Chapter 3 HYDROLOGY

3.1 INTRODUCTION

Analysis of peak rate of runoff, volume of runoff, and time distribution of flow is fundamental for design of highway drainage facilities. Most drainage facility designs require determination of a peak flow rate while others require a runoff hydrograph that provides an estimate of runoff volume. Peak flow rates are most often used for design of bridges, culverts, roadside ditches, and small storm sewer systems. Drainage systems involving detention storage, pumping stations and large or complex storm sewer systems require the development of a runoff hydrograph to estimate volume of runoff.

The relationship between the amount of precipitation on a drainage basin and the amount of runoff from the basin is complex, and too little data is available on the factors influencing the rainfall-runoff relationship to expect exact solutions. Any hydrologic analysis is only an approximation, though an error in the estimate of peak runoff effects the design. Under prediction of the peak runoff can result in a structure that is undersized and may contribute to flooding while over predicting the peak runoff may lead to an oversized drainage facility that costs more than necessary.

This chapter provides design procedures for hydrologic analysis. For a more detailed discussion refer to the publications, *Highway Drainage Guidelines - Volume II*, (AASHTO, 1999) or *Highway Hydrology - HDS-2*, (FHWA, 1996).

3.1.1 Definition

Hydrology is a science dealing with the interrelationship and movement of various forms of water on and under the earth surface and in the atmosphere. For the purpose of this manual, hydrology will deal with estimating flood magnitudes as the result of precipitation. In the design of highway drainage structures, floods are measured in terms of peak runoff or hydrograph having a discharge in cubic feet per second (cfs). Structures designed to control volume of runoff, like detention storage facilities, or situations where flood routing is used, the entire discharge hydrograph will be of interest.

3.1.2 Concept Definitions

The following terms will be important in a hydrologic analysis. These concepts will be used throughout the remainder of this chapter in dealing with different aspects of hydrologic studies.

Antecedent Moisture Conditions	Antecedent moisture conditions are the soil moisture conditions of the watershed at the beginning of a storm. These conditions affect the volume of runoff generated by a particular storm event. Notably they affect the peak discharge only in the lower range of flood magnitudes. Antecedent moisture has a rapidly decreasing influence on runoff as the flood recurrence interval becomes longer.
Depression Storage	Depression storage is the natural depressions within a watershed which store runoff. Generally after the depression storage is filled runoff will commence.
Drainage Area (A)	The area draining into a stream at a given point along the stream.
Frequency	Frequency is the number of times a flood of a given magnitude or greater can be expected to occur on average over a long period of time. Frequency analysis is the estimation of peak discharges for various recurrence intervals. Another way to express frequency is with probability. Probability analysis seeks to define the flood flow with a probability of being equaled or exceeded in any year.
Hydraulic Roughness	Hydraulic roughness is a composite of the physical characteristics which influence the flow of water across the earth's surface, whether natural or channelized. It affects both the time response of a watershed and drainage channel as well as the channel storage characteristics.
Hydrograph	The hydrograph is a graph of the time distribution of runoff from a watershed.
Hydrologic Soil Group	A group of soils having the same runoff potential under similar storm and cover conditions.
Hyetographs	The hyetograph is a graph of the time distribution of rainfall over a watershed.

Infiltration	Infiltration is a complex process of allowing runoff to penetrate the ground surface and flow through the upper soil surface. The infiltration curve is a graph of the time distribution at which this occurs.
Interception	Storage of rainfall on foliage and other intercepting surfaces during a rainfall event is called interception storage.
Lag Time	The lag time is defined as the time from the centroid of the excess rainfall to the peak of the hydrograph.
Peak Discharge	The peak discharge, sometimes called peak flow, is the maximum rate of flow of water passing a given point during or after a rainfall event or snowmelt.
Rainfall Excess	The rainfall excess is the water available to runoff after interception, depression storage and infiltration have been satisfied.
Rainfall Intensity (I)	Amount of rainfall occurring in a unit of time, converted to its equivalent in inches per hour.
Recurrence Interval	The average number of years between occurrences of a discharge or rainfall that equals or exceeds the given magnitude.
Runoff (Q)	The part of the precipitation which runs off the surface of a drainage area after all abstractions are accounted for.
Runoff Coefficient	A factor representing the portion of runoff resulting from a unit rainfall. Principally dependent on terrain, topography, slope, land use and soil type.
Time of Concentration	The time of concentration is the time it takes a drop of water falling on the hydraulically most remote point in the watershed to travel through the watershed to the point under investigation.
Ungaged Stream Sites	Locations at which no systematic records are available regarding actual stream flows.
Unit Hydrograph	A unit hydrograph is the direct runoff hydrograph resulting from a rainfall event which has a specific temporal and spatial distribution and which lasts for a unit duration of time. The ordinates of the unit hydrograph are such that the volume of direct runoff represented by the area under the hydrograph is equal to one inch of runoff from the drainage area.

3.1.3 Factors Affecting Flood Runoff

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

- Drainage basin characteristics
  - drainage area size,
  - drainage area shape,
  - drainage area orientation,
  - slope of terrain,
  - land use; consider watershed development potential,
  - geology,
  - soil type,
  - surface infiltration,
  - storage,
  - antecedent moisture condition, and
  - storage potential: overbank, ponds, wetlands, reservoirs, channel, etc.

- Stream channel characteristics
  - channel slope,
  - channel geometry,
  - channel configuration,
  - natural and artificial controls,
  - channel modification,
  - aggradation/degradation, and
  - ice and debris.
- Flood plain characteristics
  - type of soil, and
  - ground cover.
- Meteorological characteristics
  - precipitation amount,
  - time rate of precipitation, hyetograph,
  - storm cell size,
  - storm cell distribution,
  - storm direction, and
  - type of precipitation: rain, snow, hail, or combinations thereof.

3.1.4 Sources of Information

The type and source of information available for hydrologic analysis will vary from site to site. It is the responsibility of the designer to determine what information is available and applicable to a particular analysis.

All hydrologic analysis shall consider the flood history of the area and the effect of these historical floods on existing and proposed structures. The flood history shall include the historical floods and the flood history of any existing structures. Files should be reviewed for documentation of relevant communications, studies and investigations. Surveys should be conducted to provide enough field data for analysis. Typical data that is obtained in such surveys or studies are: topographic maps, aerial photographs, streamflow records, historical highwater elevations, flood discharges, and locations of hydraulic features such as reservoirs, water projects, and designated or regulatory flood plain areas.

Interagency coordination is necessary since many levels of government plan, design, and construct highway and water resource projects which might have an affect on each other. Agencies can share data and experiences within project areas to assist in the completion of accurate hydrologic analysis. Agencies include: Department of Natural Resources (DNR), U. S. Fish and Wildlife Service, U. S. Army Corps of Engineers (USACE), Watershed District & Management Organizations, Counties, Cities, Pollution Control Agency (PCA), Natural Resources Conservation Service (NRCS) previously Soil Conservation Service (SCS) and U. S. Geologic Survey (USGS).

3.2 DESIGN FREQUENCY

Design frequency should be selected commensurate with cost of the facility, amount of traffic, potential flood hazard to property, expected level of service, political considerations, and budgetary constraints as well as the magnitude and risk associated with damages from larger flood events. When long highway routes having no practical detour, where many sites are subject to independent flood events, it may be necessary to increase the design frequency at each site to avoid frequent route interruptions from floods. Consideration should be given to what frequency flood was used to design other structures along a highway corridor. In selecting a design frequency, Storm Water Management Plans should be considered where watershed changes could reasonably occur over the anticipated life of the drainage facility.

It is not economically feasible to design highway structures for the maximum runoff that a watershed is capable of producing. Therefore, a design frequency must be established. The design frequency for a given flood is defined as the reciprocal of the probability that a flood flow will be exceeded in a given year. The frequency is analogous to the recurrence interval. A 50-year recurrence interval means that every year there is a 2% chance that a 50-year flood will occur at a given point, and it could conceivably occur in several consecutive years. Over a long period of time, the 50-year flood will be equaled or exceeded on the average of once every fifty years.

While drainage structures are designed to operate for a given design frequency, performance should be checked for the review frequency. After sizing a drainage facility to pass a peak flood or the hydrograph corresponding to the design frequency, it may be necessary to review this proposed facility considering a larger discharge to insure that there are no unexpected flood hazards inherent in the proposed facility(ies). Potential impacts to consider include possible flood damage due to high embankments where overtopping is not practical, backup due to the presence of noise walls, and flood damage where a storm drain might back up. The flood damage potential due to bridges and major culverts (greater than 48") should be reviewed for the 100 year frequency. The scour potential for bridge substructures should be reviewed for the 500 year frequency or overtopping event.

3.2.1 Design Frequency Policy

The design frequency used to design a hydraulic structure is determined by the type, size, and location of the structure. Design frequency for inlets and storm sewers is based on the allowable spread on the roadway. Minor culverts (48" or less in diameter) shall be designed using a 50 year frequency. Major culverts (larger than 48" in diameter) and bridges require completion of a Risk Assessment Form (Appendix A) to determine the appropriate design frequency.

Bridges and Centerline Culverts

For all bridges over waterways, and for major centerline culverts (larger than 48"), a risk assessment shall be completed. Instead of arbitrary design frequencies, the risk assessment procedure takes into consideration capital costs and risks, and other economic, engineering, social, and environmental concerns. The risk assessment is based on the:

- the overtopping flood or the base flood, whichever is greater , or
- the greatest flood which must flow through the highway drainage structure where overtopping is not practicable. This is considered to be a 500-year frequency flood. If flood frequency data is not available, use 1.7 x 100 year flood.

Table 3.1 gives the guidelines for the recommended minimum overtopping flood criteria which should be used for a risk assessment. The risk assessment procedure is difficult to apply to small culverts. Consequently, a formal risk assessment or analysis will ordinarily not be required for minor culverts (48 in. diameter or less) unless there is significant flood damage potential. The design frequency for minor centerline culverts shall be a minimum of 50 year frequency. A copy of the risk assessment form and other information is provided in Appendix A.

Table 3.1 Minimum Overtopping Flood Frequency for Risk Assessment

PROJECTED ADT	MINIMUM OVERTOPPING FLOOD FREQUENCY
0 - 10	2 year
11 - 49	5 year
50 - 399	10 year
400 - 1499	25 year
1500 and up	50 year

Entrance Culverts

Entrance culverts shall be a minimum of 15" in diameter. They should be designed for a 10 year frequency and an overtopping area should be provided.

Storm Drains

Storm drains shall be designed to accommodate a discharge with a given return period(s) or frequency. To select the return period, assume the storm runoff should not increase the flood hazard significantly for the property. Also, the runoff should not encroach on to the street or highway so as to cause a significant traffic hazard, or limit traffic, and pedestrian movement to an unreasonable extent.

The design storm frequency for pavement drainage and the other components of the drainage system should be consistent. In order for it to be meaningful criteria, the design frequency must be tied to a design water spread. Table 3.2 gives Mn/DOT's established criteria for design frequency and allowable water spread.

Ramps, Loops, Turn Lanes, Acceleration and Deceleration Lanes should generally be designed to the same frequency as the mainline. Where the speed limit is 35 mph or less and there are no shoulders or parking lanes, the allowable spread can encroach up to ½ D for short periods of time. High capacity inlets such as slotted vane drains should be considered at locations where excessive spread may cause inconvenience or safety hazards to motorists.

