CITY OF LONGMONT
STORM DRAINAGE CRITERIA MANUAL

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

SECTION 600 STORM RUNOFF

601 INTRODUCTION

For the area within the jurisdiction of this MANUAL, two deterministic hydrological models can be used to predict storm runoff (POLICY Section 304.4), the Rational Method, and the Colorado Urban Hydrograph Procedure (CUHP) developed by the Urban Drainage and Flood Control District (UD&FCD). The detailed computational techniques for these methods are presented in this section. These methods can be employed without the use of computers or basin calibration. For certain circumstances, such as drainage basins greater than 10 square miles, a statistical or regional analysis may be required to predict the storm runoff peaks or for calibration of deterministic models (see Section 607).

602 RATIONAL METHOD

The information contained in this section was obtained from the Urban Storm Drainage Criteria Manual (Reference-1, Revised May 1, 1984). Some changes have been made to the basic data and procedures to reflect the specific requirements within the City of Longmont.

For drainage basins whose area is less than 160 acres, the design storm runoff may be analyzed using the Rational Method (POLICY Section 304.4). This method was introduced in 1889 and is still being used in many engineering offices in the United States. Even though this method has frequently come under academic criticism for its simplicity, no other practical drainage design method has evolved to a level of general acceptance by the practicing engineer. The Rational Method, when properly understood and applied, can produce satisfactory results for urban storm sewer design.

602.1 Rational Formula

The Rational Method is based on the formula:

$$Q = CIA \tag{601}$$

Q is defined as the maximum rate of runoff in cubic feet per second (actually Q has units of acre inches per hour, which is approximately equal to the units of cubic feet per second). C is a runoff coefficient which is the ratio between the maximum rate of runoff from the area and the average rate of rainfall intensity (in inches per hour) for the period of maximum rainfall of a given frequency of occurrence having a duration equal to the time of concentration. The I is the average intensity of rainfall in inches per hour for a duration equal to the time of concentration. The time of concentration usually is the time required for water to flow from the most remote point of the basin to the point being investigated. A is the contributing basin area in acres.

The basic assumptions made when applying the Rational Formula are:

1. The computed maximum rate of runoff to the design point is a function of the average rainfall rate during the time of concentration to that point.

2. The maximum rate of rainfall occurs during the time of concentration, and the design rainfall depth during the time of concentration is converted to the average rainfall intensity for the time of concentration.

3. The maximum runoff rate occurs when the entire area is contributing flow. However, this asumption has been modified from time to time when local rainfall/runoff data was used to improve calculated results.

602.2 Limitations on Methodology

The Rational Method adequately approximates the peak rate of runoff from a rainstorm in a given basin. The critics of the method usually are unsatisfied with the fact that the answers are only approximations. A shortcoming of the Rational Method is that only one point on the runoff hydrograph is computed (the peak runoff rate). Some methods, such as the John Hopkins University Inlet Method, approximates a Rational Method hydrograph by a triangle with the peak occurring at the time of concentration. The inlet method also allows for routing of the hydrograph through various drainage facilities. However, due to the uncertainties in estimating the runoff coefficient and time of concentration, the more sophisticated inlet method may not provide any better results, requires calibration, and requires additional effort for the computation. Therefore, the basic Rational Method is considered adequate for the Boulder County area.

Another disadvantage of the Rational Method is that with typical design procedures one normally assumes that all of the design flow is collected at the design point and that there is no "carry over water" running overland to the next design point. However, this is not the fault of the Rational Method, but of the design procedure. The problem becomes one of routing the surface and subsurface hydrographs which have been separated by the storm sewer system. In general, this sophistication is not warranted and a conservative assumption is made that the entire routing occurs through the storm sewer system.

Finally, whereas the Rational Method can be used for basins up to 160 acres, the size limitation is for the sum of all the sub-basins and not on the size of a single basin. The maximum size of any single basin should be considerably less and should not exceed 15 acres for offsite flow analysis and 5 acres for onsite flow analysis. These sub-basin sizes are based on a typical local street gutter capacity for the onsite analysis and the minimum size storm sewer for the offsite analysis.

602.3 Time of Concentration

One of the basic assumptions underlying the Rational Method is that runoff is a function of the average rainfall rate during the time required for water to flow from the most remote part of the drainage area to the point under consideration. However, in practice the time of concentration can be an empirical value that results in reasonable and acceptable peak flow calculations. The time of concentration relationships recommended in this MANUAL are based in part on the rainfall/runoff data collected in the Denver Metropolitan area and were developed in conjunction with the runoff coefficients recommended in this MANUAL.

For urban areas, the time of concentration consists of an inlet time or overland flow time (t-sub i) plus the time of travel (t-sub t) in the storm sewer, paved gutter, roadside drainage ditch, or drainage channel. For non-urban areas, the time of concentration consists of an overland flow time (t-sub i) plus the time of travel in a combined form, such as a small swale, channel, or drainageway. The latter portion (t-sub t) of the time of concentration can be estimated from the hydraulic properties of the storm sewer, gutter, swale, ditch, or drainageway. Inlet time, on the other hand, will vary with surface slope, depression storage, surface cover, antecedent rainfall, and infiltration capacity of the soil, as well as distance of surface flow. Thus, the time of concentration can be calculated using Equation 602 for both urban and non-urban areas:

$$t_c = t_i + t_t \qquad (602)$$

In which t_c = time of concentration (minutes)

t_i = initial, inlet, or overland flow time (minutes)

t_t = travel time in the ditch, channel, gutter, storm sewer, etc. (minutes)

To aid in the computation of t-sub c, Standard Form SF-8 has been developed to organize computation. All drainage report submittals (Section 200) shall include t-sub c calculations using SF-8 in the report.

