# 7. Hydrology



## 7.1. Hydrologic Design Policies

### *7.1.1 Introduction*

Following is a summary of policies for hydrologic analysis. For a more detailed discussion refer to the publication, Highway Drainage Guidelines, published by the American Association of State Highway and Transportation Officials.

### *7.1.2 Surveys*

Since hydrologic considerations can influence the selection of a highway corridor and the alternate routes within the corridor, studies and investigations, including consideration of the environmental and ecological impact of the project, should be undertaken. Also special studies and investigations may be required at sensitive locations. The magnitude and complexity of these studies should be commensurate with the importance and magnitude of the project and problems encountered. Typical data to be included 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 floodplain areas.

### *7.1.3 Flood Hazard*

A hydrologic analysis is prerequisite to identifying flood hazard areas and determining those locations at which construction and maintenance will be unusually expensive or hazardous

### *7.1.4 Coordination*

Since many levels of government plan, design, and construct highway and water resource projects which might have an affect on each other, interagency coordination is desirable and often necessary. In addition agencies can share data and experiences within project areas to assist in the completion of accurate hydrologic analysis.

### *7.1.5 Documentation*

Experience indicates that the design of highway drainage facilities should be adequately documented. Frequently, it is necessary to refer to plans and specifications long after the actual construction has been completed. Thus it is necessary to fully document the results of all hydrologic analysis. All documentation, computations and reporting of results shall be performed in the units (English or Metric) of the base data provided, with final design recommendations presented in metric. Published data shall not be converted. Further, design flood computation methods such as regression equations must use the units for which they were developed.

### *7.1.6 Factors Affecting Flood Runoff*

For all hydrologic analysis, the following factors should be evaluated and included when they will have a significant effect on the final results.

- Drainage basin characteristics including: size, shape, slope, land use, geology, soil type, surface infiltration, and storage.
- Stream channel characteristics including: geometry and configuration, natural and artificial controls, channel modification, aggradation - degradation, and ice and debris.
- Flood plain characteristics
- Meteorological characteristics such as precipitation amounts and type (rain, snow, hail, or combinations thereof), storm cell size and distribution characteristics, storm

direction, and time rate of precipitation (hyetograph).

### 7.1.7 Flood History

All hydrologic analysis should consider the flood history of the area and the effect of these historical floods on existing and proposed structures. The flood history should include the historical floods and the flood history of any existing structures.

### 7.1.8 Hydrologic Method

Many hydrologic methods are available. The methods to be used and the circumstances for their use are listed below in order of preference. If possible the method should be calibrated to local conditions and tested for accuracy and reliability. If applicable, a number of methods could be used and their answers compared.

### 7.1.9 Approved Methods

The following lists hydrologic methods to be used in order of preference:

- 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, the discharges based on current methods may be used subject to receipt of necessary regulatory approvals. FEMA flood studies must be validated with current information.
- Log Pearson III analyses as defined in Water Resources Council Bulletin 17B, 1981, "Guidelines for determining flood flow frequency" (WRCB 17B) shall be used for all routine designs where sufficient stream gaging record exists.
- Watershed regression equations shall be used for all routine designs at sites with watershed areas greater than the minimum specified in the regression analysis study, and in accordance with the other limitations of the regression equations, unless there are stream gage data or historical evidence suggesting other alternatives. The order of preference for use of regression equations is:
    1. Local (watershed specific) regression equations
    2. Regression equations calibrated to nearby gaged data
    3. Regression equations uncalibrated
- U.S. Army Corps of Engineers HEC-1 Flood Hydrograph package shall be used if it can be calibrated and verified with actual rainfall and runoff data. This method is data intensive.
- SCS and other unit hydrograph methods.
- Rational method shall be used only for drainage areas less than 200 acres and only if all other methods are inappropriate.

### 7.1.10 Regulatory Floodways

A comparison shall be made between published FEMA flood flows and calculated design flood flows of Section 7.1.9. In addition, survey information and datums used for FEMA flood studies should be carefully scrutinized. If calculated floods are greater than FEMA floods then the calculated floods shall be used. If FEMA flood flows are greater than those calculated, a determination of appropriate action shall be made by the Regional Hydraulics Engineer and/or the State Hydraulics Engineer. All differences between FEMA flows and those calculated shall be discussed in the Hydraulics and Hydrology Report.

### 7.1.11 Design Frequency

A design flood frequency should be selected commensurate with the facilities cost, 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. With 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. In selecting a design frequency, potential upstream land use should be considered which could reasonably occur over the anticipated life of the drainage facility. The design flood frequencies shown in Appendix A shall be used unless otherwise specified by the Statewide Hydraulics Engineer.

### 7.1.12 Economics

Hydrologic analysis should include the determination of several design flood frequencies for use in the hydraulic design. These frequencies are used to size different drainage facilities so as to allow for an optimum design, which considers both risk of damage and construction cost. Consideration should be given to what frequency flood was used to design other structures along a highway corridor.

### 7.1.13 Review Frequency

All proposed bridge or bottomless arch structures obtained using the selected design frequency shall be reviewed using the superflood to ensure that there are no unexpected flood hazards. The superflood is defined as the 500-year flood or 1.7 times the 100-year flood, whichever is less. Roadway overtopping shall be evaluated for any culvert if the headwater to diameter ratio is greater than one for its design flood.

## 7.2. Overview

### 7.2.1 Introduction

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

### 7.2.2 Definition

Hydrology is generally defined as a science dealing with the interrelationship between water on and under the earth and in the atmosphere. For the purpose of this manual, hydrology will deal with estimating flood magnitudes as the result of precipitation, which includes snowmelt and snow storage. In the design of highway drainage structures, floods are usually considered in terms of peak runoff or discharge in cubic feet per second (cfs) and hydrographs as discharge per time. For structures which are designed to control volume of runoff, like detention storage facilities, or where flood routing through culverts is used, then the entire discharge hydrograph will be of interest.

### 7.2.3 Factors Affecting Floods

In the hydrologic analysis for a drainage structure, it must be recognized that 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:

- rainfall amount and storm distribution,
- drainage area size, shape and orientation,
- ground cover,
- type of soil,
- slopes of terrain and stream(s),
- antecedent moisture condition,
- storage potential (overbank, ponds, wetlands, reservoirs, channel, etc.),
- watershed development potential,
- type of precipitation (rain, snow, hail, or combinations thereof),
- elevation, and
- mixed population events.

### 7.2.4 Sources of Information

The type and source of information available for hydrologic analysis will vary from site to site and it is the responsibility of the designer to determine what information is available and applicable to a particular analysis. Examples of sources are included in the Data Collection Chapter of this handbook.

## 7.3. Symbols and Definitions

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

**Table 7-2**

**Symbols and Definitions**

| Symbol | Definition | English Units | Metric Units |
|---|---|---|---|
| A | Drainage area | acres, sq.mi. | hectares or sq. km |
| BDF | Basin development factor | % | % |
| C | Runoff coefficient | - | - |
| $C_f$ | Frequency factor | - | - |
| CN | SCS-runoff curve number | - | - |
| $C_t$, $C_p$ | Physiographic coefficients | - | - |
| d | Time interval | hours | hours |
| DH | Difference in elevation | ft | m |
| I | Runoff intensity | in./hr | mm/hr |
| IA | Percentage of impervious area | % | % |
| $I_a$ | Initial abstraction from total rainfall | in | mm |
| K | Frequency factor for a particular return period and skew | - | - |
| L | Lag | hours | hours |
| l | Length of mainstream to furthest divide | ft | m |
| $L_{ca}$ | Length along main channel to a point opposite the watershed centroid | Miles | km |
| M | Rank of a flood within a long record | - | - |
| n | Manning roughness coefficient | - | - |
| N | Number of years of flood record | years | years |
| P | Accumulated rainfall | | mm |
| Q | Rate of runoff | cfs | cms |
| q | Storm runoff during a time interval | in | mm |
| R | Hydraulic radius | ft | m |
| RC | Regression constant | - | - |
| RQ | Equivalent rural peak runoff ratecfs | cms | |
| S or Y | Ground slope | ft/ft or % | m/m or % |
| S | Potential maximum retention storage | in | mm |
| SCS | Soil Conservation Service | - | |
| SL | Main channel slope | ft/ft or % | m/m or % |
| $S_L$ | Standard deviation of the logarithms of the peak annual floods | - | - |
| ST | Basin storage factor | % | % |
| $T_B$ | Time base of unit hydrograph | hours | hours |
| $t_c$ or $T_c$ | Time of concentration | min or hours | min or hours |
| $T_L$ | Lag time | hours | hours |
| $T_r$ | Snyder's duration of excess rainfall | hours | hours |
| UQ | Urban peak runoff rate | cfs | cms |
| V | Velocity | ft/s | m/s |
| X | Logarithm of the annual peak | - | - |

## 7.4. Concept Definitions

### 7.4.1 Introduction

Following are discussions of concepts which 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.

### 7.4.2 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. As floods become more rare, antecedent moisture has a rapidly decreasing influence on runoff.

### 7.4.3 Base Discharge

Base discharge is the portion of total flow that is contributed largely from groundwater (also called base flow and dry-weather flow).

### 7.4.4 Calibration

Calibration is a process of varying the parameters or coefficients of a hydrologic method so that it will estimate peak discharges and hydrographs consistent with local rainfall and streamflow data.

### 7.4.5 Depression Storage

Depression storage is the natural depressions within a watershed which store runoff. Generally after the depression storage is filled runoff will commence.

### 7.4.6 Frequency

Frequency is the number of times a flood of a given magnitude can be expected to occur on an average over a long period of time. Frequency analysis is then 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 equalled or exceeded in any year.