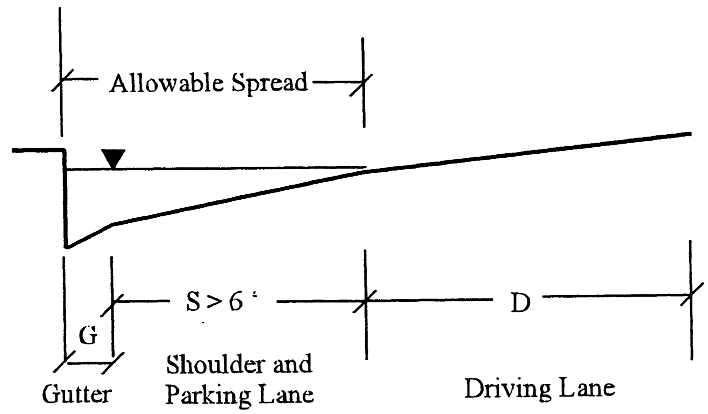

Figure 3.1 Allowable Spread for Shoulders of 6 Feet or more

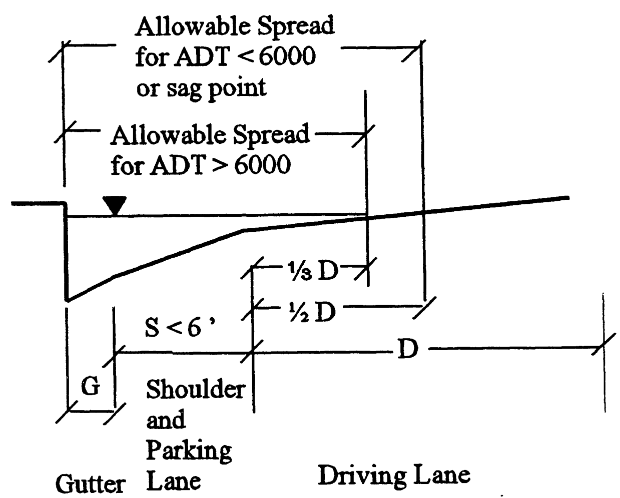

Figure 3.2 Allowable Spread for Shoulders Less than 6 Feet

Table 3.2 Design Frequency for Storm Drains

PROJECTED 2-WAY ADT (vehicles per day, VPD)	DESIGN FREQUENCY (Year)	ALLOWABLE SPREAD[1]
> 6000	10	P, S, or a D
	50 Year at sag point[2]	P, S, or ½ D
2000-6000	10	P, S, or ½ D
1000-1999	5	P, S, or ½ D
< 1000	3	P, S, or ½ D

[1] P = Parking lane, S = Shoulder of 6′ or more, D = Driving lane if there is no shoulder.

[2] Sag Point refers to a true sag where flooding of 2' or more can occur.

3.2.2 Rainfall vs. Flood Frequency

Drainage structures are designed for a designated flood frequency. However, certain hydrologic procedures use rainfall and rainfall frequency as the basic input. Thus it is commonly assumed in most cases that the 10-year rainfall will produce the 10-year flood. In general, rainfall is used to design storm drains and culverts with small drainage areas. The rational method and SCS methods use the assumption that a rainfall of a certain frequency will produce a flood of the same frequency.

Rainfall intensity-duration curves have been developed for the common design frequencies. Rainfall intensity data for Minnesota generated from HYDRO-35 is provided in Section 3.5.4. Appendix B contains 24 hour rainfall amounts for various recurrence intervals in Minnesota based on the U.S. Weather Bureau's *Technical Publication No. 40* (Hershfield, 1961). Additional data on rainfall can be found in other sources. Data on historical storms may be available from the State Climatologist.

3.3 HYDROLOGIC PROCEDURE SELECTION

Streamflow measurements are usually unavailable for determining flood frequency relationships at a site. In such cases, it is accepted practice to estimate peak runoff rates and hydrographs using statistical or empirical methods. The design discharge for other structures on the stream and historical data should be reviewed, and consideration given to previous studies and Flood Insurance Studies. Mn/DOT's practice shall be to use the discharge that best reflects local project conditions with the reasons documented.

3.3.1 Peak Flow Rates or Hydrographs

Determination of peak runoff rates for design conditions is generally adequate for conveyance systems such as storm drains or open channels. However, if the design includes storage basins or complex conveyance networks, a flood hydrograph is usually required. The development of runoff hydrographs is usually accomplished using computer programs.

3.3.2 Hydrologic Procedures Options

The recommended hierarchy for selecting a method of computing discharge for design of highway structures in Minnesota and the circumstances for their use are listed below and shown in Table 3.3. Where feasible, the hydrologic model should be calibrated to local conditions and tested for accuracy and reliability. Consider design discharges for other structures in the area and historical data for the area. In general the results from different hydrologic models should not be averaged.

- Rational method shall be used only for drainage areas less than 200 acres, and preferably only for developed areas.
- Both the graphical peak discharge and tabular hydrograph method available in TR-55 are simplified procedures are derived from TR-20.
- The TR-55 graphical method may be used for small rural watersheds (1 - 2000 acres).
- TR-55's tabular hydrograph method can be applied to larger watershed areas by splitting a non-homogenous watershed into homogenous subareas.
- Minnesota USGS regression equations should be used for routine designs of bridges and culverts with 54" or larger widths and in accordance with the other limitations of the regression equations, unless there is gaging station data or historical evidence suggesting other alternatives.
- Log Pearson III analyses of stream gaging station data should be used for all routine designs with gaging station data, provided there is at least 10 years of continuous record for 10-year discharge estimates and 25 years for 100-year discharge estimates.
- The SCS unit hydrograph method contained in TR-20 should be used for storage routing or storage design.
- Suitable computer programs such as HEC 1, TR-20, TR-55, and HYDRAIN may be used for hydrologic calculations. Other computer programs that incorporate the recommended methodology may also be used. Output from all programs must be reviewed to see that the answers are reasonable.
- The 100-year discharges specified in the FEMA flood insurance study shall be used to analyze impacts of a proposed crossing on a regulatory floodway. However, if these discharges are deemed to be outdated, discharges based on current methods may be used subject to receipt of necessary regulatory approvals.

Table 3.3 Selection of Discharge Computation Method

FACILITY DESCRIPTION	METHOD				
	Frequency Analysis	Regression Equations	SCS Hydrograph	SCS Peak Discharge	Rational
Stream Flow, Channels, Bridges, & Culverts Greater then 48"	customary method	alternate method	complex facilities or hydrograph needed	alternate method	
Culverts 48" and Smaller		alternate method	complex facilities or hydrograph needed	customary method	alternative method
Storm Drains, Roadside Ditches, & Side Culverts			complex facilities or hydrograph needed		customary method
Detention Basins			customary method		preliminary evaluation, or permit review

3.4 TIME OF CONCENTRATION

The time of concentration (t_c) is the time required for water to flow from the hydraulically most remote point of the drainage area to the point under investigation. Water moves through a watershed as a combination of overland and channelized flow. The type of flow that occurs is a function of the conveyance system and is best determined by field inspection. t_c is an important variable in many hydrologic methods, including the Rational and Soil Conservation Service (SCS) procedures. For the same watershed, the shorter the t_c, the larger the peak discharge for similar land use. Many of the computation methods for determining time of concentration are available within most commonly used computer applications.

3.4.1 Total Time of Concentration

To obtain the total time of concentration, the channel flow time, or travel time is calculated and added to the overland flow time. The units must be consistent for all values in Equation 3.1. The Rational Equation normally uses t_c in minutes, SCS procedures generally use t_c in hours. The total time of concentration is:

$$t_c = t_o + t_t \tag{3.1}$$

Where: t_c = total time of concentration
t_o = overland flow time
t_t = travel time

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

When designing a drainage system, the overland flow path is not necessarily perpendicular to the contours shown on available mapping. Often the land will be graded and swales will intercept the natural contour and conduct the water to streets which reduces the time of concentration. Generally the overland flow path is less than 200 feet in urban areas and 400 feet in rural areas.

For storm drainage systems, the designer is often concerned with two different times of concentration: one for inlet spacing and the other for pipe sizing. The time of concentration (t_c) for inlet spacing is the time for water to flow from the hydraulically most distant point of the drainage area to the inlet, which is known as the inlet time. Usually this is the sum of the time required for water to move across the pavement or overland back of the curb to the gutter, plus the time required for flow to move through the length of the gutter to the inlet. The time of concentration for the second downstream inlet and each succeeding inlet should be determined independently, the same as the first inlet. Travel time between inlets is not considered. When the total time of concentration for pavement drainage inlets is less than seven minutes, a minimum of seven minutes should be used to estimate the duration of rainfall.

When determining pipe size, the time of concentration (t_c) for any point along a storm drain is the time to reach the inlet at the upper end of the line plus the time of flow through the storm drain from the upper end of the storm drain to the point in question. In general, where there is more than one source of runoff to a given point in a storm drainage system, the longest t_c is used to estimate the intensity (I). Exceptions to this generality, include a large inflow area at some point along the system, the t_c for that area may produce a larger discharge than the t_c for the summed area with the longer t_c. The designer should be aware of this possibility when joining drainage areas and determine which drainage area governs.

3.4.2 Travel Time

Travel time is the length of time it takes to travel as channelized flow. Manning's Equation and the Continuity Equation calculate velocity not time. After determining the average flow velocity in the pipe or channel, the travel time is obtained by dividing channel length by average pipe velocity.

$$t_t = \frac{L}{60V} \tag{3.2}$$

Where: t_t = travel time (min)
L = length which runoff must travel (ft)
V = estimated or calculated velocity (ft/s)

3.4.3 Selection of Method

When selecting a method for determining the time of concentration, consider the conditions for which the equation was developed and how they compare to the drainage area being evaluated; what hydrologic computation procedures will be applied; and what type of facility is being designed. The methods for determining time of concentration included in this section are applicable for many different hydrologic computation procedures, though when SCS methods are used to compute discharge, time of concentration should be determined using the methods recommended by the Natural Resource Conservation Service (NRCS).

Table 3.4 Methods for Calculating Time of Concentration

METHOD	COMMENTS
Kinematic Wave Equation	Overland flow time. Requires iterative solution.
Manning's Kinematic Solution	Overland flow time. The maximum flow length is 300'.
Manning's Equation • Overland Flow • Triangular Gutter Flow • Pipe Flow	Flow velocity for non-pressure flow in pipes, open channels, gutter flow or overland flow.
Continuity Equation	Flow velocity for pressure flow in pipes.

3.4.4 Kinematic Wave Equation

HEC-12 (FHWA, 1984) recommends the kinematic wave equation as the most realistic method for estimating overland flow time of concentration.

$$t_o = \frac{56L^{0.6}n^{0.6}}{i^{0.4}S^{0.3}} \tag{3.3}$$

Where: t_o = time of overland flow (sec)
L = overland flow length (ft)
n = Manning's roughness coefficient
i = rainfall rate (in/hr)
S = average slope of the overland area

When using this equation, both the time of concentration and rainfall intensity are unknowns and trial and error iterations are required. A value for rainfall intensity (i) is assumed and the related time of concentration found. The assumed rainfall intensity must then be checked against the rainfall Intensity-Duration-Frequency curve for the frequency of the event chosen for the particular design problem, and the procedure repeated until the assumed rainfall intensity is in agreement with the intensity associated with the time of concentration.

3.4.5 Manning's Kinematic Solution

For sheet (overland) flow for a distance of less than 300 feet, TR-55 (SCS, 1986) recommends using Manning's kinematic solution to compute t_o. This simplified form of the Manning's kinematic solution is based on the following assumptions: shallow steady uniform flow, constant intensity of rainfall excess (rain available for runoff), rainfall duration of 24 hours, and minor infiltration effect on travel time.

$$t_o = \frac{0.007(nL)^{0.8}}{P_2^{0.5} s^{0.4}} \tag{3.4}$$

Where: t_o = overland flow time (hr)
n = Manning's roughness coefficient, Table 3.5
L = flow length (ft)
P_2 = 2-year, 24-hour rainfall (in)
s = slope of hydraulic grade line (ft/ft), assumed equivalent to land slope

Table 3.5 Roughness Coefficients (Manning's n) For Sheet Flow

SURFACE DESCRIPTION		n[1]
Smooth Surfaces (concrete, asphalt, gravel, or bare soil)		0.011
Fallow (no residue)		0.05
Cultivated Soils	Residue cover ≤ 20%	0.06
	Residue cover > 20%	0.17
Grass	Short grass prairie	0.15
	Dense grasses[2]	0.24
	Bermuda grass	0.41
Range (natural)		0.13
Woods[3]	Light underbrush	0.40
	Dense underbrush	0.80

[1] The n values are a composite of information complied by Engman (1986).
[2] Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
[3] When selecting n, consider cover to a height of about 0.1 ft, the only part of the plant cover that will obstruct sheet flow.