602.3.1 Non-Urbanized Basins

The initial or overland flow time (t-sub i) in non-urbanized watersheds may be calculated using Equation 603 or Figure-601.

$$t_i = 1.8\,(1.1 - C_5)\sqrt{L}\;/\;3\sqrt{S} \qquad (603)$$

Where t_i = initial or overland flow time (minutes)

C_5 = runoff coefficient for 5-year frequency (from Table-601)

L = length of overland flow, (feet, 500-feet maximum)

S = average basin slope (percent)

Equation 603 is considered adequate for distances up to 500-feet. For longer basin lengths, the time of concentration needs to be considered in combination with the travel time (t-sub t) which is calculated using the hydraulic properties of the swale, ditch, or channel. For preliminary work, the travel time (t-sub t) can be estimated with the help of Figure-602. The time of concentration is then the sum of the initial flow time (t-sub i) and the travel time (t-sub t). The minimum t-sub c recommended for non-urban watershed is 10-minutes. The process of calculating the time of concentration in non-urbanized basin is illustrated in the following example:

Example No. 1: Time of Concentration in Non-Urban Basin

Given: A 15 acre non-urbanized rangeland watershed. Watershed has a length of 660-feet and an average slope of 1.0 percent. Uppermost 400-feet of watershed has an average land slope of 2.0 percent.

Required: Time of concentration

Solution:

Step 1: First find the runoff coefficient for clay type soil grassland for a 5-year storm. From Table-601

$C_5 = 0.10$

Step 2: Find the initial (overland) flow time for the uppermost 400-feet of the watershed. From Figure-601, t-sub i for the 400-foot sub-watershed length, slope and C-sub 5 is

$t_i = 30$ minutes

Step 3: Find the travel time for the remaining 260-feet of the watershed length. From Figure-602, the average travel velocity for the given watershed, slope, and "Short Grass Pasture" curve is

$V = 0.7$ ft/sec.

The travel time can be calculated using this velocity and 260-feet of travel length.

$t_t = L/60V = 260 \text{ ft.} / (60 \text{ sec/min})(0.7 \text{ ft/sec})$

$t_t = 6$ minutes

Step 4: Combine t_i and t_t to find the time of concentration t_c.

$$t_c = t_i + t_t \quad (602)$$

$t_c = 30 + 6 = 36$ minutes (greater than 10 minutes minimum)

602.3.2 Urbanized Basins

Overland flow in urbanized basins can occur from the back of the lot to the street, in parking lots, in greenbelt areas, or within park areas. It can be calculated using the procedure described in Section 602.3.1 except the travel time t-sub t to the first design point or inlet is estimated using the "Paved Area (Sheet Flow) & Shallow Gutter Flow" line in Figure-602. The time of concentration for the first design point in an urbanized basin using this procedure should not exceed the time of concentration calculated using Equation 604, which was developed using the rainfall/runoff data collected in the Denver region.

$$t_c = L/180 + 10 \quad (604)$$

In which t_c = time of concentration at the first design point in an urban watershed (minutes)

L = watershed length (feet)

Normally, Equation 604 will result in a lesser time of concentration at the first design point and will govern in an urbanized watershed. For subsequent design points, the time of concentration is calculated by accumulating the travel times in downstream drainageway reaches. The minimum t-sub c recommended for urbanized area is 10 minutes.

The inlet time can be estimated by calculating the various overland distances and flow velocities taken from the most remote point. A common mistake is to assume travel velocities that are too small. Another common error is to not analyze the portion of basin which would result in the longest computed time of concentration. This error is most often encountered in long basins, or a basin where the upper portion contains grassy park land and the lower developed urban land.

When studying a tract of land proposed for subdivision, the overland flow path should not necessarily be taken perpendicular to the contours since the land will be graded and swales will often intercept the natural contour and conduct the water to the streets, thus increasing the time of concentration.

Example No. 2: Time of Concentration in Urbanized Basin

Given: a 100 acre single family residential development. Watershed has a total length of 1000-feet. The upper overland flow portion is from back of a lot and is 100-feet in length at an average slope of 2.0 percent. The overland flow is mostly over grass and a short driveway, having a composite 5-year runoff coefficient of 0.35. The lower 900-feet of the basin has an average slope of 1.0 percent.

Find: Time of concentration at the first storm sewer inlet located 1000-feet from the top of the watershed.

Solution:

Step 1: Find the initial (overland) flow time for the uppermost 100-feet at 2 percent slope. From Figure-601

t_i = 11 minutes

Step 2: Using "Paved Area (Sheet Flow) & Shallow Gutter Flow" curve on Figure-602 find the average flow velocity for the remaining 900-feet at 1.0 percent slope

V = 2.0 ft/sec

Step 3: Calculate the travel time t_t using the velocity found in Step 2.

t_t = L/60V = 900-feet / (60 sec/min)(2.0 ft.sec)

t_t = 7.5 minutes

Step 4: Calculate the time of concentration using Equations 602 and 604. Select the smaller time of the two as the time of concentration at the first inlet.

$$t_c = t_i + t_t$$

$$t_c = 11.0 + 7.5 = 18.5 \text{ minutes}$$

or

$$t_c = L/180 + 10$$

$$t_c = 1000/180 + 10 = 15.6 \text{ minutes}$$

Since the Equation 604 gives the smaller time of concentration, it controls. Thus, at first inlet use

$$t_c = 15.6 \text{ minutes}$$

Step 5: Continue the time of concentration calculations in the downstream direction. The flow calculated at each design point is then used to calculate the flow velocity in the downstream pipe, gutter, swale, or channel. This flow velocity is then used to calculate the time of travel to the next downstream design point.