### 7.4.7 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.

### 7.4.8 Hydrograph

The hydrograph is a graph of the time distribution of runoff from a watershed.

### 7.4.9 Hyetographs

The hyetograph is a graph of the time distribution of rainfall over a watershed.

### 7.4.10 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.

### 7.4.11 Interception

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

### 7.4.12 Lag Time

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

### 7.4.13 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.

### 7.4.14 Rainfall Excess

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

### 7.4.15 Stage

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

### 7.4.16 Time of Concentration

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

### 7.4.17 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.

## 7.5. Design Frequency

### *7.5.1 Overview*

Since it is not economically feasible to design a structure for the maximum runoff a watershed is capable of producing, a design frequency must be established. The frequency with which a given flood can be expected to occur is the reciprocal of the probability or chance that the flood will be equaled or exceeded in a given year. If a flood has a 20 percent chance of being equaled or exceeded each year, over a long period of time, the flood will be equaled or exceeded on an average of once every five years. This is called the return period or recurrence interval (RI). Thus the exceedence probability equals 100/RI.

The designer should note that the 5-year flood is not one that will necessarily be equaled or exceeded every five years. There is a 20 percent chance that the flood will be equaled or exceeded in any year; therefore, the 5-year flood could conceivably occur in several consecutive years. The same reasoning applies to floods with other return periods. The designer should also note that a structures design life does not equate to the return period for hydraulic design.

### *7.5.2 Design Frequency*

**Cross Drainage**

A drainage facility should be designed to accommodate a discharge with a given return period(s) for the following circumstances. The design should be such that the backwater or the headwater caused by the structure for the design storm does not:

- increase the flood hazard, including erosion, significantly for property upstream or downstream of the structure,
- adversely affect the structure, or
- adversely affect the highway embankment.

Based on these design criteria, a design involving temporary roadway overtopping for floods larger than the design event is acceptable practice. Usually, if overtopping is allowed, the structure may be designed to accommodate a flood of some lower frequency without overtopping.

**Storm Drains**

A storm drain should be designed to accommodate a discharge with a given return period(s) for the following circumstances. The design should be such that the storm runoff does not:

- increase the flood hazard significantly for property (flood hazard to property includes the effects of flow concentration downstream of the facility),
- encroach on to the street or highway so as to cause a significant traffic hazard or limit traffic, emerging vehicle, or pedestrian movement to an unreasonable extent.

Based on these design criteria, a design involving temporary street or road inundation for floods larger than the design event is acceptable practice.

### *7.5.3 Review Frequency*

After sizing a drainage facility using a flood and sometimes the hydrograph corresponding to the design frequency, it should be necessary to review this proposed facility with a base discharge. This is done to insure that there are no unexpected flood hazards inherent in the proposed facility(ies). The review flood should be the 100- year event. In some cases, a flood event larger than the 100-year flood super-flood is used for analysis to ensure the safety of the drainage structure and downstream development.

### *7.5.4 Rainfall vs. Flood Frequency*

Drainage structures are designed based on some flood frequency. However, certain hydrologic procedures use rainfall and rainfall frequency as the basic input. Thus it is commonly assumed that the 10-year rainfall will produce the 10-year flood. Depending on antecedent soil moisture conditions, and other hydrologic parameters this may be true or there may not be a direct relationship between rainfall and flood frequency.

### *7.5.5 Rainfall Curves*

Rainfall data are available for many geographic areas. From these data, rainfall intensity-duration curves have been developed for the commonly used design frequencies. Appendix B at the end of this chapter contains the curves available at this time for the Department.

### *7.5.6 Discharge Determination*

Estimating peak discharges of various recurrence intervals is one of the most common engineering challenges faced by drainage facility designers. The problem can be divided into two general categories:

- Gaged sites - the site is at or near a gaging station and the streamflow record is of sufficient length to be used to provide estimates of peak discharges. A complete record is usually defined as one having at least 10 years of continuous of record. Twenty-five years of record is considered optimum. In Alaska, this is a rare situation.
- Ungaged sites - the site is not near a gaging station and no streamflow record is available. This situation is very common in Alaska.

This chapter will address hydrologic procedures that can be used for both categories.

## 7.6. Hydrologic Procedure Selection

### *7.6.1 Overview*

Streamflow measurements for determining a flood frequency relationship at a site are usually unavailable; in such cases, it is accepted practice to estimate peak runoff rates and hydrographs using statistical or emperical methods. In general results from using several methods should be compared, not averaged. The Department practice shall be to use the discharge that best reflects local project conditions with the reasons documented. The Department use for each procedure is outlined with each hydrologic procedure given below. The designer shall be responsible for knowing and understanding the method and limitations of the procedure used.

### *7.6.2 Peak Flow Rates or Hydrographs*

A consideration of peak runoff rates for design conditions is generally adequate for conveyance systems such as storm drains or open channels. However, if the design must include flood routing (e.g., storage basins or complex conveyance networks), a flood hydrograph is usually required. Although the development of runoff hydrographs (typically more complex than estimating peak runoff rates) is often accomplished using computer programs, some methods are adaptable to nomographs or other desktop procedures.

### *7.6.3 Hydrologic Procedures*

**Log Pearson III Flood Frequency**

With adequate continuous or synthesized stream gage data the log Pearson III is considered to be the most reliable method for estimating flood frequency relationships and should be used for all designs. Data can be obtained from the local U.S.G.S. office in your area or from the Regional Hydraulics Engineer.

**Regression Equations**

Peak flow can be calculated by using watershed regression equations developed for specific geographic regions. The equations are in the form of a log-log formula, where the dependent variable would be the peak flow for a given frequency, and the independent variables may be variables such as area, slope, and other meteorological, physical or site specific data.

**U.S. Army Corps of Engineers HEC-1 Flood Hydrograph** package can be used if it can be calibrated and verified with actual rainfall and runoff data.

**SCS Synthetic Unit Hydrograph**

The Soil Conservation Service has developed a synthetic unit hydrograph procedure which has been widely used for developing rural and urban hydrographs. The unit hydrograph used by the SCS method is based upon an analysis of a large number of natural unit hydrographs from a broad cross section of geographic locations and hydrologic regions. Rainfall is a necessary input.

**Snyder's Unit Hydrograph**

This method developed in 1938 has been used extensively by the Corps of Engineers and provides a means of generating a synthetic unit hydrograph. Rainfall is a necessary input. Rational Method - Provides peak runoff rates for small urban and rural watersheds of less than 200 acres, but is best suited to urban storm drain systems. It should be used with caution at all times but especially if the time of concentration exceeds 30 minutes. Rainfall is a necessary input.

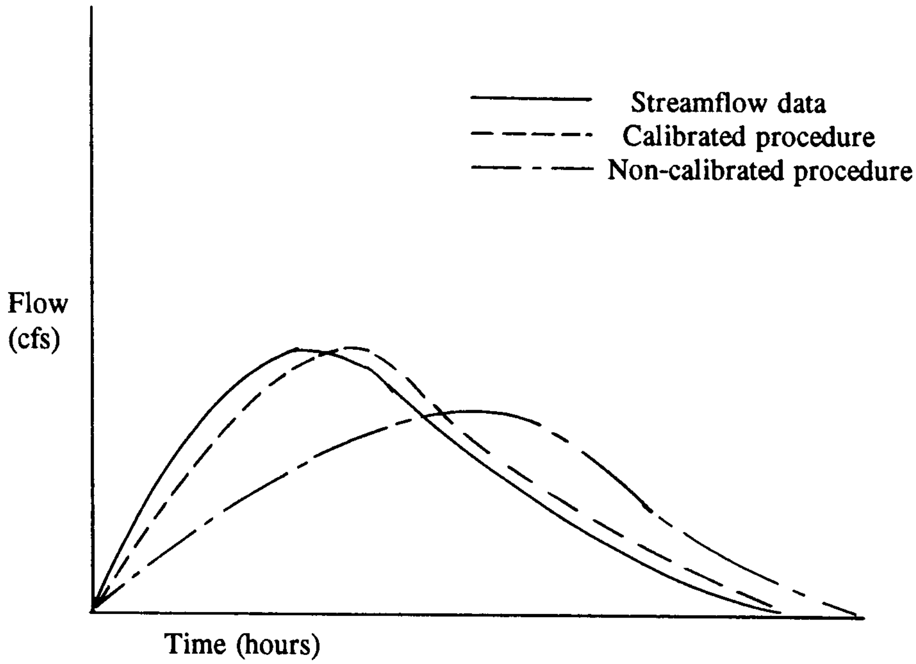
## 7.7. Calibration

### *7.7.1 Hydrologic Accuracy*

The accuracy of the hydrologic estimates will have a major affect on the design of drainage or flood control facilities. Although it might be argued that one hydrologic procedure is more accurate than another , practice has shown that all of the methods discussed in this chapter can, if calibrated, produce acceptable results consistent with observed or measured events. What should be emphasized is the need to calibrate the method for local conditions. This calibration process can result in much more accurate and consistent estimates of peak flows and hydrographs.

### *7.7.2 Illustration*

Following is an illustration of a hydrograph resulting from flow data as compared to a hydrograph resulting from using a non-calibrated and calibrated hydrologic procedure. It can be seen that the calibrated hydrograph, although not exactly duplicating the hydrograph from streamflow data, is a much better representation of the streamflow hydrograph than the non-calibrated hydrograph.



### 7.7.3 Calibration Process

The calibration process can vary depending on the data or information available for a local area, and the procedure used.