3.4.6 Manning's Equation

For watersheds with storm drains or channels, the travel time must be added into the total time of concentration. Manning's Equation is used to determine the average velocity, usually determined for bank-full flow conditions. The velocity calculated from Mannings equation is plugged into Equation 3.2 to calculate the travel time. Open channels are assumed to begin where the surveyed cross sections have been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets.

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \tag{3.5}$$

Where: V = mean velocity of flow (ft/s)
n = Manning's roughness coefficient
R = hydraulic radius (ft) = Area (ft²) / Wetted Perimeter (ft)
S = slope of the hydraulic grade line (ft/ft)

3.4.7 Overland Flow

Manning's Equation is often used to calculate the flow velocity in order to estimate time of concentration for shallow concentrated flow overland. The SCS Upland Method is a graphical version of this equation, and a t_o calculated with the following assumption can be used with the SCS Peak Flow method. Determine velocity from Equation 3.5 with the assumption hydraulic radius (R) is equal to the flow depth (d). Table 3.6 gives approximate values for "d" and "n" based on type of cover. Apply Equation 3.2 to calculate t_o for a known length and velocity.

Table 3.6 Roughness Coefficients (Manning's n) For Overland flow

SURFACE DESCRIPTION	MANNING'S N	
	FLOW DEPTH d = 0.1 feet	FLOW DEPTH d = 0.2 feet
Forest with heavy litter, hay meadow & brush	0.13	0.20
Cultivated: minimum tillage, contoured or strip cropped & woodland	0.065	0.10
Pasture or cultivated straight row crop	0.045	0.07
Nearly bare and untilled	0.03	0.05
Paved area	0.015	0.025

Source: SCS Hydrology Guide for Minnesota Figure 4-1

Simplified Manning's Equations are available (SCS, 1986) for grassed waterway (unpaved areas) and paved areas to determine travel time for shallow concentrated flow.

Unpaved $$V = 16.1345(S)^{0.5} \quad (3.6)$$

Paved $$V = 20.3285(S)^{0.5} \quad (3.7)$$

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

3.4.8 Triangular Gutter Flow

The travel time for gutter flow is estimated using an average velocity of the flow. HEC-12 (FHWA, 1984) contains a nomograph for determining the velocity in a triangular gutter section given the watercourse slope, gutter cross slope and water spread. The equation can be modified for composite cross sections. For a triangular channel with uniform inflow per length and zero flow at the upstream end, the average velocity will occur where the spread is 65% of the maximum. For a triangular channel Manning's equation becomes:

$$V = \frac{1.12}{n} T^{2/3} S_x^{2/3} S^{1/2} \quad (3.8)$$

Where: V = mean velocity of flow (ft/s)
n = Manning's roughness coefficient
T = spread across cross section (ft)
S_x = cross slope (ft/ft)
S = slope of the hydraulic grade line (ft/ft)

3.4.9 Pipe Flow

For ordinary conditions, storm drains should be sized on the assumption that they will flow full or practically full for the design discharge. For non-pressure flow, the velocity can be determined using Manning's equation. For circular pipes flowing full, the equation becomes:

$$V = \frac{0.590}{n} D^{2/3} S^{1/2} \qquad (3.9)$$

Where: V = mean velocity of flow (ft/s)
n = Manning's roughness coefficient
D = diameter of circular pipe (ft)
S = slope of the hydraulic grade line (ft/ft)

3.4.10 Continuity Equation

If the pipes of a storm drainage system will operate under pressure flow, the continuity equation should be used to determine the velocity.

$$V = \frac{Q}{A} \qquad (3.10)$$

Where: V = mean velocity of flow (ft/s)
Q = discharge in pipe (cfs)
A = area of pipe (ft^2)

3.5 RATIONAL METHOD

The rational method is commonly used to calculate the peak flow from small drainage areas. It is recommended for estimating the design storm peak runoff for areas up to 200 acres.

3.5.1 Application

The rational formula applies best to developed areas with significant amount of pavement or gutters and is typically used for designing storm drain systems, and small or sideline culverts. The rational formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration. The equation is:

$$Q = CIA = \left(\sum CA\right)I \tag{3.11}$$

Where: Q = discharge (cfs)
C = runoff coefficient representing a ratio of runoff to rainfall
I = rainfall intensity (in/hr)
A = drainage area (acres)

The peak discharge calculated by the rational formula is very sensitive to the values selected for the parameters. The designer must use good engineering judgment in estimating values that are used in the method. The first step in applying the rational method is to obtain a good topographic map and define the boundaries of the drainage area in question. A field inspection should be conducted to determine if natural drainage divides have been altered. Restrictions to the natural flow such as highway crossings and dams that exist in the drainage area should be investigated to determine how they affect the design flows.

3.5.2 Limitations

The assumptions of the rational method which limit its use to small drainage areas of up to 200 acres include:

- The rate of runoff resulting from any rainfall intensity is maximum when the rainfall intensity lasts as long or longer than the time of concentration of the drainage area. This assumption limits the size of the drainage basin that can be evaluated by the rational method. For large drainage areas, the time of concentration can be so large that constant rainfall intensities for such long periods do not occur and shorter more intense rainfalls can produce larger peak flows.
- The frequency of peak discharges is the same as that of the rainfall intensity for the given time of concentration. Frequencies of peak discharges depend on rainfall frequencies, antecedent moisture conditions in the watershed, and the response characteristics of the drainage system. For small and largely impervious areas, rainfall frequency is the dominant factor. For larger drainage basins, the response characteristics control.
- The fraction of rainfall that becomes runoff (C) is independent of rainfall intensity or volume. This assumption is reasonable for impervious areas, such as streets, rooftops, and parking lots. For pervious areas, the fraction of runoff varies with rainfall intensity and the accumulated volume of rainfall. The selected runoff coefficient must be appropriate for the storm, soil, and land use conditions.
- The peak rate of runoff is sufficient information for the design.

3.5.3 Runoff Coefficient

The runoff coefficient (C) requires engineering judgment and understanding on the part of the designer to apply correctly. Typical coefficients represent the integrated effects of many drainage basin parameters. The selected value should incorperate storm, soil and land use conditions. The runoff coefficients for various types of surfaces are shown in Table 3.7. The total CA value is to be based on a ratio of the drainage areas associated with each C value as follows:

$$Total(CA) = \left(\sum CA\right) = A_1C_1 + A_2C_2 + \ldots + A_nC_n \tag{3.12}$$

Selection of the runoff coefficients (C) requires good engineering judgement. The designer should document justification for the C values used at a given site. It is not appropriate to use the lowest value in a range, unless warranted by local site conditions. Considerations for selecting C include rainfall intensity, slope, soil type, direct connection of impervious area, future land use, detention effects.

- The coefficients provided in Table 3.7 are applicable for storms of 5 yr to 10 yr frequencies. Less frequent, higher intensity storms will require a higher coefficient because infiltration and other losses have a proportionally smaller effect on runoff.
- As the slope of the drainage basin increases, the selected C value should also increase. This is caused by the fact that as the slope of the drainage area increases, the velocity of overland and channel flow will increase allowing less opportunity for water to infiltrate the ground surface. Thus, more of the rainfall will become runoff from the drainage area.
- Consider soil type and infiltration rates when selecting C. Sandy soils infiltrate more resulting in less runoff, while clay soils produce more runoff.
- In determining the runoff coefficient (C) values for the drainage area, consider future changes in land use that might occur during the service life of the proposed facility that could result in an inadequate drainage system.
- The effects of upstream detention facilities and local stormwater management plans should be taken into account.

Table 3.7 Runoff Coefficients for Rational Formula

TYPE OF DRAINAGE AREA			RUNOFF COEFFICIENT
Business	Downtown areas		0.70 - 0.95
	Neighborhood areas		0.50 - 0.70
Residential	Single-family areas		0.30 - 0.50
	Multi-units	Detached	0.40 - 0.60
		Attached	0.60 - 0.75
	Suburban		0.25 - 0.40
	Apartment dwelling areas		0.50 - 0.70
Industrial	Light areas		0.50 - 0.80
	Heavy areas		0.60 - 0.90
Parks, cemeteries			0.10 - 0.25
Playgrounds			0.20 - 0.35
Railroad yard areas			0.20 - 0.40
Unimproved Urban Areas			0.10 - 0.30
Lawns	Sandy soil	flat, 2%	0.05 - 0.10
		average, 2 - 7%	0.10 - 0.15
		steep, 7%	0.15 - 0.20
	Heavy soil	flat, 2%	0.13 - 0.17
		average, 2 - 7%	0.18 - 0.22
		steep, 7%	0.25 - 0.35
Streets	Asphaltic		0.70 - 0.95
	Concrete		0.80 - 0.95
	Brick		0.70 - 0.85
Drives and walks			0.75 - 0.85
Roofs			0.75 - 0.95
Rural	Average infiltration rates sandy & gravel soils	Cultivated	0.20
		Pasture	0.15
		Woodlands	0.10
	Average infiltration rates; Loams and similar soils with no clay pans	Cultivated	0.40
		Pasture	0.35
		Woodlands	0.30
	Below average infiltration rates; heavy clay soils; soils with a clay pans near the surface; shallow soil above impervious rock	Cultivated	0.50
		Pasture	0.45
		Woodlands	0.40

Source: data for urban type drainage areas ASCE, 1960

3.5.4 Rainfall Intensity

The average rainfall intensity (I) is the intensity of rainfall in inches per hour for a duration equal to the time of concentration. The time of concentration is the time required for water to flow from the hydraulically most remote point of the drainage area to the point under investigation. Methods for computing time of concentration are given in Section 3.4.

Intensity is the rate of rainfall over an interval of time such that intensity multiplied by duration equals amount of rain. For example, an intensity of 5 inches per hour for a duration of 5 minutes indicates a total rainfall amount of 5 X 5/60 = 0.42 inches. Rainfall frequency relations will vary from one area to another. An approach to selecting the values is to review the sources for a particular area and then select the sources that seems most appropriate.

The value for Intensities (I) for storm durations of up to 60 minutes and for typical recurrence frequencies are given in Table 3.8. This table correspond to the 3 zones of the state shown on Figure 3.3. The intensity-duration-frequency (IDF) curves for each zone are provided in Figures 3.4, 3.5 and 3.6 The curves were developed using the procedure from NWS technical memorandum *HY-DRO-35* (Frederick, et al. 1977).

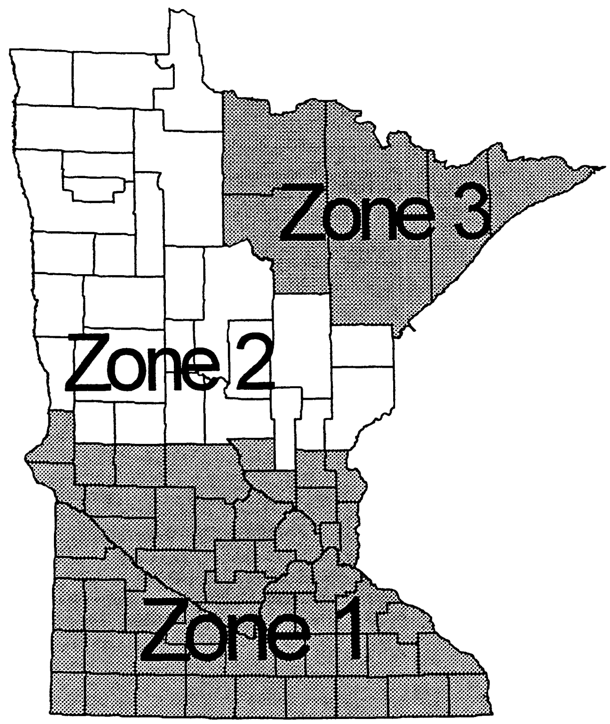

Figure 3.3 Minnesota Zones for Intensity-Duration-Frequency (IDF) Curves

Table 3.8 Rainfall Intensity - Duration - Frequency Tables for Minnesota

RAINFALL INTENSITY (in/hr)

ZONE 1 - SOUTHERN					
Event Frequency (Years)	Rain Duration or Time of Concentration (Minutes)				
	5	10	15	30	60
2	5.2	4.1	3.4	2.3	1.4
3	5.6	4.5	3.8	2.6	1.6
5	6.3	5.1	4.3	2.9	1.9
10	7.1	5.8	4.9	3.4	2.2
25	8.3	6.8	5.8	4.0	2.6
50	9.3	7.7	6.5	4.5	2.9

