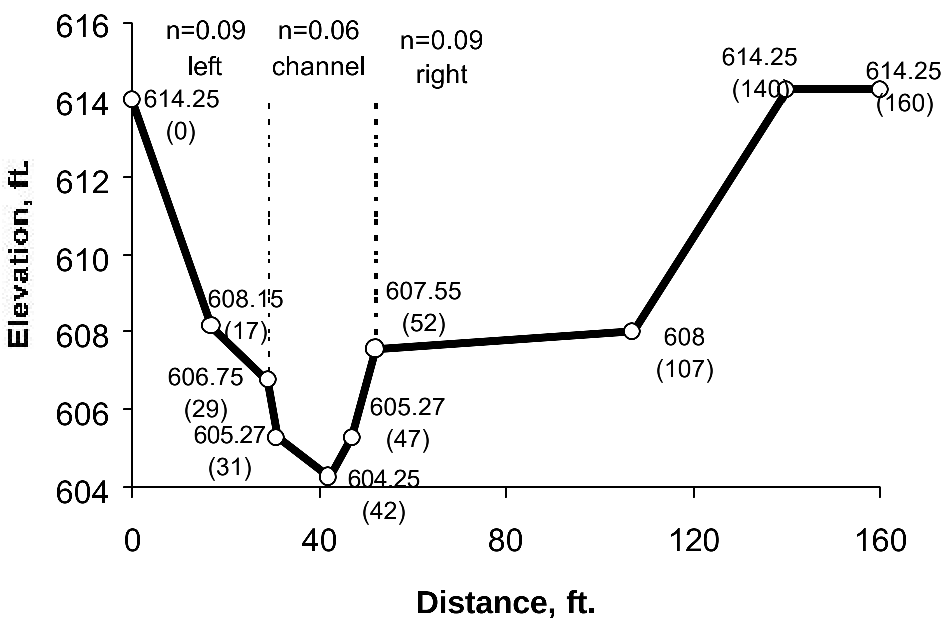


Figure 8.4 - Example Cross Section

Following is an example of a normal depth solution:

Example 8.1: Given a discharge of 345 cfs, a stream slope of .004 ft/ft, and the following cross section, compute the normal depth solution.

Solution:

– Assume a water surface elevation - 608.7

– Compute hydraulic properties for each "subsection" of the cross section:

– The area (A) and wetted perimeter (P) can be computed for the assumed water surface elevation of 608.7, by using simple algebra (see figure 8.4). The hydraulic radius (R) is defined by Area (A)/wetted perimeter (P).

  left overbank: n=.09; A=15.4 sq. ft, P=13.7 ft, R=1.13 ft

  channel: n=.06; A=79.9 sq. ft, P=24.1 ft, R=3.32 ft

  right overbank: n=.09; A=52.2 sq. ft, P=58.8 ft, R=.89

– Compute Q for each subsection, using Manning's equation (15), sum all of the Q's to obtain a total Q:

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$$

Qleft:	17 cfs
Qchannel:	278 cfs
Qright:	50 cfs
Q total:	**345 cfs**

– If a range of tailwater elevations and discharges are required, additional elevations can be selected, and discharges computed.

elevation	604.25	606.00	606.30	607.00	607.30	608.00	609
computed Q, cfs	0	34	50	95	120	200	425

– Plot results on a stage versus discharge curve (See figure 8.5).

  The graph can be used to determine an elevation for a given Q.

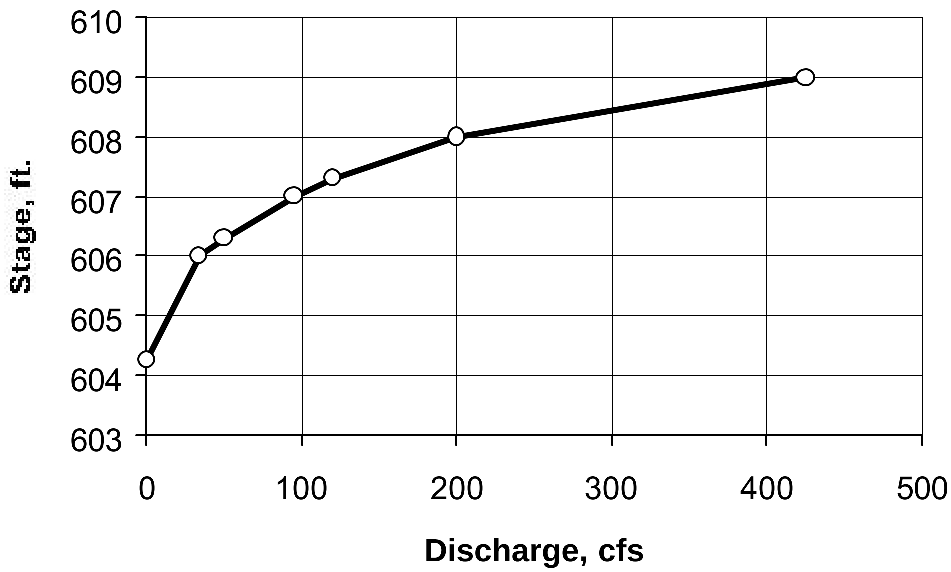


Figure 8.5 - Rating Curve for Example

Important Note:

A normal depth solution is applicable only if there are no downstream restrictions which may cause a backwater at the site (see figure 8.6). Restrictions can include bridges, culverts, dams, and natural and man -made restrictions. In addition, the stream slope should be fairly uniform, and the field-surveyed cross sections must be representative of the stream. The cross sections must be taken perpendicular to the flow as it would occur during flood conditions.

3. **Detailed Hydraulic Analysis**

As noted above, a normal-depth solution is not appropriate in all instances. For those times when a downstream restriction is causing a backwater at the site, a more in -depth analysis is required. Such an analysis can be done using computations by hand (which is quite tedious) or through the use of one of numerous computer programs (Some of the computer programs available include HEC-RAS (reference 53), HEC-2, WSPRO, and WSP-2).

4. **Equivalent Hydraulic Grade Line**

If no other information is available, it is possible to approximate the tailwater elevation by using the equivalent hydraulic grade line:

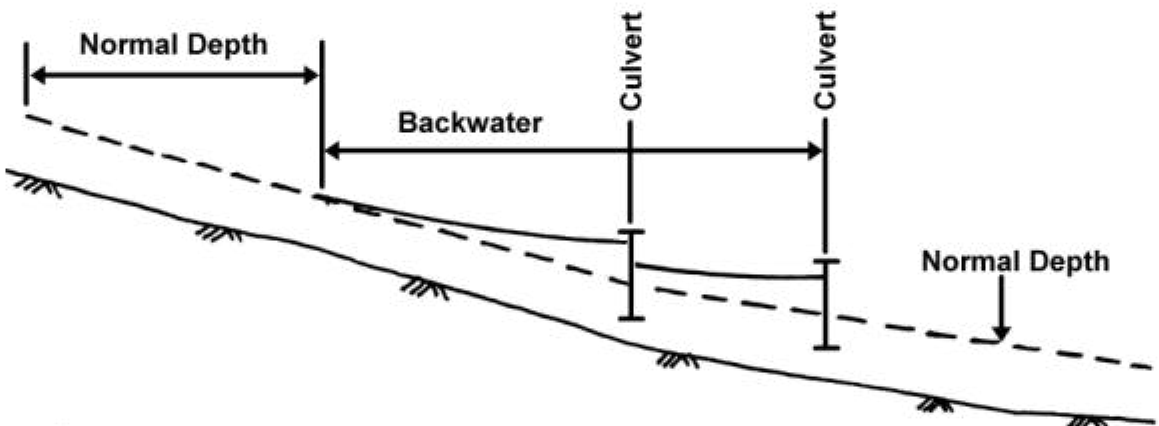
$$ho = (D + d_c)/2 \qquad (29)$$

where: **ho** - the vertical dimension from the culvert invert to the outlet equivalent hydraulic grade line.
D - diameter of the culvert
d_c - critical depth in the culvert for the given discharge

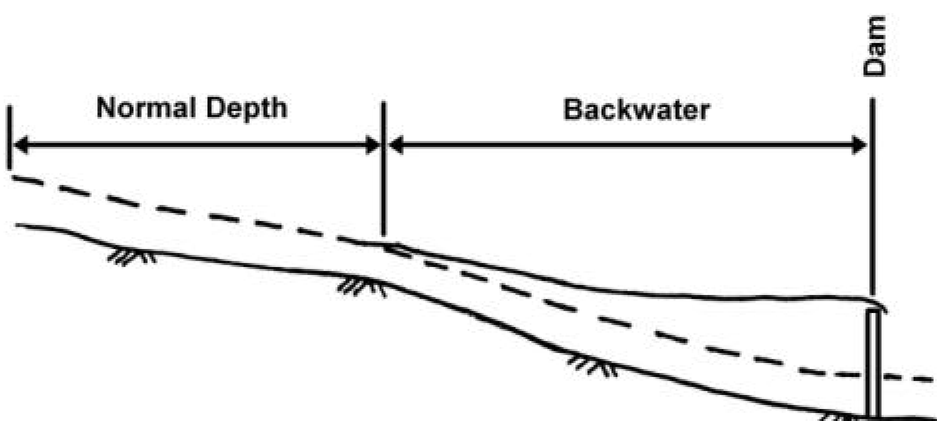
Critical depth is defined as the minimum specific energy for a given discharge. Specific energy being the total energy head ($Y + v^2/2g$) above the culvert invert or channel bottom. For depths of flow greater than d_c, the flow is sub-critical, or tranquil. If the depth of flow is less than d_c, the flow is super-critical, or rapid. For a constant discharge, as the depth decreases, the flow area decreases, which results in increased velocity. On the other hand, as the depth increases, the flow area increases, and the velocity decreases. (Figure 8.7 shows a plot of the depth versus specific energy for a constant discharge.)

Preferably, the hydraulic grade line should not be used in place of tailwater computations. It should be used as a comparison to the tailwater determined by computations. The equivalent hydraulic grade line should be used if it exceeds the tailwater elevation computed by other methods.

Figure 8.8 shows a typical chart for determining d_c for a given culvert and discharge. Additional charts are included in Appendix E.

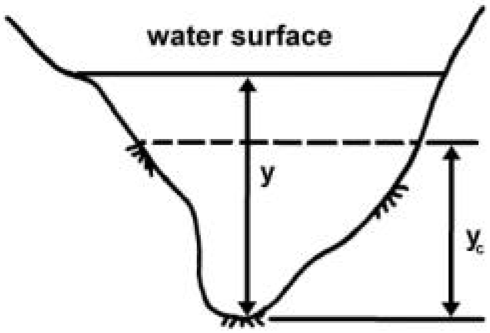


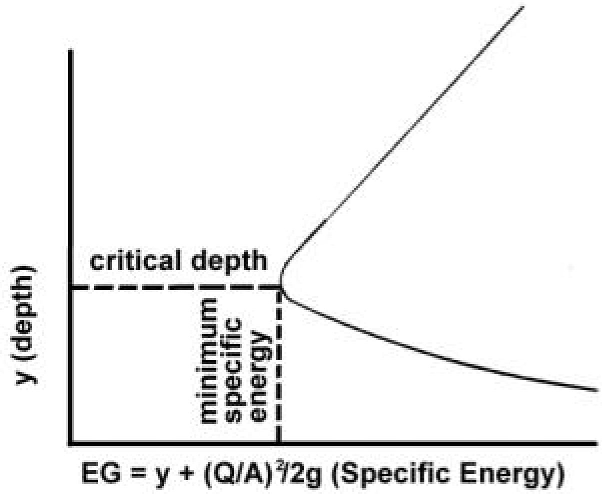
a. Backwater Effects Due to Downstream Culverts/Bridges



b. Backwater Effects Due to a Downstream Dam

Figure 8.6 - Effects of Backwater Due to Downstream Restrictions





a. Typical Cross Section **b. Specific Energy Diagram**

Figure 8.7 - Critical Depth

Example 8.2: Given: a 5-foot diameter corrugated metal pipe with an outlet invert elevation of 580.2 feet. The total discharge is 250 cfs, and the computed tailwater elevation is 584.0 feet. Compute the equivalent hydraulic grade-line.

1. From figure 8.8, for a discharge of 250 cfs d_c is 4.4 feet.
2. ho = (D + d_c)/2 = (5 + 4.4)/2 = **4.7**
3. The equivalent hydraulic grade-line elevation:

   Outlet invert + ho = 580.2 + 4.7 = **584.9**

In example 8.2, the equivalent hydraulic grade line exceeds the tailwater elevation of 584.0 feet that was computed. The "tailwater" elevation that should be used in the analysis of the culvert for the discharge of 250 cfs is 584.9 feet and not 584.0 feet.

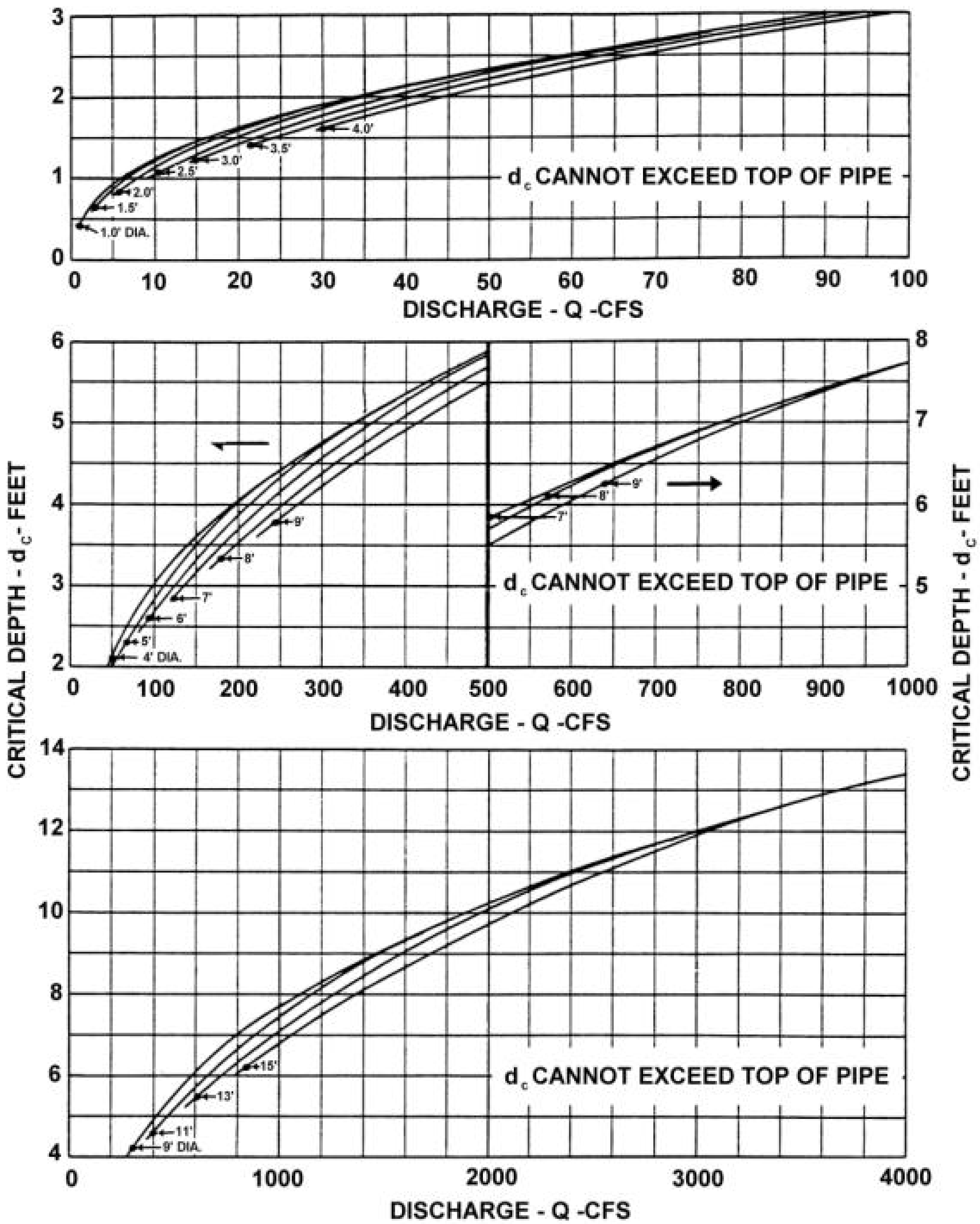


Figure 8.8 - Critical Depth Chart for Circular Pipe

(Source: FHWA, 1985, reference 13)

High-water marks

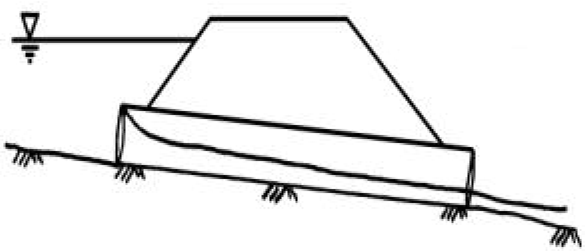
Hydraulics and hydrology involve considerable judgement on the part of the designer in estimating the various coefficients that are needed to determine the discharge and the anticipated water-surface elevation. Since these are not exact sciences, the importance of comparing the preliminary results with what has occurred in the past cannot be stressed enough. High-water marks are a good indicator of how the watercourse will function during a flood.

The validity of the computations is destroyed if the "100-year" elevation that has been computed is exceeded each spring. If high-water marks do not support the water surface elevations determined by the computations, the friction slope (S_f) and "n" values should be reviewed, as these are the values which typically induce errors into the normal-depth solution.

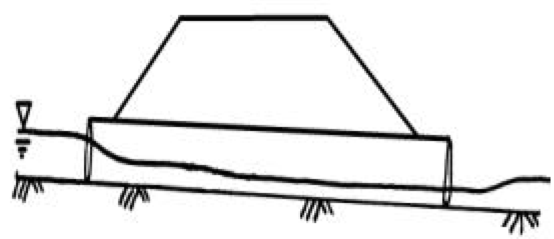
INLET & OUTLET CONTROL - CULVERT FLOW

Before the basic orifice equation is discussed, it is necessary to get a brief overview of culvert (orifice) flow, which is be classified into two categories:

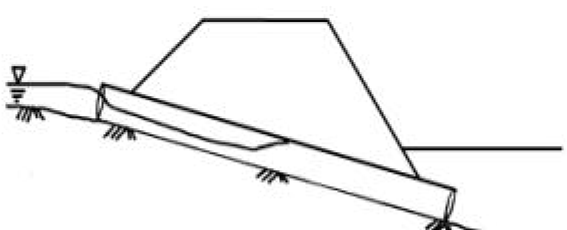
1. Inlet Control (figure 8.9)
2. Outlet Control (figure 8.10)



Inlet Submerged



Inlet and Outlet Unsubmerged



Outlet Submerged

Figure 8.9 - Culverts Flowing Under Inlet Control

Inlet Control

The capacity of a culvert that is flowing under inlet control is governed only by the cross-sectional area of the culvert and the inlet configuration (sharp-edge inlet, rounded inlet, a headwall, mitered, or protruding from fill). The tailwater and length of the culvert have no impact on the upstream water surface elevation (In other words, the water can get out and away from the culvert faster than it can get into the culvert.)

Inlet control conditions will typically occur for culverts that are fairly steep, or for an inlet that is very restrictive.

Outlet Control

For culverts flowing under **outlet control**, the upstream water surface elevation is controlled by a combination of:

1. Downstream water-surface elevation
2. Culvert length
3. Culvert material
4. Culvert slope
5. Inlet configuration (sharp, rounded wingwalls)

The downstream water-surface elevation (tailwater) can be controlled by downstream restrictions or the flow-carrying capacity of the channel. The tailwater is a very critical factor in the design of an outlet structure. Earlier, a method of computing a tailwater elevation was discussed.

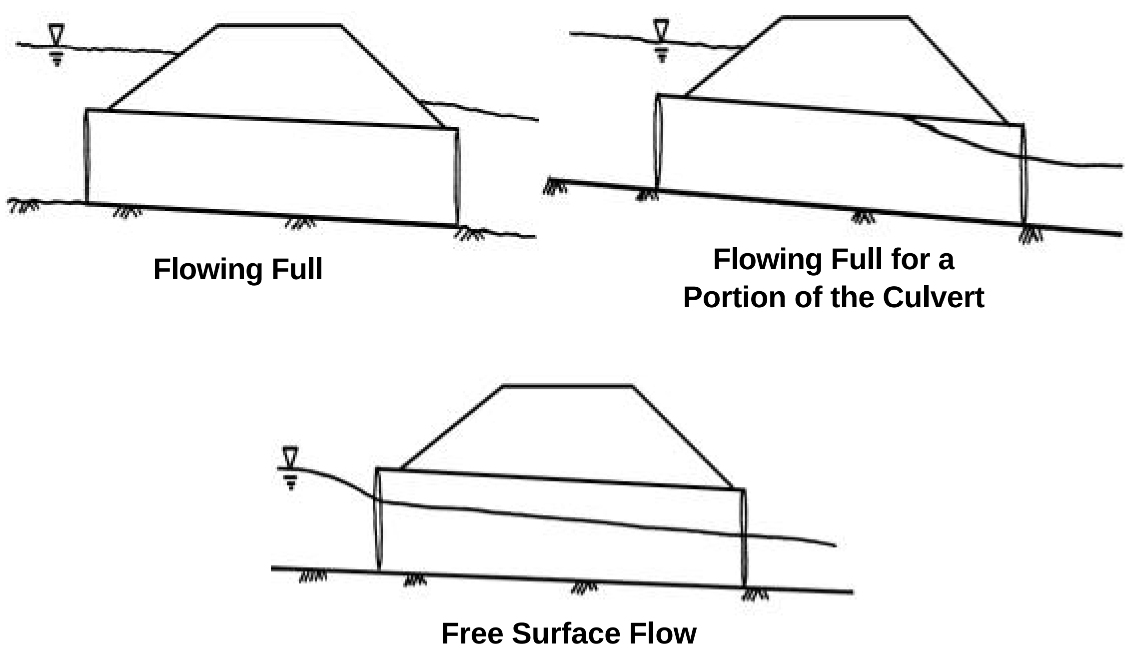


Figure 8.10 - Culverts Flowing Under Outlet Control

In some instances, it is not readily apparent whether a culvert is functioning under inlet or outlet control. For this reason, it is highly recommended that the outlet structure be analyzed for both inlet and outlet control to ensure that the structure will be functioning as designed.