1. If streamflow data are available for an area, the hydrologic procedures can be calibrated to these data. The process would involve generating peak discharge and hydrographs for different input conditions (e.g., slope, area, antecedent soil moisture conditions) and comparing these results to the gages data. Changes in the model would then be made to improve the estimated values as compared to the measured values.
2. After changing the variables or parameters in the hydrologic procedure the results should be 7.checked against another similar gaged stream or another portion of the streamflow data that were not used for calibration.
3. If some local agency has developed procedures or equations for an area based on streamflow data, general hydrologic procedures can be calibrated to these local procedures. In this way the general hydrologic procedures can be used for a greater range of conditions (e.g., land uses, size, slope).
4. The calibration process should only be undertaken by personnel highly qualified in hydrologic procedures and design.
5. Should it be necessary to use unreasonable values for variables in order for the model to produce reasonable results, then the model should be considered suspect and its use carefully considered (e.g., having to use terrain variables that are obviously dissimilar to the geographic area in order to calibrate to measured discharges or hydrographs.).

## 7.8. Regional Regression Equations

### 7.8.1 Introduction

Regional regression equations are a commonly accepted method for estimating peak flows at ungaged sites or sites with insufficient data. Also, they have been shown to be accurate, reliable, and easy to use as well as providing consistent findings when applied by different hydraulic engineers (Newton and Herrin, 1982). The method provided in this manual is termed the watershed characteristics method. Regression studies are statistical practices used to develop runoff equations. These equations are used to relate such things as the peak flow or some other flood characteristic at a specified recurrence interval to the watershed's physiographic, hydrologic and meteorological characteristics. As such it should be noted that the regression analysis is a separate study and is not part of the analysis contained in this manual for devising a flood-frequency curve at an ungaged site, i.e., the regression analysis only provides the equation for application. It is important to note that the same data set used to develop a set of regression equations be used for the application of the equations. That is, if specific temperature and rainfall maps were used to develop the equations, the same maps should be used for applying the equations and not different, albeit newer or more current information.

### 7.8.2 Department Application

The preferred regression equations for use by the Department are those prescribed in "Magnitude and Frequency of Floods in Alaska and Conterminous Basins of Cananda", U.S. Geological Survey, Jones, S.H. and Fahl, C.B. Water Resources Investigations Report 93-4179. The regression equations presented in this report were developed from flood data from gaging stations that have operated for at least eight years on non-urban and unregulated streams, and on streams that are unaffected by the failure of natural dams (e.g.snow or ice dams), or from rapid melting due to volcanic activity.

### 7.8.3 Hydrologic Regions

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 watershed, channel and meterological characteristics; often termed hydrologically homogeneous geographic areas. Because of the distance between stream gages and sometimes due to mixed population flood events such as rain on snowpack, the regional boundaries cannot be considered as precise. Problems related to hydrologic boundaries may occur in selecting the appropriate regression equation, such as when a watershed lies totally within a hydrologic region, but close to a hydrologic region boundary. Care must be exercised in using regression equations. A thorough hydrologic investigation as discussed in Chapter 6, Data Collection, and comparison with other flood frequency methods is important for validating the results of regression equations.

### *7.8.4 Regression Equation Calibration*

The results of flood frequency regression equations may be calibrated using flood frequency information from a nearby gaged watershed which has similar basin characteristics and is within the same hydrologic region, and which has sufficient data to perform a Log Pearson Type III (LP-III) analysis . Each regression equation for a particular return interval is applied to the gaged watershed, the results of which are compared to the LP-III results. The resulting calibration ratio is then applied to the results of the regression equations from the ungaged watershed in order to calibrate them.

## 7.9. Example Problem: Flood Frequency from Regional Regression Equations

(Example is under development, illustrating use of USGS Regression Equation and calibration technique using a model gaged watershed)

## 7.10. Analysis of Stream Gage Data

### *7.10.1 Introduction*

Although the distribution is uneven, gaging stations exist throughout Alaska 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 in time, say at least 10 years, a frequency analysis may be made according to the following discussion. 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.

### *7.10.2 Application*

The Department shall use the stream gage analysis findings for design when there is sufficient years of measured or synthesized stream gage record. The preferred method for analyzing stream gage data is the Log Pearson Type III method as provided in Bulletin 17B, “Guidelines for Determining Flood Flow Frequency”, United States Water Resources Council, 1981. The Gumbel graphical method, which can be found in many hydrology textbooks, may be used primarily as a check to insure errors are not made—particularly with the frequency estimates of larger floods. Where series discrepancies (20% +) are encountered in the findings between the two methods, special studies may be required. These special studies may consist of such things as comparison with regression equations, application of other flood-frequency methods, and the collection and analysis of historical data. Outliers shall be placed into perspective using the procedure found in Water Resources Council Bulletin 17B.

### *7.10.3 Transposition of Data*

Although it is not the preferred method, the transposition of design discharges from one basin to another basin with similar hydrologic characteristics may be accomplished by multiplying the design discharge by the direct ratio of the respective drainage areas raised to an exponent. Application of the exponent is based on the relative sizes of the watersheds and requires judgement. 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. Following is an example using an exponent of 0.7. The preferred method for transposition of data uses the regression equations (Jones and Fahl, 1993) calibrated to a gaged watershed as discussed in Section 7.10.4.

**Table 7-11**

**Exponent for Hydrologic Regions**

| Watershed | $Q_{25}$ | Area, sq.mi. |
|---|---|---|
| Gaged Watershed A | 62,000 | 737 |
| Gaged Watershed B | 38,000 | 734 |
| Gaged Watershed C | 45,000 | 971 |
| Ungaged Watershed D | - | 450 |

A: $62,000\ (450/737)^{0.7} = 43,895$ cfs
B: $38,000\ (450/734)^{0.7} = 26,980$ cfs
C: $45,000\ (450/971)^{0.7} = 26,266$ cfs
D: $Q_{25} = (43,895 + 26,980 + 26,266)/3 = 32,380$ cfs

## 7.11. Unit Hydrograph: Gaged Data

### *7.11.1 Introduction*

It is sometimes useful or necessary to estimate the runoff hydrograph associated with the peak discharge of a desired frequency. Several methods are available to develop a design hydrograph. At sites where gaged data are available, a unit hydrograph can be developed from corresponding rainfall and runoff data.

A unit hydrograph is a hydrograph of the runoff resulting from a hypothetical storm that has a specified duration, e.g., 1 hour, and that produces exactly one inch of runoff over the drainage area. The unit hydrograph techniques described below can be used to approximate the rainfall-runoff response of a particular watershed and to predict the flood hydrograph that would result from a design storm of a desired frequency.

### *7.11.2 Application*

When a hydrograph, as opposed to merely peak flow, is neccessary for design, and when actual hydrographs for the design peak flows are not available, but other hydrographs are available, the Department shall use the following unit hydrograph method.

### *7.11.3 Characteristics*

In order to develop a unit hydrograph for a watershed, corresponding rainfall and runoff records must be available for the area. These records consist of flood discharge data at the desired site as well as corresponding rainfall data from the contributing watershed for the same flood event. Discharge data are plotted against time to produce a flood hydrograph. Rainfall records are usually obtained as rainfall mass curves that can be used to develop a graph of rainfall intensity over time (termed a rainfall hyetograph).

The procedure for constructing a unit hydrograph are given below.

### *7.11.4 Procedure*

Following is the procedure for construction of a unit hydrograph from rainfall and runoff data.

**Step 1**

Determination of Direct Runoff Hydrograph. The normal stream baseflow is subtracted from the flood hydrograph to produce a direct runoff hydrograph by drawing a straight line from the beginning of the rising portion of the hydrograph to a point directly under the peak, at the same slope as the baseflow curve prior to the beginning of the flood hydrograph. A second straight line is then drawn from the end of the first line to connect to the recession limb of the hydrograph at a point where the baseflow is equal in magnitude to that where the hydrograph began. The direct runoff hydrograph is then determined by subtracting the baseflow from the flood hydrograph.

**Step 2**

Determination of Direct Runoff Volume. The direct runoff is determined as the area under the direct runoff hydrograph. The volume is determined by dividing the time base of the hydrograph into convenient time increments, determining the average discharge for each time increment, and multiply the length of the time increment by the average discharge to obtain the volume for that increment. This procedure is repeated for each time increment and the incremental volumes are summed to obtain the total direct runoff volume. Another approach would be to planimeter the area under the runoff hydrograph. This volume, usually in cubic feet, is converted into inches by dividing by the total watershed area in square feet and converting this depth into inches.

**Step 3**

<u>Determination of Unit Hydrograph Ordinates.</u> The ordinates of the unit hydrograph are determined by dividing each ordinate of the flood hydrograph by the volume of direct runoff (in inches). The time base of the unit hydrograph will be the same, as well as the general shape. Only the magnitude of the discharge coordinates will be different. If the direct runoff volume is recomputed as described above, the total volume should equal one inch.

**Step 4**

<u>Determination of Unit Storm Duration.</u> The storm that produced the unit hydrograph calculated above will have a specified duration. This duration is the length of time that the storm produced direct runoff. For example, if all of the rainfall during the first 15 minutes percolated into the ground, the first 15 minutes would not be part of the storm duration. The unit storm duration can be determined using the rainfall hyetograph and the direct runoff hydrograph. Because of the complexity of most rainfall records and the difficulty in accurately determining the amount of rainfall that is lost through infiltration, depression storage, etc., two fairly simple adjustments are made to determine the total depth of rainfall that becomes direct runoff.