ZONE 2 - NORTHWEST AND CENTRAL					
Event Frequency (Years)	Rain Duration or Time of Concentration (Minutes)				
	5	10	15	30	60
2	5.0	3.9	3.2	2.1	1.3
3	5.5	4.3	3.6	2.4	1.5
5	6.2	4.9	4.1	2.8	1.8
10	7.0	5.6	4.7	3.2	2.1
25	8.2	6.6	5.5	3.8	2.5
50	9.2	7.3	6.2	4.3	2.8

ZONE 3 - NORTHEAST					
Event Frequency (Years)	Rain Duration or Time of Concentration (Minutes)				
	5	10	15	30	60
2	4.9	3.6	3.0	1.9	1.2
3	5.3	4.0	3.2	2.2	1.4
5	5.9	4.5	3.7	2.5	1.6
10	6.6	5.0	4.2	2.8	1.8
25	7.7	5.9	4.9	3.4	2.2
50	8.5	6.6	5.4	3.8	2.5

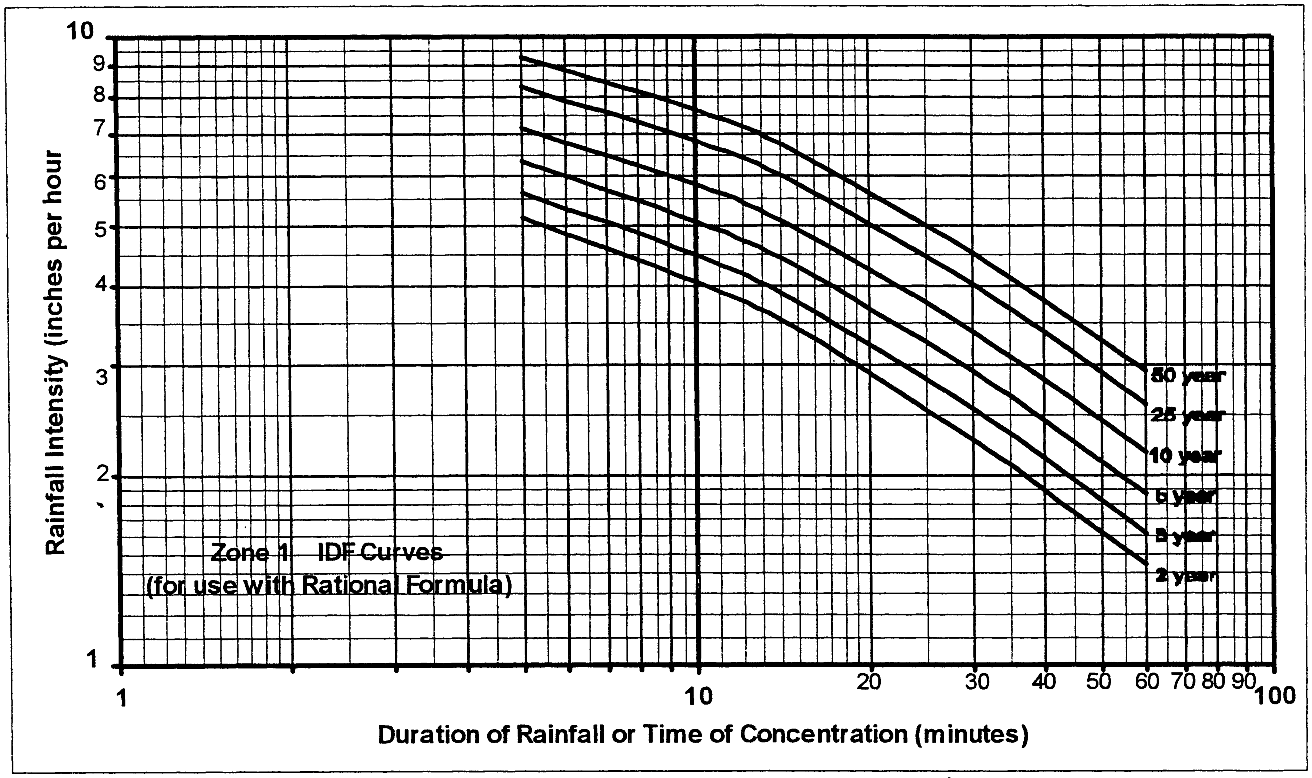

Figure 3.4 Zone 1 Southern Minnesota Rainfall Intensity - Duration - Frequency (IDF) Curves

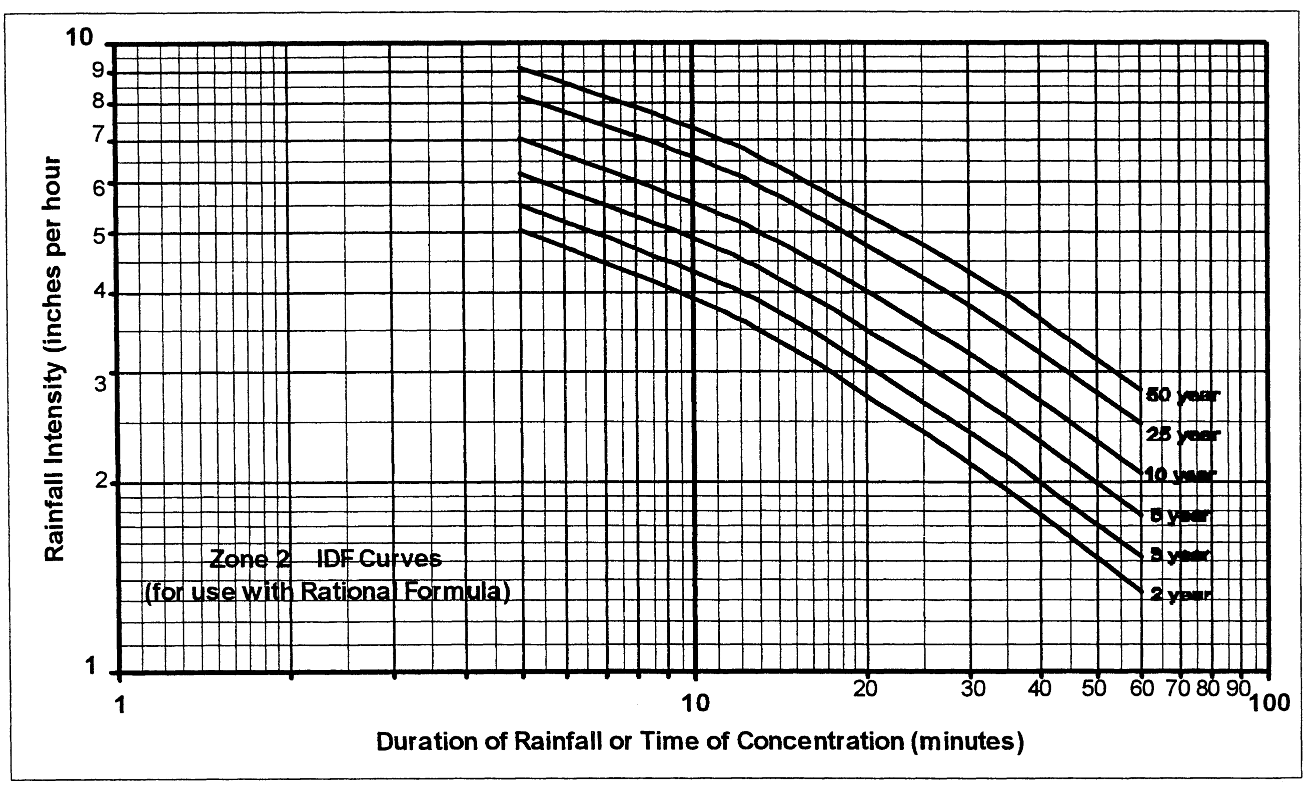

Figure 3.5 Zone 2 Northwest and Central Minnesota Rainfall Intensity - Duration - Frequency (IDF) Curves

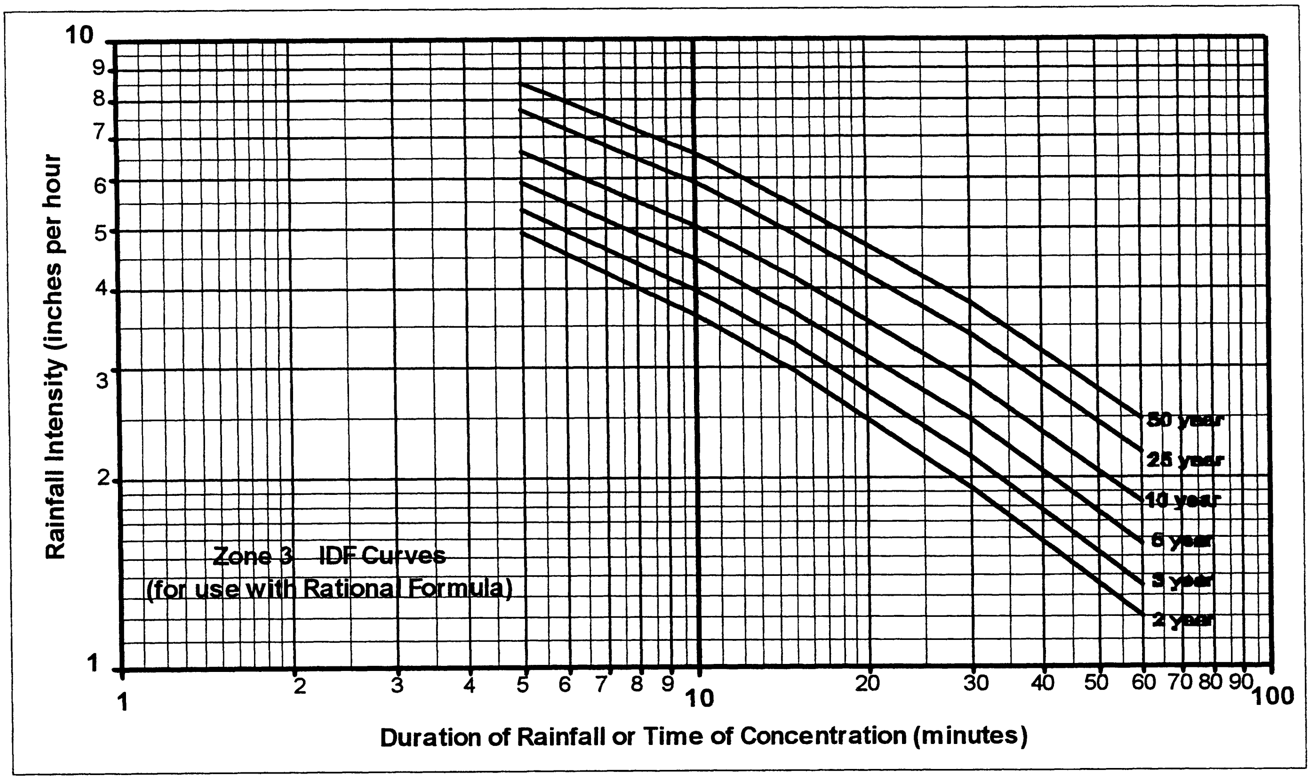

Figure 3.6 Zone 3 Northeast Minnesota Rainfall Intensity - Duration - Frequency (IDF) Curves

3.6 SCS METHOD

The SCS Method was developed by the U. S. Soil Conservation Service (SCS) before the name of the agency changed to the Natural Resources Conservation Service (NCRS). The technique estimates the rate of runoff from the following parameters: drainage area, runoff factor (Curve Number), time of concentration, and rainfall. The SCS approach considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. Direct runoff can be calculated for any storm, either real or fabricated, by subtracting infiltration and other losses from the rainfall to obtain the precipitation excess. A peak flow can be determined graphically, or a hydrograph can be generated. The hydrograph procedure is used primarily for the design and analysis of storage areas. The peak flow procedure is generally used for designing culverts. The SCS procedures are widely used to determine runoff, peak flow, and hydrographs for ungaged watersheds.

The runoff and hydrograph procedures are explained in the SCS *National Engineering Handbook* (SCS, 1985), Section 4 (NEH-4). Additional information is in the *Hydrology Guide for Minnesota* (SCS, ----), *SCS Technical Release No. 55* (SCS, 1986), and *SCS Engineering Field Manual, Chapter 2* (SCS, 1989).