The individual travel times are then accumulated in a downstream direction to calculate the time of concentration at each successive downstream design points.

602.4 Intensity

The intensity, I, is the average rainfall rate in inches per hour for the period of maximum rainfall of a given frequency having a duration equal to the time of concentration.

After the design storm frequency has been selected, and the rainfall zone within which the property lies has been identified, the rainfall intensity can be obtained from Table-505 or from Figures-502, -503, or -504. Refer to Section 500 Rainfall of this MANUAL for additional information.

602.5 Runoff Coefficient

The runoff coefficient, C, represents the integrated effects of infiltration, evaporation, retention, flow routing, and interception, all which effect the time distribution and peak rate of runoff. Determination of the coefficient requires judgement and understanding on the part of the engineer. Table-601 presents the recommended values of C for the various recurrence frequency storms. The values are presented for different surface characteristics as well as for different aggregate land uses.

Sometimes a composite runoff coefficient is desirable computed on the basis of the percentage of different types of surface in the drainage area. This procedure is often applied to typical "sample" blocks as a guide to selection of reasonable values of the coefficient for an entire area. Suggested coefficients with respect to surface type are also given in Table-601 under the column labelled "Percent Impervious". Where land use features are known, a composite C analyis will result in more accurate results. Refer to the Storm Sewers Section of this MANUAL for further discussion of the use of the Rational Method.

Table-601 lists runoff coefficients that vary with recurrence frequency. The coefficients were developed using the available rainfall and runoff information in the Denver region and were designed to work in conjunction with the time of concentration recommendations in Section 602.3. Use of these coefficients and procedure outside of the semi-arid climates found in areas such as the Denver region may not be valid. Because the coefficients vary with frequency, no further adjustments are needed for large storms.

602.6 Application of the Rational Method

The first step in applying the Rational Method is to obtain a topographic map and define the boundaries of all the relevant drainage basins. Basins to be defined include all basins tributary to the area of study and sub-basins in the study area. A field check and possibly field surveys should be made for each basin. At this stage of planning, the possibility for the diversion of trans-basin waters should be identified.

The major storm drainage basin does not always coincide with the minor storm drainage basin. This is often the case in urban areas where a low flow will stay next to a curb and follow the lowest grade, but when a large flow occurs the water will be deep enough so that part of the water will overflow street crowns and flow into a new sub-basin. For an example of how to apply the Rational Method refer to the Storm Sewer Section of this MANUAL (Section 805).

602.7 Major Storm Analysis

When analyzing the major runoff occurring on an area that has a storm sewer system sized for the initial storm, care must be used when applying the Rational Method. Normal application of the Rational Method assumes that all of the runoff is collected by the storm sewer. For the initial storm design, the time of concentration is dependent upon the flow time in the sewer. However, during the major storm runoff, the sewers will probably be at capacity and could not carry the additional water flowing to the inlets. This additional water then flows overland past the inlets, generally at a lower velocity than the flow in the storm sewers.

If a separate time of concentration analysis is made for the pipe flow and surface flow, a time lag between the surface flow peak and the pipe flow peak will occur. This lag, in effect, will allow the pipe to carry a larger portion of the major storm runoff then would be predicted using the initial storm time of concentration. The basis for this increased benefit is that the excess water from one inlet will flow to the next inlet downhill, using the overland route. If that inlet is also at capacity, the water will often continue on until capacity is available in the storm sewer. The analysis of this aspect of the interaction between the storm sewer system and the major storm runoff is complex. The simplified approach of using the initial storm time of concentration for all frequency analysis is acceptable for Boulder County.

603 EFFECTIVE RAINFALL

Effective rainfall is that portion of precipitation which runs off the land to drainageways after a rainstorm. Those portions of precipitation that do not reach drainageways are called abstractions, and include: interception by vegetation, evaporation, infiltration, storage in all surface depressions, and long-time surface retention. The total design rain falling to the earth may be obtained from the Rainfall part of the MANUAL (Section 500). This section illustrates a method for estimating the amount of rainfall that actually becomes runoff when a design rainstorm is used.

603.1 Pervious-Impervious Areas

All parts of a basin can be considered either pervious or impervious. The pervious part of a drainage basin is that area where water can readily infiltrate into the ground. The impervious part is that area where water does not readily infiltrate into the ground, such as areas which are paved or covered with buildings and sidewalks. In urban hydrology the percent of pervious and the percent of impervious land is important. As urbanization occurs, the percent of impervious area increases and the rainfall-runoff relation changes significantly. The total amount of runoff volume normally increases, the time to the runoff peak rate decreases, and the peak runoff rates increase as the area urbanizes.