Orifice/Culvert Flow

An orifice can be in the form of a pipe or box culvert or some other opening. The basic equation for orifice flow is given by:

$$Q = CA\,(2gH)^{1/2} \quad \text{(reference 7)} \qquad (30)$$

where: **Q** - discharge (outflow) in cubic feet/second
C - discharge coefficient
A - cross sectional area of orifice, square feet
g - acceleration due to gravity, 32.2 ft/sec/sec
H - head on the orifice, feet

Note: See figures 8.11a and 8.11b for the definition of head. For free flowing outlet (figure 8.11a), **H** is the difference in elevation between the upstream water surface elevation, and the center of the orifice.

For submerged flow, **H** is the difference between the upstream and downstream water surface elevations (figure 8.11b).

Equation 30 is arranged so the discharge can be computed for a given H. It is possible to rearrange (30) to a form in which it is straightforward to compute H for a given Q.

$$H = (Q/CA)^2/2g \qquad (31)$$

In equations 30 & 31, the C coefficient is obtained from reference material. The discharge coefficient (C) is a function of many variables such as size, shape and sharpness of the opening, the type of material the orifice is made of, head on the orifice, and the extent to which the flow is already suppressed.

a. Free Flowing Outlet **b. Tailwater-Influenced Outlet**

Figure 8.11 - Head Differential

The C coefficient can range from .15 for a long corrugated metal pipe, to 0.95 for a short tube (orifice) with rounded edges. A typical value for C would be about 0.6, for sharp edged orifices. Values of C can be found in Brater and King "Handbook of Hydraulics" (reference 7). If no references are available, the following C coefficients may provide some guidance:

Table 8.1 - C coefficients for orifices and culverts

orifices	**C**
Sharp opening	0.60
suppressed on sides and bottom	0.70
suppressed on sides, top, and bottom	0.90
Orifice with rounded corners	0.95
culverts	
4-ft CMP no headwall, 60 ft long	0.54
4-ft CMP with headwall, 60 ft long	0.57
4-ft concrete pipe, 60 ft long, square edge	0.75
4-ft concrete pipe, 60 ft long, beveled edge	0.82

The following equation is another form of equation 31:

$$H = K\,(Q/A)^2/2g \qquad (32)$$

where: **K** is a total loss coefficient which can be computed by knowing the length of the culvert, the culvert material, and the entrance geometry (references 12 & 45).

$$K = 1 + k_e + k_f \qquad (33)$$

where: $\mathbf{k_e}$ - entrance loss coefficient (Table 8.2)
$\mathbf{k_f}$ - friction loss coefficient:

$$k_f = \frac{29.1 n^2 L}{R^{4/3}} \qquad (34)$$

where: **n** – Manning's roughness coefficient (table 8.2)
L - length of culvert
R - hydraulic radius, (area/wetted perimeter)

Outlet Control, Full or Partly Full Entrance Loss Coefficients

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

Table 8.2 - Entrance Loss Coefficients

(Source: reference 12)

Table 8.3 - Selected Manning's n Values

Material			Manning n
Concrete Pipe:	w/ smooth walls		.011 - .013
	w/ rough walls		.015 - .017
Concrete Box:	w/ smooth walls		.012 - .015
	w/ rough walls		.014 - .018
Corrugated Metal Pipes and Boxes:			
	Annular corrugations	2-2/3" x 1/2"	.027 - .023
		3" x 1"	.028 - .027
		6" x 2"	.035 - .032
	Helical corrugations	2-2/3" x 1/2"	.012 - .024

(Source: FHWA, reference 12)

Substituting equation 34 into equation 33:

$$K = 1 + k_e + \frac{29.1 n^2 L}{R^{4/3}} \qquad (35)$$

To demonstrate how this equation above may be used, the following example is given.

Example 8.3: Compute the K value for a 3 -foot circular concrete pipe, 50 feet long, with a square-edge entrance. Using the computed K value, determine the head needed to carry 70 cfs. (The tailwater submerges culvert outlet.)

For a 3-foot circular concrete pipe:

Area (A) $= Pi(D)^2/4 = 3.14(3)^2/4 = 7.07$ square feet

Wetted Perimeter (WP) $= Pi(D) = 3.14(3) = 9.42$ feet

Hydraulic Radius (R) $= A / P = 7.07 ft^2 / 9.42 ft = 0.75$ feet

Use equation 35 to compute total loss coefficient:

$$K = 1 + k_e + \frac{29.1 n^2 L}{R^{4/3}}$$

1. From table 8.2, **ke = <u>0.5</u>;** From table 8.3, n = 0.012

2. $kf = \frac{\mathbf{29.1\,(n)^2} L}{R^{4/3}} = \frac{29.1\,(0.012)^2\,50}{0.75^{4/3}} = \underline{\mathbf{.31}}$

3. $K = 1 + ke + kf = 1 + 0.5 + 0.31 = \underline{\mathbf{1.81}}$

4. To compute the head needed to pass the given discharge (Q), equation 32 is used:

   $H = K(Q/A)^2 / 2g$

Q = 70 cfs	(given)
K = 1.81	(computed in step 3)
A = 7.07 sq. ft.	(area of 3-foot pipe)
g = 32.2 ft./sec./sec.	(gravity constant)

   $H = 1.81(70/7.07)^2/64.4 = 1.81(9.9)^2/64.4 = \underline{\mathbf{2.76}}$ ft.

The computed H is the head differential between the tailwater and the headwater.

If the tailwater elevation is 584.0 feet, the outlet control computations produce an upstream stage equal to:

   H + TWEL = (2.76 + 584.0) = **586.76 ft.**

In addition to obtaining C values from tables, or computing K values, it is possible to use nomographs that have been developed for most types of culverts and pipes. Figure 8.13 is an example of a nomograph. The nomograph allows the designer to determine a head (H) for a given discharge or vice-versa. Using example 8.3 above, the outlet control nomograph (figure 8.13) indicates H as 2.76. The "H" that was computed using the equation or the nomograph is added to the tailwater elevation to determine the upstream water surface elevation (see figure 8.12).

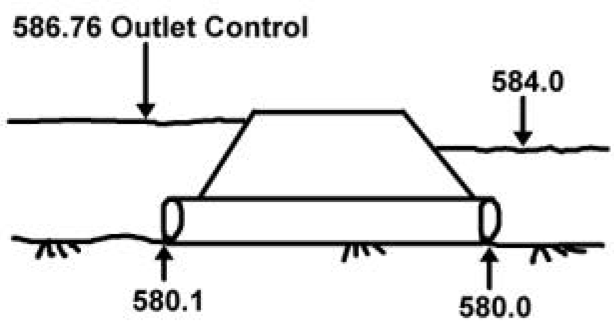


Figure 8.12 - Head Differential for Culvert Example

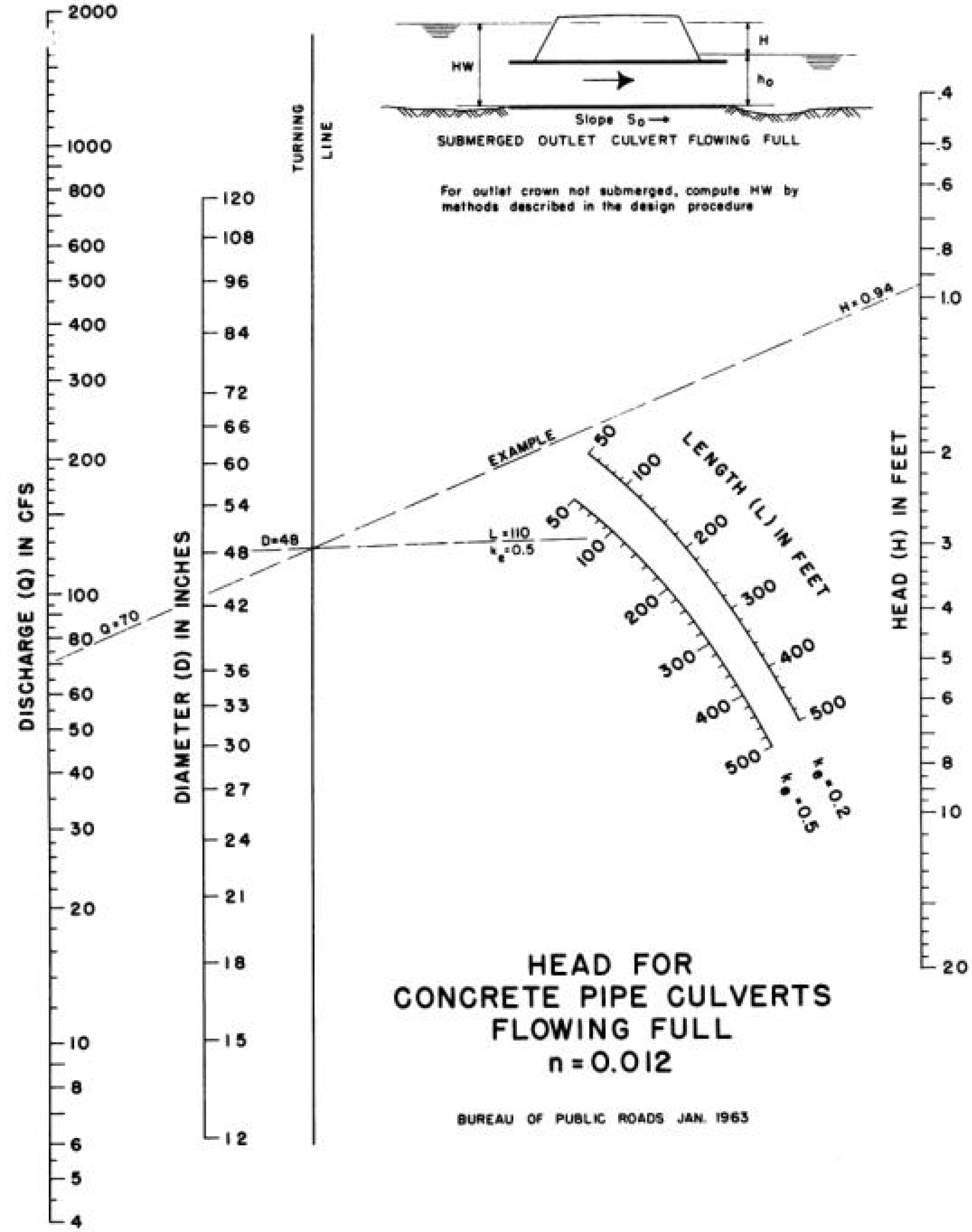


Figure 8.13 - Outlet-Control Nomograph

(Source: reference 12)

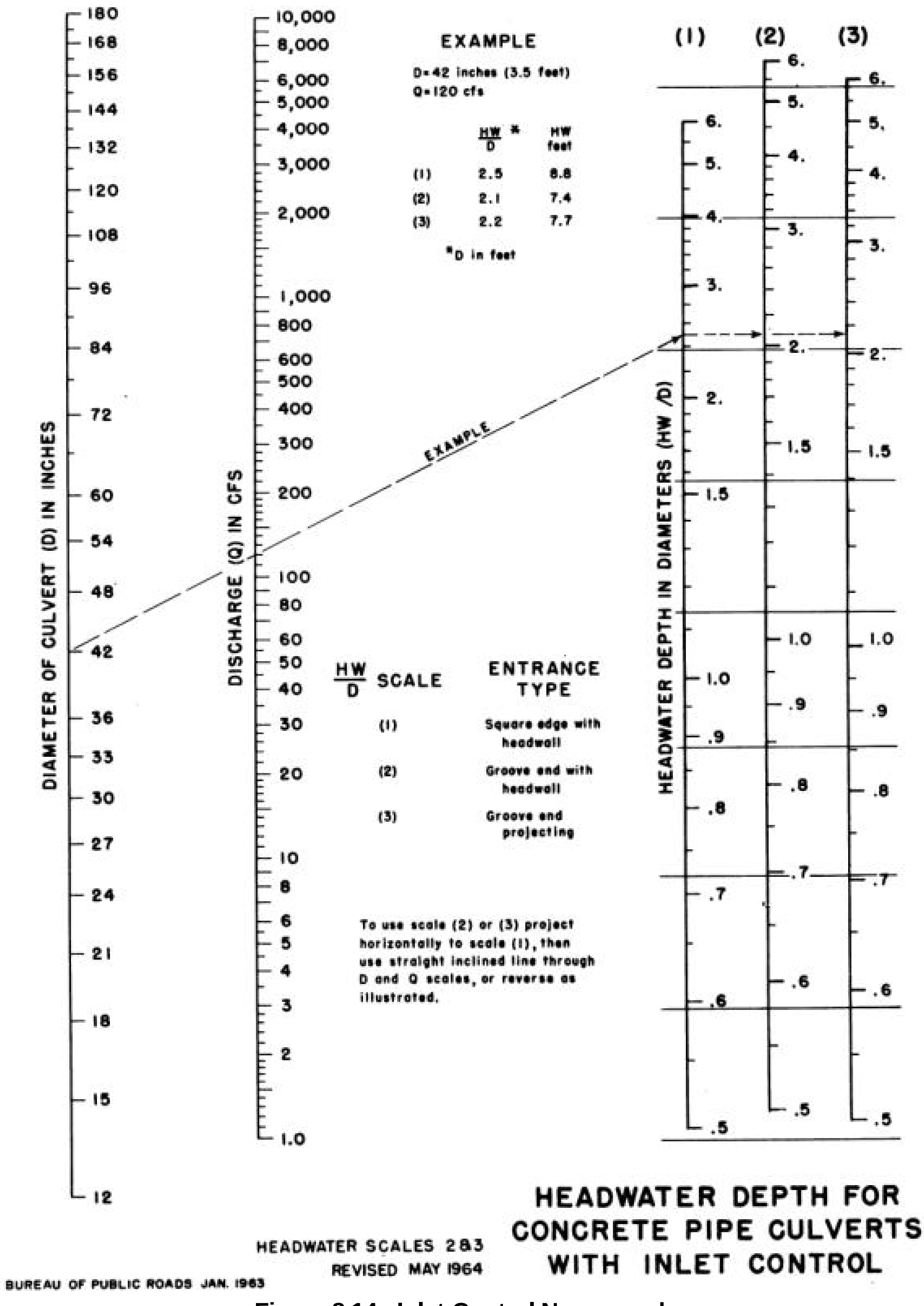


Figure 8.14 - Inlet-Control Nomograph

(Source: reference 12)

It was noted earlier that a culvert can function under either inlet or outlet control. The head (H) of 2.76 feet computed in the example 8.3 was determined assuming outlet control. It is also necessary to determine the upstream stage due to inlet control. Figure 8.14 is a typical inlet control nomograph for a culvert. Other nomographs are available in reference 12.

Example 8.4: Inlet-control computations
Given a 3-foot circular concrete pipe with a square edge entrance, determine the head needed to carry 70 cfs, for inlet control.

Using the nomograph in figure 8.14:

HW/D = 2.1 (Q = 70 cfs, Diameter = 36 inches)

where: **HW** - headwater in feet above the upstream invert of the culvert
D - diameter of the culvert

thus : **HW** = 2.1(D) = 2.1(3.0) = **6.3 feet**

If the upstream invert elevation of the culvert is 580.1, inlet control would produce an upstream stage was equal to:

HW + Upstream invert elevation = 6.3 + 580.1 = 586.4 feet

See figure 8.15 for a comparison of upstream stages for inlet and outlet control for examples 8.3 and 8.4.

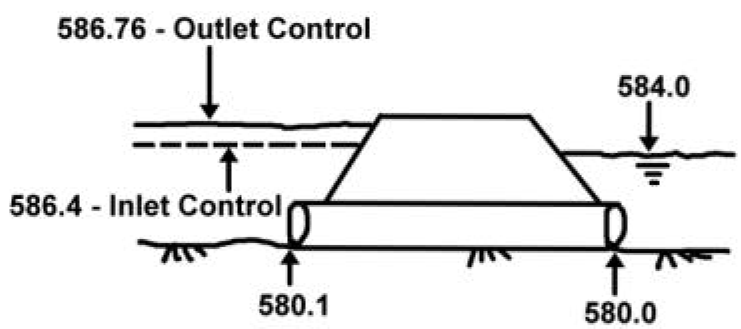
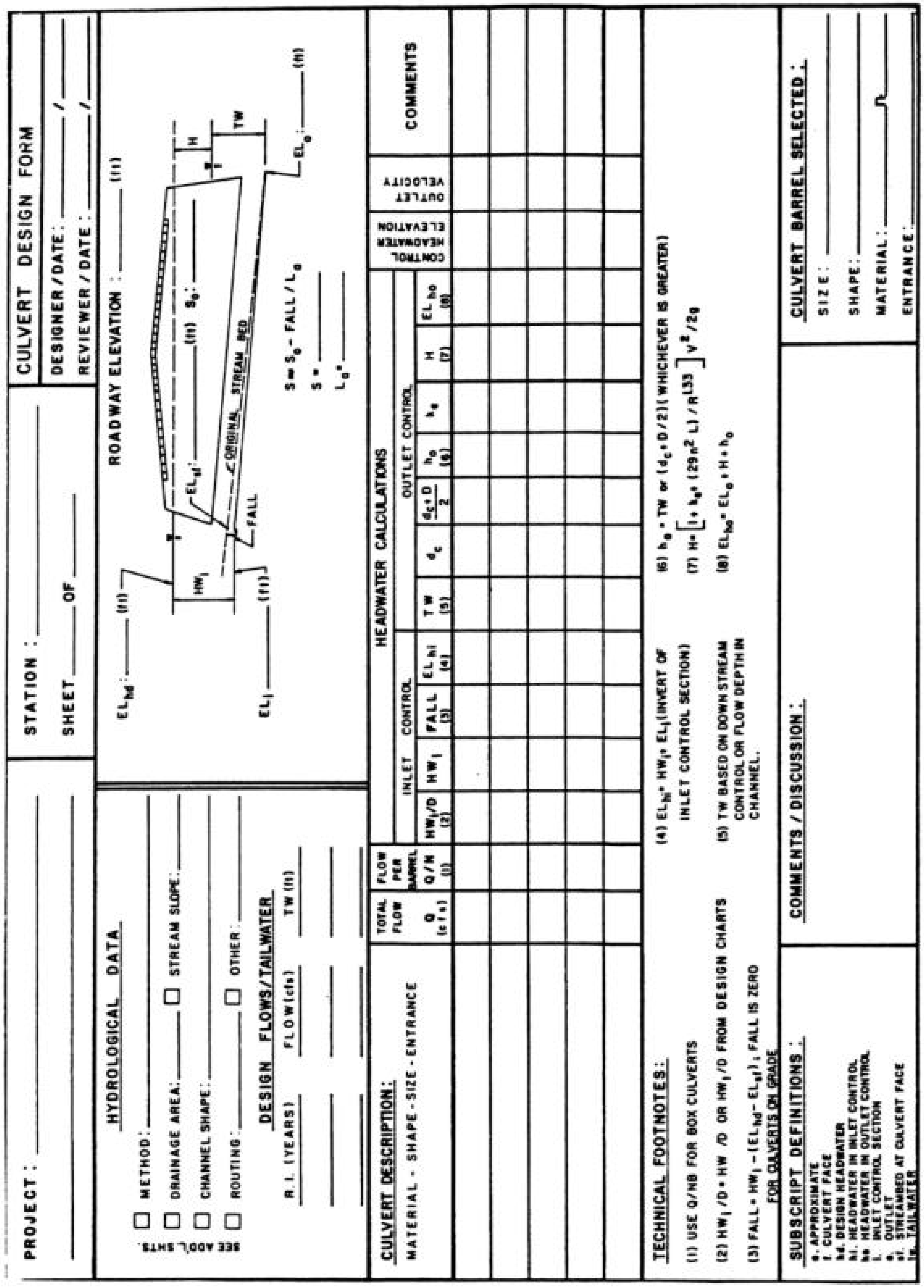


Figure 8.15 - Comparison of Inlet and Outlet Control for Example

In this example, the stage produced under outlet control exceeds the stage produced under inlet control, Thus, the culvert is functioning under outlet control. If tailwater conditions were different, or if a different discharge was analyzed, the control could shift to inlet control.