3.6.1 Application

In synthetic hydrograph applications, runoff is often referred to as rainfall excess or effective rainfall. This is defined as the amount of rainfall that exceeds the capability of the land to infiltrate or otherwise retain the rain water. The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope. Land use is the watershed cover, and it includes both agricultural and nonagricultural uses. Items such as type of vegetation, water surfaces, roads and roofs are all part of the land use. Land treatment applies mainly to agricultural land use, and it includes mechanical practices such as contouring or terracing and management practices such as rotation of crops.

The SCS method uses a combination of soil conditions and land-use (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area when the soil is not frozen. The higher the CN, the higher the runoff potential. A relationship between rainfall and runoff was derived by NCRS from experimental plots for numerous soils and vegetative cover conditions. The equation was developed mainly for small watersheds for which only daily rainfall and watershed data are ordinarily available. The SCS runoff equation is therefore a method of estimating direct runoff from 24-hour storm rainfall events.

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad \textbf{(3.13a)}$$

Initial abstraction (I_a) is all losses that occur before runoff begins, including surface storage, interception, and infiltration. Based on studies of experimental watershed data a relationship between I_a and S was developed, where initial abstraction (I_a) is 20% of the maximum retention. Substituting 0.2S for I_a, the SCS rainfall-runoff equation becomes:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad \textbf{(3.13b)}$$

S is related to the soil and cover conditions of the watershed through the Curve Number (CN):

$$S = \frac{1000}{CN} - 10 \qquad \textbf{(3.14)}$$

Where: Q = accumulated direct runoff (inches)
P = accumulated rainfall (inches)
I_a = initial abstraction (inches)
S = potential maximum retention after runoff begins (inches)
CN = SCS Curve Number

3.6.2 Limitations

The following limitations apply to SCS runoff procedures:

- Runoff from snowmelt or rain on frozen ground cannot be estimated using the procedures given in this section.
- The Curve Number procedure is less accurate when runoff is less than 0.5 inch.
- The SCS runoff procedures apply only to direct surface runoff. In some cases, large sources of subsurface flow can contribute to the runoff.
- The TR-55 graphical method may be used only for hydrologically homogeneous watersheds because the procedure is limited to a single watershed subarea where the drainage area is greater than 1.0 acre and less than 2,000 acres.
- The TR-55 tabular hydrograph method can be used for a heterogeneous watershed that is divided into homogeneous subareas. The hydrographs from each subarea can be routed and combined.
- The runoff equation assumes the initial abstraction (I_a) to be 20% of the maximum retention based on studies on small watersheds. I_a consists of interception, initial infiltration, surface depression storage, evapotranspiration, and other factors. This approximation can be significant in urban areas because the combination of impervious areas with pervious areas can imply a significant initial loss that may not take place. The opposite effect can occur if the impervious areas have surface depressions that store some runoff.
- The watershed should have one main stream. If more than one exists, the branches must have nearly equal T_c values.
- The watershed must be hydrologically similar; i.e., able to be represented by a weighted CN. Land use, soils, and cover are distributed uniformly throughout the watershed or subwatershed.
- If the computed T_c is less than 0.1 hour, use 0.1 hour. If the computed T_c is greater than 10 hours, peak discharge should be estimated by using the NEH-4 procedures (SCS, 1985), which are automated in the TR-20 computer program.
- Use the same procedure to estimate time of concentration (T_c) when calculating the peak discharge for both the present and developed conditions of a watershed.
- If depression storage constitutes more than one-third of the total drainage area or if it intercepts the drainage, the procedures in NEH-4 should be used.
- When the weighted CN is less than 40 or more than 98, use another procedure to estimate peak discharge.
- If the drainage area is greater than 10 square miles, the point rainfall should be adjusted using the procedure from Chapter 4, NEH-4 (SCS, 1985) or TP-40 (Hershfield, 1961).

3.6.3 Rainfall

The SCS method is based on a 24-hour storm event with a specified time distribution. For design purposes, a Type I or Type II time distribution may be used in Minnesota. The storm distributions (Figure 3.7) are "typical" time distributions which the NCRS has developed using rainfall records from around the country. To use a distribution it is necessary for the user to obtain the 24-hour rainfall value. Appendix B contains 24 hour rainfall amounts for various return periods. Historical storm events can also be used as input.

For Minnesota, the local office of the NCRS recommends the Type I storm distribution for rural watersheds greater than 30 acres. The Type II storm distribution is recommended for urban watersheds, for small watersheds (less than 30 acres), and steep drainage areas.

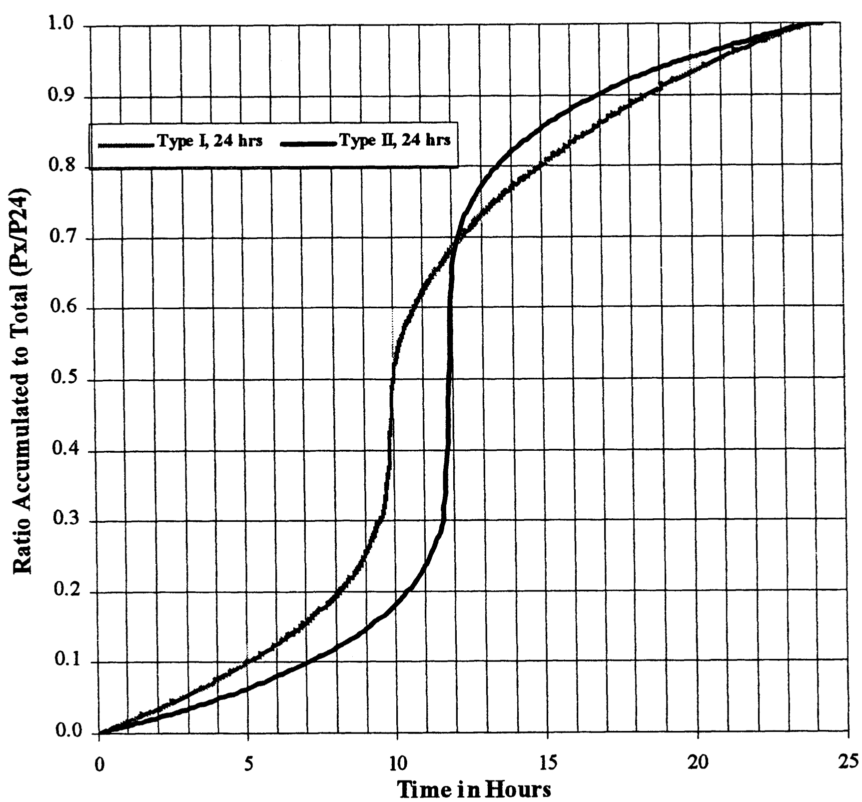

Figure 3.7 Type I and Type II Design Storm Curves
Source: Developed from SCS, 1986 Exhibit 5

3.6.4 Hydrologic Soil Group

Soil properties influence the relationship between runoff and rainfall since different soil groups have different rates of infiltration. Based on these infiltration rates, the Natural Resources Conservation Service (NRCS) has divided soils into four hydrologic soil groups as follows:

Group A Low runoff potential - Soils having high infiltration rates even when thoroughly wetted, consisting chiefly of deep, well to excessively drained sands and/or gravel. These soils have a high rate of water transmission and would result a low runoff potential. Minimum infiltration rate: 0.30 to 0.45 inch per hour.

Group B Soils having moderate infiltration rates when thoroughly wetted, consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission. Minimum infiltration rate: 0.15 to 0.30 inch per hour.

Group C Soils having slow infiltration rates when thoroughly wetted, consisting chiefly of soils with a layer that impedes the downward movement of water, or soils with moderately fine to fine texture and a slow infiltration rate. These soils have a slow rate of water transmission. Minimum infiltration rate: 0.05 to 0.15 inch per hour.

Group D High runoff potential - Soils having very slow infiltration rates when thoroughly wetted, consisting chiefly of clay soils with a high swelling potential; soils with a high permanent water table; soils with claypan or clay layer at or near the surface; and shallow soils over nearly impervious materials. These soils have a very slow rate of water transmission. Minimum infiltration rate: 0 to 0.05 inch per hour.

Soil classifications for Minnesota soils are given in Table 3.9. Where the Soil Class is given as B/D the first letter represents the drained condition and the second letter is the undrained condition. Detailed information on the soil type may be found in the county soil survey report or can be requested from the soils engineer. Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected.

3.6.5 Curve Number

The SCS runoff procedure uses a Curve Number (CN) based on the watershed's soil and cover conditions. The CN describes average conditions that are useful for design purposes. Tables 3.10 and 3.11 provide curve numbers (CN) for urban and agricultural land uses. In selecting the curve number, the Minnesota NCRS recommends that ratings as to "Poor" or "Good" are based largely on the proportion of dense vegetation in the rotation. "Good" will generally be used for cultivated land in Minnesota except where land is very droughty or severely abused.

Curve numbers are determined based upon soil hydrologic groups present in the drainage area, and are also influenced by land use type (e.g., cultivated versus forested). These soil hydrologic groups are discussed in detail in Section 3.6.4. The soil hydrologic groups (A-D) are primarily a function of soil texture and structure as well as moisture content. Group A soil is most permeable, while groups B-D are progressively less permeable, with group D exhibiting the least infiltration and most runoff per unit land area. The influence of the soil hydrologic groups upon the curve numbers for various land use types is shown in Table 3.11. The soil hydrologic group, condition of the land, and its use are all important for determining the curve number and estimating runoff.

The most accurate way to determine the curve number is to identify the soil group or groups in the drainage area in question using a county soil survey from Natural Resources Conservation Service (NRCS). This should be available in a local or county NRCS office and/or it may be stored in the Mn/DOT District Office. As of the year 2000, approximately 75% of the state of Minnesota was mapped by NRCS into soil classifications with the respective soil groups referenced accordingly. Table 3.9 is similar to the soil classification and soil hydrologic group reference data that can normally be found in the NRCS county soil survey in a table referred to as "Water Features" near the end of the document. Note that each soil classification is assigned a soil hydrologic group in Table 3.9.

When the SCS method of predicting runoff is used, the NRCS soil survey data should be employed whenever possible as it is the best soil information available in most cases. If soil hydrologic group data is unavailable through the county surveys, another good data source is the NRCS "STATSGO" GIS database. The scale of this data base map is 1:250,000 and views of the drainage area being investigated can be manipulated using Arc View. Hydrologic soil groups are presented as a GIS theme in Arc View using the STATSGO database.