When analyzing an area for design purposes, the probable future percent of impervious area must be estimated. Table-601 is presented as a guide for estimating the total percent of imperviousness. Although some of the runoff models use "connected impervious" areas in estimating effective rainfall, the CUHP runoff model was developed using the total, impervious area in the watershed. Thus, all references to impervious area and all calculations in this MANUAL are based on total impervious areas.

Figure-603 is provided to assist in estimating the percent impervious values for low and medium density residential areas. The impervious values are based on the observed trends in the urbanized portions of the Denver Metropolitan area and reflect the densities found in built-up areas, but do not account for parks or large open spaces.

603.2 Depression and Retention Losses

Rain water that is collected and held in small depressions and does not become part of the general runoff is called depression storage. Most of this water eventually infiltrates or is evaporated. Depression losses include water intercepted by trees and bushes and water that is detained on the surface and does not run off until the storm is over. The water that is held in depressions on roofs and roads and eventually evaporates is considered a part of depression losses. The CUHP method in Section 604 uses numerical values of depression and retention in calculating the effective rainfall. Table-602 can be used as a guide to estimating the amount of depression and retention storage.

When an area is analyzed for depression and retention storage, the various pervious and impervious storage values must be considered and accumulated according to the percent of aerial coverage. An area can be designed to have an artifically high depression and retention storage and thus reduce the maximum rate of runoff from that area. This can be done by using flat or depressed areas in parks and open spaces and on flat roofs.

603.3 Infiltration

The flow of water through the soil surface is called infiltration. In urban hydrology much of the infiltration occurs on areas covered with lawn grass. Urbanization can increase or decrease the total amount of infiltration (Reference-29).

Soil type is the most important factor in determining the infiltration rate. When the soil has a large percent of well-graded fines, the infiltration rate is low. In cases of extremely tight soils, there may be essentially no infiltration from a practical standpoint. If the soil has several layers or horizons, the least permeable layer will control the maximum infiltration rate. The soil cover also plays an important role in determining the infiltration rate. Vegetation, and lawn grass in particular, tends to increase infiltration by loosening the soil near the surface. Other factors affecting infiltration rates include: slope of land, temperature, quality of water, age of lawn, and soil compaction (Reference-30).

As the rainfall continues the infiltration rate decreases. When a rainfall occurs on an area that has little antecedent moisture (i.e., the ground is dry), the infiltration rate is much higher than with a high antecedent moisture, such as from a previous storm or from irrigation. Although antecedent precipitation is very important when calculating runoff from smaller storms in non-urbanized areas, the data collected in the urbanized basins indicates that antecedent precipitation has a limited effect on runoff in the urbanized areas within the UD&FCD. The effects are also assumed to be limited within Boulder County.

There are many infiltration models in use by the urban storm runoff hydrologists. These models range widely in complexity; however, because of the climatic condition in the semi-arid region and because runoff from urban watersheds is not very sensitive to infiltration refinements, the infiltration model proposed by Horton (References-31 and -32) was found to provide a good balance between simplicity and reasonable physical description of the infiltration process. Horton's infiltration model is described by Equation 605.

$$f = f_o + (f_i - f_o)\, e^{-at} \qquad (605)$$

In which f = infiltration rate (inches/hour)

f_i = initial infiltration rate (inches/hour)

f_o = final infiltration rate (inches/hour)

e = natural logarithm base

a = decay coefficient (1/second)

t = time in seconds

Horton's equation is illustrated graphically in Figure-604. In developing this equation Horton observed that infiltration is high early in the storm and eventially decays to a steady state constant value as the pores in the soil become saturated. The coefficients and initial and final infiltration values are site specific and depend on the soils and vegetative cover complex. These values can be developed for each site if sufficient rainfall/runoff observations are made. However, such an approach is not practical. The UD&FCD has analyzed a considerable amount of rainfall/runoff data. On the basis of this analysis, the values in Table-602 are recommended for use in conjunction with the 1982 version of the CUHP. Most frequently found within Boulder County are SCS Hydrologic Soil

Groups are C and D. However, areas of Group A and B soils may also be found within Boulder County. The engineer should consult SCS Soil surveys whenever available.

To calculate the amount of maximum infiltration depths that may occur at each time increment, it is necessary to integrate Equation 605 and calculate the values for each time increment. Very little accuracy is lost if instead of integrating Equation 605, the infiltration rate is calculated at the center of each time increment and is then multiplied by the unit time increment to calculate the infiltration depth. This was done for the four SCS Hydrologic Soil Groups and the results are presented in Table-603. Although Tables-602 and -603 provide recommended values for various Horton equation parameters, these recommendations are being made only for the urbanized or urbanizing watersheds in the Denver metropolitan area.

603.4 Example Calculation

That portion of rainfall that becomes runoff during or soon after a storm is called effective rainfall. The abstractions from rainfall that determine the effective precipitation are functions of infiltration, retention, and depression storage, intensity of rainfall, percent of imperviousness, etc. An example of estimating the effective rainfall is presented below using Table-604 as an aid in calculations.

Example No. 3: Effective Rainfall Calculation

Col. 1: For the design location select a rainfall time interval, usually 5-minutes (see Section 604.4).

Col. 2: Select a design frequency and the design storm using the procedure described in Section 500 RAINFALL.

Pervious Area, Columns 3 through 6

Col. 3: Tabulate Increments of infiltration for each time period. In this example, Table-603 values for SCS Hydrologic Soil Group D were used.