A form has been developed by the Federal Highway Administration to be used to analyze a culvert. The form (figure 8.16) is organized to allow the designer to determine both inlet and outlet control stages.



PROJECT: ____________________

STATION: ____________ SHEET ____ OF ____

CULVERT DESIGN FORM

DESIGNER / DATE: ________ / ________

REVIEWER / DATE: ________ / ________

HYDROLOGICAL DATA

SEE ADD'L SHTS.

☐ METHOD: ____________

☐ DRAINAGE AREA: ________ ☐ STREAM SLOPE: ________

☐ CHANNEL SHAPE: ____________

☐ ROUTING: ________ ☐ OTHER: ________

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)

$S = S_o - FALL / L_a$

S = ________

L_a = ________

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW	FLOW PER BARREL	HEADWATER CALCULATIONS											CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
			INLET CONTROL				OUTLET CONTROL									
	Q (cfs)	Q/N (1)	HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)			

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29n^2 L) / R^{1.33}\right] V^2 / 2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ____________

SHAPE: ____________

MATERIAL: ____________ n

ENTRANCE: ____________

Figure 8.16 - Culvert Design Form
(Source: reference 12)

Low Flow (Free surface flow)

If the culvert is flowing greater than 75% full, the procedure outlined earlier provides reasonable results. In some instances, the capacity of the culvert will be much greater than the discharge, in which case, the culvert will function as an "open channel" (see figure 8.17).

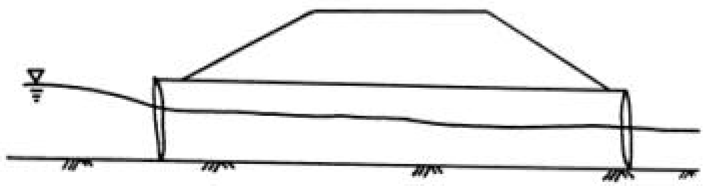


Figure 8.17 - Culvert Functioning Under "Free-Surface" Flow

To estimate the losses through a culvert functioning as an open channel, it is possible to use equations 32 and 35. Where:

$$K = 1 + k_e + \frac{29.1\, n^2\, L}{R^{4/3}} \qquad (32)$$

$$H = K\,(Q/A)^2\,/2g \qquad (35)$$

For culverts flowing partially full, the hydraulic radius (R) and area (A) are adjusted to account for the culvert not flowing full.

As an example, if the culvert in figure 8.17 were a 4-foot by 4-foot concrete box culvert 50 feet long with a square edge headwall and a tailwater depth of 2.5 feet, the adjusted hydraulic radius, area, and K value would be computed as follows:

Area(adjusted) = width x tailwater depth = 4 ft. x 2.5 ft. = 10 ft ^{2}

Hydraulic Radius (adjusted) = wetted perimeter$_{adj}$/area$_{adj}$

= (2 x depth + width)/ 10 ft ^{2}

= (2 x 2.5 ft + 4 ft)/ 10 ft ^{2}

= 1.125 ft.

Adjusted K $= 1 + 0.5 + \frac{29.1(0.012)^2(50)}{(1.125)^{4/3}} = 1.67$

To compute the head loss through the culvert, equation (35) would be used after inserting adjusted K and A values.

$$H = K_{adj}\,(Q/A_{adj})^2/2g = 1.67\,(Q/10)^2/64.4$$

Appendix F contains information needed to compute adjusted areas and wetted perimeters for circular and pipe-arch culverts.

To accurately determine the losses through a culvert that is going "low flow," step-backwater computations would be required through the culvert. The computations would include the energy losses due to friction and the entrance losses. There are computer and hand-held-calculator programs available for step -backwater computations. One such computer program is a culvert analysis program, **HY-8**, which was developed by Pennsylvania State University in cooperation with the Federal Highway Administration (FHWA). This program can be downloaded from the DEQ homepage at https://www.michigan.gov/egle/about/Organization/Water-Resources/transportation/ hydraulic-programs-and-report-guidelines. Even if a computer program is used to design an outlet structure, it is essential for the designer to be able to review the results to determine if they are reasonable.

Additional nomographs and more information on culvert flow is contained in "Hydraulic Design of Highway Culverts" (reference 12). The publication may be obtained through:

National Technical Information Service
U.S. Department of Commerce
5285 Port Royal Road
Springfield, VA 22162
(703) 487-4600

Weirs

The other type of outlet structure is classified as a weir. Weirs can be used as the primary outlet structure, or they can function as the emergency spillway. Weirs can be broken into two categories: sharp crested and broad crested.

Sharp-Crested Weir

A sharp-crested weir (see figure 8.18) has a "sharp" upstream corner. The flow over a sharp-crested rectangular weir is defined as:

$$Q = CLH^{3/2} \tag{36}$$

where: **Q** - discharge, cfs
C - discharge coefficient
L - effective length of the weir, ft.
H - head, difference in feet between the crest of the weir and the upstream water energy gradient.

The discharge coefficient (C) for the sharp-crested weir can be calculated using:

$$C = 3.27 + \frac{0.4h}{P} \tag{37}$$

where : **P** - height of weir crest above the channel bottom
h - head of energy gradient above crest

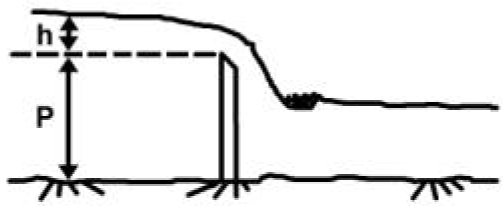


Figure 8.18 - Sharp-Crested Weir

The effective weir length (L) may have to be corrected to account for end contractions (see figure 8.19). End contractions refer to a weir configuration in which flow must contract to get over the weir.

$$L = L' - 0.1nh \tag{38}$$

where: **L'** - measured weir length
n - number of end contractions
h - head of energy gradient above crest

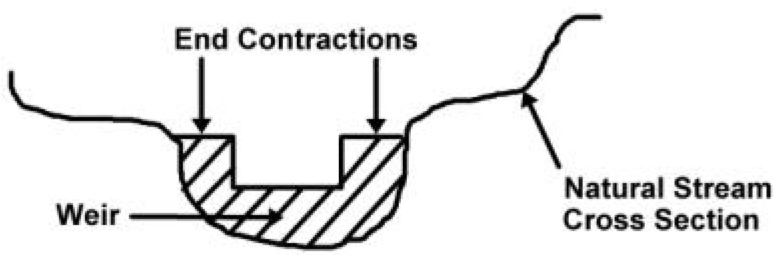


Figure 8.19 - End Contractions For Sharp-Crested Weir

Following is an example of discharge computations for a sharp crested weir.

Example 8.5: Given a sharp-crested weir, with a crest that is 4 feet above the channel bottom. The measured weir length is 15 feet with contraction on each end of the weir. Compute the discharge for a head of 2 feet.

1. Compute C: Equation (37) $C = 3.27 - 0.4h/P$
   $= 3.27 - 0.4(2/4)$
   $= 3.07$

2. Compute L: Equation (38) $L = L' - 0.1nh$
   $= 15 - 0.1(2)2$
   $= 14.6$ ft

3. Compute discharge: Equation (36) $Q = CLH^{3/2}$

   $Q = 3.07(14.6)(2)^{3/2}$ = **<u>127</u> <u>cfs</u>**

Triangular sharp-crested weir

The triangular weir is able to provide very accurate measurement of flows. Flow over the triangular weir is defined as:

$$Q = 2.5 \tan (a/2)\, H^{2.5} \qquad (39)$$

where: **a** - angle of the notch of the weir (figure 8.20)

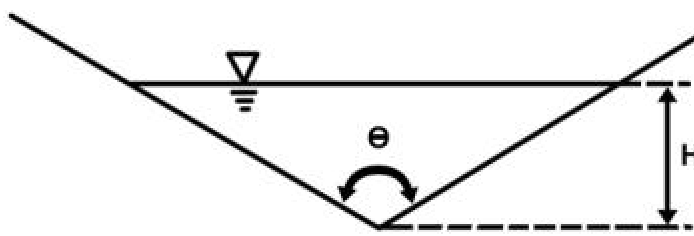


Figure 8.20 - Triangular Weir

Example 8.6: Given a= 120 degrees, and H = 3.0 feet.

Compute the discharge.

Equation (39) $Q = 2.5 \tan (120/2)\, (3)^{2.5}$

$= \underline{67.5}\ \underline{cfs}$

Broad-Crested Weir

A broad-crested weir differs from a sharp-crested weir in that the weir is wide enough to support the water as it flows over the weir (see figure 8.21). The discharge over a broad - crested weir is determined using:

$$Q = CLH^{3/2}$$

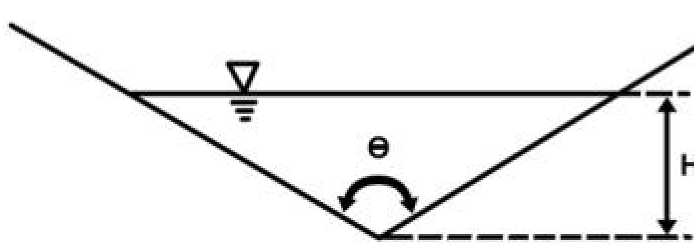


Figure 8.21 - Broad-Crested Weir

Discharge Coefficients (C)

The discharge coefficient can be obtained from figure 8.24. Depending on the shape of the weir and amount of head on the weir, the C value may range from 2.4 to 3.1. If the head greatly exceeds the width of the weir, the broad-crested weir will function as a sharp-crested weir, and the C may reach 3.32. For highway/dam embankments a C value of 2.8 to 3.0 is generally used.

Weir Length (L)

The length of the weir that is used in the equation should be reduced for any obstructions or high ground that may reduce the efficiency of the weir (see figure 8.22). Typically, broad-crested weirs will be relatively long. As a result, end contractions should not be a major factor. Equation (38) can be used to reduce the effective weir length due to end contractions.

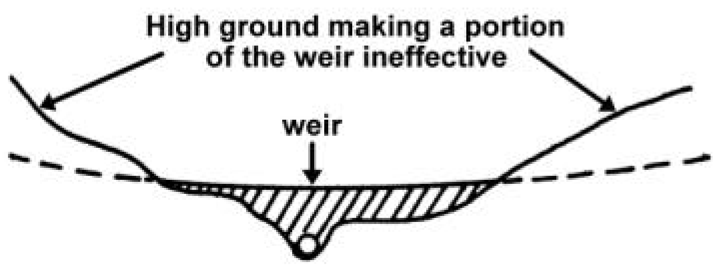


Figure 8.22 - Weir length Ineffective Due to High Ground

Submergence of Weir

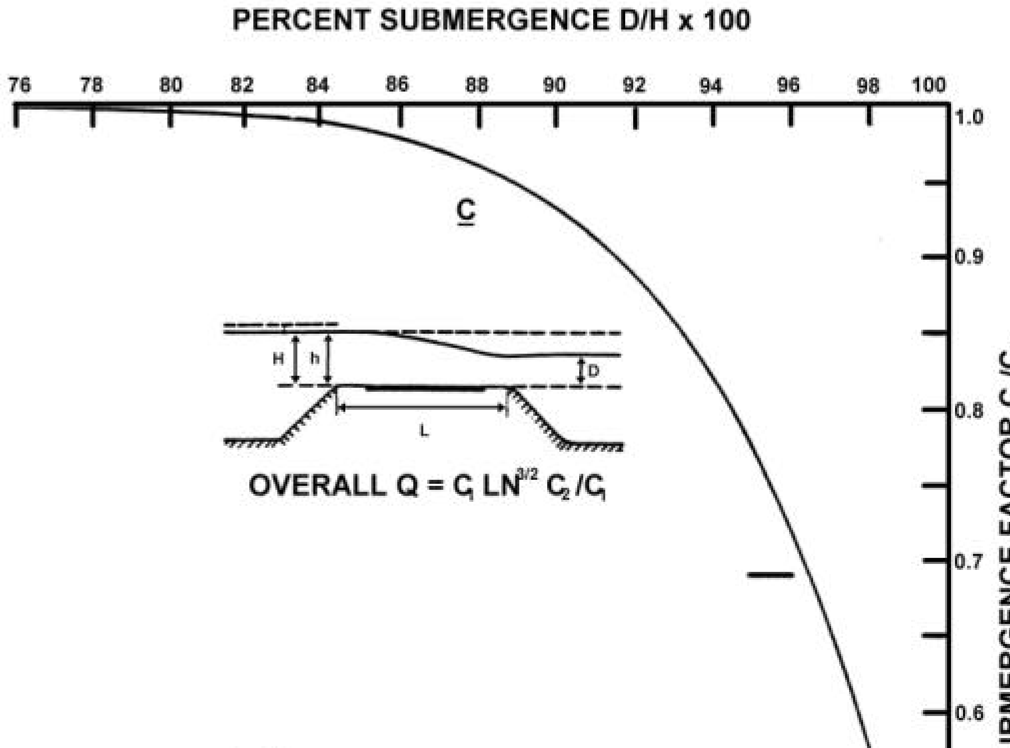
In addition to impacting the flow-carrying capacity of culverts, high tailwater can also impact weirs. The weir equations that have been shown assume that the weir is free flowing. However, if the tailwater rises high enough, the weir will be submerged, and the weir flow-carrying capacity will be reduced (see figure 8.23). If submergence occurs, the weir calculations will have to be adjusted to account for the high tailwater. For sharp-crested weirs, the discharge can be adjusted for submergence using the following equation:

$$Qs/Q = [1.0 - (hs/h)^n]^{0.385} \qquad (39)$$

where: **Qs** - discharge for a submerged weir
Q - discharge computed using weir equations
hs - tailwater depth above the weir crest
h - head upstream of the weir
n - exponent, 1.5 for rectangular, 2.5 for a triangular weir.

For broad-crested weirs, figure 8.23 can be used to adjust the computed discharge due to submergence.

Figure 8.23 - Submergence Factors



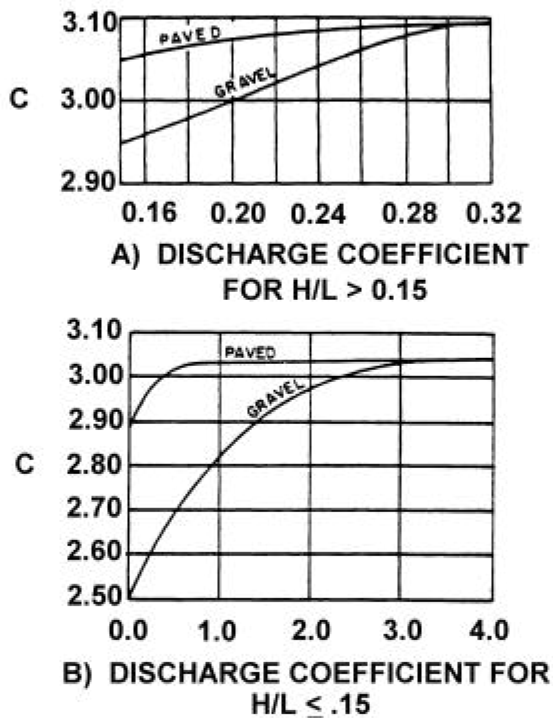


Figure 8.24 - Weir Discharge Coefficients for Road Embankments

(Source: FHWA, reference 12)

Sloping weirs

In many instances, an embankment or an emergency spillway will not be flat. In such cases it will be necessary to determine an h, to be used in the weir formula. One method that gives reasonable results involves breaking the weir into subsections and computing an average height for each of the subsections. In figure 8.25, the weir is broken into three subsections. The flow through each of the subsections is determined using:

$Q = CLH^{3/2}$

where: **H** - the average height within the subsection $(H_1 + H_2)/2$
L - length of the subsection
C - discharge coefficient

Once the flow is determined for each of the subsections, the total discharge is obtained by summing all of the subsections.

$Q_t = Q_1 + Q_2 + Q_3$

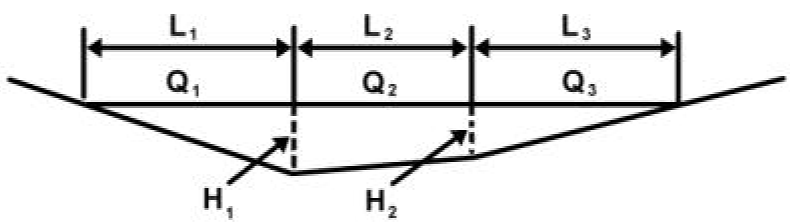


Figure 8.25 - Irregular Weir

Rating Curves

When a range of discharges are being analyzed, it will be necessary to develop a rating curve for the structure. A rating curve is a plot on a graph showing the upstream stage for a given discharge. The rating curve may be complicated if both weir flow and culvert flow will be occurring. The following steps can be used to develop a rating curve.

1. Assume an upstream water surface elevation.
2. Compute the discharge for the assumed WSEL.
3. (Compute both culvert and weir flows if each will be occurring at the assumed WSEL.)
4. Plot the results (discharge computed for the assumed WSEL).
5. Repeat until the curve is adequately defined.
6. Develop a "combined rating curve."

The rating curves for the culvert(s) and weir(s) are added together to form one rating curve. The "addition" can be done either graphically (see figure 8.28) or mathematically.

The following example will incorporate the orifice and weir equations that have been discussed, and a rating curve will be developed.

Example 8.7: Given the culvert and spillway configuration below, develop a rating curve for the structure up to 120 cfs. A tailwater rating curve computed in normal-depth example can be used to determine the tailwater.

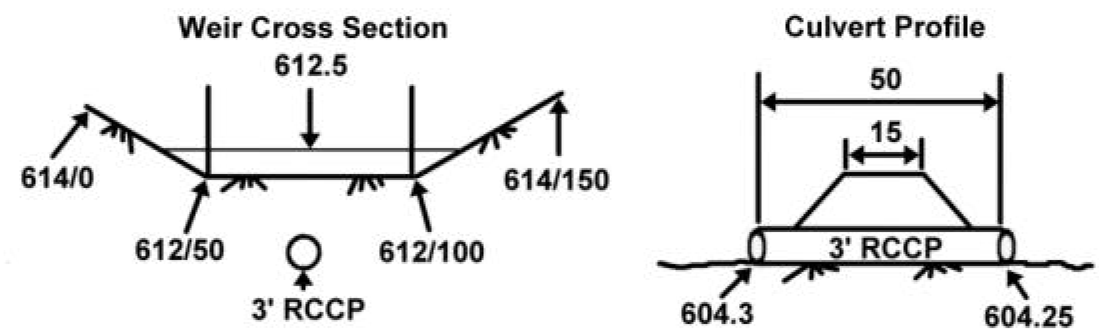


Figure 8.26 - Example Culvert and Spillway Configuration

1. Tailwater rating curve - (from example 8.1):

elevation	604.25	606.00	606.30	607.00	607.30	608.00	609.00
computed Q, cfs	0	34	50	95	120	200	425

2. Culvert Data & Rating Curve

total loss coefficient (K) = 1.81 (from example 8.3, page 124)
Area = 7.07 sq. ft. (from example 8.3)

Outlet Control computations:

equation (31): $H = K\,(Q/A)^2/2g$

$$= 1.81\,(Q/7.07)^2\,/2(32.2)$$

- rating curve based on outlet control computations

Q, cfs	34	50	80	95	120
H, ft	0.65	1.41	3.60	5.07	8.10

- inlet control (HW/D values) from nomograph (figure 8.14)

	inlet control		outlet control		
Q (cfs)	H (ft)	HW (ft)*	TWEL (ft)	H (ft)	HW (ft)*
0	0	604.3	604.3	0	604.3
34	3.0	607.3	606.0	0.65	606.7
50	4.1	608.4	606.3	1.41	607.7
80	7.7	612.0	606.8	3.60	609.4
95	9.5	613.8	607.0	5.07	612.1
120	14.4	618.7	607.3	8.10	615.4

*note: For inlet control, HW = H + 604.3 (upstream invert).
For outlet control, HW = H + TWEL.

In this example, inlet control will govern, as the computed headwater for inlet control conditions exceeds the outlet control headwater elevations. Figure 8.27 is a plot of the rating curve for the culvert.