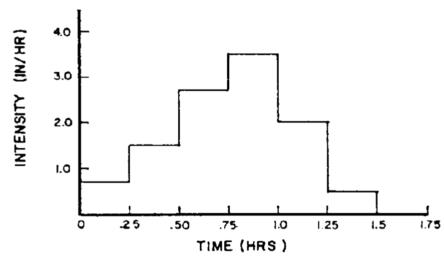
1. The time of the beginning of rainfall and the time of the beginning of the corresponding flood hydrograph are compared. Any rainfall occurring prior to the beginning of runoff is assumed to be lost due to initial abstractions and infiltration and is subtracted from the storm.

2. The remaining volume of rainfall (the area under the hyetograph) is calculated and compared to the volume of direct runoff. The rainfall volume will be greater if infiltration and other losses have occurred. The Phi index method is used to subtract out these losses. The Phi index is defined as the rate of rainfall above which the rainfall volume equals the runoff volume. A straight line is drawn to indicate a constant loss rate; the position of this straight line is drawn such that the remaining volume under the hyetograph equals the volume of direct runoff computed from the direct runoff hydrograph.

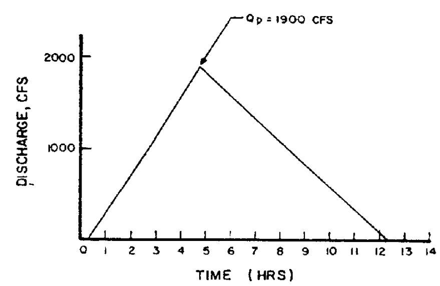
The duration of the unit storm is the time during which the storm produces direct runoff. It is easily determined from the rainfall hyetograph once the two adjustments described above have been made. Once a unit hydrograph for a particular storm duration has been determined, unit hydrographs for other durations can be developed.

## 7.12. Example Problem - Unit Hydrograph

Figure 7-1 shows a rainfall hyetograph and the resulting direct runoff hydrograph (baseflow subtracted) produced by a particular storm over a watershed of 5940 acres.




Rainfall Hyetograph




Direct Runoff Hydrograph

**Figure 7-1**

**Rainfall Hyetograph and Direct Runoff Hydrograph**

A. <u>Determination of Direct Runoff Volume</u>

$$\text{Volume} = 0.5 \times 1900 \text{ ft}^3\text{/sec} \times 3600 \text{ sec/hr} \times (12.25 - 0.25) \text{ hr}$$

$$= 41{,}000{,}000 \text{ ft}^3$$

Equivalent Depth of Rainfall

Depth = $[(4.1 \times 10^7 \text{ ft}^3)/5940 \text{ acres})]$

$\times [(1 \text{ acre}/43560 \text{ ft}^2)] \times (12\text{in/ft})$

= 1.9 inches

B. Determination of Unit Hydrograph Ordinates

$Q_p$/1.0 inch = 1900 cfs/1.9 inches = 1000 cfs

Check Volume:

Volume = $0.5 \times 1000 \text{ ft}^3/\text{sec} \times 3600 \text{ sec/hr}$

$\times (12.25 - 0.25)$ hr

= $2.16 \times 10^7 \text{ ft}^3$

Depth = $[(2.16 \times 10^7 \text{ ft}^3)/5940 \text{ acres})]$

$\times (1 \text{ acre}/43560 \text{ ft}) \times (12 \text{ in/ft})$

= 1.0 inch

C. Determination of Storm Duration

Since the direct runoff hydrograph begins at 0.25 hr, the first increment of rainfall from 0 to 0.25 hr satisfies the initial abstractions (interception, evaporation, and depression storage). The remaining volume of rainfall is:

Volume (depth) = 0.25 hr x (1.5 + 2.7 + 3.5 + 2.0 + 0.5) in/hr
= 2.55 inches

Since the direct runoff volume (depth) is 1.9 inches, the remaining 0.65 (2.55 - 1.90) inches must be lost due to infiltration. The phi index is adjusted by trial and error to yield the desired volume of excess rainfall. For a phi index = 0.5 in/hr.

Excess rainfall = (1.0 + 2.2 + 3.0 + 1.5) x 0.25
= 1.9 inches

The duration of the storm producing this rainfall is one hour.

## 7.13. Snyder Synthetic Unit Hydrograph

### *7.13.1 Introduction*

The Snyder Synthetic Unit Hydrograph method, developed in 1938, has been used extensively by the Corps of Engineers and provides a means of generating a synthetic unit hydrograph. This method is valuable when gaged data are not available for a specific location, however, it may yield overly conservative results.

### *7.13.2 Equations and Procedure*

Following is the procedure to develop a unit hydrograph using the Snyder unit hydrograph method.

**Step 1**

The following data should be obtained:

- watershed area in square miles (A),
- length along the main channel to point on the watershed divide in miles (L),
- length along the main channel to a point opposite the watershed centroid in miles ($L_{ca}$),
- values of physiographic coefficients $C_t$ and $C_p$ from below. Considerable judgment should be exercised in selecting $C_t$ and $C_p$ values since conditions may vary greatly between watersheds within the same river basin.

$C_t$ and $C_p$ Values

(Table is under development)

**Step 2**

Determine the lag time, $T_L$, of the unit hydrograph. The lag time is the time from the centroid of the excess rainfall to the hydrograph peak. Snyder derived the following empirical equation for lag time where $T_L$ is the lag time in hours, $C_t$ is the empirical coefficient defined above, L is the length along the main channel from outlet to divide in miles, and $L_{ca}$ is the length along the main channel from the outlet to a point opposite the watershed centroid in miles.

$$T_L = C_t (LL_{ca})^{0.3} \quad (7.1)$$

**Step 3**Determination of hydrograph duration. The relationship developed by Snyder for the duration of the excess rainfall, $T_r$ in hours, is a function of the lag time computed above.

$$T_R = T_L/5.5 \quad (7.2)$$

This always results in an initial value for $T_R$ of $T_L/5.5$. However, a relationship has been developed to adjust the computed lag time for other durations. This is necessary because the equation above results in inconvenient values of unit hydrograph duration. The adjustment relationship is:

$$T_L(_{adj.}) = T_L + 0.25(T'_R - T_R) \quad (7.3)$$

Where $T_L(_{adj.})$ is the adjusted lag time for the new duration, $T'_R$.

The peak discharge for the unit hydrograph is determined from the equation below:

$$Q_p = [640C_pA] / [T_L(_{adj.})] \qquad (7.4)$$

Where $Q_p$ is the peak discharge in cfs, $C_p$ is the empirical coefficient defined above, and A is the watershed area in square miles.

The time base, $T_B$, of the unit hydrograph was determined by Snyder to be approximately equal to:

$$T_B = 3 + T_L(_{adj.})/8 \qquad (7.5)$$

Where $T_B$ is the time of the synthetic unit hydrgraph in days. This relationship, while reasonable for larger watersheds, may not be applicable for smaller watersheds. A more realistic value for smaller watersheds is to use 3 to 5 times the time to peak as a base for the unit hydrograph. The time to peak is defined as the time from the beginning of the rising limb of the hydrograph to the peak discharge.

The time widths of the unit hydrograph at discharges equal to 50 percent and 75 percent of the peak discharges, $W_{50}$ and $W_{75}$ respectively (in hours), have been found to be approximated by the following equations:

$$W_{50} = 735(Q_p/A)^{-1.075} \qquad (7.6)$$

and

$$W_{75} = 434(Q_p/A)^{-1.075} \qquad (7.7)$$

Using the values computed in the previous steps, the unit hydrograph can now be sketched, remembering that the total volume of runoff must equal 1 inch. A rule of thumb to assist in sketching the unit hydrograph is that the $W_{50}$ and $W_{75}$ time widths should be apportioned with one third to the left of the peak and two thirds to the right of the peak.

## 7.14. Example Problem - Snyder Hydrograph

### *7.14.1 Example*

A synthetic unit hydrograph is to be constructed for a watershed of 875 square miles, where L = 83 miles, $L_{ca}$ = 40.6 miles, $C_t$ = 1.32, and $C_p$ = 0.63.

A 3-hour unit hydrograph is desired. Using the equations from Section 7.21, the following values are established.

$T_L$ = 15.1 hours

$T_R$ = 2.75 hours

$T_b$ = 117.6 hours

$W_{50}$ = 21.7 hours

$W_{75}$ = 12.8 hours

$T_L$(adj.) = 15.2 hours

$Q_p$ = 23,210 cfs

Compared to the hydrograph widths at 50 and 75 percent of the peak flow, a time base of 117.6 hours is very long. To obtain a more realistic value, it is assumed that the time base is 4.5 times the time to peak, or

$T_b$ = 75 hours

### *7.14.2 Snyder Hydrograph*

These points are plotted (•) and a smooth hydrograph shape is fitted with the key dimensions. The volume under the hydrograph is then computed with the discharge ordinates being scaled from the figure. The total volume computed is 1.128 inches, which is larger than the required 1 inch. The surplus volume, over 1 inch, must be deducted from the unit hydrograph in a reasonable and systematic way. A procedure for adjusting the volume to one inch is given in FHWA Hydraulic Engineering Circular #19. The final hydrograph is shown in Figure 7-2 with Adjusted $T_R$ = 3.0 hours and $T_p$ = 16.7 hours.

Note: Adjustment of the hydrograph volume may be facilitated by use of straight line connectors instead of a smooth curve.