Col. 4: The total pervious depression storage is selected from Table-602 and appears as the total at the bottom of Column 4. For each time period the depression storage in Column 4 is found by subtracting infiltration, Column 3, from precipitation, Column 2. If the result is negative, there is no excess for this period, and Column 4 and Column 5 are zero. If the result is positive, then enter the amount in Column 4 as the depression storage for that time period or the amount available as determined by deducting the total accumulated amount from the total assumed value shown at the bottom of Column 4. When the cumulative amount in Column 4 equals the total shown at the bottom of the column, the depression storage is fully used and all remaining values are zero. Note that the last value of detention storage will usually be less than Column 2 minus Column 3 and will be determined by finding the difference between the maximum amount and the amount of infiltration through the previous time periods.

Col. 5: Effective precipitation for the pervious area in Column 2 minus Column 3 and 4, positive values only.

Col. 6: Column 5 times the (decimal) percent of the pervious area gives the area-weighted depth of water that will run off in each time increment for the pervious area.

Impervious Area, Columns 7 through 10

Col. 7: Enter the total assumed impervious depression storage selected from Table-602 at the bottom of Column 7. The impervious depression storage in Column 7 is then either the amount of precipitation in Column 2 or the amount available as determined by deducting the total accumulated amount from the total assumed value shown at the bottom of Column 7. When the total assumed amount is fully used, all remaining values are zero. The depression and retention losses are obtained from Table-602.

Col. 8: After all of the impervious storage has been filled, an impervious loss will still occur. To account for this, Column 8 is computed by taking 5 percent of Column 2 less Column 7.

Col. 9: Effective precipitation for the impervious area is Column 2 less Column 7 and 8.

Col. 10: Column 9 times the (decimal) percent of impervious area gives the area-weighted depth of water that will run off in each time increment for the pervious area.

Col.11: Add Columns 10 and 6 to obtain the average effective rainfall. This is the "design effective rainfall" that will be applied to the unit hydrograph, Section 604, to obtain the design storm runoff hydrograph.

604 COLORADO URBAN HYDROGRAPH PROCEDURE

The information contained in this section was obtained from the Urban Storm Drainage Criteria Manual (Reference-1, Revised May 1, 1984). Some changes have been made to the basic data and procedures to reflect the specific requirements of the Boulder County area.

For basins that are larger than 160 acres, the Colorado Urban Hydrograph Procedure (CUHP) shall be used (POLICY - Section 304.4).

604.1 History of CUHP

The unit hydrograph principle was originally developed by Sherman in 1932 (Reference-20). The synthetic unit hydrograph, which is used for analysis when there is no rainfall-runoff data for the basin under study, as is often the case in the Denver region, was developed by Snyder in 1938 (Reference-21). The presentation given in this section is termed the Colorado Urban Hydrograph Procedure (CUHP) because coefficients and the form of the equation are based upon data collected in Colorado and on studies conducted or financed by the UD&FCD. The data for use in the development of the 1982 version of the CUHP were collected by the U.S. Geological Survey between 1969 and 1981 under a cooperative agreement with the UD&FCD. Data collection activities are being continued under a similar cooperative agreement between the District and USGS; however, the number of stations has been reduced. The goal of the currently ongoing data collection effort is to develop a long term data base for further refinements to the hydrologic techniques.

604.2 Unit Hydrograph Theory

A unit hydrograph is defined as the hydrograph of 1-inch of direct runoff from the tributary area resulting from a unit storm. A unit storm is a rainfall of such duration that the period of surface runoff is not appreciably less for any rain of shorter duration. The unit hydrograph thus represents the integrated effects of factors such as tributary area, shape, street pattern, channel capacities, and street and land slopes (References-22 through -27).

To apply the unit hydrograph, the effective precipitation depths for the "unit storm" periods are multiplied by the ordinates of the unit hydrograph and added to obtain a design storm runoff hydrograph. The basic premise of the unit hydrograph is that individual hydrographs resulting from the successive increments of rainfall excess that occur throughout a storm period will be proportional in discharge throughout their length, and that when properly arranged with respect to time the ordinates of the individual unit graphs can be added to give ordinates representing the total storm discharge. The hydrograph of total storm discharge is obtained by summing the ordinates of the individual hydrographs (principle of superposition).

The derivation and application of the unit hydrograph are based on the following assumptions:

1. The rainfall intensity is constant during the storm that produces the unit hydrograph.

2. The rainfall is uniformly distributed throughout the whole area of the drainage basin.

3. The base or time duration of the design runoff due to an effective rainfall of unit duration is constant.

4. The ordinates of the design runoff with a common base time are directly proportional to the total amount of direct runoff represented by each hydrograph.

5. The effects of all physical characteristics of a given drainage basin, including shape, slope, detention, infiltration, drainage pattern, channel storage, etc..., are reflected in the shape of the unit hydrograph for that basin.

604.3 Equations

There are four basic equations used in defining the limits of the synthetic unit hydrograph. The first equation defines the lag time of the basin in terms of time to peak, t-sub p, which for the CUHP method, is defined as the time from the center of the unit storm duration to the peak of the unit hydrograph as shown in Figure-605.

$$t_p = C_t \left[\left(L\, L_{ca} / \sqrt{S} \right) \right]^{0.48} \qquad (606)$$

In which t_p = time to peak of the unit hydrograph from midpoint of unit rainfall in hours.

L = length along stream from study point to upstream limits of the basin in miles.

L_{ca} = length along stream from study point to a point along stream adjacent to the centroid of the basin in miles.