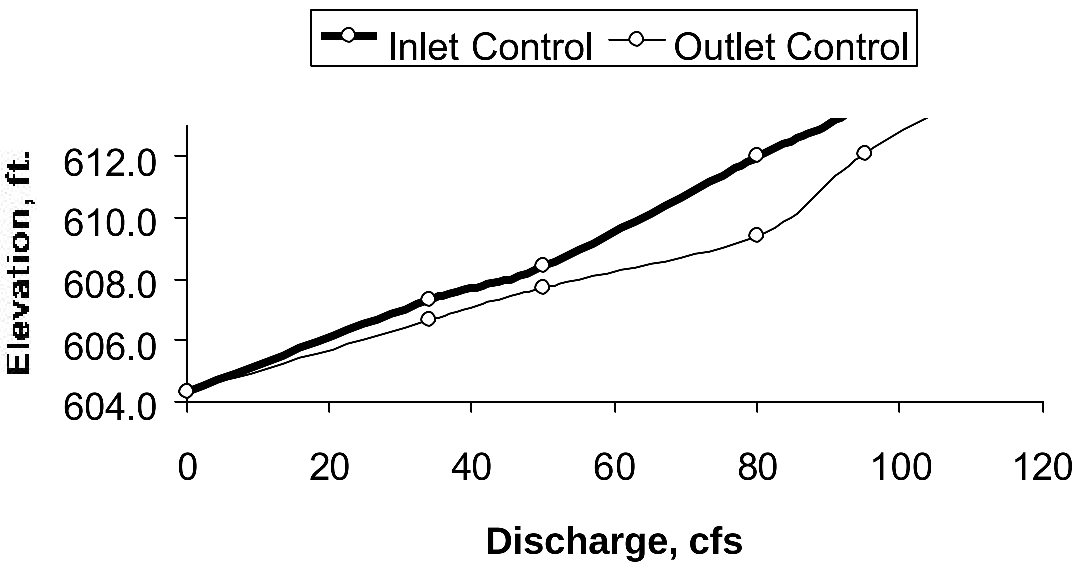


Figure 8.27 - Culvert Rating Curve

3. Weir data & rating curve

Compute discharges for various selected upstream stages using equation (36):

$Q = CLH^{3/2}$

- Select stage = **612.5 feet**, compute discharge for each subsection, (see figure 8.25).
  Select "C" values from figures 8.23 and 8.24.

$Q_{total} = Q_1 + Q_2 + Q_3 = 12.5(2.6)(.5/2)^{1.5} + 50(2.7)(.5)^{1.5} + 12.5(2.6)(.5/2)^{1.5}$

Q = **52 cfs** @ 612.5 feet

- Select stage = **613.0 feet**, compute discharge:

$$Q = 25(2.7)(1/2)^{1.5} + 50(2.82)(1)^{1.5} + 25(2.7)(1/2)^{1.5} = \mathbf{189\ cfs}$$

Additional stages can be selected to produce the following stage-discharge relationship:

Elevation, ft	612.0	612.5	613.0	613.5	614.0
Discharge, cfs	0	52	189	403	702

4. Combined Rating Curve

Once rating curves have been developed for the culvert and the weir, it is possible to add the two curves together to obtain a combined rating curve.

Elevation, ft	604.3	607.0	609.0	610.0	611.0	612.0	612.4	612.6
Culvert, cfs	0	30	55	63	72	80	84	85
Weir, cfs	0	0	0	0	0	0	36	80
Total Q, cfs	0	30	55	63	72	80	120	165

From the rating curve, the stage upstream of the structure for a discharge of 120 cfs, is 612.4. The culvert would be carrying 84 cfs, and the weir would be carrying 36 cfs. Figure 8.28 is a plot of the combined rating curve.

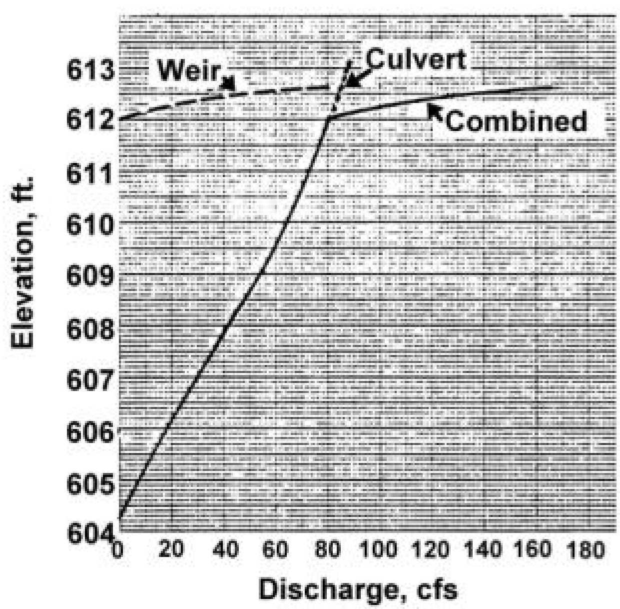


Figure 8.28 - Combined Rating Curve

Riser Pipes

In some instances, riser pipes may be used to control the outflow from a basin (see figure 8.29). The inlet will function as a weir until the weir is submerged from the effects of the downstream outlet structure (culvert). Once the weir is submerged, the culvert hydraulic will determine the rate of outflow discharge.

Following are some guidelines for the design of the riser pipe:

1. The area of the riser = 1.5 x area of the outlet pipe.
2. An anti-vortex wall is essential for the riser outlet to function as designed. For box risers, the anti-vortex wall consists of a back training wall and a dike extending from the wall to the embankment. For circular risers, the anti-vortex wall may be a splitter wall or a tangent wall (see figure 8.30). The height of these walls should be equal to the maximum headwater or two outlet pipe diameters, whichever is less.
3. The crest of the riser pipe should be a minimum distance of 2 times the diameter of the outlet pipe above the outlet invert (see figure 8.29).

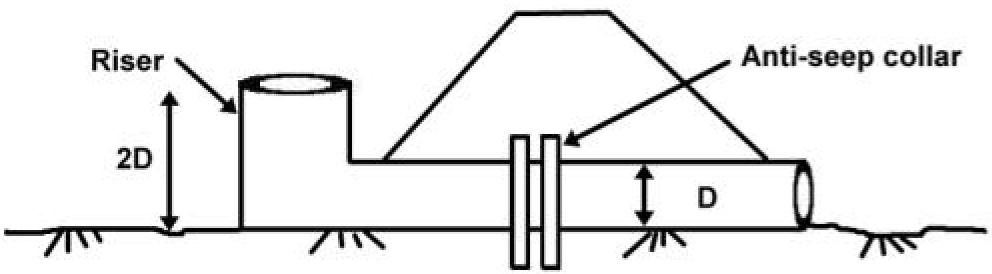


Figure 8.29 - Riser Pipe

Rectangular Riser **Circular Riser** **Circular Riser**

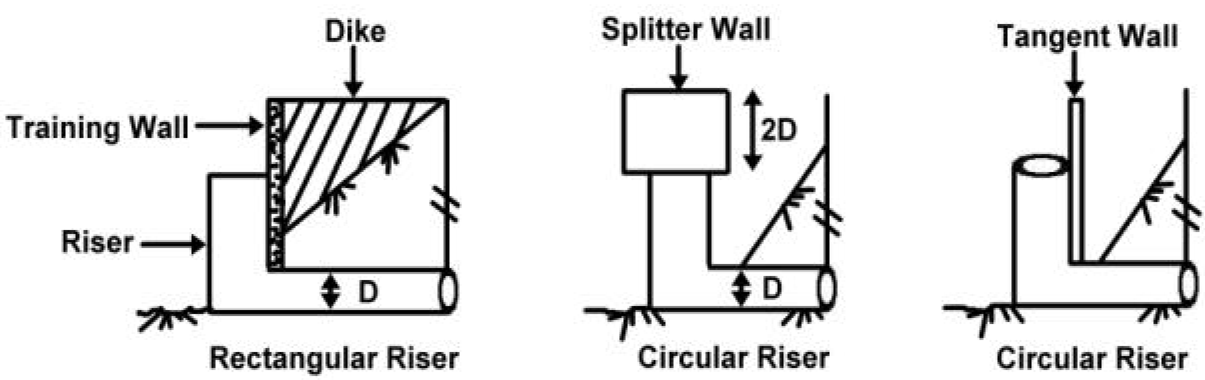


Figure 8.30 - Anti-Vortex Wall on Riser Pipe

The flow over the inlet can be determined using the weir equation:

$Q = CLH^{3/2}$

where: **C** - discharge coefficient, is a function of head. A value of 3.3 is reasonable for most heads on the weir. For more information on discharge coefficients for riser pipes, see reference 32.
L - weir length (see figure 8.31).
H - stage in feet above crest

The length to be used to compute the weir flow is computed to be :

L = Pi x D - obstructions (circular riser) (40)

where: **Pi** - 3.14156
D - inside diameter of pipe (cmp riser)
- inside diameter of female pipe joint (Reinforced concrete riser).

obstructions - include the thickness of the splitter wall (t).

L = W + 2B - obstructions (box riser) (41)

W - see figure 8.31
B - see figure 8.31

These are inside dimensions. If the riser lip is rounded, the dimensions are measured from the high point on the rounded lip.

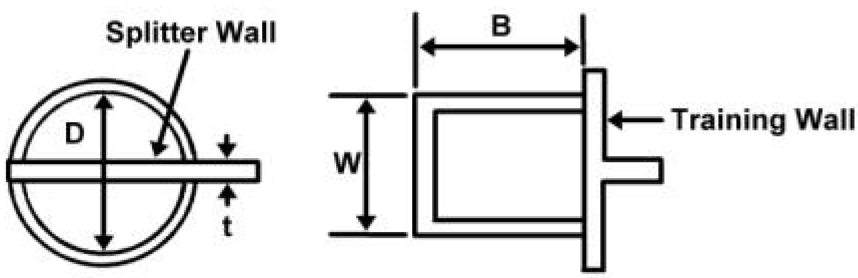


Figure 8.31 - Weir Length for Riser Pipe

As the discharge increases, the stage upstream of the riser pipe may eventually become controlled by the hydraulics of the outlet culvert. Thus, to establish a rating curve for an outlet structure that includes a riser pipe, it will be necessary to:

1. Establish a tailwater rating curve.
2. Establish a rating curve for the outlet culvert including the effects of the tailwater.
3. Establish a rating curve for the riser pipe including the effects of the backwater from the outlet culvert.
4. For a given discharge, the resulting detention-pond elevation can be obtained from the rating curve.

Example 8.8: Using the rating curve developed in example 8.7 above, develop a rating curve for a 4-foot diameter riser pipe (see figure 8.32) with crest at elevation 611.0 feet. The splitter wall is 4 inches thick.

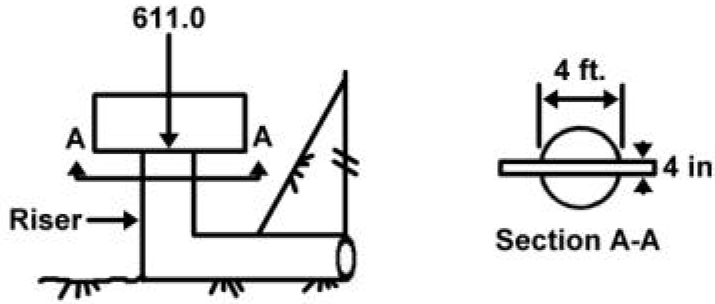


Figure 8.32 - Example Riser Pipe Problem

– **Rating Curve for Culvert** (From example 8.7)

Elevation, ft	604.3	607.0	609.0	610.0	611.0	612.0	612.4	612.6
Culvert, cfs	0	30	55	63	72	80	84	85

- Develop a Rating Curve for Riser

$Q = CLH^{3/2}$

$L = Pi \times D$ - obstructions

$= 3.3\{(Pi \times 4) - (4/12 \times 2)\}H^{3/2}$

$= 3.3\{11.9\}(Stage - 611.0)^{3/2}$

- **Rating Curve for Riser Pipe**

Stage, feet	611.0	611.5	612.0	612.4	612.6	613.0
H, feet	0.0	0.5	1.0	1.4	1.6	2.0
Q, cfs	0.0	14.0	39.0	65.0	79.0	111.0

Note: This rating curve does not include the effects of the backwater from the culvert.

- The riser pipe rating curve is plotted along with the culvert rating curve (figure 8.33).

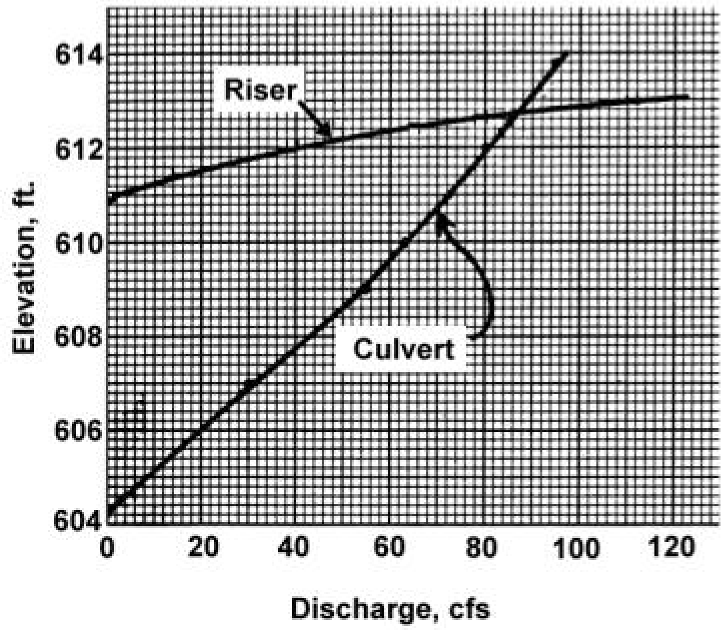


Figure 8.33 - Riser Pipe Rating Curve

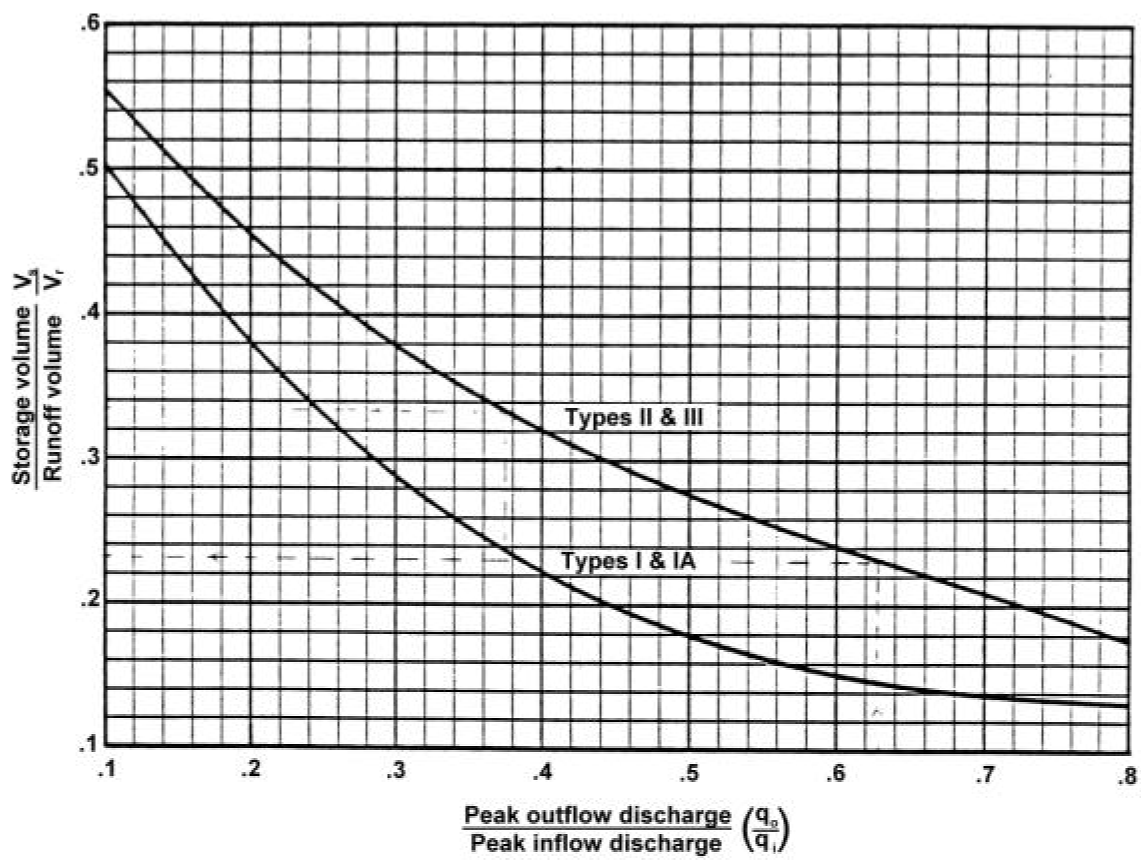
From the above figure, it can be seen that at about elevation 612.8 feet, the stage on the detention pond shifts from being controlled by the riser pipe to the outlet culvert. If the backwater caused by the outlet culvert is ignored, the outlet capacity of the structure may be significantly overestimated, and the potential for structural failure will increase.

CHAPTER 9: STORAGE-VOLUME REQUIREMENTS

As a watershed develops, there is potential for an increase in runoff and peak discharges. One of the most common techniques for minimizing the increase in runoff and discharges is the requirement of the construction of a detention pond. This section will cover the volume requirements and the routing techniques used in designing a detention pond.

Urban Hydrology for Small Watersheds, TR-55 (reference 46) contains a quick method for estimating the size of a detention pond. Figure 9.1 shows a relationship between the peak inflow and peak outflow to the storage volume and runoff volume. By knowing three values (such as runoff volume, peak inflow, and peak outflow), it is possible to estimate the fourth value (storage volume required).

In examples 7.2 and 7.3, a volume of runoff and a peak flow had been computed for a particular site. If peak outflows are to be reduced at this site, figure 9.1 can be used to estimate the storage required.



Where: V_s - Storage volume, acre-feet
V_r - Runoff volume, acre-feet
q_o - Peak outflow discharge, cfs
q_i - Peak inflow discharge, cfs

Figure 9.1 - Approximate Detention Basin Routing

(Source: Soil Conservation Service, Reference 46)

Example 9.1: The volume of runoff is computed to be 2.79 sq.mi.-inches (148.8 acre-feet). The peak inflow is 430 cfs. Estimate the storage volume needed to reduce the peak outflow to 270 cfs.

- Peak inflow	= 430 cfs
- Peak outflow	= 270 cfs
- Ratio of outflow to inflow	= 270 cfs/ 430 cfs = .628
- From figure 9.1, using type II rainfall, ratio of storage volume/runoff volume	= **0.233**
- Required storage volume	= 0.233 x 2.79 sq.mi.-inches
	= 0.650 sq. mi. - inches
	= 34.7 acre-feet use **35.0 acre - feet**

Example 9.2: If a detention pond containing 50 acre-feet of storage was being proposed on the above watershed, estimate the impact on the peak outflow.

- runoff volume = 2.79 sq.mi.-inches = 148.8 acre-feet
- storage volume provided = 50 acre-feet
- ratio of storage volume/runoff volume = 50/148.8 = 0.336
- from figure 9.1, peak outflow/peak inflow = **.372**
- peak inflow - 430 cfs
- thus: peak outflow = 0.372 x 430 cfs = **160 cfs**

where: **V_s - Storage volume, acre-feet**
V_r - Runoff volume, acre-feet
q_o - Peak outflow discharge, cfs
q_i - Peak inflow discharge, cfs

It is noted in the TR-55 manual that this procedure will result in storage volume requirements that are conservative. This technique may overestimate the volume requirement by as much as 25 percent. A detailed hydrologic analysis and routing would provide a more accurate estimate of the storage requirements and peak outflow.

ROUTING TECHNIQUE

There are different methods available to perform a routing through a detention pond. The primary idea behind a routing is to determine the impact that the detention pond will have on the inflowing flood peak by using the **continuity equation.** The equation can be thought of as **Inflow** (to the detention basin) minus **outflow** (from the detention basin) equals **change in storage** (in the detention basin). In equation form:

$$dt\,(I_{ave} - O_{ave}) = S \qquad (42)$$

where:
dt - a time interval
I_{ave} - average inflow during time interval
O_{ave} - average outflow during time interval
S - change in storage during time interval

Storage - Indication method

Of the various methods available to route a flood flow through a detention pond, the storage-indication method will be the only one discussed briefly in this guidebook. If additional information is needed on other routing techniques, it is suggested that the reader refer to the SCS's National Engineering Handbook Section 4 Hydrology (reference 49).

Equation 42 that has been rearranged is used in the storage-indication method:

$$I_{ave} + S_1/dt - O_1/2 = S_2/dt + O_2/2 \quad (43)$$

where:
- **S_1** - is the storage at t_1 (the beginning of the routing interval).
- **O_1** - is the outflow at t_1
- **S_2** - is the storage at t_2 (the end of the routing interval).
- **O_2** - is the outflow at t2.
- **I_{ave}** - is the average inflow for the time interval $(I_1 + I_2)/2$.

The following steps can be used to route a flood hydrograph through a detention basin by "hand", using the storage indication method.

1. **Develop a rating curve for the outlet structure**. The rating curve can be developed using procedures discussed in Chapter 8. The rating curve will show the discharge for a given elevation for the outlet structure. The discharge should be expressed in units that are consistent with the storage volume. Units of acre-feet will result in numbers that are considerably smaller than if cubic feet are used. (One acre-foot = 43560 cubic feet, one cubic foot/second = .083 acre-feet /hour).

2. **Develop storage-elevation curve for the detention pond**. The amount of storage available within the detention pond is computed for a range of elevations. The storage units should be consistent with discharge rating curve (such as acre-feet or cubic feet).