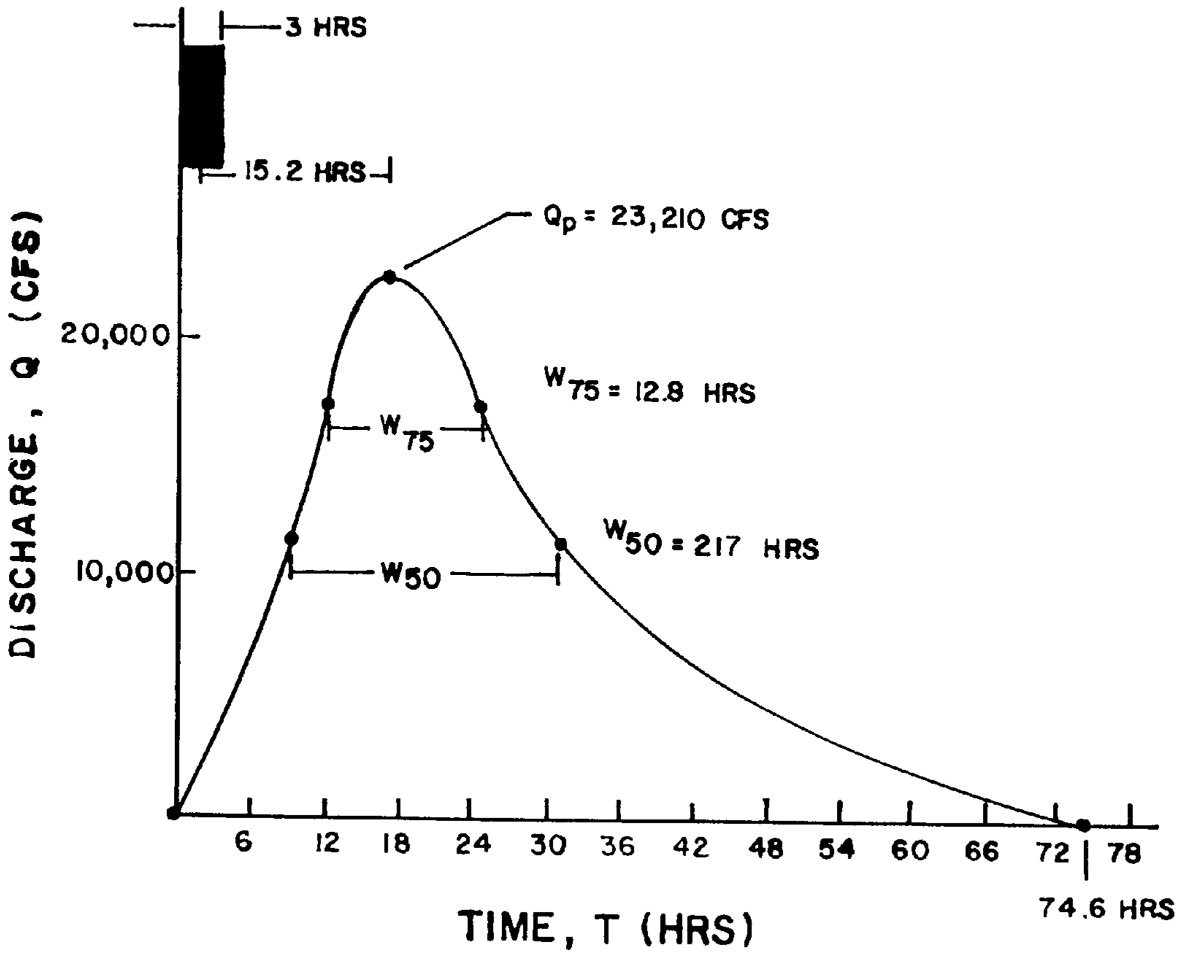



**Figure 7-2**

**Example Problem - Final Hydrograph**

## 7.15. SCS Unit Hydrograph

### *7.15.1 Introduction*

Techniques developed by the U. S. Soil Conservation Service for calculating rates of runoff require the same basic data as the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. The SCS approach, however, is more

sophisticated in that it considers also the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. With the SCS method, the direct runoff can be calculated for any storm, either real or fabricated, by subtracting infiltration and other losses from the rainfall to obtain the precipitation excess. Details of the methodology can be found in the SCS National Engineering Handbook, Section 4. Simplified procedures are presented in SCS Engineering Division, Technical Release 55.

### *7.15.2 Application*

Where hydrographs of design peak flows are required for basins of 100 acres to 1300 acres (approximately two square miles), the SCS Unit Hydrograph methods may be used. The regional regression equations discussed in Section 7.8 may be more appropriate for basins of two square miles or more. Care should be taken in applying this method to areas of frozen ground and where snowmelt peaks govern.

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

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

### *7.15.3 Equations and Concepts*

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

Drainage Area - The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas it might be necessary to divide the area into subdrainage areas 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 the flood flows. Also a field inspection of existing or proposed drainage systems should be made to determine if the natural drainage divides have been altered. These alterations could make significant changes in the size and slope of the subdrainage areas.

Rainfall - The SCS method is based on a 24-hour storm event which has a Type I or Type II time distribution. The Type I storm distribution is a “typical” time distribution which the SCS has prepared from rainfall records for the Pacific maritime climate. The more intense Type II distribution is typical for a continental climate which could be applied to portions of interior Alaska, albeit cautiously. Figure 7-3 on the next page shows this distribution. To use this distribution it is necessary for the user to obtain the 24-hour rainfall value (from the figures in Appendix B) for the frequency of the design storm desired. Then multiply this value by 24 to obtain the total 24-hour storm volume in inches.

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

$$Q = (P - I_a)^2 / (P - I_a) + S \qquad (7.8)$$

Where:

$Q$ = accumulated direct runoff, inches

$P$ = accumulated rainfall (potential maximum runoff), inches

$I_a$ = initial abstraction including surface storage, interception, and infiltration prior to runoff, inches

$S$ = potential maximum retention, inches The relationship between $I_a$ and S was developed from experimental watershed data. It removes the necessity for estimating $I_a$ for common usage. The empirical relationship used in the SCS runoff equation is:

$$I_a = 0.2S \qquad (7.9)$$

Substituting 0.2S for $I_a$ in equation 7.20, the SCS rainfall-runoff equation becomes:

$$Q = (P - 0.2S)^2 / (P + 0.8S) \qquad (7.10)$$

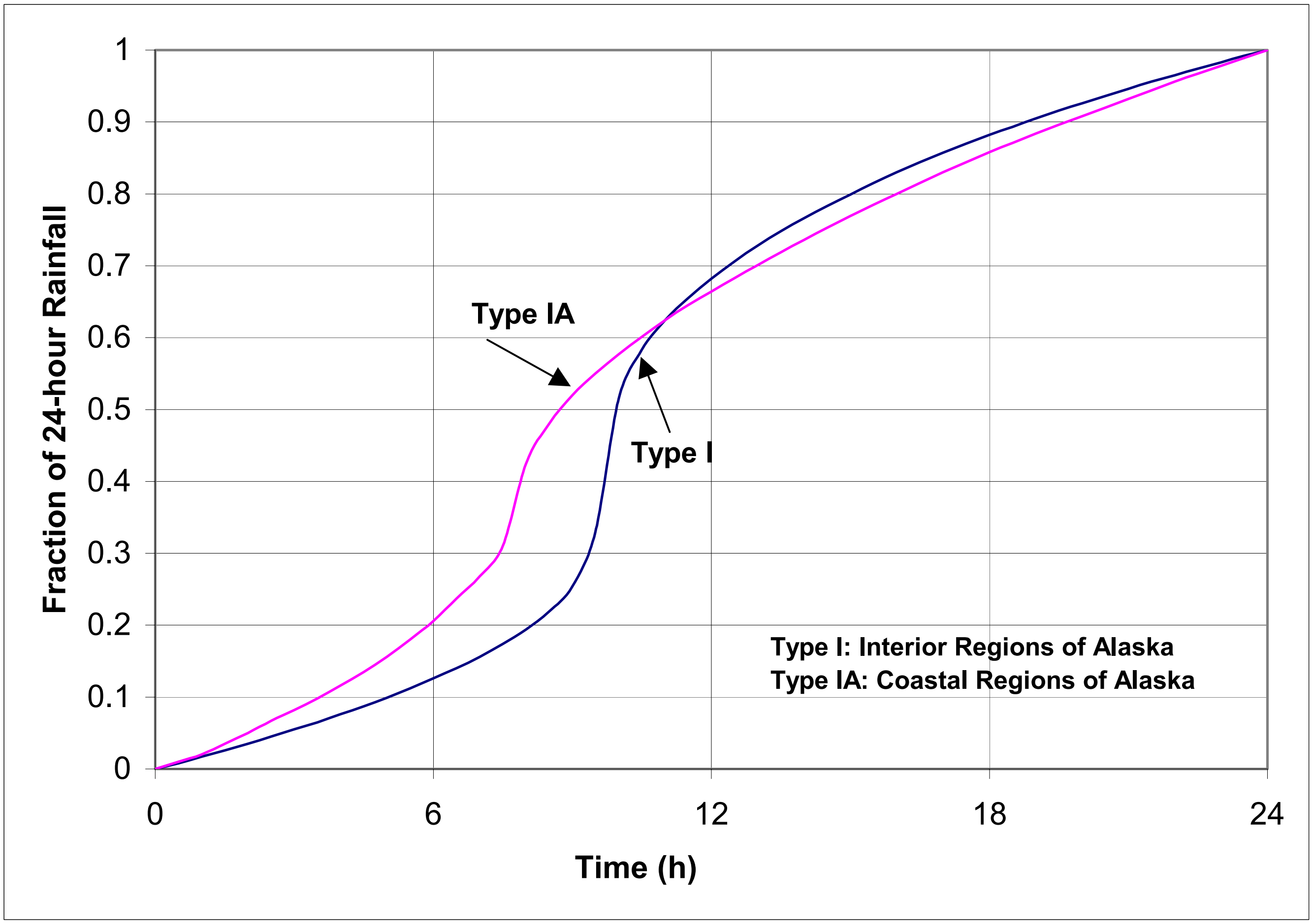



**Figure 7-3**

**Design Storm Curve**

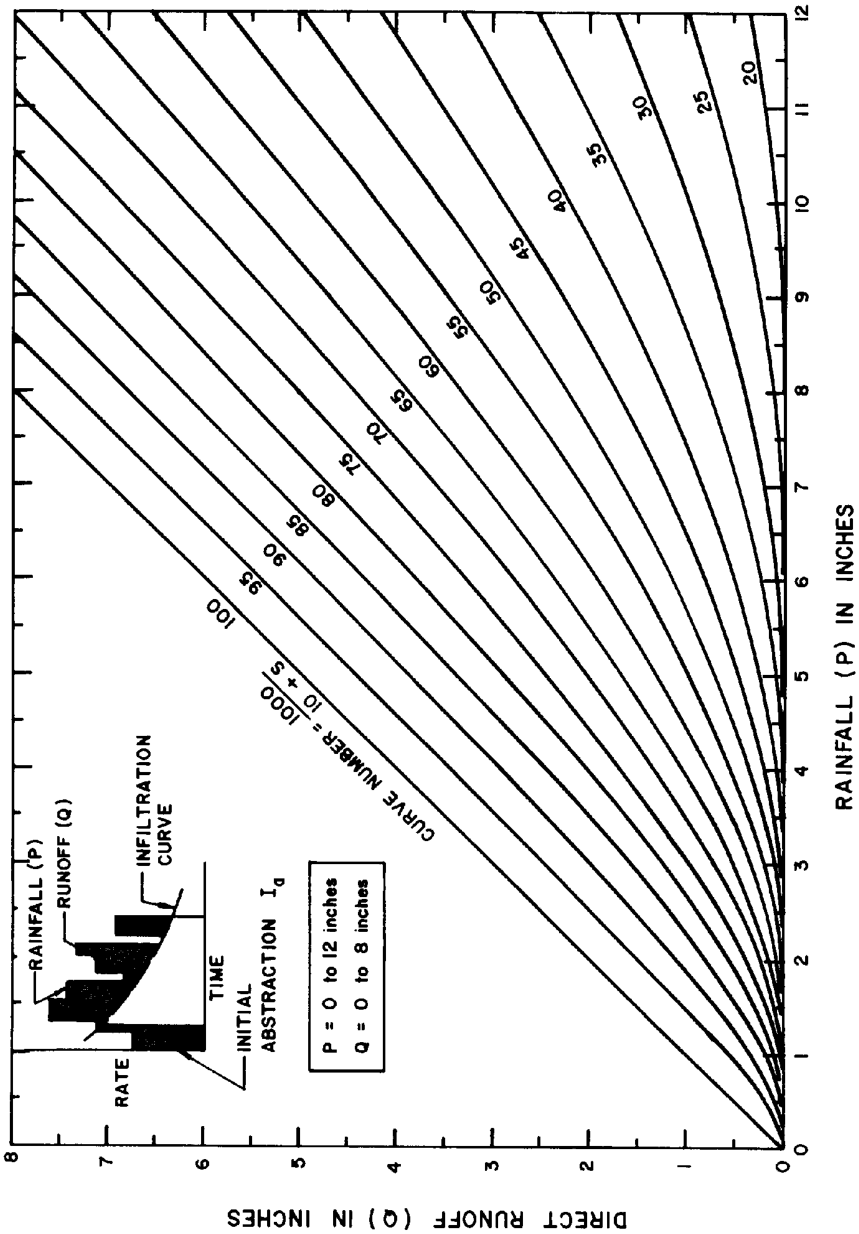



**Figure 7-4**

**SCS Relation between Direct Runoff, Curve Number and Precipitation**

**Source: HEC 19**

Figure 7-4 shows a graphical solution of this equation which enables the precipitation excess from a storm to be obtained if the total rainfall and watershed curve number are known. For example, 4.8 inches of direct runoff would result if 6.5 inches of rainfall occurs on a watershed with a curve number of 85.

### 7.15.4 Procedure

Following is a discussion of procedures that are used in the hydrograph method and recommended tables and figures.

**Runoff Factor**

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

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

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

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

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

The following pages give a series of tables related to runoff factors. The first tables (Tables 7-12 -7-15) gives curve numbers for various land uses. These tables are based on an average antecedent moisture condition i.e., soils that are neither very wet nor very dry when the design storm begins. Curve numbers should be selected only after a field inspection of the watershed and a review of zoning and soil maps. Table 7-16 gives conversion factors to convert average curve numbers to wet and dry curve numbers. Table 7-17 gives the antecedent conditions for the three classifications.

## Table 7-12

## Runoff Curve Numbers[1]

## Urban Areas

| Cover description | | Curve numbers for hydrologic soil groups | | | |
|---|---|---|---|---|---|
| Cover type and hydrologic condition | Average percent impervious area[2] | A | B | C | D |
| Fully developed urban areas (vegetation established) | | | | | |
| Open space (lawns, parks, golf courses, cemeteries, etc.)[3] | | | | | |
| Poor condition (grass cover <50%) | | 68 | 79 | 86 | 89 |
| Fair condition (grass cover 50% to 75%) | | 49 | 69 | 79 | 84 |
| Good condition (grass cover > 75%) | | 39 | 61 | 74 | 80 |
| Impervious areas: | | 98 | 98 | 98 | 98 |
| Paved parking lots, roofs, driveways, etc. (excluding right-of-way) | | | | | |
| Streets and roads: | | | | | |
| Paved; curbs and storm drains (excluding right-of-way) | | 98 | 98 | 98 | 98 |
| Paved; open ditches (including right-of-way) | | 83 | 85 | 89 | 91 |
| Gravel (including right of way) | | 76 | 85 | 89 | 91 |
| Dirt (including right-of-way) | | 72 | 82 | 87 | 89 |
| Western desert urban areas: | | | | | |
| Natural desert landscaping (pervious areas only) | | 63 | 77 | 85 | 88 |
| Artificial desert landscaping (impervious weed barrier, desert shrub with 1 –2 inch sand or gravel mulch and basin borders) | | 96 | 96 | 96 | 96 |
| Urban districts: | | | | | |
| Commercial and business | 85 | 89 | 92 | 94 | 95 |
| Industrial | 72 | 81 | 88 | 91 | 93 |
| Residential districts by average lot size: | | | | | |
| 1/8 acre or less (town houses) | 65 | 77 | 85 | 90 | 92 |
| 1/4 acre | 38 | 61 | 75 | 83 | 87 |
| 1/3 acre | 30 | 57 | 72 | 81 | 86 |
| 1/2 acre | 25 | 54 | 70 | 80 | 85 |
| 1 acre | 20 | 51 | 68 | 79 | 84 |
| 2 acres | 12 | 46 | 65 | 77 | 82 |
| Developing urban areas: | | | | | |
| Newly graded areas (pervious areas only, no vegetation) | | 77 | 86 | 91 | 94 |
| Idle lands (CN's are determined using cover types similar to those in Table 7-14. | | | | | |

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

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

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

## Table 7-13

## Cultivated Agricultural Land[1]

| Cover description | | Curve numbers for hydrologic soil group | | | | |
|---|---|---|---|---|---|---|
| Cover type | Treatment[2] | Hydrologic condition[3] | A | B | C | D |
| Fallow | Bare soil | - | 77 | 86 | 91 | 94 |
| | Crop residue | Poor | 76 | 85 | 90 | 93 |
| | cover (CR) | Good | 74 | 83 | 88 | 90 |
| Row | Straight row (SR) | Poor | 72 | 81 | 88 | 91 |
| Crops | | Good | 67 | 78 | 85 | 89 |
| | SR + CR | Poor | 71 | 80 | 87 | 90 |
| | | Good | 64 | 75 | 82 | 85 |
| | Contoured © | Poor | 70 | 79 | 84 | 88 |
| | | Good | 65 | 75 | 82 | 86 |
| | C + CR | Poor | 69 | 78 | 83 | 87 |
| | | Good | 64 | 74 | 81 | 85 |
| | Contoured & | Poor | 66 | 74 | 80 | 82 |
| | Terraced (C & T) | Good | 62 | 71 | 78 | 81 |
| | C&T + CR | Poor | 65 | 73 | 79 | 81 |
| | | Good | 61 | 70 | 77 | 80 |
| | Small grain SR | Poor | 65 | 76 | 84 | 88 |
| | | Good | 63 | 75 | 83 | 87 |
| | SR + CR | Poor | 64 | 75 | 83 | 86 |
| | | Good | 60 | 72 | 80 | 84 |
| | C | Poor | 63 | 74 | 82 | 85 |
| | | Good | 61 | 73 | 81 | 84 |
| | C + CR | Poor | 62 | 73 | 81 | 84 |
| | | Good | 60 | 72 | 80 | 83 |
| | C&T | Poor | 61 | 72 | 79 | 82 |
| | | Good | 59 | 70 | 78 | 81 |
| | C&T + CR | Poor | 60 | 71 | 78 | 81 |
| | | Good | 58 | 69 | 77 | 80 |
| | Close-seeded SR | Poor | 66 | 77 | 85 | 89 |
| | or broadcast | Good | 58 | 72 | 81 | 85 |
| | Legumes or C | Poor | 64 | 75 | 83 | 85 |
| | Rotation | Good | 55 | 69 | 78 | 83 |
| | Meadow C&T | Poor | 63 | 73 | 80 | 83 |
| | | Good | 51 | 67 | 76 | 80 |

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

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

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

Poor: Factors impair infiltration and tend to increase runoff.