S = weighted average slope of basin along the stream to upstream limits of the basin in feet per foot.

C_t = coefficient reflecting time to peak.

The time from the beginning of unit rainfall to the peak of the unit hydrograph is determined by:

$$T_p = 60t_p + 0.5t_u \tag{607}$$

In which T_p = time from beginning of unit rainfall to peak of hydrograph in minutes.

t_u = time of unit rainfall duration in minutes.

The unit peak of the unit hydrograph is defined by:

$$q_p = 640\ C_p/t_p \tag{608}$$

In which q_p = peak rate of runoff in cfs per square mile.

C_p = coefficient related to peak rate of runoff.

Once q_p is determined, the peak of the unit hydrograph for the basin is computed by:

$$Q_p = q_p A \tag{609}$$

In which Q_p = peak of the unit hydrograph in cfs.

A = area of basin in square miles.

604.4 Unit Storm Duration

For most urban studies the unit storm duration, t-sub u, should be 5-minutes. However, the unit duration may be increased for larger basins. The unit duration should be incremented for convenience in multiples of 5-minutes (i.e., 10- or 15-minutes) with the maximum unit duration recommended at 15-minutes for the 1982 version of the CUHP. The unit storm duration for values larger than 5-minutes should not exceed one-third of t-sub p. As an example, if the basin has a t-sub p = 35-minutes then an appropriate unit storm duration would be 5-minutes or 10-minutes (i.e., less than or equal to 1/3 t-sub p).

604.5 Basin Size Limitations

The rainfall/runoff data used in the development of the 1982 version of the CUHP was obtained primarily from small basins. Basin sizes ranged from 0.15 square miles to 3.08 square miles. Although some extrapolation is justified, unlimited extrapolation of how the basin response to rainfall is not justified. The maximum size of a basin to be analyzed with a single unit hydrograph should be less than 5 square miles. Whenever a larger basin needs to be studied, the basin should be subdivided into sub-basins of 5 square miles or less and individual sub-basin storm hydrograph be routed downstream using appropriate channel routing procedures. The routed hydrographs are then added to develop a single conposite storm hydrograph.

604.6 Basin Shape Limitations

The basin shape can have a profound effect on the final results and, in some instances, can result in underestimates of peak flows. Experience with the 1982 version of the CUHP has shown that whenever basin length is increased faster than basin area, the storm hydrograph peak will tend to decrease. Although hydrologic routing is an integral part of runoff analysis, the data used to develop the 1982 version of the CUHP is insufficient to conclude that the observed CUHP response with increasing basin length is valid. For this reason, irregularly shaped or very long basins (i.e., basin length to width ratio of 4 or more) should be divided into more regularly shaped sub-basins. A composite basin storm hydrograph can be developed using appropriate routing techniques and by adding of the individual sub-basin storm hydrographs.

604.7 Basin Slope Limitations

The 1982 version of the CUHP was developed using data from basins having a range of major drainageway slopes between 0.005 ft/ft and 0.037 ft/ft. Caution is required when extrapolating beyond this range. In natural and grass lined drainageways, channels become unstable when a Froude Number of 1.0 is approached. As a result, there are natural processes at work that limit the time to peak of a unit hydrograph as the drainageway becomes steeper. To account for this phenomena, the recommended slope used in Equation 606 for natural drainageways and existing man made grass lined channels shall be adjusted using Figure-606.

A typical range of slopes for existing improved channels is 0.004 ft/ft to 0.007 ft/ft. For preliminary estimating purposes a slope of 0.005 ft/ft should be used for grass lined channels that are to be designed using this MANUAL.

This MANUAL also limits the Froude Number to 0.8 for riprap lined channels. For preliminary estimating purposes where riprap channels are contemplated, a slope of 0.10 should be used with Equation 606. When a riprap channel is in existence, use the measured average channel profile slope.

In concrete lined channels and buried conduits, the velocities can be very high. For this reason, the average ground slope (i.e., not flow line slope) should be used when concrete lined channels and/or storm sewers are part of the drainageways.

Where the flow line slope varies along the channel, calculate a weighted basin slope for use with Equation 606, by first segmenting the major drainageway into reaches having similar longitudinal slopes. Then calculate the weighted slope using Equation 610.

$$S = \left[\frac{L_1S_1^{0.24}+L_2S_2^{0.24}+\ldots.+L_nS_n^{0.24}}{L_1+L_2+L_3+\ldots.Ln}\right]^{4.17} \qquad (610)$$

In which $S_1,S^2,\ldots\ldots S_n$ = slopes of individual reaches in ft/ft (after adjusting using Figure-606).

$L_1,S_2,\ldots\ldots L_n$ = lengths of corresponding reaches.

604.8 Basin Land Use Consideration

A lumped parameter model such as a CUHP relies on data from basins having relatively uniform land use. For basins having zones of differing land use, the basin should be subdivided into sub-basins having relatively uniform land use. As an example, if a lower half of a watershed has been urbanized and the upper half is to remain as open space, two distinct hydrographs should be developed. The upper sub-basin hydrograph will be based on the coefficients for undeveloped land and the lower sub-basin hydrograph will be based on coefficients for the developed area.

604.9 Determination of C-sub t and C-sub p Coefficients

The value of C-sub t in Equation 606 may be determined using Figure-607. Note that the curve in Figure-607 can be represented by parabolic equations having the percent impervious (I-sub a) as an independent variable. Three sets of unique coefficients describe the curve, each of the three ranges of imperviousness. The mathematical description of the I-sub a vs C-sub t curve was developed so that the CUHP procedure could be computerized.