3. **Combine the curves developed** in steps 1 & 2 to form a relationship between storage and discharge. Table 9.1 is an example of storage volumes and discharges for a range of elevations.

Table 9.1 - Storage-Discharge Relationship

Elevation	Storage	Discharge
	100,000	
~~feet~~	~~cubic feet~~	~~cfs~~
604.3	0.0	0
606.0	8.3	61
607.0	13.1	74
608.0	17.9	87
609.0	23.1	100
610.0	28.3	200
611.0	33.5	420
612.0	38.8	750

4. **Select a routing interval (dt).** Typically for small watersheds the routing interval will be less than an hour, usually 0.25 hour to 0.5 hour.

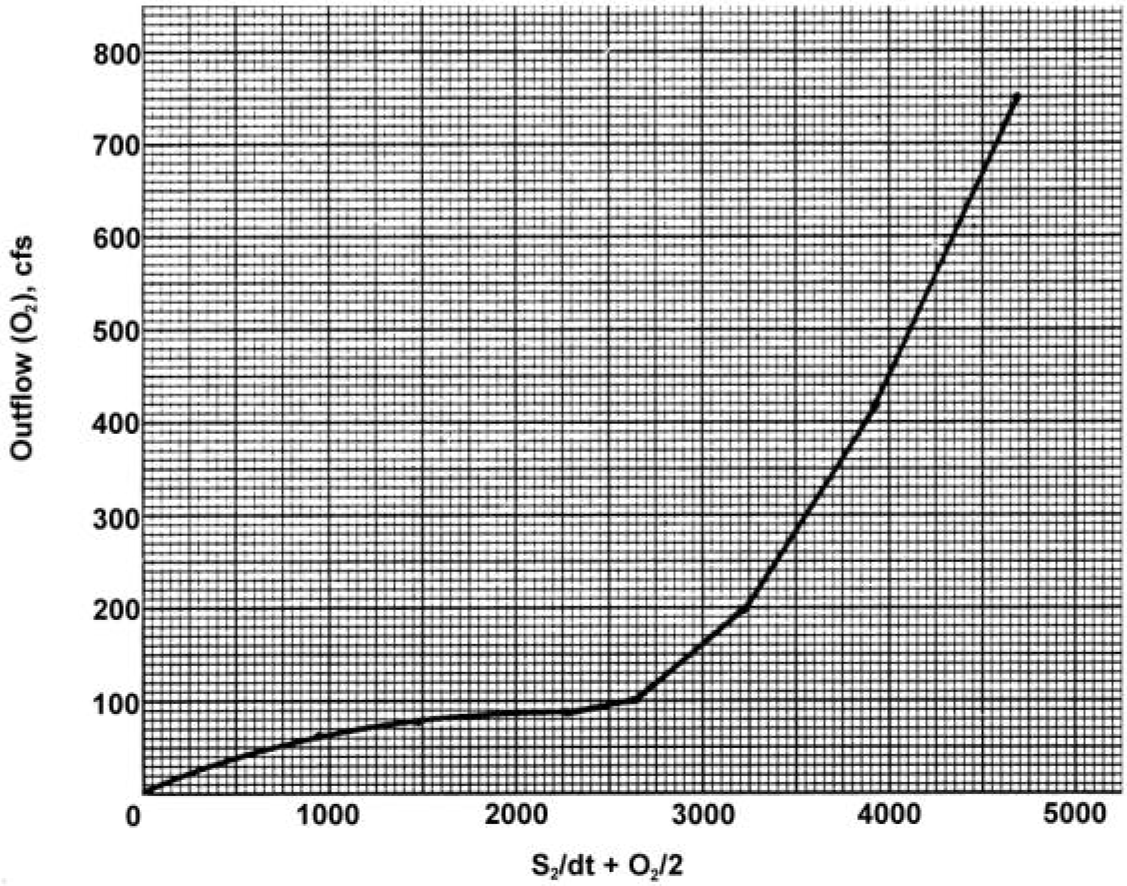
5. **Prepare the working curves**, and plot O_2 versus $S_2/dt + O_2/2$ (figure 9.2). Using the storage-discharge relationship in Table 9.1, an example working curve is developed below.

Table 9.2 - Working Curve (dt = 0.25 hrs. = 900 seconds)

Elevation	Discharge 0_2*	Storage S_2*	$0_2/2$	S_2/dt	$S_2/dt + 0_2/2$
604.3	0	0.0	0	0	0
606.0	61	8.3	30	**922	952
607.0	74	13.1	37	1456	1493
608.0	87	17.9	44	1989	2023
609.0	100	23.1	50	2567	2617
610.0	200	28.3	100	3144	3244
611.0	420	33.5	210	3722	3932
612.0	750	38.8	375	4311	4686

NOTES: * 02 is the outflow, cfs at the given elevation.
S2 is the storage 100,000 cubic feet, at the given elevation.
** S2/dt = 830,000 cubic ft/900 seconds = 922 cubic ft/second.

Where: **O_2 - Outflow in cfs**
S_2/dt - storage/routing interval, cubic feet/second



Where: O_2 - Outflow in cfs

S_2/dt - storage/routing interval, cubic feet/second

Figure 9.2 - Working Curve For Example

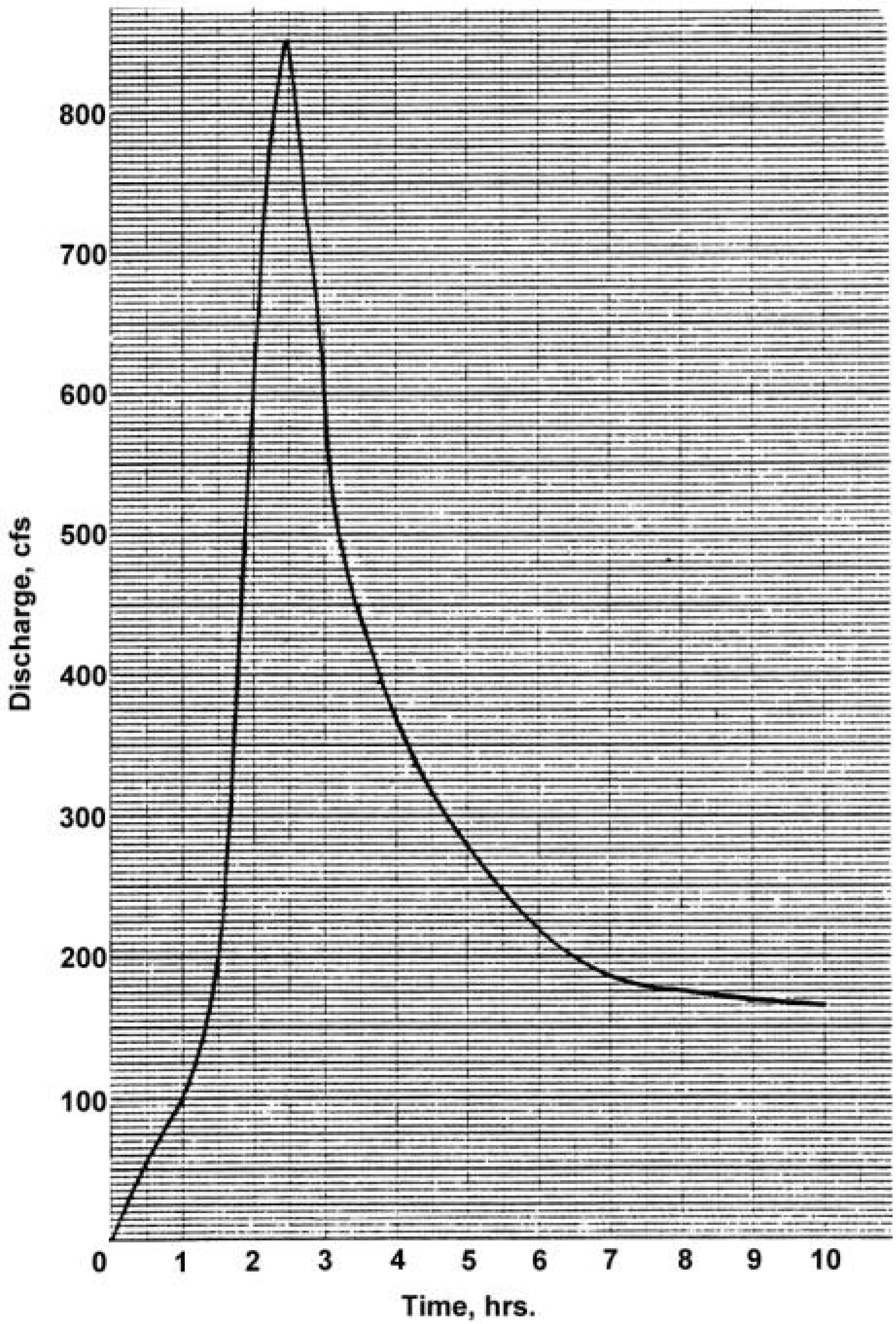


Figure 9.3 - Inflow Hydrograph for Routing Example

6. **Set-up operations table**. Column 1 contains the time at the selected increment, and column 2 contains the inflow taken from the inflow hydrograph (figure 9.3). The average inflow in column 3 is the average of the flow at the current time and the previous time interval. As an example, at a time of 0.50 hrs, the average inflow of 43 cfs is the average between 0.25 hrs (30 cfs) and 0.50 hrs (55 cfs).

Table 9.3 - Operations Table

(1) Time (hrs)	(2) Inflow (cfs)	(3) Avg. Inflow (cfs)	(4) $S2/dt=O_2$ (cfs)	(5) O_2 (cfs)
0.00	0	0		
0.25	30	15		
0.50	55	43		
0.75	80	68		
1.00	100	90		
1.25	138	119		
1.50	190	164		
1.75	380	285		
2.00	610	495		
2.25	785	698		
2.50	850	818		
2.75	730	790		
3.00	620	676		
3.25	485	552		
3.50	440	463		
3.75	395	418		
4.00	370	383		
4.25	335	353		
4.50	315	325		

The last two columns (4 & 5) will be completed during the routing.

7. **Route the inflow through the detention pond**.

   The routing would include:

   a) Determine inflow, storage, and outflow for initial conditions. In many cases, the initial inflow, outflow, and storage will be 0.
   b) Subtract outflow (column 5) from column 4 and add average inflow (column 3) for the next time increment. The computed value is placed in column 4 for the next time increment. (In the table below under initial conditions, columns 4 and 5 are each 0. At the time of 0.25 hours, the average inflow is 15 cfs. Column 4, at the time of 0.25 hours, is equal to 0 - 0 + 15 = 15). As a further example, at the time of 0.75 hours, column 4 shows a value of 123 cfs; from figure 9.2, the outflow in column 5 is 11 cfs; the average inflow at time 1.00 hour is 90 cfs. Column 4 at 1.00 hours is 123 - 11 + 90 = 202).

Operations Table

(1) Time (hrs)	(2) Inflow (cfs)	(3) Avg. Inflow (cfs)	(4) S2/dt+02 (cfs)	(5) 0 (cfs)
0.00	0	0	0	0
0.25	30	15	15	1
0.50	55	43	57	2
0.75	80	68	~~123~~	~~11~~
1.00	100	~~90~~	*202	

* (123 - 11 + 90)

c) From the plot of $S_2/dt + O_2$ vs. O_2, determine the outflow O_2, for the computed value of $S_2/dt + O_2$. As examples, from figure 9.2, when $S_2/dt + O_2 = 123$, the outflow is 11 cfs; when $S_2/dt + O_2 = 202$, $O_2 = 19$ cfs.

Operations Table

(1) Time (hrs)	(1) Time (hrs)	(1) Time (hrs)	(1) Time (hrs)	(1) Time (hrs)
0.00	0.00	0.00	0.00	0.00
0.25	0.25	0.25	0.25	0.25
0.50	0.50	0.50	0.50	0.50
0.75	0.75	0.75	0.75	0.75
1.00	1.00	1.00	1.00	1.00

d) Repeat the steps until routing is complete.

The results of the partial routing indicate that the peak inflow has been reduced from a discharge of 850 cfs, to an outflow of 433 cfs. From the outlet rating curve (table 9.1) the maximum stage on the detention pond is 611.1 feet.

If required, the routing could be continued until the entire outflow hydrograph is developed.

Operations Table

(1)	(2)	(3)	(4)	(5)
Time	**Inflow**	**Avg. Inflow**	**S2/dt+02**	**0**
(hrs)	(cfs)	(cfs)	(cfs)	(cfs)
0.00	0	0	0	0
0.25	30	15	15	1
0.50	55	43	57	2
0.75	80	68	123	11
1.00	100	90	202	19
1.25	138	119	302	27
1.50	190	164	439	33
1.75	380	285	724	50
2.00	610	495	1169	70
2.25	785	698	1797	81
2.50	**850**	818	2534	95
2.75	730	790	3229	200
3.00	620	675	3704	343
3.25	485	552	3913	413
3.50	440	463	3963	**433**
3.75	395	418	3948	422
4.00	370	383	3809	408
4.25	335	353	3854	392

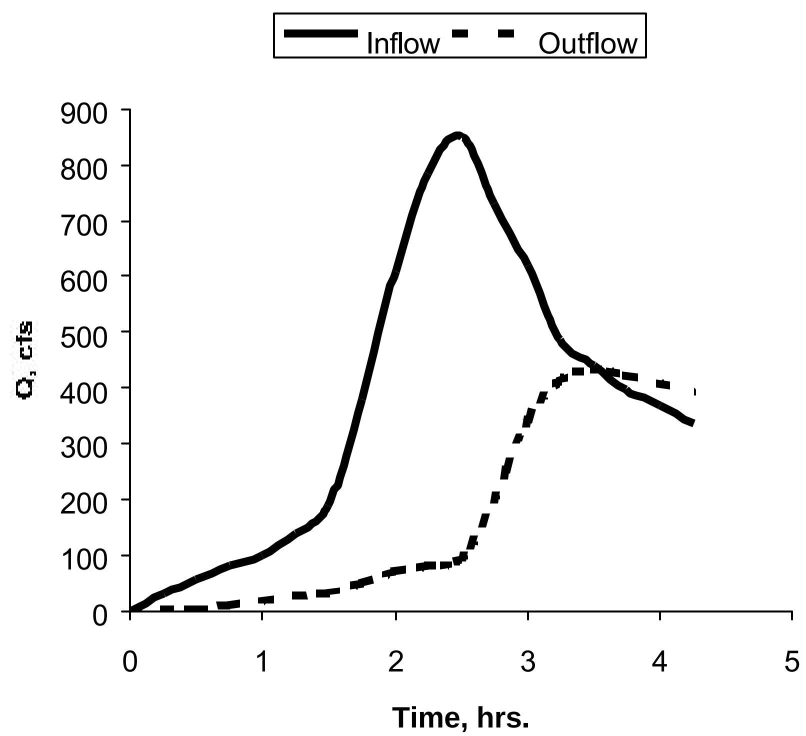


Figure 9.4 - Partial Inflow and Outflow Hydrographs for Routing Example

COMPUTER PROGRAMS

There are computer programs available that will perform detailed hydrologic analyses and routing. A big advantage to using a computer program is that once the computer model is "set-up", numerous options and scenarios can be analyzed with very little additional effort. The more popular computer programs include:

1. HEC-1 - developed by the Corps of Engineers. The program will generate hydrographs and perform routing (reference 44). This program can be downloaded at: *link removed, no longer valid*.
2. TR-20 - developed by the Soil Conservation Service. The program will generate hydrographs and perform routing (reference 47).
3. DAMBRK - developed by the National Weather Service. The program is an unsteady-state model (flow rate is not assumed to be constant) which combines hydraulic and hydrologic techniques. It is able to accurately model flood-waves as they move downstream. Originally developed to model flood-wave produced by dam failures, the program can be used to route flows down channels, through basins, and will produce a flood profile (reference 15).
4. ILLUDAS - originally developed at the Road Research Laboratory in England, and later enhanced by the Illinois State Water Survey. The program uses a simplified routing technique, the runoff the characteristics from the basin, along with the rainfall to size storm sewers, to compute the storage needed to prevent sewer capacity from being exceeded. Suggested as a preliminary planning tool. (reference 43).
5. SWMM - Storm Water Management Model - developed under the sponsorship of the U.S. Environmental Protection Agency is a comprehensive urban stormwater analysis model that computes runoff, pollutant transport, detention storage, and treatment. The model is complex and is not really suited for analysis of a single detention pond, but is more applicable for analysis of complete storm sewer systems. (reference 42).
6. Others - In recent years other models have been developed that also generate runoff hydrographs, and design detention basins. Technical magazines such as "Civil Engineering" will typically carry information and advertisements relating to new developments in stormwater management modeling.

No matter which computer program that is used in the analysis and design of a detention basin, it is still the designer's responsibility to become completely familiar with the program, and to check the results. The results provided by the computer program are a function of the experience of the user and the data input. At no time should computer programs be used "blindly"; the results should **not** be treated as gospel.

CHAPTER 10: OPERATION & MAINTENANCE

OPERATION & MAINTENANCE

A stormwater-management system that is functioning properly will require regular maintenance. If sediment and other particulates are not being deposited and retained, the basin is not providing any water-quality benefits. For a detention/retention system to function as designed, it is essential that a regular inspection and maintenance program be in place. The operation & maintenance of the stormwater facilities should not be limited to occurring only when complaints are received. Instead, the maintenance program should be both "preventive" and "corrective." In addition to responding to complaints, there should also be a "preventive" portion of the inspection program which can discover small maintenance problems that can be solved with a minimal amount of effort.

There are a wide variety of maintenance problems that can be associated with detention facilities including weed & grass control, sedimentation, erosion, and outlet blockage.

The results of a survey by the American Public Works Association, 1980 are shown in table 10.1.

Table 10.1 - Maintenance Problems of Detention Facilities

Problem Type	Relative Degree of Severity 100 - most severe 0 - no problem
Weed Growth	100
Maintaining Grass	93
Sedimentation	87
Bank Deterioration	79
Mosquito Control	77
Outlet Stoppage	76
Soggy Surfaces	71
Inflow Water Pollution	69
Algal Growth	68
Fence Maintenance	66
Unsatisfactory Emergency Spillway	60
Dam Failure, Leakage	55

Some of these problems can be minimized during design. However, no matter how well a facility is designed and constructed, maintenance will be required.

Following is a list of items that should be included in a maintenance program:

1. Inspection of Outlet

Blockages. A common source of pond failure is blockage of the outlet structure. A blocked outlet will reduce the outflow capacity of a basin, and will increase the chance of structural failure. In an extended detention facility, a blocked outlet will result in shallow water being stored. Shallow water will cultivate significant weed and algae growth, and will make maintenance extremely difficult.

Thus, any inspection program must regularly check the outlet structure for blockages due to sediment or debris. Once discovered, the blockages must be removed. The inspection should be conducted at least monthly, and more frequently during the spring runoff season.

Structural Condition. The outlet structure should be inspected for cracks and spalling (deterioration) of the concrete, erosion of the embankments, differential settlement, seepage, and scour at the inlet/outlet. Any of these problems could lead to failure of the outlet structure if not corrected. The structural condition should be inspected at least annually, or following a major flood event. The Dam Safety Guidebook (reference 28) prepared for the Federal Emergency Management Agency provides some good guidance relative to inspection of the structure. A copy of this guidebook may be available from the DEQ, Land and Water Management Division, Dam Safety Unit (telephone # 517-284-5570). Even though the Guidebook was prepared with dams in mind, many of the terms and checklists are applicable to the outlet structures for detention basins.

2. Dredging and Removal of Sediment.

Typically sediment will have to be removed every 5 to 10 years. The maintenance schedule will vary from basin to basin, as it is dependent on the watershed, and the size of the basin. A recommended practice in designing a detention basin is to include extra volume of storage to account for the volume of sediment that will be deposited in the basin. How much "extra" volume was included in the basin will be a factor in determining the frequency of clean-out. In addition, if development is occurring in the watershed, it will likely be necessary to remove sediment more frequently, as increased development will increase sediment.

It is extremely important that the design of the basins include an access point, which will allow the removal of sediment. It is also important to have a place to put the dredged materials, either on-site or off-site.

3. Mowing.

The detention ponds can be maintained as a meadow, which would require mowing at least twice a year. However, in residential areas, the mowing frequency may have to be increased to 10 to 14 times a year for "aesthetic" reasons. Thus, mowing can be a large maintenance expense. It is suggested that slow-growing, water-tolerant species, such as K-31 tall fescue and crownvetch be used to minimize the need for mowing.

4. Algae and Aquatic Plants

Since wet detention ponds will receive and store stormwater that contains nutrients, they will be able to support algae and aquatic plants. A properly designed wet detention pond will limit the plant growth to the edges of the pond.

It will be virtually impossible to eliminate the growth of algae in a wet detention pond. To try to control the algae growth it is possible to:

a) "Harvest" the algae through the use of special machinery.

b) Chemicals are available that control the growth of algae. However, the use of chemicals can contaminate the receiving waters, and thus should be avoided if possible.

c) In some instances, minnows and small fish have been used to control the growth of algae. The introduction of fish will require that the pond be designed to support fish over the winter.

d) Install a mechanical aerator to reduce odors and the growth of algae.

e) Drain the pond and clean out the bottom, which will remove the nutrients that are responsible for the growth of algae.