Curve numbers are often finalized after field inspection of the watershed and review of aerial photographs, zoning maps, and soil type maps. Future changes in land use that might occur during the service life of the proposed facility should be evaluated as such changes could lead to a larger volume of runoff and higher peak discharges. Also, consider the effects of upstream detention facilities and local stormwater management plans may be taken into account. Compute a weighted curve number for the drainage area. Sum up the products of area and CN for each surface/soil type, and divide by the total drainage area.

$$\text{Weighted } CN = \frac{CN_1 A_1 + CN_2 A_2 + \ldots + CN_n A_n}{Total A} \tag{3.15}$$

Where: CN = Curve Number
A = Area

Urban development has its own unique charactersitics. Impervious surfaces such as roads, parking lots, and buildings cover a significant amount of an urban watershed. The urban CN's given in Table 3.11 apply for typical land use, assuming percentages of impervious area specified in the table. Curve numbers with other percent imperviousness can be calculated using a weighted CN approach. The impervious areas are assumed to have a CN of 98 and pervious areas are assumed to be equivalent to open space in good condition.

$$CN_w = CN_p(1-f) + f(98) \tag{3.16}$$

Where: CN_w = weighted Curve Number
CN_p = pervious Curve Number from Table 3.11
f = fraction of imperviousness (decimal value)

The urban CN's listed in Table 3.11 were developed with the assumption that impervious areas are directly connected to the drainage system. The impervious area is considered connected if the runoff flows directly into the drainage system, or flows as concentrated shallow flow over a pervious area and then into a drainage system. Runoff from an unconnected impervious area is spread over a pervious area as sheet flow and may result in reduced peak runoff rates and volumes of direct flood runoff. An adjusted CN can be calculated with Equation 3.17 if the total impervious area is less than 30%. TR-55 (SCS, 1986) includes more information on the procedure to modify the CN for impervious area.

$$CN_c = CN_p + \left(\frac{P_i}{100}\right)(98 - CN_p)(1 - 0.5R) \qquad \text{For } P_i \le 30\% \tag{3.17}$$

Where: CN_c = composite Curve Number
CN_p = pervious Curve Number from Table 3.11
P_i = percent impervious (%)
R = ratio of unconnected impervious area to the total impervious area

Table 3.9 Hydrologic Soil Classifications for Minnesota

Aastad	B	Blue Earth	B/D	Dalbo	B	Flak	C	Hidewood	C	Lohnes	A
Aazdahl	B	Bluffton	C/D	Darfur	B/D	Flaming	A	Hillet	C/D	Lomax	B
Adolph	B/D	Bold	B	Darnen	B	Flandreau	B	Hiwood	A	Loxley	A/D
Adrian	A/D	Boone	A	Dassel	B/D	Flom	B/D	Hixton	B	Lupton	A/D
Afton	C/D	Boots	A/D	Dawson	A/D	Floyd	B	Holdingford	C	Lura	C/D
Ahmeek	C	Borup	B/D	Deerwood	B/D	Foldahl	B	Houghton	A/D	Maddock	A
Alcester	B	Braham	B	Derinda	C	Forada	B/D	Hubbard	A	Madelia	B/D
Allendale	B	Brainerd	C	Dickey	A	Fordville	B	Huntsville	B	Mahtowa	C/D
Almena	C	Bremer	C	Dickinson	B	Forman	B	Ihlen	B	Malachy	B
Alstad	B	Brickton	C	Dickman	A	Formdale	B	Indus	D	Marcus	B/D
Alvin	B	Brill	B	Dinsdale	B	Fossum	A/D	Insula	D	Markey	A/D
Amery	B	Brodale	C	Divide	B	Foxhome	B	Isan	A/D	Marlean	B
Ames	C/D	Brookings	B	Dodgeville	B	Fram	B	Isanti	A/D	Marna	D
Ankeny	B	Brophy	A/D	Doland	B	Freeon	B	Jackson	B	Marquette	A
Anoka	B	Browntown	C/D	Donaldson	B	Freer	C	Joliet	D	Marshan	B/D
Antigo	B	Burkhardt	B	Donnan	C	Frontenac	B	Joy	B	Marysland	B/D
Arcola	C	Burnsville	B	Doran	C	Fulda	C/D	Judson	B	Mavie	B/D
Aredale	B	Buse	B	Dorchester	B	Gale	B	Kamrar	B	Maxcreek	B/D
Arenzville	B	Calamine	C/D	Dorset	B	Galva	B	Kanaranzi	B	Maxfield	B/D
Arland	B	Calco	C/D	Dovray	C/D	Garnes	B	Karlstad	A	Mayer	B/D
Arveson	A/D	Campia	B	Downs	B	Garwin	C/D	Kasota	C	Mazaska	C/D
Arvilla	B	Canisteo	C/D	Dubuque	B	Glencoe	B/D	Kasson	C	McDonaldsville	C/D
Athelwold	B	Carlos	A/D	Duelm	A	Glyndon	B	Kato	C	McIntosh	B
Atkinson	B	Caron	A/D	Duluth	C	Gonvick	B	Kegonsa	B	McPaul	B
Auburndale	C/D	Cashel	C	Dunbarton	D	Gotham	A	Kennebec	B	Medary	C
Augsburg	B/D	Cathro	A/D	Dundas	B/D	Granby	A/D	Kenyon	B	Meehan	A/D
Automba	B	Cannahon	D	Dunnville	B	Grays	B	Kilkenny	B	Menagha	A
Badger	C/D	Chaseburg	B	Dusler	C	Greenwood	A/D	Kingsley	B	Meridian	B
Barbert	D	Chaska	B/D	Eckman	B	Grimstad	B	Kingston	B	Merton	B
Barnes	B	Chelsea	A	Edison	B	Grogan	B	Kittson	C	Merwin	A/D
Baroda	D	Chetek	B	Edwards	B/D	Growton	B	Klinger	B	Mesaba	C
Barrington	B	Chilgren	C	Egeland	B	Grygla	B/D	Kranzburg	B	Metogga	A/D
Barronett	B/D	Clarion	B	Elderon	B	Guckeen	C	Kratka	B/D	Milaca	C
Barrows	B/D	Clontarf	B	Eleva	B	Halder	C	LaPrairie	B	Millerville	A/D
Barto	B/D	Cloquet	B	Ely	B	Hamar	A/D	Lamont	B	Millington	B
Baudette	B	Clyde	B/D	Embden	B	Hamel	C	Lamoure	C	Minneiska	C
Bearden	C	Collinwood	C	Emmert	A	Hamerly	C	Langhei	B	Minneopa	B
Beauford	D	Colo	B/D	Enloe	D	Hangaard	A/D	Langola	B	Minnetonka	D
Becker	B	Colvin	C/D	Enstrom	B	Hanska	C	Lasa	A	Moland	B
Bellechester	A	Comfrey	B/D	Erin	B	Hantho	B	Lawler	B	Moody	B
Beltrami	B	Conic	C	Estelline	B	Harps	B/D	Lawson	B	Moose Lake	A/D
Bena	A	Capaston	D	Estherville	B	Harpster	B/D	LeSueur	B	Mora	C
Benoit	B/D	Cordova	C/D	Etter	B	Hatfield	B/D	Lemond	B/D	Mosomo	A
Beotia	B	Cormant	A/D	Everly	B	Hattie	C	Lerdal	C	Mt. Carroll	B
Bergland	D	Crippin	B	Eyota	A	Haug	B/D	Lester	B	Muscatine	B
Bertrand	B	Crocker	A	Fairhaven	B	Havana	B	Letri	B/D	Muskego	A/D
Beseman	A/D	Corfton	B	Fargo	C	Hayden	B	Lilah	A	Nebish	B
Billett	B	Cromwell	A	Farrar	B	Hayfield	B	Linder	B	Nemadji	B
Biscay	B/D	Curran	C	Faxon	B/D	Hecla	A	Lindstrom	B	Nereson	B
Bixby	B	Cushing	B	Fayette	B	Hegne	C/D	Lino	B	Nessel	B
Blackhoof	C/D	Cutfoot	A	Fedji	A	Hesch	B	Lismore	B	Newfound	C
Blomford	B/D	Cylinder	B	Fieldon	B/D	Heyder	B	Litchfield	A	Newglarus	B
Blooming	B	Dakota	B	Finchford	A	Hibbing	C	Lobo	D	Newry	B

Source: SCS Hydrology Guide for Minnesota

Table 3.9 (continued) Hydrologic Soil Classifications for Minnesota

Newson	A/D	Palms	A/D	Rockton	B	Shields	C	Tama	B	Vlasaty	C
Nicollet	B	Palsgrove	B	Rockwell	B/D	Shooker	C	Taopi	C	Wacousta	B/D
Nokasippi	D	Parent	B/D	Rockwood	C	Shorewood	C	Tara	B	Wadena	B
Nokay	C	Parnell	C/D	Rolfe	C/D	Shullsburg	C	Tawas	A/D	Wahpeton	C
Nordness	B	Pelan	B	Roliss	B/D	Sinai	C	Taylor	C	Waldorf	C/D
Normania	B	Percy	B/D	Rondeau	A/D	Singsaas	B	Tell	B	Warba	B
Northcote	C/D	Perella	B/D	Ronneby	C	Sioux	A	Terril	B	Warman	B/D
Nowen	B/D	Plainfield	A	Rosemount	B	Skyberg	C	Tilfer	B/D	Waskish	A/D
Noyes	C/D	Poinsett	B	Rosendale	B	Sletten	B/D	Timula	B	Watab	C
Nutley	C	Pomroy	B	Roseville	B	Soderville	A	Toddville	B	Watseka	A
Nymore	A	Poppleton	A	Rosholt	B	Sogn	D	Toivola	A	Waubay	B
Oak Lake	B	Port Byron	B	Rothsay	B	Sparta	A	Tonka	C/D	Waubeek	B
Ocheyedan	B	Prebish	C/D	Rushmore	B/D	Spencer	C	Torning	B	Waucoma	B
Ogilvie	B/D	Primghar	B	Ryan	D	Spicer	B/D	Towner	B	Waukee	B
Okoboji	B/D	Protvin	C	Sac	B	Spillville	B	Trent	B	Waukegan	B
Oldham	C/D	Quam	B/D	Salinda	A	Spooner	C/D	Tripoli	B/D	Waukon	B
Omega	A	Quetico	D	Santiago	B	Spottswood	B	Trosky	B/D	Webster	B/D
Onamia	B	Racine	B	Sargeant	D	Storden	B	Truman	B	Whalan	B
Ontonagon	D	Radford	B	Sartell	A	Strandquist	B/D	Twig	A/D	Wheatville	B
Opole	B	Ransom	B	Sattre	B	Stronghurst	B	Udolpho	B/D	Whitewood	C/D
Orion	B	Rasset	B	Sawmill	B/D	Stuntz	C	Ulen	B	Wildwood	C/D
Oronoco	B	Rauville	C/D	Schapville	C	Suamico	A/D	Upsala	C	Wilmonton	B
Osakis	B	Readlyn	B	Schley	B	Svea	B	Urness	B/D	Winger	B/D
Oshawa	C/D	Redby	B	Seaforth	B	Sverdrup	B	Vallers	C	Wyndmere	B
Ossian	B/D	Renova	B	Seaton	B	Swenoda	B	Vasa	B	Zell	B
Ostrander	B	Renshaw	B	Seelyeville	A/D	Syrene	B/D	Ves	B	Zimmerman	A
Otter	B/D	Rib	C	Shakopee	C/D	Talcot	B/D	Vienna	B	Zumbro	A
Otterholt	B	Richwood	B	Shawano	A	Tallula	B	Viking	D	Zwingle	D
Paget	C	Rifle	A/D	Shible	B						

Source: SCS Hydrology Guide for Minnesota

Table 3.10 SCS Curve Numbers for Rural Land Uses

Cover		Condition or Rotation	Acres Per Practice	Curve Numbers for Moisture Condition II				Product
				A Soils	B Soils	C Soils	D Soils	
Fallow	Straight Row			77	86	91	94	
Row Crops	Straight Row	Poor		72	81	88	91	
		Good		67	78	85	89	
		Mulch till		61	76	84	87	
	Contoured[2]	Poor		70	79	84	88	
		Good		65	75	82	86	
		Mulch till		62	73	80	85	
	C and T[1]	Poor		66	74	80	82	
		Good		62	71	78	81	
		Mulch till		61	70	77	80	
Small Grain	Straight Row	Poor		65	76	84	88	
		Good		63	75	83	87	
		Mulch till		58	74	82	86	
	Contoured[2]	Poor		63	74	82	85	
		Good		61	73	81	84	
		Mulch till		59	72	80	83	
	C and T[1]	Poor		61	72	79	82	
		Good		59	70	78	81	
		Mulch till		58	69	77	80	
Legumes or Rotation Meadow	Straight Row	Poor		66	77	85	89	
		Good		58	72	81	85	
	Contoured[2]	Poor		64	75	83	85	
		Good		55	69	78	83	
	C and T[1]	Poor		63	73	80	83	
		Good		51	67	76	80	
Pasture		Poor		68	79	86	89	
		Fair		49	69	79	84	
		Good		39	61	74	80	
Meadow (Permanent)		Good		30	58	71	78	
Wood or Forest Land		Poor		45	66	77	83	
		Fair		36	60	73	79	
		Good		25	55	70	77	
Farmsteads		- - -		59	74	82	86	
Roads (including R/W)	Dirt Surface	- - -		72	82	87	89	
	Hard Surface	- - -		74	84	90	92	
Impervious Surface		- - -		100	100	100	100	
Water Surface (lakes, ponds)		- - -		100	100	100	100	
Swamp	Open water[3]	- - -		85	85	85	85	
	Vegetated[4]	- - -		78	78	78	78	
Residential	Low Density	- - -		47	65	76	82	
	Medium Density	- - -		54	70	79	84	
	High Density	- - -		70	81	87	90	
Commercial and Industrial		- - -		86	91	93	94	

Total Acres = ____________ Product Total = ____________

$$\text{Weighted Runoff Curve Number} = \frac{\text{Product Total}}{\text{Total Acres}} = \underline{\hspace{6cm}} = \underline{\hspace{6cm}}$$

Source: SCS Hydrology Guide for Minnesota Figure 3-1

[1] Contoured and graded terraces or land with less tan 2% slope

[2] Includes level terraced areas (runoff corrected by volume)

[3] 1/3 of swam surface is open water

[4] Swamp has no open water and the design is a 25-year frequency or less.