The value of C-sub p to be used in Equation 608 may be determined using Figure-608. The curve in Figure-608 is also represented with a parabolic equation. To determine C-sub p, first obtain the value of the Peaking Parameter P from Figure-608. Then calculate C-sub p using Equation 612:

$$C_p = P\ C_t A^{0.15} \qquad (612)$$

In which C_t = coefficient from Figure-607.

P = peaking parameter from Figure-608.

A = basin area in square miles.

604.10 Unit Hydrograph Shape

The shape of the unit hydrograph is a function of the physical characteristics of the watershed. The shape incorporates the effects of watershed size, shape, degree of development, slope, type and size of drainage system, soils, and many other watershed factors. The shape of the unit hydrograph is also dependent on the temporal and spatial distribution of rainstorms and will vary with each storm event. As a result, a unit hydrograph based on rainfall/runoff data is an approximation that provides the engineer or hydrologist with a reasonable unit hydrograph shape for a given hydrologic region.

Equations 606 through 613 are used to define the peak discharge and its location for the unit hydrograph. The widths of the unit hydrograph at 50 and 75 percent of the peak can be estimated using Figure-609. Note that the unit hydrograph width at 50 and 75 percent of the peak are given in hours. The two equations shown on Figure-609 mathematically describe the lines on the figure.

A study of many unit hydrographs generated using recorded rainfall and runoff events indicates that, as a general rule, 0.35 of the width at 50 percent of peak is to the left of the peak and 0.65 of the width is to the right of the peak. At 75 percent of the peak, 0.45 of the width is left of the peak and 0.55 of the width is to the right of the peak. However, on some hydrographs this rule requires modification. Whenever the above rule results in the hydrograph at 50 percent of peak being to the left of the peak by more than 0.6 T-sub p (T-sub p =

the distance from zero to the peak of the unit hydrograph); the x-coordinate at 50 percent of peak should be placed at 0.6 T-sub p, and at 75 percent of the peak the x-coordinate should be placed at 0.424 T-sub p. Figure-605 shows how a typical unit hydrograph may be shaped to best approximate the trends found in the rainfall/runoff data.

604.11 Hydrograph Calculation Example

Once the unit hydrograph has been determined (Section 604.10) and the effective precipitation from a design storm determined (Section 603.4), the design storm hydrograph can be calculated. The time units of the unit hydrograph should be the same as the time units of the excess precipitation.

In general, the procedure is as follows. Set up a table such as Table-605 putting time intervals in the first column and unit hydrograph ordinates in the second column. Place the design effective precipitation values as determined in Column 11 of Table-604 across the top, and then multiply the first excess precipitation value (0.06 in example) times all the unit hydrograph ordinates in Column 2 and put answers in the third column. Next multiply the second excess precipitation value (0.11) times the unit hydrograph ordinates lagged one time unit as shown in Column 4. Multiply each succeeding precipitation value times the unit hydrograph value and lag them appropriately in the table. Finally, add up all the multiplied values horizontally to obtain the design storm runoff hydrograph.

A specific example of generating a storm hydrograph using the CUHP methodology is as follows:

Example No. 4: CUHP Storm Hydrograph

Given: A basin that has the following characteristics:
Watershed Area (A) = 0.38 square miles = 243 acres
Watershed Length (L) = 1.28 miles
Distance to Centroid (L-sub ca) = 0.52 miles
Impervious Area (I_a) = 44%
Slope along Waterway (S) = 0.0102 ft/ft

Required: A storm hydrograph for the 100-year design storm.

Solution:

Step 1: Determine C-sub t using procedure given in Section 604.9. From the given percent of impervious cover (i.e., 44%) find the value of C-sub t from Figure-607

$C_t = 0.091$

Step 2: Determine t-sub p using Equation 606

$$t_p = C_t\, [(L\, Lca/ \sqrt{S}\,)]^{0.48}$$

$$= (0.091)[1.28 \times 0.52/ \sqrt{0.0102}\,]^{0.48}$$

$$t_p = 0.225 \text{ hours (13.5 minutes)}$$

Step 3: Determine if either a 5-minute unit hydrograph or a longer unit hydrograph is required

$t_u \leq 1/3t_p$ (5-minute minimum)

$(1/3)\ (13.5) = 4.5$

Use t_u = 5-minutes (0.083 hours)

Step 4: Determine C-sub p using Figure-608, the value of imperviousness, the given basin area, the C-sub t found in Step 2 and Equation 612. From Figure-608:

$P = 6.21$

Then

$C_p = P\ C_t A^{0.15}$

$= 6.21 \times 0.091 \times (0.38)^{0.15}$

$= 0.49$

Step 5: Determine q_p using Equation 608

$q_p = 640\ C_p/t_p$

$= 640 \times 0.49/0.225$

$= 1394$ cfs/sq. mi.