Extended detention basins should not experience problems with the growth of algae, as water is not retained in the basin. However, if the bottom of the basin is constructed without a slope, the bottom may remain wet, and wetland vegetation may begin to grow. If it is desired to maintain the bottom of the basin, the bottom slope should be constructed with at least a 2% slope.

5. Fences

In some instances fences are used to limit access to the basin or the outlet structure. As a safety precaution, the fence should be inspected periodically to be sure that it is functioning as it was intended.

Figure 10.1 gives a sample checklist that may be used to identify problem areas and to recommend solutions. It is suggested that about 3% to 5% of the construction cost of the facility be allocated annually to finance the maintenance program.

FINANCING OF STORMWATER-MANAGEMENT FACILITIES

A major factor in the success or failure of a stormwater-management facility is the availability of adequate finances to operate and maintain the facility. Historically, local governments and drain commissioners have been responsible for solving local drainage problems. The funding for the drainage work has usually been in the form of property taxes or a special assessment district based on contributing drainage area. Typically, maintenance of drainage structures have been given a low priority primarily due to limited funding.

CHECK LIST
OPERATION AND MAINTENANCE INSPECTION RECORD

Name of Project ______________________ Date of Inspection ______________

Project Location ____

Type of Inspection ____

Reservoir Inspection: Satisfactory ______________ Unsatisfactory ____________

Item	**Acceptable**	**Unacceptable**	**Required Maintenance**
1. Vegetation			
2. Fences			
3. Principal Spillway			
4. Trash Racks			
5. Gates, Valves or Stoplogs			
6. Diversion Structure			
7. Energy Dissipators			
8. Reservoir Area			
9. Embankment Conditions			
10. Fill Areas			
11. Condition of Concrete			
12. Outlet Channel			
13. Pump Station			

REMARKS:

Signature of Inspector ________________________

Figure 10.1 - Maintenance Check List

(Source: references 28 & 35)

STORMWATER UTILITY

Throughout the country, various communities have developed methods of funding the maintenance of stormwater-management facilities. One method that is being utilized is the creation of a stormwater utility. These utilities provide services of flood control, drainage, and stormwater management, and are financed with user charges (reference 22). The user fees are typically based upon the runoff that would be anticipated from the property. In other words, a commercial property with paved parking lots would be required to pay more than a residential development due to the greater runoff potential.

The stormwater utility is different from property taxes in that tax-exempt properties (churches, schools, etc.) would be assessed the "user fee." Based on a 1990 survey by the Maryland Department of Environment (reference 34) the median stormwater-utility annual charge for single family residences was $25.80; the charges ranged from $12.84 in Roseville, Minnesota, to $89.40 in Bellevue, Washington. (The City of Ann Arbor had an annual utility charge of $18.24).

For nonresidential parcels, it is difficult to set a "typical" rate, as rates vary with the degree of impervious area. Some communities charge per square foot of impervious area (Louisville charges $1.75 per 2500 square feet of impervious area), while others charge based on type of development (see table 10.2).

There are a wide variety of methods that a stormwater-management utility can use to assess a "user fee." The degree of impervious area is considered in most methods. Whatever method is used, the primary benefit of the utility is a stable source of funds available for the operation and maintenance of the stormwater management system.

Table 10.2 - Rate Schedule for the City of Seattle

Class	Impervious Surface Percentage	Rate
Residential		$ 26.07/parcel/yr
Very Light	0-10	$ 26.07/parcel/yr
Light	10-20	$ 60.83/acre/yr
Moderate	20-45	$126.01/acre/yr
Moderate heavy	45-65	$242.33/acre/yr
Heavy	65-85	$308.51/acre/yr
Very Heavy	85-100	$404.10/acre/yr
County Roads	NA	$ 90.44/acre/yr
State Highways	NA	$ 66.85/acre/yr

Special Property Tax

In the City of Novi, Oakland County, voters passed a 1/2-mill property-tax increase for stormwater facility maintenance. The City also collects fees from developers for connecting to the stormwater system. Obviously, the biggest "hurdle" in using a special property tax is getting the approval of the voters to pass the millage. Before asking voters to vote to increase their taxes, it will be necessary to try to educate the public to the concept and benefits of stormwater management.

Lump-Sum Payment by Developers

At the time a facility design is reviewed and approved, the community may require an "front-end" payment which is earmarked for the specific site. The payment can be in the form of a permanent maintenance deposit which is invested and the interest used to fund all future maintenance costs. The other approach is a payment to cover all maintenance for a given period of time, such as 10 years.

Special Assessment District

Special assessment districts may be established by the local government, or they may be established by the County Drain Commissioner under the Michigan Drain Code, 1956 P.A. 40, as amended. Under this concept, property owners within an established drainage district are assessed a fee for the maintenance of stormwater management facilities.

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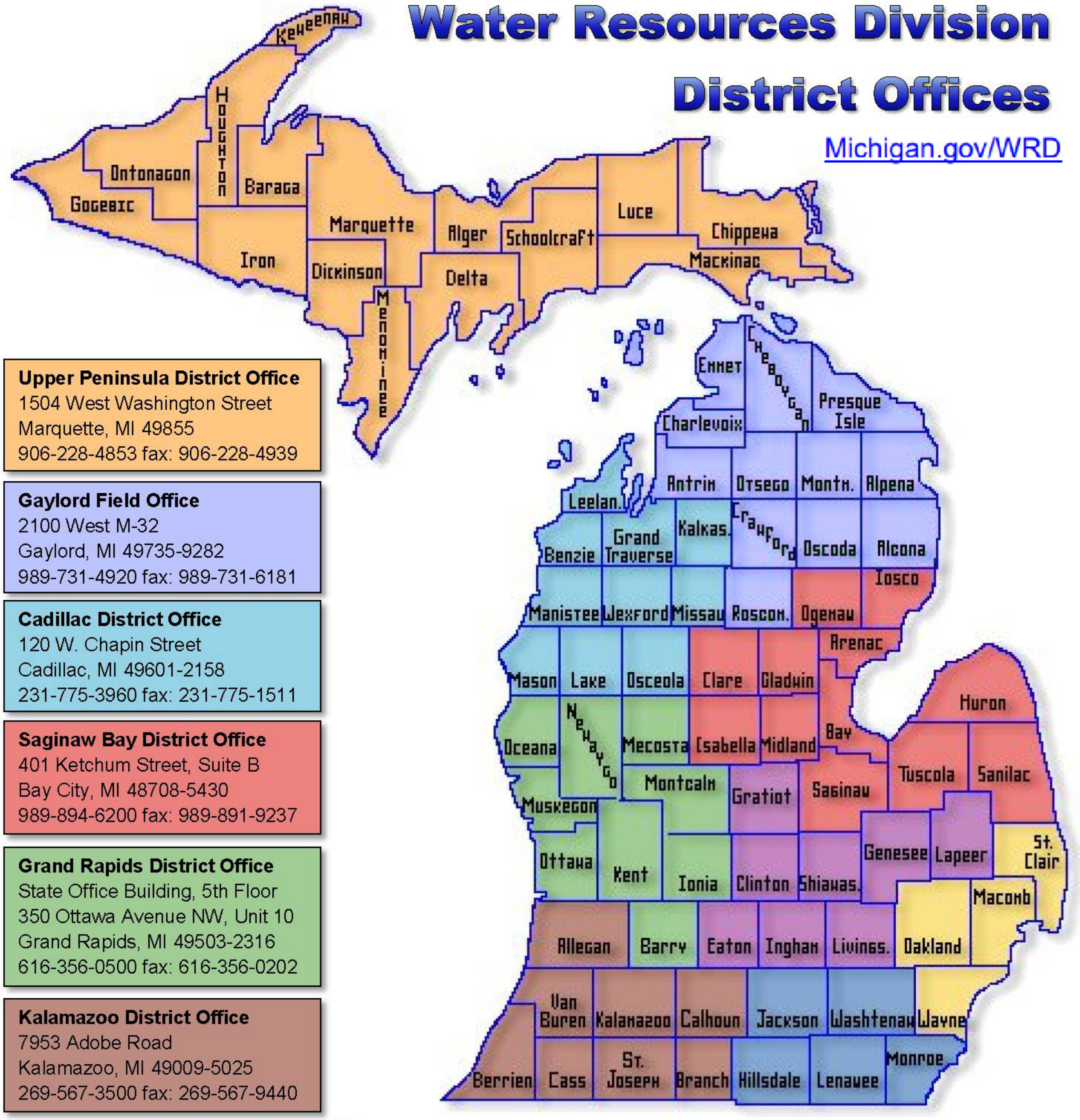
APPENDIX A - DEQ OFFICES

https://www.michigan.gov/-/media/Project/Websites/egle/Documents/Programs/WRD/About-Us/district-offices.pdf

Water Resources Division

District Offices

Michigan.gov/WRD



Upper Peninsula District Office
1504 West Washington Street
Marquette, MI 49855
906-228-4853 fax: 906-228-4939

Gaylord Field Office
2100 West M-32
Gaylord, MI 49735-9282
989-731-4920 fax: 989-731-6181

Cadillac District Office
120 W. Chapin Street
Cadillac, MI 49601-2158
231-775-3960 fax: 231-775-1511

Saginaw Bay District Office
401 Ketchum Street, Suite B
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989-894-6200 fax: 989-891-9237

Grand Rapids District Office
State Office Building, 5th Floor
350 Ottawa Avenue NW, Unit 10
Grand Rapids, MI 49503-2316
616-356-0500 fax: 616-356-0202

Kalamazoo District Office
7953 Adobe Road
Kalamazoo, MI 49009-5025
269-567-3500 fax: 269-567-9440

Lansing District Office
525 W. Allegan (Constitution Hall, 1S)
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586-753-3700 fax: 586-753-3751