Table 3.11 SCS Curve Numbers for Urban Land Uses

Cover	Condition or Rotation	Acres Per Practice	Curve Numbers for Moisture Condition II				Product
			A Soils	B Soils	C Soils	D Soils	
Cultivated Land	Without conservation treatment		72	81	88	91	
	With conservation treatment		62	71	78	81	
Pasture or Range Land	Poor		68	79	86	89	
	Good		39	61	74	80	
Meadow	Good		30	58	71	78	
Wood or Forest Land	Poor: thin stand, no mulch		45	66	77	83	
	Good		25	55	70	77	
Open Spaces (lawns, parks, golf courses, cemeteries)	Good: 75% or more grass cover		39	61	74	80	
	Fair: 50% to 75% grass cover		49	69	79	84	
Commercial and Business Areas (85% impervious)			89	92	94	95	
Industrial Districts (72% impervious)			81	88	91	93	
Residential	≤ 1/8 acre lot 65% impervious		77	85	90	92	
	1/4 acre lot 38% impervious		61	75	83	87	
	1/3 acre lot 30% impervious		57	72	81	86	
	½ acre lot 25% impervious		54	70	80	85	
	1 acre lot 20% impervious		51	68	79	84	
Paved Parking Lots, Roofs, Driveways			98	98	98	98	
Streets and Roads	paved with curb and storm drain		98	98	98	98	
	gravel		76	85	89	91	
	dirt		72	82	87	89	
Marsh			85	85	85	85	
Other							

Total Acres = ____________ Product Total = ____________

$$\text{Weighted Runoff Curve Number} = \frac{\text{Product Total}}{\text{Total Acres}} = \underline{\qquad\qquad\qquad} = \underline{\qquad\qquad\qquad}$$

Source: SCS Hydrology Guide for Minnesota Figure 3-2

1 For land uses with impervious areas, curve numbers are computed assuming that 100% of runoff from impervious areas is directly connected to the drainage system. Pervious areas (lawn) are considered to be equivalent to lawns in good condition and the impervious areas have a CN of 98

2 Includes paved streets

Table 3.12 Rainfall Groups For Antecedent Runoff Conditions

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

Source: Soil Conservation Service

Antecedent Runoff Condition (ARC) is an index of runoff potential before a storm event, and attempts to account for variation in CN at a site from storm to storm. (ARC may also be called the Antecedent Moisture Condition, AMC). Table 3.12 describes the three conditions

The average antecedent runoff condition is generally recommended for design purposes. The wet and dry conditions are used for simulating historic conditions. The wet antecedent runoff condition may also be used for a conservative design. Table 3.13 shows the converted curve numbers for other antecedent moisture conditions.

Table 3.13 ARC Conversion

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

Source: Soil Conservation Service

3.6.6 Peak Discharge Procedure

The NCRS developed a graphical procedure to estimate the peak discharge from a small watershed. Rainfall, curve number, time of concentration, and drainage area, are required. Rainfall is estimated from the type of storm distribution, and 24-hour rainfall for the watershed. The watershed drainage area must be greater than 1.0 acre and less than 2,000 acres. If the drainage area is outside these limits, use TR-20 procedures. The peak discharge equation is:

$$q_p = q_u A Q F_p \tag{3.18}$$

Where: q_p = peak discharge (cfs)
q_u = unit peak discharge (cms/in)
A = drainage area (mi^2)
Q = direct runoff (in)
F_p = pond and swamp adjustment factor
Only use adjustment factor for ponds or swamps that are not in the t_c flow path.
(See Table 3.14)

The peak discharge can be calculated using the following procedure:

Table 3.14 Adjustment Factor (F_p) for pond and swamp areas

Percent of pond and swamp area (%)	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

Source: SCS, 1986

Step 1 Determine drainage area (A). The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas, it might be necessary to divide the area into subareas to account for major land use changes, obtain analysis results at different points within the drainage area, or locate stormwater drainage facilities and assess their effects on flood flow. When subareas are used, a hydrograph procedure is required to determine the combined discharge. Also a field inspection of existing or proposed drainage systems should be made to determine if the natural drainage divides have been altered. These alterations could make significant changes in the size and slope of the sub-drainage areas.

Step 2 Determine curve number (CN).

A. Determine soil group from Table 3.9.
B. Use Table 3.10 or 3.11 to find CN
C. If area is urbanized, determine if default assumptions are valid. If the % impervious is not the same as the default values listed in Table 3.11, or impervious areas are not directly connected to the drainage system then adjust the CN using Equations 3.16 and 3.17.
D. Calculate Weighted CN, Equation 3.15.
E. Consider Antecedent Moisture Condition. If Average (II) is not valid, use Table 3.13 to adjust the CN value.

Step 3 Determine the design frequency according to guidance in Section 3.2.

Step 4 Determine Time of Concentration t_c. Time of concentration (t_c) influences the shape and the peak of the runoff hydrograph. The SCS defines t_c as the time required for water to travel from the most hydraulically distant point in a watershed to the outlet. SCS methods generally require t_c in hours. Procedures to calculate Time of Concentration are in Section 3.4.

Step 5 Determine P, the 24 hour rainfall (inches), for the design frequency from Appendix B.

Step 6 Find the initial abstraction (I_a) from Table 3.15 using the watershed CN.

Step 7 Calculate I_a/P, this ratio is a parameter that indicates how much of the total rainfall is needed to satisfy the initial abstraction. The larger the I_a/P ratio, the lower the unit peak discharge (q_u) for a given t_c. If the computed I_a/P ratio is outside the range of 0.1 to 0.50, then the limiting values should be used; i.e., use 0.1 if less than 0.1 and use 0.5 if greater than 0.5.

Step 8 Determine q_u, unit peak discharge, from Figure 3.8 or 3.9 depending on the rainfall distribution type. If the ratio falls between the limiting values, use linear interpolation

Step 9 Find Q, runoff (inches), using Equations 3.13b and 3.14.

Step 10 Calculate q_p, peak discharge (cfs), from Equation 3.18. Apply F_p (Table 3.14) if applicable.

Table 3.15 I_a Values for Runoff Curve Numbers

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

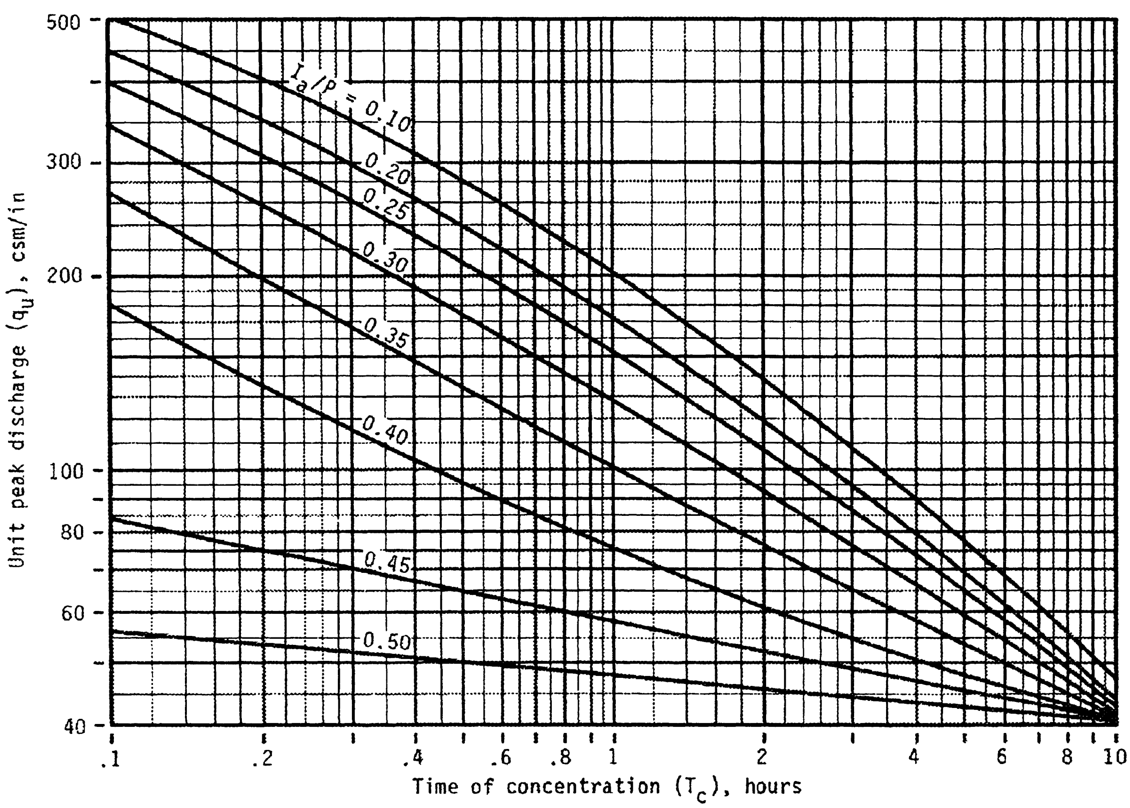

Figure 3.8 Unit Peak Discharge (q_u) for SCS Type I Rainfall Distribution
Source: Engineering Field Manual, Chapter 2 (SCS, 1989)

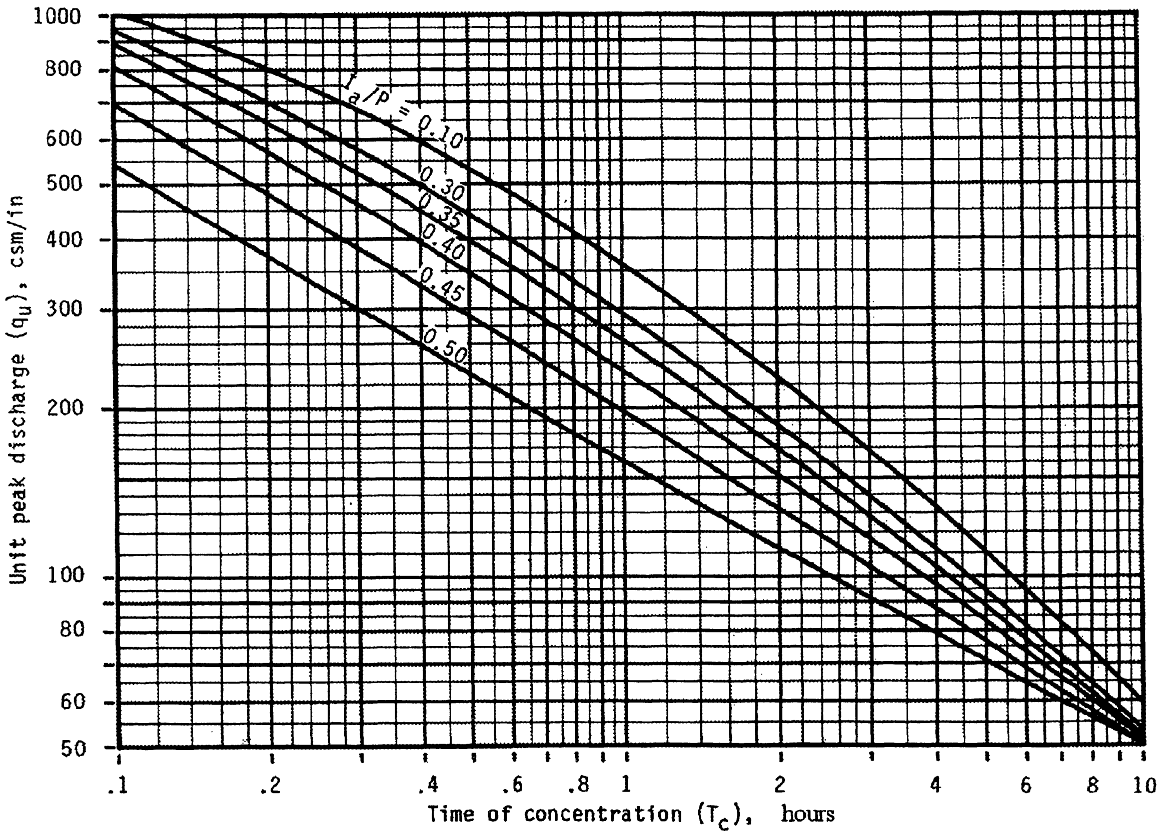

Figure 3.9 Unit Peak Discharge (q_u) for SCS Type II Rainfall Distribution
Source: Engineering Field Manual, Chapter 2 (SCS, 1989)

3.6.7 SCS Hydrograph Procedure

The SCS hydrograph is a synthetic unit hydrograph developed by analysis of many natural unit hydrographs. A unit hydrograph represents the time distribution of flow resulting from one inch of direct runoff occurring over the watershed in a specified time. The hydrograph shape is dependant on the peak discharge for a given rainfall and the basin lag. Basin lag is the time from the center of mass of rainfall excess to the hydrograph peak. The hydrograph can be used to model drainage basins of various sizes and shapes. Steep slopes, compact shape, and an efficient drainage network tend to make lag time short and peaks high; flat slopes, elongated shape, and an inefficient drainage network tend to make lag time long and peaks low.