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

## Table 7-14

## Other Agricultural Lands[1]

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

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

[2] Poor: < 50% ground cover or heavily grazed with no mulch

Fair: 50 to 75% ground cover and not heavily grazed

Good: > 75% ground cover and lightly or only occasionally grazed

[3] Poor: < 50% ground cover

Fair: 50 to 75% ground cover

Good: > 75% ground cover

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

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

[6] Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.

Fair: Woods grazed but not burned, and some forest litter covers the soil.

Good: Woods protected from grazing, litter and brush adequately cover soil.

**Table 7-16**

**Conversion from Average Antecedent Moisture Conditions**

**To Dry And Wet Conditions**

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

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

## Table 7-17

## Rainfall Groups For Antecedent Soil Moisture Conditions

## During Growing And Dormant Seasons

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

Source: Soil Conservation Service

**Time of Concentration**

**Triangular Hydrograph Equation**

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

$$L = 0.6\ t_c \qquad (7.11)$$

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

$$L = (l^{0.8}\ (S + 1)^{0.7}) / (1900\ Y^{0.5}) \qquad (7.12)$$

Where:

L = lag, hrs

l = length of mainstream to farthest divide, ft

Y = average slope of watershed, %

S = 1000/CN - 10

CN = SCS curve number

Figure 7-5 is a graphical solution that can be used to estimate the lag time from a natural, homogeneous watershed with the same curve number. Then the lag time can be corrected for the effects of urbanization by using Figures 7-6 and 7-7. The amount of modifications to the hydraulic flow length usually must be determined from topographic maps or aerial photographs following a field inspection of the area. The modification to the hydraulic flow length not only includes pipes and channels but also the length of flow in streets and driveways.

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

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

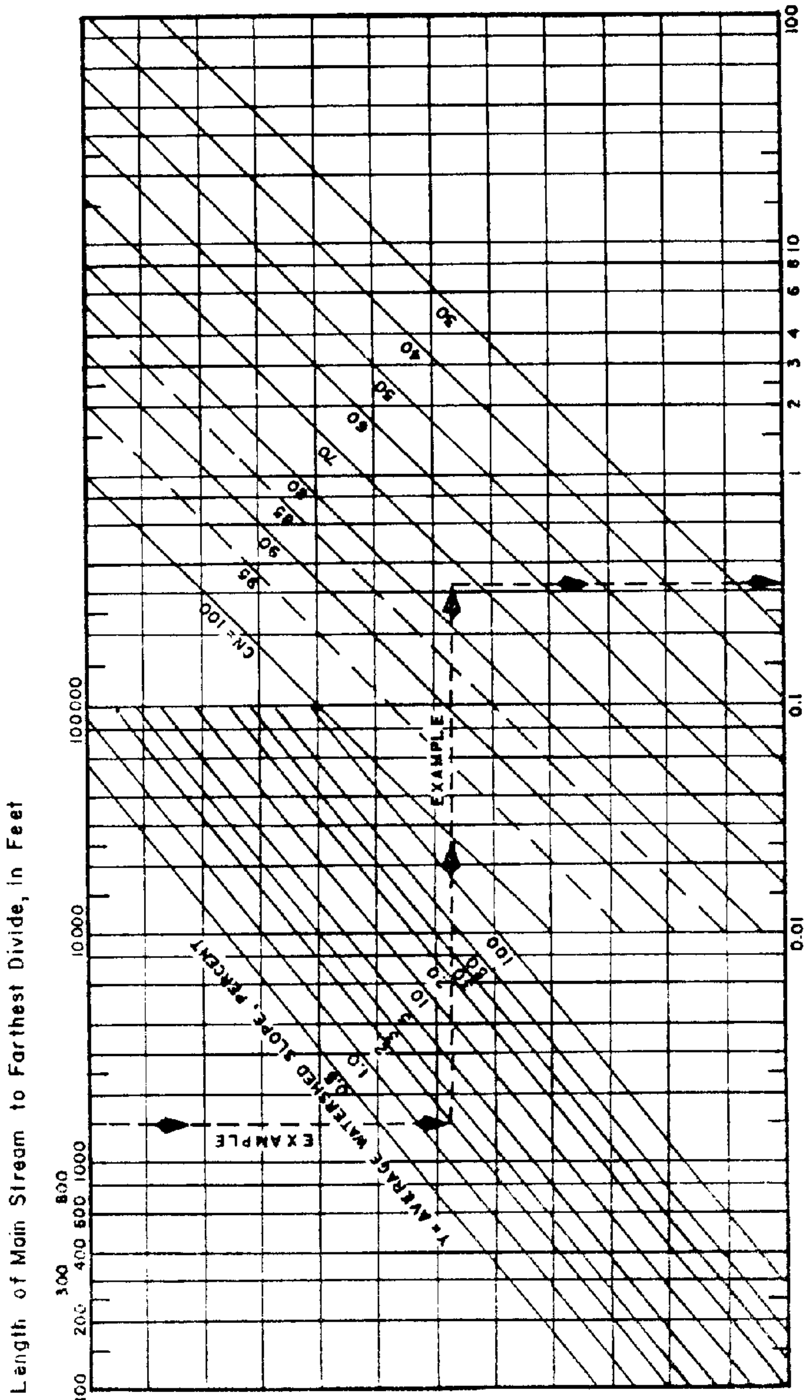



**Figure 7-5**

**Graphical Solution - SCS Lag Time**

**Source: Soil Conservation Service**

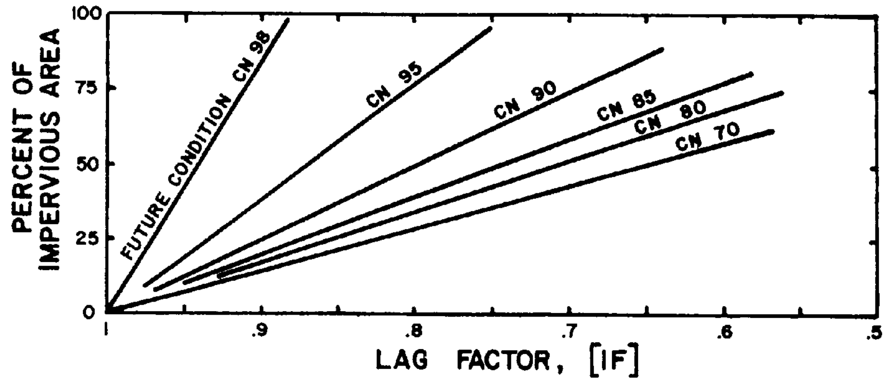



Figure 7-15 Factors For Adjusting Lag When Impervious Areas Occur In Watershed

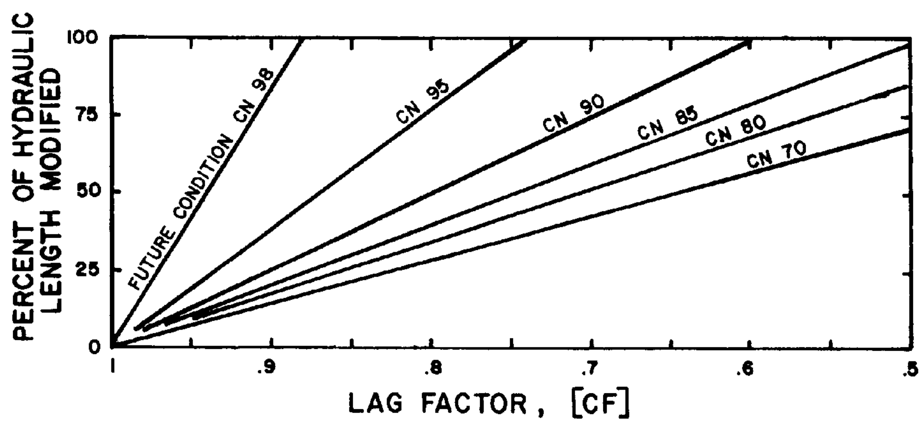



**Figure 7-7**

**Factors For Adjusting Lag when The Main Channel Has Been Hydraulically Improved**

**Source: HEC 19**

$$q_p = (484\ A\ (q)) / (d/2 + L) \qquad (7.13)$$

Where:

$q_p$ = peak rate of discharge, cfs

A = area, $mi^2$

q = storm runoff during time interval, inches

d = time interval, hrs

L = watershed lag, hrs

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

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

## 7.16. Example Problem(s) – SCS Unit

## 7.17. Hydrograph

(Example problem is under development using an example from TR55)

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## Appendix A: Design Flood or Storm Selection Guidelines

### Design Flood Frequencies

| Type of Structure | Return Period (Exceedance Probability) |
|---|---|
| Culverts in Designated Flood Hazard areas* | 100 years (1%) |
| Culverts on Primary Highways | 50 years (2%) |
| Culverts on Secondary Highways with high D.H.V.'s or providing Sole Area Access | 50 years (2%) |
| Culverts on Secondary Highways of less importance | 10 years (10%) |
| Channel Changes in Designated Flood Hazard Areas | 100 years (1%) |
| Channel Changes along Primary Highways & important Secondary Highways | 50 years (2%) |
| Channel Changes along less important Secondary Highways | 25 years (4%) |
| Trunk Storm Sewers Lines on Primary Highways | 50 years (2%) |
| All other Trunk Storm Sewer Lines | 25 years (4%) |
| Storm Sewer Feeder Lines | 10 years (10%) |
| Side Ditches, Storm Water Inlets and Gutter Flow | 10 years (10%) |
| Side Ditches, Storm Water Inlets and Gutter Flow in Depressed Roadway Sections | 50 years (2%) |
| Bridges in Designated Flood Hazard Areas* | 100 years (1%) |
| Bridges on all Highways | 50 years (2%) |
| Scour at Bridges, Design | 100 years (1%) |
| Scour at Bridges, Check | 1.7x100 years or<br>500 years (0.2%) |

* Unless local ordinance requires a greater design frequency.