DEQ Water Resources Division 517-284-5567

APPENDIX B

RAINFALL FREQUENCY FOR MICHIGAN

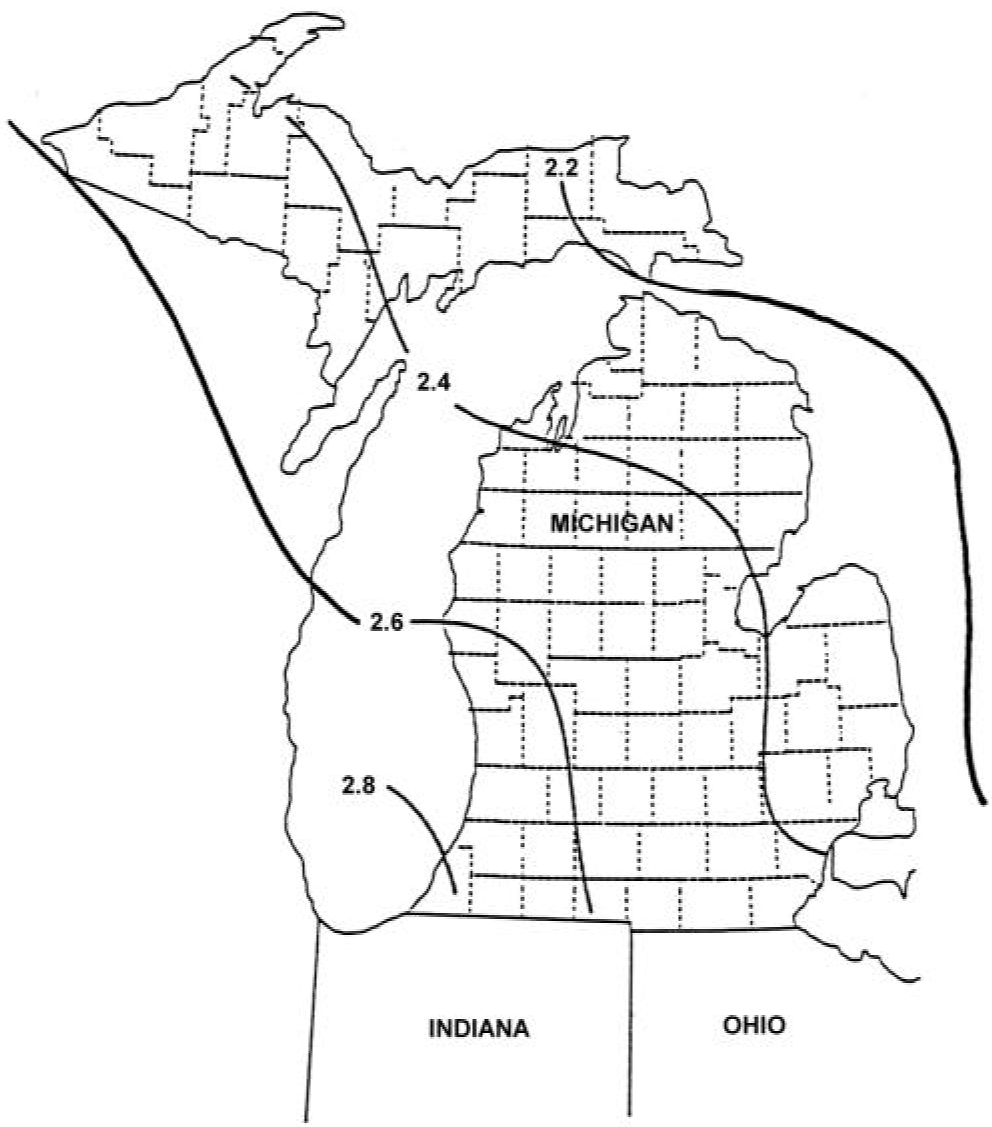


Figure B.1 - 2-year, 24-hour Rainfall

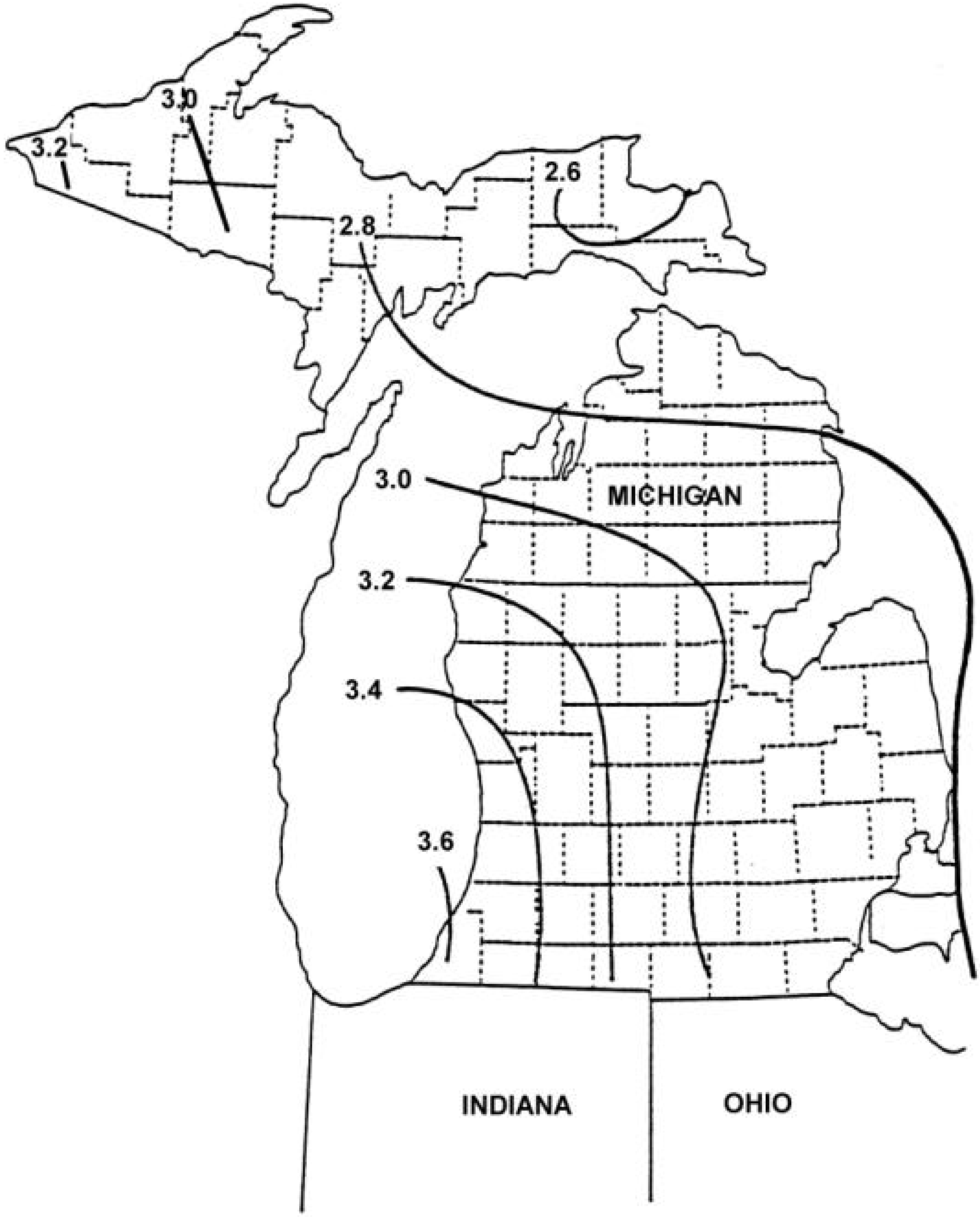


Figure B.2 - 5-year, 24-hour Rainfall

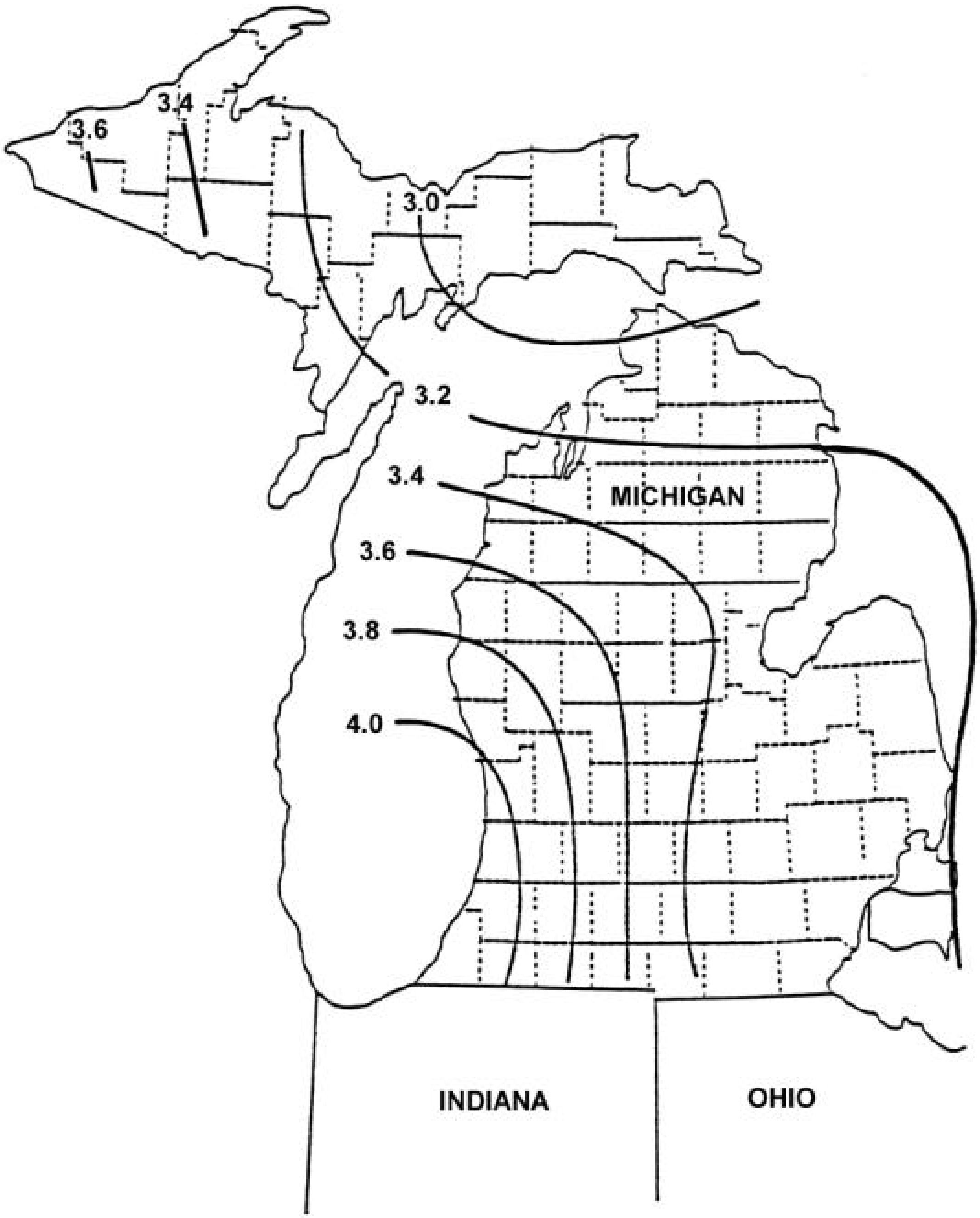


Figure B.3 - 10-year, 24-hour Rainfall

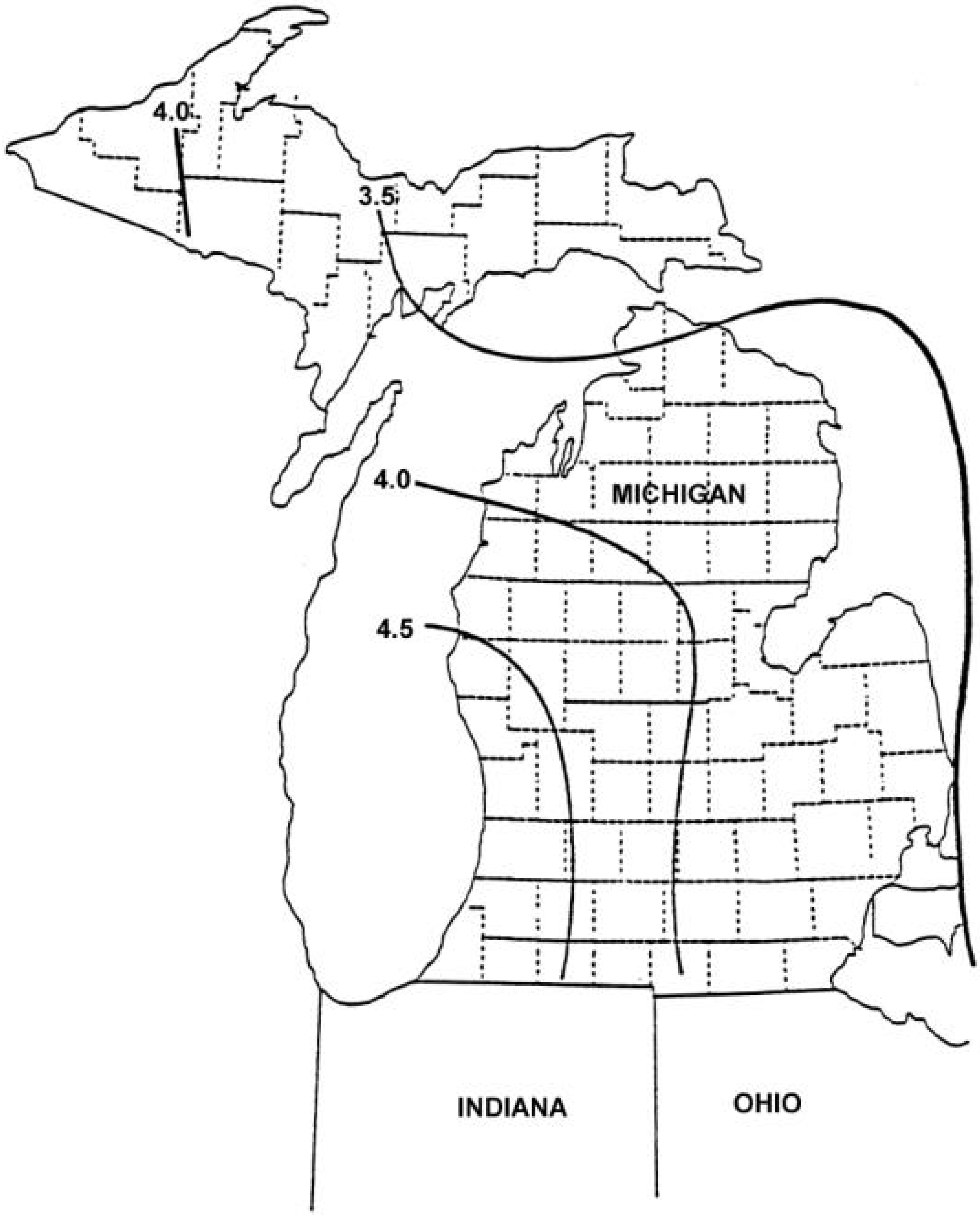


Figure B.4 - 25-year, 24-hour Rainfall

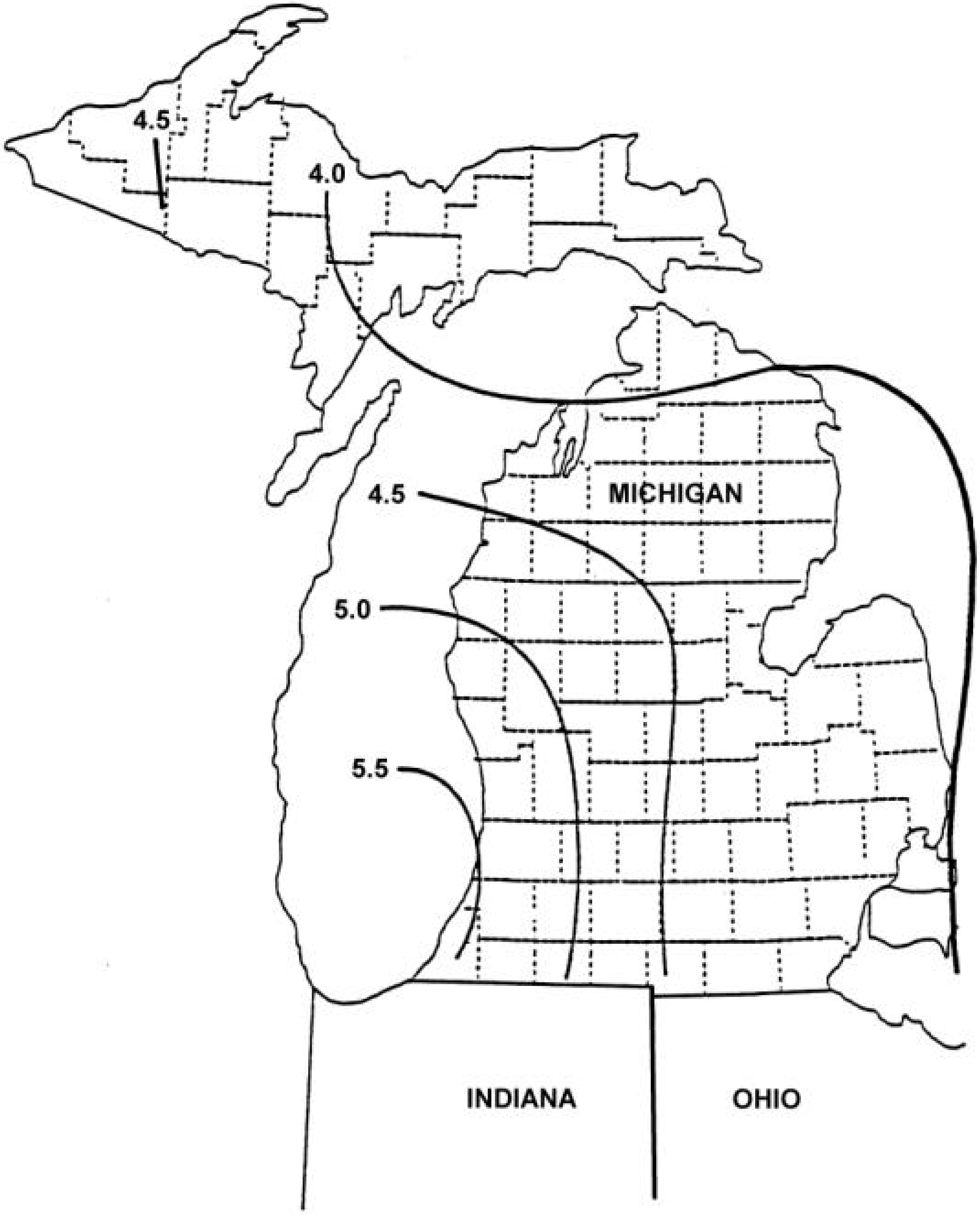


Figure B.5 - 50-year, 24-hour Rainfall

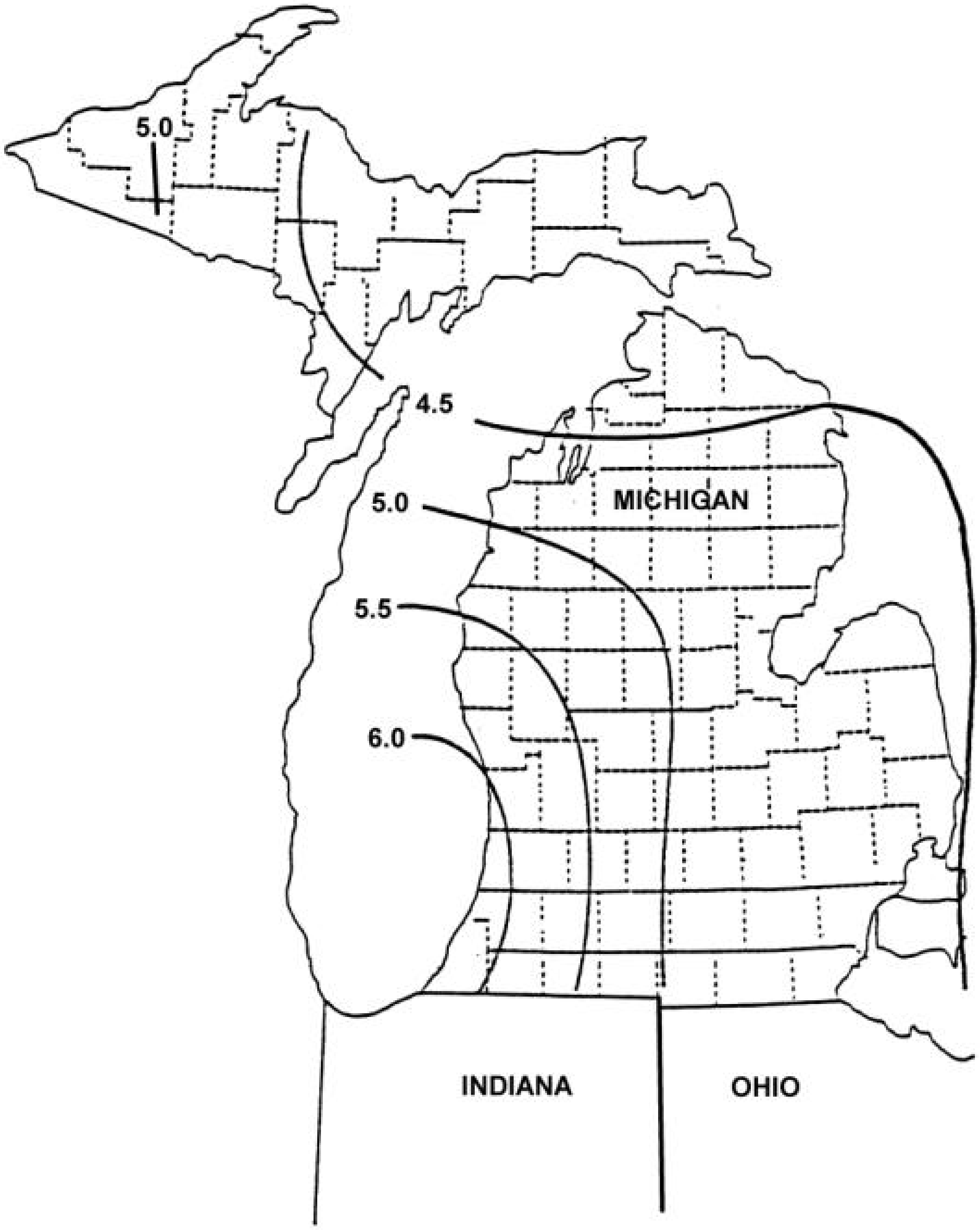


Figure B.6 - 100-year, 24-hour Rainfall

APPENDIX C - HYDROLOGIC SOIL GROUPS FOR MICHIGAN SOILS

soil series	Hyd. Group	soil series	Hyd. Group	soil series	Hyd. Group	soil series	Hyd. Group
Abbaya	B	Blue Lake	A	Chesaning	B	Elmdale	B
Abscota	A	Bohemian	B	Chestonia	D	Elston	B
Adrian	D/A	Bonduel	C	Chippeny	D	Elyers	D/B
Alcona	B	Bono	D	Cohocrab	D/B	Emmet	B
Algansee	B	Boots	D/A	Coloma	A	Ensign	D
Allendale	B	Borski	B	Colonville	C	Ensley	D/B
Allouez	B	Bowers	C	Colwood	D/B	Epoufette	D/B
Alpena	A	Bowstring	D/A	Conover	C	Epworth	A
Alstad	C	Boyer	B	Coral	C	Ermatinger	D/B
Amasa	B	Brady	B	Corunna	D/B	Esau	A
Angelica	D/B	Branch	B	Coupes	B	Escanaba	A
Arkona	B	Brassar	C	Covert	A	Essexville	D/A
Arkport	B	Breckenridge	D/B	Crosier	C	Evart	D
Arnheim	D	Brems	A	Croswell	A	Fabius	B
Ashkum	D/B	Brevort	D/B	Cunard	B	Fairport	C
Assinins	B	Brimley	B	Cushing	B	Fence	B
Aubarque	D/C	Bronson	B	Dawson	D/A	Fibre	D/B
Aubbeenaubbee	B	Brookston	D/B	Deer Park	A	Filion	D
Au Gres	B	Bruce	D/B	Deerton	A	Finch	C
Aurelius	D/B	Burleigh	D/A	Deford	D/A	Fox	B
Avoca	B	Burr	D	Del Rey	C	Frankenmuth	C
Bach	D/B	Cadmus	B	Detour	B	Frenchefts	B
Badaxe	S	Capac	C	Dighton	B	Freda	D
Banat	B	Carbondale	D/A	Dixboro	B	Froberg	D
Barry	D/B	Carlisle	D/A	Dora	D/B	Fulton	D
Battlefield	D/A	Caasopolis	B	Dowagiac	B	Gaastra	C
Beavertail	D	Cathro	D/A	Dresden	B	Gagetown	B
Beechwood	C	Celina	C	Dryburg	B	Gay	D/B
Belding	B	Ceresco	B	Dryden	B	Geneses	B
Belleville	D/B	Champion	B	Duel	A	Gilchrist	A
Benona	A	Channahon	D	Dungridge	B	Gilford	D/B
Bergland	D	Channing	B	East Lake	A	Gladwin	A
Berville	D/B	Charity	D	Eastport	A	Glawe	D/B
Biscuit	D/B	Charlevoix	B	Edmore	D	Glendora	D/A
Bixby	B	Chatham	B	Edwards	D/B	Glynwood	C
Bixlet	C	Cheboygan	B	Eel	B	Gogebic	B
Blount	C	Chelsea	A	Eleva	B	Gogomain	D/B

(Source: References 39 and 49)

Appendix C - Hydrologic Soil Groups for Michigan Soils

soil series	Hyd. Group	soil series	Hyd. Group	soil series	Hyd. Group	soil series	Hyd. Group
Goodman	B	Kalkaska	A	Macomb	B	Mussey	D/B
Gotham	D/B	Kallio	C	Mancelona	A	Nadeau	S
Grace	B	Karlin	A	Manistee	A	Nahma	D/B
Granby	D/A	Kawbawgam	C	Manitowish	B	Napoleon	D/A
Graftan	A	Kakkawlin	C	Markey	D/A	Nappanee	D
Graveraet	B	Kendallville	B	Marlefts	B	Nester	C
Graycalm	A	Kent	D	Martinsrills	B	Net	C
Grayling	A	Keowns	D/B	Martisco	D/B	Newaygo	B
Greenwood	D/A	Kerston	D/A	Matherton	B	Newton	D/A
Grindstone	C	Keweenaw	A	Maumee	D/A	Nottawa	B
Grousehaven	D	Kibble	B	McBride	B	Nunica	C
Guardlake	A	Kidder	B	Mecosta	A	Oakville	A
Guelph	B	Kilmanagh	C	Melita	A	Ockley	B
Gutport	D	Kingsville	D/A	Menagha	A	Oconto	B
Hagensville	C	Kinross	D/A	Menominee	A	Ocqueoc	A
Halfaday	A	Kiva	A	Metyin	D/A	Ogemaw	D/C
Hatmaker	C	Klacking	A	Metamora	B	Okee	B
Henrietta	D/B	Kokomo	D/B	Metma	B	Oldman	C
Hessel	D/B	Koontz	D	Miami	B	Olentangy	D/A
Herringer	D/C	Krakow	B	Michigamme	C	Omega	A
Hillsdale	B	Lacora	D/B	Millsdale	D/B '	Omena	B
Hodenpyl	B	Lamson	D/B	Milton	C	Onaway	B
Houghton	D/A	Landes	B	Minoa	C	Onota	B
Hoytville	D/C	Lapesr	B	Minocqua	D/B	Ontonagon	D
Huntington	B	Larry	D	Minong	D	Ormas	B
Ingalls	B	Leelanau	A	Misery	C	Oshtemo	B
Ingersoll	B	Lenawee	D/B	Mitiwanga	C	Otisco	A
Ionia	B	Leoni	B	Moltke	B	Ottokee	A
Iosco	B	Liminga	A	Monico	C	Owosso	B
Isabella	B	Linwood	D/A	Monitor	C	Paavola	B
Ishpeming	A	Locke	B	Montcalm	A	Padus	B
Ithaca	C	Lode	B	Moquah	B	Palms	D/A
Jacobsville	D	Londo	C	Morley	C	Parkhill	D/B
Jeddo	D/C	Longtie	B	Morocco	B	Paulding	D
Jesso	C	Loxley	D/A	Mudsock	D/H	Pelkie	A
Johnswood	B	Lupron	D/A	Munising	B	Pella	D/B
Kalamazoo	B	Mackinac	B	Munuscong	D/B	Pemene	B

Two soil groups such as D/B indicates the undrained/drained condition

(Source: References 39 and 49)

Appendix C - Hydrologic Soil Groups for Michigan Soils

soil series	Hyd. Group	soil series	Hyd. Group	soil series	Hyd. Group	soil series	Hyd. Group
Pence	B	Roselms	D	St. Ignace	D	Wakefield	B
Pendleton	C	Rousseau	A	Stambaugh	B	Wallace	B
Pequaming	A	Rubicon	A	Steuben	B	Wallkill	D/C
Pertin	B	Rudyard	D	Sturgeon	B	Warners	D/C
Perrinton	C	Ruse	D	Sugar	B	Wasepi	S
Peri:	D	Saganing	D/A	Summerville	D	Washtenaw	D/C
Peshekee	D	Sanilac	B	Sundell	B	Watton	C
Petticoat	B	Saranac	D/C	Sunfield	B	Waucedah	D
Pewamo	D/C	Sarona	B	Superior	D	Wauseon	D/B
Pickford	D	Satago	D	Tacoosh	D/B	Wautoma	D/B
Pinconning	D/B	Saugatuck	C	Tallula	B	Wega	B
Pinnebog	D/A	Saylesville	C	Tamarack	B	Wes=bury	C
Pipestone	B	Sayher	A	Tappan	D/B	Whalan	B
Plainfield	A	Scalley	B	Tawas	D/A	Wheatley	D/A
Pleine	D	Schoolcraft	B	Teasdale	B	Whitaker	C
Ponozzo	C	Sebewa	D/B	Tedrow	B	Whitehall	B
Posen	B	Selfridge	B	Tekenink	B	Willette	D/A
Poseyville	C	Selkirk	C	Thetford	A	Winneshiek	B
Potagannissing	D	Seward	B	Thomas	D/B	Winterfield	D/A
Poy	D	Shebeon	C	Tobico	D/A	Wisnet	D/S
Proctor	B	Shelldrake	A	Toledo	D	Witbeck	D/B
Randolph	C	Shelter	B	Tonkey	D/B	Wixom	S
Rapson	B	Shiawassee	C	Toogood	A	Wolcott	D/B
Remus	B	Shinrock	C	Trenary	B	Woodbeck	B
Rensselaer	D/B	Shoals	C	Trimountain	B	Yalmer	B
Richter	B	Sickles	D/B	Tula	C	Ypsi	C
Riddles	B	Sims	D	Tuscola	B	Zeba	B
Rifle	D/A	Sisson	B	Tustin	B	Ziegenfuss	D
Riggsville	C	Skanee	C	Twining	C	Zilwaukee	D
Rimer	C	Sleeth	C	Tyre	D/A	Zimmerman	A
Riverdale	A	Sloan	D/B	Ubly	B		
Rockbottom	B	Solona	C	Velvet	C		
Rockcut	B	Soo	D/C	Vestaburg	D/A		
Rodman	A	Sparta	A	Vilas	A		
Ronan	D	Spinks	A	Volinia	B		
Rondeau	D/A	Springlake	A	Wainola	B		
Roscommon	D/A	St. Clair	D	Waiska	B		

Two soil groups such as D/B indicates the undrained/drained condition

(Source: References 39 and 49)

APPENDIX D - METHOD FOR ESTIMATING "n" VALUES

The following is a procedure that may be used in estimating "n" values to be used in open-channel hydraulic computations. The procedure is discussed in further detail in reference 8, and involves selecting a basic value and then, through a series of steps, modifying the value to account for irregularity, variation in size, obstructions, vegetation, and meander.

Step 1. Selection of the basic n value (n_1) for a straight, uniform, smooth channel. The basic n value is determined based only on the materials forming the channel.