Hydrograph development and routing is generally done with a computer. TR-20 (SCS, 1982) is a computer program developed by NCRS that develops hydrographs, routes the hydrographs through storage, and combines hydrographs. TR-20, or another computer program that utilizes TR-20 or TR-55 methodology, is generally recommended when a hydrograph is needed for design. NCRS used results from running TR-20 to develop a simplified graphical procedure to obtain the peak discharge for small watersheds. This procedure is outlined in *Hydrology Guide for Minnesota* (SCS), *SCS Engineering Field Manual, Chapter 2* (SCS, 1989), and *SCS Technical Release No. 55* (SCS, 1986).

A brief explanation of some aspects of the SCS procedure and the TR-20 computer model are given below. Further information can be found in the TR-20 manual and NEH-4. Consult the User's Manual to use TR-20 or another computer program to develop a hydrograph using the SCS procedures. The SCS procedure to estimate the peak rate of discharge is:

$$q_p = \frac{484AQ}{T_p} \tag{3.19}$$

Where: q_p = peak rate of discharge (cfs)
A = area (mi^2)
Q = storm runoff (inches)
T_p = time to peak (hrs)

$$T_p = \frac{D}{2} + L \tag{3.20}$$

Where: D = storm duration, hrs
L = watershed lag, hrs (on average L = 0.6 t_c)

Once q_p and T_p have been determined, a synthetic unit hydrograph can be constructed. The constant 484, or peak rate factor, is valid for the SCS dimensionless unit hydrograph and is the basis for the graphical procedures and the default hydrograph in TR-20. This constant affects the shape of the hydrograph and has been known to vary from about 600 in steep terrain to 300 in very flat swampy area. NEH-4 contains further information on the hydrograph procedures.

The TR-20 computer program is a single event model which computes direct runoff resulting from a synthetic or natural rainstorm. It develops hydrographs from runoff and routes the flow through stream channels and reservoirs. It combines the routed hydrograph with those from tributaries, and computes the peak discharges, their times of occurrence and the water surface elevations at any desired cross-section or structure. The watershed should be divided into as many subareas as necessary to define hydrologic and structural effects. Tributaries, watershed shape, valley slope, runoff curve number homogeneity, and water storage areas can all determine where subareas are required. Each sub-watershed is assumed to be hydrologically homogeneous. For each sub-watershed, the user will be required to enter the drainage area in square miles, the runoff curve number, and the time of concentration in hours. For best results in using the TR-20 computer program, the time of concentration for each subarea should be of the same magnitude, with the longest t_c less than 3 times the shortest t_c. The largest sub-watershed area should be less than 5 times the smallest sub-watershed area. The output from the program can be affected by the main time increment which is user supplied. In general, use the time increment equal to 0.1 to 0.2 times the shortest t_c, but not less than 0.1 hour. The user may need to try several different time increments.

3.7 USGS REGRESSION EQUATIONS

Regional regression equations are a commonly accepted method for estimating peak flows at ungaged sites or sites with insufficient data. Regression studies are statistical practices used to develop runoff equations. These equations are used to relate peak flow at a specified recurrence interval to the watershed's physiographic, hydrologic and meteorological characteristics The U.S. Geological Survey (USGS) developed the regression equations for Minnesota and published them in *Techniques for Estimating Magnitude and Frequency of Floods in Minnesota* (Guetzkow, 1977). Since then regression analysis was redone and new equations developed and published twice. Once with the same title *Techniques for Estimating the Magnitude and Frequency of Floods in Minnesota* (Jacques and Lorenz, 1988) developed in 1987 and published in 1988, and also as *Techniques for Estimating Peak Flow on Small Streams in Minnesota* (Lorenz, Carlson and Sanocki, 1997).

Regression analyses use stream gage data to define hydrologic regions. These are geographic regions having very similar flood frequency relationships and, as such, commonly display similar characteristics. Separate regression analysis were performed and equations developed at 2-, 5-, 10-, 25-, 50-, and 100-year recurrence intervals for each hydrologically similar region. The most significant characteristic of the equations is the drainage area size above the point of interest. Additional basin characteristics such as main channel slope, lake storage, total storage and mean annual runoff are included as independent variables when they were determined to be statistically significant.

3.7.1 Application

The USGS Regression Equations may be used to design culverts and bridges in Minnesota. The USGS Regression Equations, obtained from multiple-regression analyses of gaging-station data in each hydrologic region, can be used to obtain flood-frequency estimates for ungaged sites on unregulated streams. Peak discharges for selected recurrence intervals can be computed from the empirical equations that relate flood magnitude to basin characteristics. Historical observations and other methods, should be used to evaluate the results. Where there is stream gage data, the findings from a Log Pearson III method should govern, provided there is at least 10 years of stream gage record. Reasonable and prudent judgment along with consideration of the standard regression error shall be used in reaching a design decision. Typical regression equation format:

$$Q_i = aA^b S^c \left(St + 1\right)^d \tag{3.21}$$

Where: Q_i = flood frequency estimate for recurrence interval i
A = drainage area in square miles
S = main channel slope in feet per mile
St = percent of drainage area occupied by lakes, ponds or wetlands
a, b, c, d = variables

The regression equations, hydrologic region boundaries, variable definitions, procedures for use and discussion on accuracy and limitations are provided in the USGS reports listed in Section 3.7 and will not be duplicated in this manual.

3.7.2 Limitations

The basin characteristics of the stream being analyzed should be within the limits of those used to develop the equations. Each USGS report has a table showing the upper and lower limits for basins characteristics used in the regression analysis; care should be exercised using equation outside of the specified limits. The regression equations are intended to be used on streams which are not significantly affected by manmade regulation, urbanization, or diversion. The equations should not be used immediately downstream of a lake or ponding area, but may be used to determine the peak inflow to be routed through an impoundment.

3.7.3 Procedure

There are three sets of equations that can be applied to a particular site. These include the 1997 regression equations developed by Lorenz, Carlson and Sanocki; 1987 regression equations developed by Jacques and Lorenz; and the 1977 regression equations developed by Guetzkow. Although the later equations were intended to supersede earlier equations, it is advisable to compute the discharge using multiple methods, and compare to other historical data or methods. The 1987 procedure divides the state into 4 hydrologic regions, where the 1977 procedure used 8 different hydrologic regions in the state and the 1997 procedures use 6 regions. Although the later reports used gaging stations with longer periods of records and more stations, it is valid to use the earlier 1977 and 1987 equations as a check.

Problems related to hydrologic boundaries may occur in selecting the appropriate regression equation. Because of the distance between stream gages, the regional boundaries cannot be considered as precise. The watershed of interest may lie partly within two or more hydrologic regions or it may lie totally within a hydrologic region, but close to a hydrologic region boundary. In these instances care must be exercised in using regression equations. A field visit is recommended to first collect all available historical flood data as well as to compare the project's watershed characteristics with those of the abutting hydrologic regions. For drainage areas near regional divides, the equations for both regions should be evaluated and the results compared. The coefficients in the equations can also be used to transfer flood flows from a gaged site to an ungaged site on the same stream. The regional boundaries generally follow watershed basin divides.

3.8 ANALYSIS OF STREAM GAGE DATA

Many gaging stations exist throughout Minnesota where data can be obtained and used for hydrologic studies. If a project is located near one of these gages and the gaging record is of sufficient length of time, a frequency analysis may be made. The most important aspect of applicable station records is the series of annual peak discharges. It is possible to apply a frequency analysis to that data for the derivation of flood-frequency curves. Such curves can then be used in several different ways.

- If the subject site is at or very near the gaging site and on the same stream and watershed, the discharge for a specific frequency from the flood-frequency curve can be used directly.
- If the facility site is nearby or representative of a watershed with similar hydrologic characteristics, transposition of frequency discharges is possible.
- If the flood-frequency curve is from one of a group of several gaging stations comprising a hydrologic region, then regional regression relations may be derived. Regional regression relations are usually furnished by established hydrologic agencies and the designer will not be involved in their development.

3.8.1 Application

Mn/DOT may use a frequency analysis of stream gage data at appropriate locations when there is sufficient years of measured stream gage record. The preferred method of estimating flood frequency curves from stream gage data is a statistical method, which makes use of the Log Pearson Type III frequency distribution.

The analysis of gaged data permits an estimate of the peak discharge for the desired return period at a particular site. Experience has shown that statistical frequency distributions may be more representative of naturally occurring floods and can be reliable when used for prediction. Although several different distributions are used for frequency analysis, experience has shown the Log-Person Type III distribution to be one of the most useful. The Log-Pearson III distribution and the process of fitting it to a particular data sample are described in detail in Water Resources Council Bulletin 17B, *Guidelines for Determining Flood Flow Frequency*, (WRC, 1981). Special handling of outliers, historical data, incomplete data, and zero flow years is covered in detail in Bulletin 17B.

There are two alternative methods for determining the value of the skew coefficient to be used in calculating the Log-Pearson Type III curve fit. The value of skew that is calculated directly from the gage data using the above formula is called the station skew. This value may not be a true representation of the actual skew of the data if the period of record is short or if there are extreme events in the period of record. Often, the station skew and the generalized skew can be combined to provide a better estimate for a given sample of flood data.

3.8.2 Transferring Gaged Data

Gaged data may be transferred to an ungaged site on the same river as the gaged site provided such data are nearby, within the same hydrologic region, and there are no major tributaries or diversions between the gage and the site of interest. These procedures make use of the constants obtained in developing the regression equations. To transfer discharge data from a gaged site to an ungaged site on the same stream:

$$Q_u = Q_g \left(\frac{A_u}{A_g} \right)^B \tag{3.22}$$

Where: Q_u = flood frequency estimate to ungaged site
Q_g = flood frequency estimate for gaged site
A_u = drainage area for ungaged site
A_g = drainage area for gaged site
B = exponent for drainage area from the appropriate regression equation

This transfer relation can be used where drainage area size differs by no more than 50%. If other basin characteristics differ significantly, they should also be included by taking the ratio of the parameters raised to the power given in the regression equation. If the period of record at the gaged site is short, a weighted average of the results of the transfer equation and regression equation should be used.

The transposition of design discharges from one basin to another basin with similar hydrologic characteristics is accomplished by multiplying the design discharge by the direct ratio of the respective drainage areas raised to the drainage area exponent given in the appropriate regression equation. Thus on streams where no gaging station is in existence, records of gaging stations in nearby hydrologically similar watersheds may be used. The discharge for such an ungaged stream may be determined by the transposition of records using a similar procedure. This procedure is repeated for each available nearby watershed and the results are averaged to obtain a value for the desired flood frequency relationships in the ungaged watershed.

3.9 REFERENCES

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