**NOTE**: In addition to the exceedance probability used for design purposes the Federal Highway Administration under Executive Order #11988 and the State of Alaska under Administrative Order #46 (A.O. #46) require the evaluation of a structure's ability to pass an event with an exceedance probability of 1% (Q100). This evaluation is required on all tidal and fresh water stream encroachments (i.e. 100 year tidal surge and/or 100 year flood). A.O. #46 further requires the evaluation of flood-related erosion-prone and mudslide (i.e. mud flow) hazard areas. In the case of erosion, this includes currents of water exceeding anticipated cyclical levels, or an unusually high water level in a natural body of water, accompanied by a severe storm, or by an unanticipated force of nature, such as a flash flood or an abnormal tidal surge, or by some similarly unusual and unforeseeable event which results in flooding. For mudslides this includes periods of unusually heavy or sustained rain.

## Appendix B:  Rainfall Curves

(Table is under development.)

## Appendix C: Soil Classifications

(Table is under development)

## Appendix D: Travel Time Estimation

### Introduction

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

Following is a discussion of procedures and equations for calculating travel time and time of concentration.

### Travel Time

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

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

$$T_t = L/(3600V) \qquad \text{(D1)}$$

Where:

$T_t$ = travel time, hr

L = = flow length, ft

V = average velocity, ft/s

3600 = conversion factor from seconds to hours.

### Time of Concentration

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

$$T_c = T_{t1} + T_{t2} + \ldots T_{tm} \qquad \text{(D2)}$$

Where:

$T_c$ = time of concentration, hr

m = number of flow segments.

### Sheet Flow

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

For sheet flow of less than 300 feet, use Manning's kinematic solution (Overton and Meadows 1976) to compute $T_t$:

$$T_t = [0.007\,(nL)^{0.8} / (P_2)^{0.5} s^{0.4}] \qquad \text{(D3)}$$

Where:

$T_t$ = travel time, hr

n = Manning's roughness coefficient (Table D-1)

L = flow length, ft

$P_2$ = 2-year, 24-hour rainfall, in

s = slope of hydraulic grade line (land slope), ft/ft

**Table D-1 - Roughness Coefficients (Manning's n) For Sheet Flow**

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

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

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

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

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

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

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

**Shallow Concentrated Flow**

After a maximum of 300 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from Figure D-1 on the next page, in which average velocity is a function of watercourse slope and type of channel. For slopes less than 0.005 ft/ft, use equations given below for Figure D-1.

Average velocities for estimating travel time for shallow concentrated flow using Figure D-1.

Unpaved $V = 16.1345(s)^{0.5}$ (D4)

Paved $V = 20.3282(s)^{0.5}$ (D5)

Where:

V = average velocity, ft/s

S = slope of hydraulic grade line (watercourse slope), ft/ft

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

After determining average velocity using Figure D-1, use equation D1 to estimate travel time for the shallow concentrated flow segment.

**Open Channels**

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

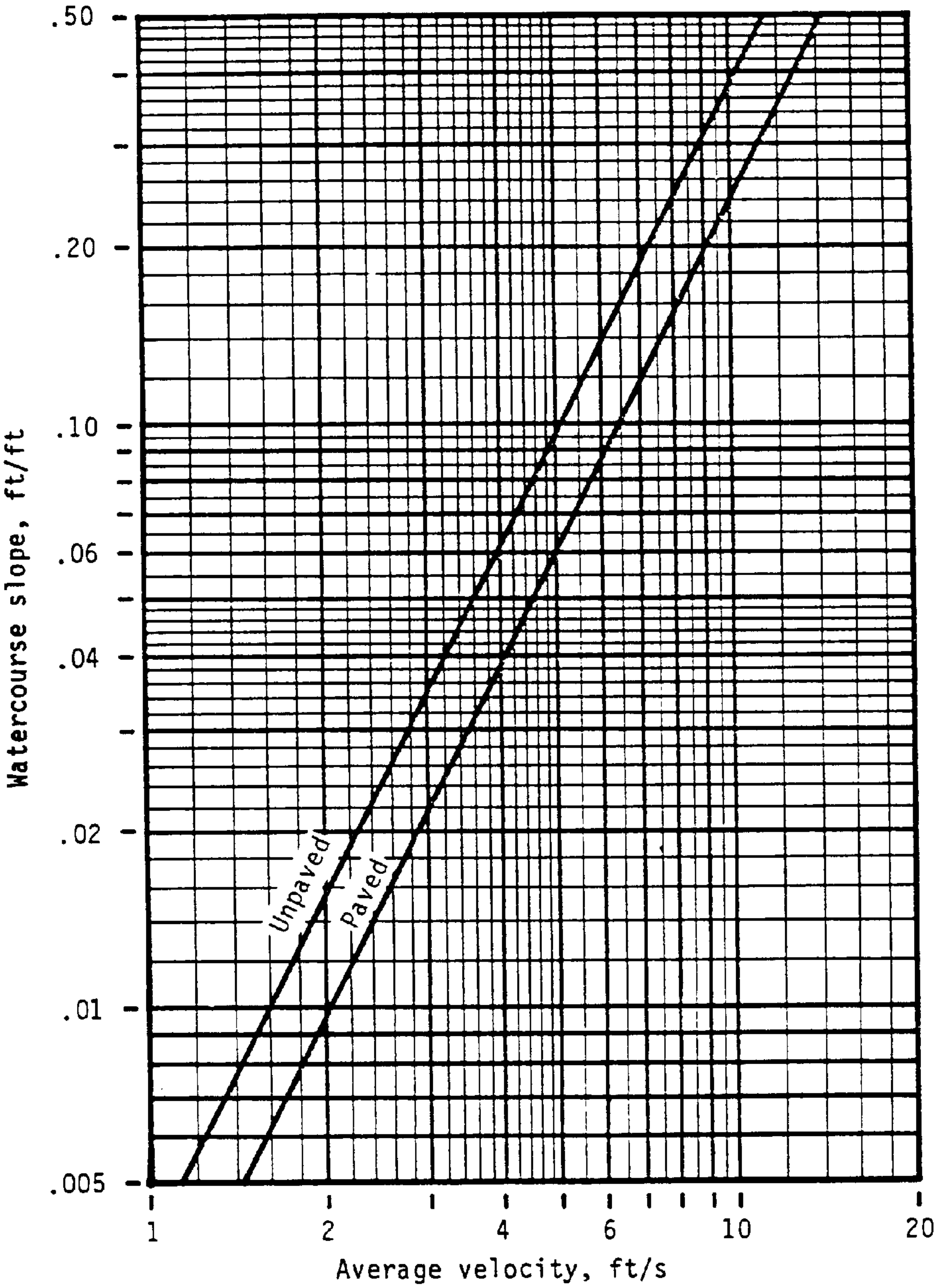


**Figure D-1**

**Average Velocities for Estimating Travel Time**

**For Shallow Concentrated Flow**

**Source: SCS TR-55**

Manning's equation is

$$V = (1.49\ r^{2/3}\ s^{1/2})/n \qquad \text{(D6)}$$

where:

$V$ = average velocity, ft/s

$r$ = hydraulic radius, ft (equal to $a/p_w$)

$a$ = cross sectional flow area, ft$^2$

$p_w$ = wetted perimeter, ft

$s$ = slope of the hydraulic grade line, ft/ft

$n$ = Manning's roughness coefficient

After average velocity is computed using equation D6, $T_t$ for the channel segment can be estimated using equation D1.

**Reservoir or Lake**

Sometimes it is necessary to compute a $T_c$ for a watershed having a relatively large body of water in the flow path. In such cases, $T_c$ is computed to the upstream end of the lake or reservoir, and for the body of water the travel time is computed using the equation:

$$V_w = (gD_m)^{0.5} \qquad \text{(D7)}$$

Where:

$V_w$ = the wave velocity across the water, ft/s

$g$ = 32.2 ft/s$^2$

$D_m$ = mean depth of lake or reservoir, ft

Generally, $V_w$ will be high 8 - 30 ft/s.

One must not overlook the fact that equation D7 only provides for estimating travel time across the lake and for the inflow hydrograph to the lake's outlet. It does not account for the travel time involved with the passage of the inflow hydrograph through spillway storage and the reservoir or lake outlet. This time is generally much longer and is added to the travel time across the lake. The travel time through lake storage and its outlet can be determined by the storage routing procedures in Storage Chapter.

For additional discussion of equation D7 see King's *Handbook of Hydraulics*, fourth edition, page 8-50, or *Elementary Mechanics of Fluids*, by Hunter Rouse, John Wiley and Sons, Inc., 1946, page 142.

Equation D7 can be used for swamps with much open water, but where the vegetation or debris is relatively thick less than about 25 percent open water, Manning's equation is more appropriate.

**Limitations**

- Manning's kinematic solution should not be used for sheet flow longer than 300 feet. Equation D3 was developed for use with the four standard rainfall intensity-duration relationships.
- In watersheds with storm drains, carefully identify the appropriate hydraulic flow path to estimate $T_c$. Storm drains generally handle only a small portion of a large event. The rest of the peak flow travels by streets, lawns, and so on, to the outlet. Consult a standard hydraulics textbook to determine average velocity in pipes for either pressure or nonpressure flow.
- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. Detailed storage routing procedures should be used to determine the outflow through the culvert.