Channel material	Basic n
Earth	0.020
Cut in rock	0.025
In fine gravel	0.024
In coarse gravel	0.028

Step 2. Selection of modifying value for surface irregularity (n_2). The selection is based on the degree of roughness or irregularity of the surfaces of channel sides and bottom.

Degree of irregularity	Surfaces comparable to	Modify value
Smooth	The best obtainable for the materials involved.	**0.00**
Minor	Good dredged channels; slightly eroded or scoured side slopes of canals or drainage channels.	**0.01**
Moderate	Fair to poor dredged channels; moderately sloughed or eroded side slopes.	**0.01**
Severe	Badly sloughed banks of natural channels; unshaped, jagged and irregular surfaces of channels excavated in rock.	**0.02**

(Source: "Guide for Selecting Roughness Coefficient "n" Values For Channels," USDA, Soil Conservation Service, December 1963, and Reference 8)

Step 3. Selection of modifying value for variations in shape and size of cross sections (n_3). Shape changes causing the greatest turbulence are those in which the main flow shifts from side to side in short distances.

Character of variation in size and shape of cross sections
Modifying value

Changes in size or shape occurring gradually	**0.000**
Large and small sections alternating occasionally or shape changes causing occasional shifting of main flow from side to side	**0.005**
Large and small sections alternating frequently or shape changes causing frequently or shape changes causing frequent shifting of main flow from side to side	**0.010** to **0.015**

Step 4. Selection of modifying value for obstructions (n_4). Based on the presence of obstructions such as debris, stumps, boulders, and logs.

Relative effect of obstructions	~~**Modifying value**~~
Negligible	0.000
Minor	0.010 to 0.015
Appreciable	0.020 to 0.030
Severe	0.040 to 0.060

Step 5. Selection of modifying value for vegetation (n_5).

Vegetation and flow conditions comparable to:	**Degree of effect on n**	**Range in modifying value**
Dense growths of flexible turf grasses, where the average depth of flow is 2 to 3 times the height of vegetation.	Low	**0.005** to **0.010**
Turf Grasses where the average depth of flow is 1 to 2 times the height of vegetation. Stemmy grasses, weeds or tree seedlings with moderate cover where the average depth of flow is 2 to 3 times the height of vegetation. Brushy growths, moderately dense, along side slopes of channel, little vegetation along the channel bottom.	Medium	**0.010** to **0.025**
Turf grasses where the average depth of flow is about equal to the height of vegetation. Dormant season, willow or cottonwood trees 8 to 10 years old, inter-grown with some weeds and brush, no vegetation is in foliage. Growing season, some weeds in full foliage along side slopes, little vegetation along the channel bottom.	High	**0.050** to **0.100**
Turf grasses where depth of flow is less than one half the height of vegetation. Growing season, weeds and brush in full foliage along side slopes; dense growth of cattails along channel bottom.	Very high	**0.050** to **0.100**

Step 6. Determination of the modifying value for meandering of channel (n_6). The value is multiplied by the sum total of the basic n value and the modifying values determined in steps 1 through 5.

~~Ratio lm/ls~~	~~Degree of meandering~~	~~Modifying value~~
1.0 to 1.2	Minor	0
1.2 to 1.5	Appreciable	0.15 n_s
1.5 and greater	Severe	0.30 n_s

where: **l_m** - the meander length of the reach
l_s - the straight length of the channel in the reach
n_s - sum of steps 1 through 5 ($n_1+n_2+n_3+n_4+n_5$)

Step 7. Add all of the values together to obtain the n value for the reach.

Computed n value = ($n_1+n_2+n_3+n_4+n_5+n_6$)

Example estimation of "n" value

(1)	channel in earth	0.020
(2)	banks moderately sloughed	0.010
(3)	gradual size changes	0.000
(4)	Minor obstructions	0.010
(5)	Low vegetation	~~0.005~~
	subtotal (ns)	0.045
(6)	Appreciable meander (.15ns)	~~0.007~~
	computed n value	**0.052**

APPENDIX E

CRITICAL DEPTH CHARTS

Figure No.	Description
E.1	Circular Section
E.2	Rectangular Section
E.3	Corrugated Metal Pipe Arch

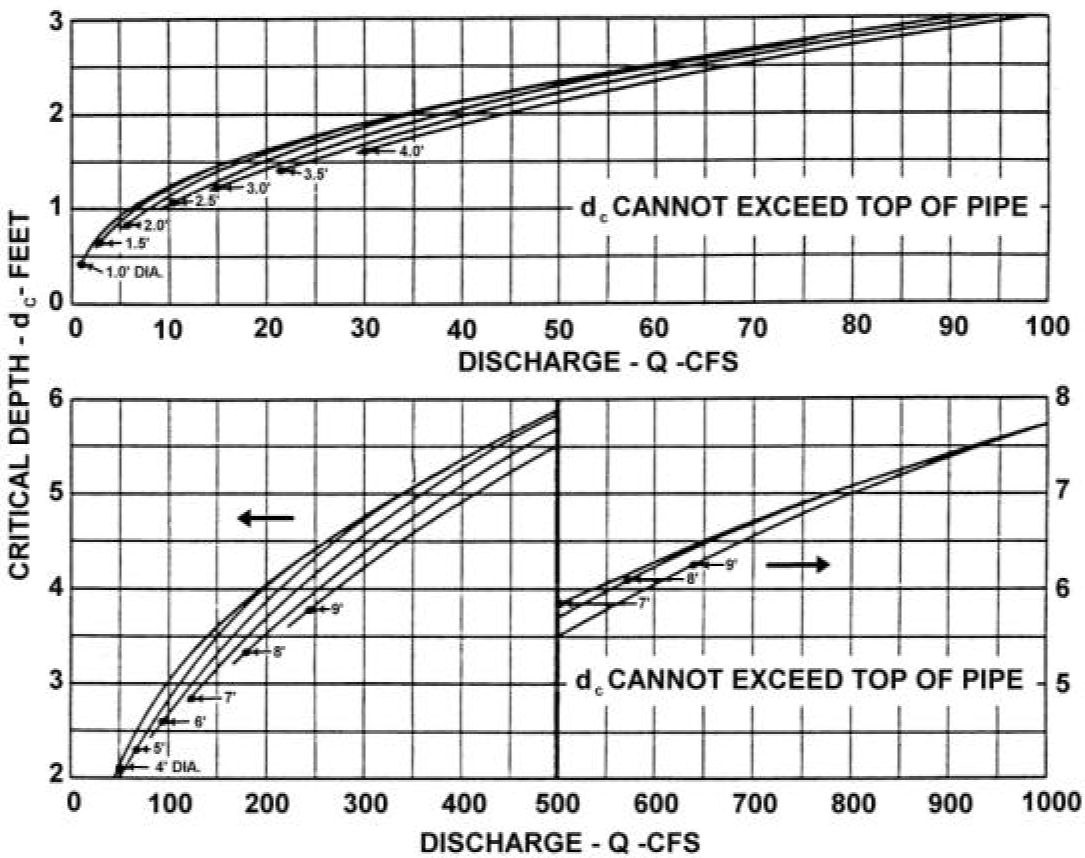


Figure E.1 - Critical Depth Chart for Circular Pipe

(Source: Reference 12)

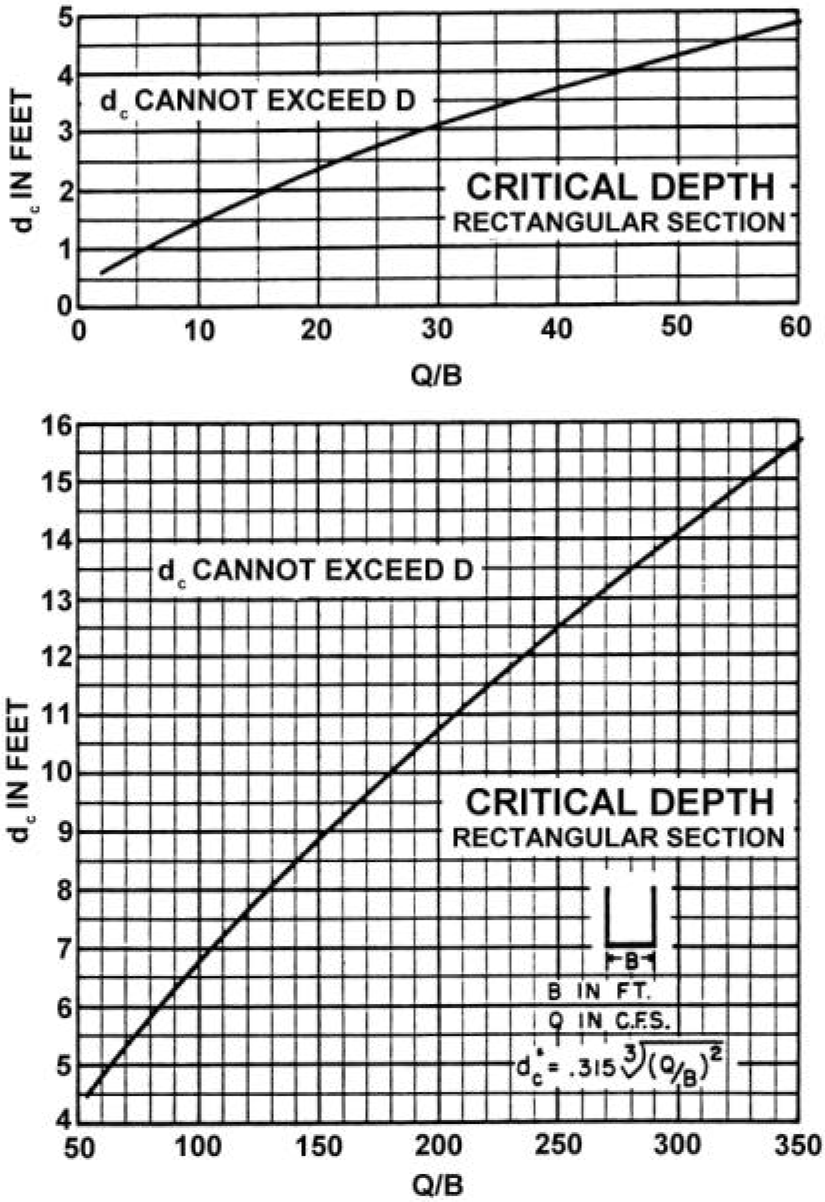


Figure E.2 - Critical Depth for Rectangular Section

(Source: Reference 12)

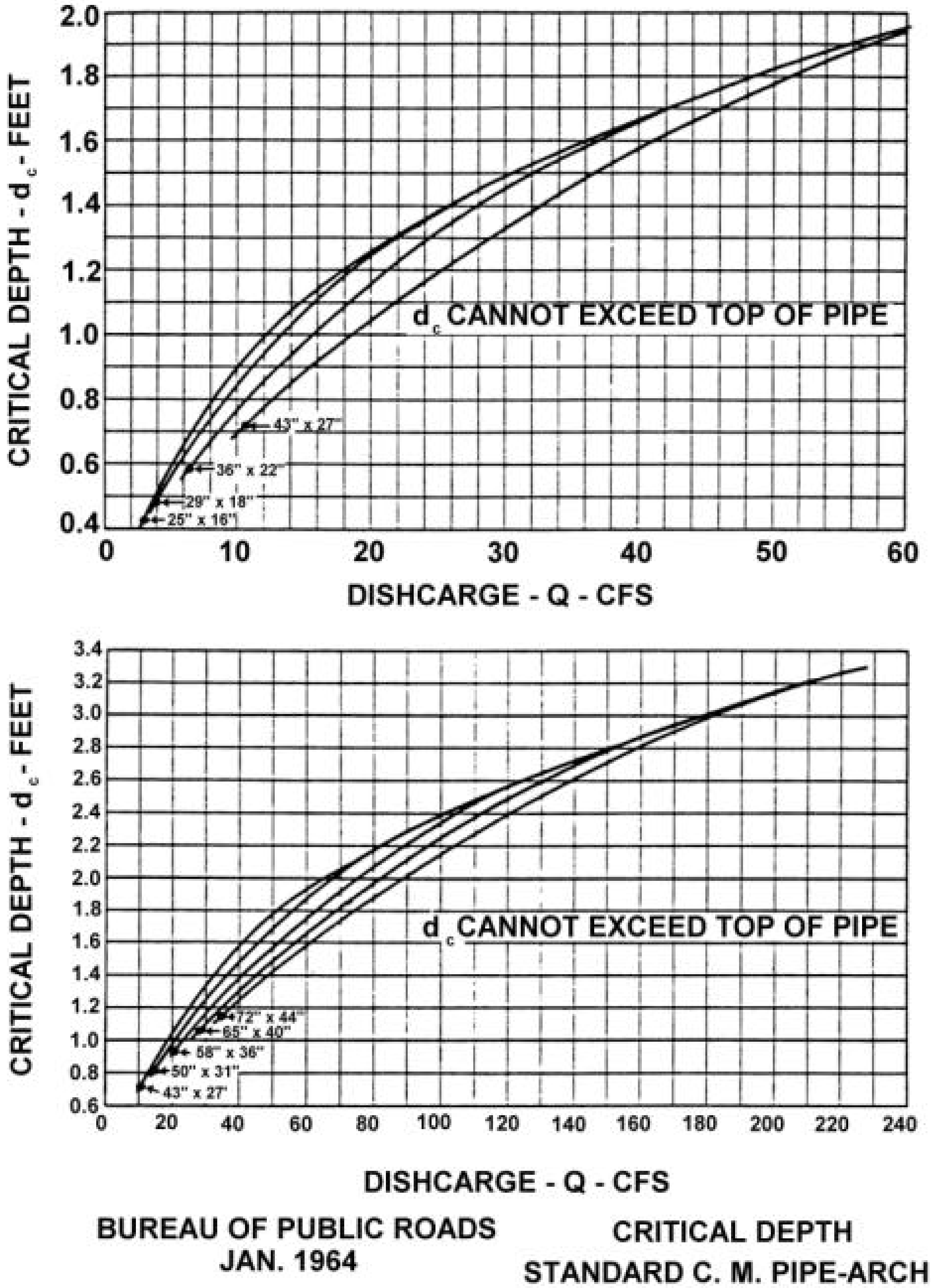
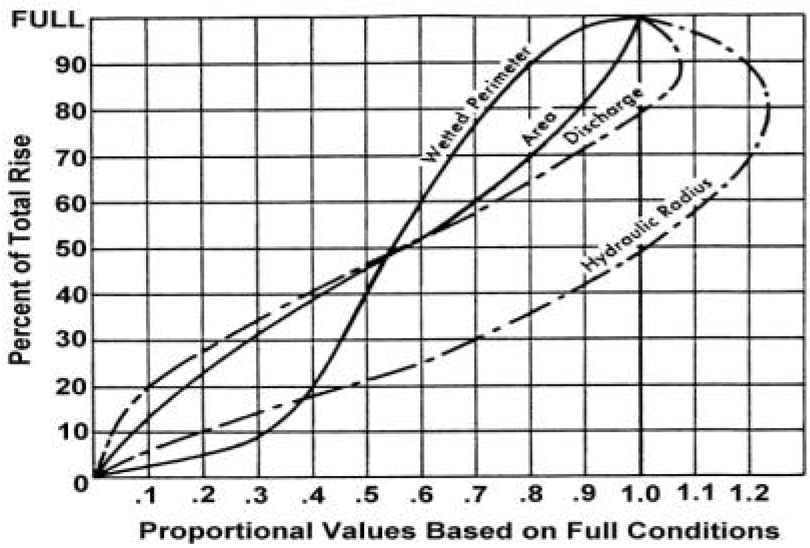


Figure E.3 - Corrugated Metal Pipe Arch

(Source: Reference 12)

APPENDIX F-1
HYDRAULIC PROPERTIES OF CULVERT SECTIONS
Corrugated Steel Pipe-Arches



Hydraulic properties of corrugated
steel and structural plate pipe-arches.

Full-Flow data for Corrugated Steel Pipe-Arches

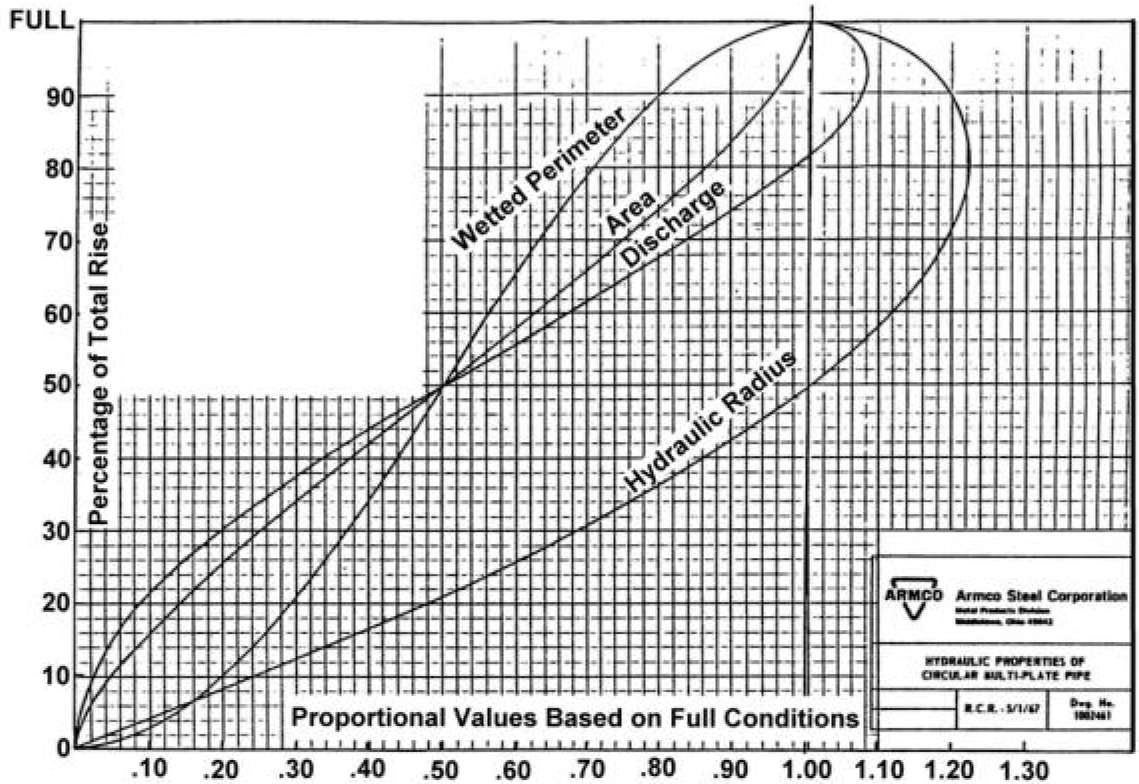
Corrugations 2 2/3 x ½ in.

Dimensions in inches			Waterway Area in Sq. Ft.	Hydraulic Radius A/(Pi)D in Feet
	Pipe-Arch			
Pipe Diam.	Span	Rise		
15	18	11	1.1	0.280
18	22	13	1.6	0.340
21	25	16	2.2	0.400
24	29	18	2.8	0.446
30	36	22	4.4	0.560
36	43	27	6.4	0.679
*42	50	31	837	0.791
*48	58	36	11.4	0.907
*54	65	40	14.3	1.012
*60	72	44	17.6	1.120
*66	79	49	21.3	1.233
*72	85	54	25.3	1.342
Corrugations 3 x 1 in.				
66	73	55	22	1.273
72	81	59	26	1.379
78	87	63	31	1.518
84	95	67	35	1.592
90	103	71	40	1.698

*These sizes apply to both types of corrugations: 2-2/3 x ½ and 3 x 1 in.

(Source: Reference 2)

APPENDIX F-2
HYDRAULIC PROPERTIES OF CULVERT SECTIONS
Circular Pipe



Periphery	Area ($ft.^2$)	Hydraulic Radius	Periphery	Area ($ft.^2$)	Hydraulic Radius	Periphery	Area ($ft.^2$)	Hydraulic Radius	Periphery	Area ($ft.^2$)	Hydraulic Radius
60°	19	1.23	138°	105	2.89	216°	259	4.54	294°	483	6.20
66°	23	1.36	144°	114	3.01	222°	271	4.67	300°	503	6.32
72°	28	1.49	150°	124	3.14	228°	289	4.80	306°	523	6.45
78°	33	1.61	156°	134	3.27	234°	305	4.92	312°	544	6.58
84°	38	1.74	162°	145	3.40	240°	321	5.05	318°	565	6.71
90°	44	1.87	168°	156	3.52	246°	337	5.18	324°	587	6.83
96°	50	2.00	174°	168	3.65	252°	354	5.31	330°	609	6.96
102°	57	2.12	180°	179	3.78	258°	371	5.43	336°	631	7.09
108°	64	2.25	186°	192	3.91	264°	389	5.66	342°	654	7.22
114°	71	2.38	192°	204	4.03	270°	407	5.69	348°	678	7.34
120°	79	2.50	198°	218	4.14	276°	425	5.82	354°	701	7.47
126°	87	2.63	204°	231	4.29	282°	441	5.94	360°	725	7.60
132°	96	2.76	210°	245	4.41.	288°	469	6.07			

(Source: Reference 2)