Step 6: Determine Q_p using Equation 609

$Q_p = q_p A$

$= 1394 \times 0.38$

$= 530$ cfs

Step 7: Determine T_p using Equation 607

$T_p = 60\ t_p + 0.5\ t_u$

$= (60 \times 0.225) + (0.5 \times 5.0)$

$= 16.0$-minutes

Step 8: Determine the width of the unit hydrograph and the portion of that width ahead of Q-sub p at 50 and 75 percent of Q-sub p using Figures-605 and -609

W50% = 0.359 hours (21.0-minutes)

W75% = 0.186 hours (11.2-minutes)

W50% ahead of Q_p = 0.35 x 21

= 7.4-minutes *

W75% ahead of Q_p = 0.45 x 11.2

= 5.0-minutes

*Note: 7.4-minutes is less than 0.6 T-sub p = 9.6-minutes; therefore use 7.4-minutes and 5.0-minutes at W50% and W75% respectively. If the 7.4-minutes were greater than 0.6 T-sub p, the alternate procedure would have been used.

Step 9: Sketch the unit hydrograph (Figure-610) using T-sub p, Q-sub p, W50%, W75%, and the portions of W50% and W75% ahead of Q-sub p determine above. Compute Unit Hydrograph Volume:

(243 acres) x (1-inch/12-inch/foot) = 20.2 ac. ft. Planimeter the unit hydrograph to obtain the volume. Adjust recession leg of the hydrograph unit 20.2 ac. ft. (plus or minus 5%) is obtained. Volume of the first trial was 17.2 ac. ft. which is 15% to low. The unit hydrograph was revised as shown in Figure-610 and a volume of 20.3 ac. ft. was measured which is within the required 5% accuracy.

Step 10: Obtain design effective precipitation values from Table-604 of the section discussing "Effective Rainfall". (Example No. 3, Section 603.4).

Step 11: Set up Table-605.

Step 12: Multiply the precipitation value at the top of Column 3 by each of the unit hydrograph ordinates and put in Column 3 for the corresponding time. Next multiply the precipitation value in Column 4 by each of the unit hydrograph ordinates and place in Column 4 lagged by one time increment from the corresponding unit hydrograph time. Proceed to multiply each of the precipitation values times the unit hydrograph ordinates, each time lagging the new hydrograph by one more time unit.

Step 13: Column 25 is the design storm hydrograph obtained by horizontally summing the individual hydrographs in Column 3 through 24. Note that in this example, time zero is the beginning of excess rainfall and not the beginning of rainfall. This is important when routing and adding several hydrographs from different basins. It may be necessary to adjust the start of each hydrograph so that they are all consistent with the beginning of rainfall.

605 CHANNEL ROUTING OF HYDROGRAPHS

Whenever a large (i.e., greater than 160 acres) or a non-homogeneous basin is being investigated, the basin should be divided into smaller and more homogeneous sub-basins. The storm hydrograph for each sub-basin can then be calculated using the procedures described in Section 604. The user then must route and combine the individual sub-basin hydrographs to develop a storm hydrograph for the entire watershed. There are several methods commonly used in channel routing which include:

a. Direct Translation
b. Convex
c. Muskingum
d. Storage-Discharge (Modified Puls)
e. Kinematic Wave
f. Diffusion Wave
g. Dynamic Wave

Only the Direct Translation and Convex methods will be presented in detail in this section. The last three methods are probably the most accurate of the routing techniques; however, they require a high speed computer to use. The Muskingum method is similar to the Convex method and the Storage-Discharge (Modified Puls) method is somewhat cumbersome for hand calculations.

605.1 Direct Translation

In the direct translation method of routing, the hydrograph is merely shifted in time to account for the travel time along a watershed reach. The shape of the hydrograph is not modified.

When calculating the translation velocities, the appropriate maximum Froude Number for the type of waterway must be used. Table-606 contains the recommended limits for calculating the translation velocity of a hydrograph. The flow used in calculating the translation velocity is the peak flow of the hydrograph. The peak flow is used in conjunction with the drainageway shape, slope, and roughness to calculate the normal depth velocity (i.e., translation velocity).

Example No. 5: Direct Translation Channel Routing

Given: A sample watershed shown in Table-607. Channel reach A-B is 550-feet long, trapezoidal, grass lined channel having a 5-foot bottom, 4:1 side slopes, 0.006 ft/ft longitudinal slope and Manning roughness n = .040.

Channel reach B-C is 3000-feet long, natural waterway, that can be approximately described as trapezoidal having a 20-foot bottom, 3:1 side slopes, 0.008 ft/ft longitudinal slope and Manning roughness n = 0.045.

Table-607 giving the design storm hydrographs for sub-basins 1, 2, and 3 (at design points A and B). The storm hydrograph for the sub-basin tributary to design point C is assumed to be very small and will not affect final results.

Required: Composite storm hydrographs at design points B and C calculated using direct translation method.

Solution:

Step 1: Set up Table-608.

Step 2: Using the procedure for calculating normal depth described in Section 700 "Open Channels", calculate normal depth and velocity for each A-B using the peak flow of the hydrograph. Check against the criteria in Table-606.

$(Qn)/(S^{1/2}b^{8/3}) = (156 \times 0.04)/(.006)^{1/2}(5)^{8/3}) = 1.10$

From Table-701 at Z = 4, y/b = 0.526

$y = 0.526b = (.526 \times 5) = 2.63$ ft

$\text{Area } (A) = (5 \times 2.63) + 4(2.63)^2 = 40.8 \text{ ft}^2$

$V = Q/A = 156/40.8 = 3.82$ fps

$F = V/\sqrt{Ag/T}$; where T - water surface width

$T = (2 \times 4 \times 2.63) + (5) = 26.0$ ft

$F = 3.82/\sqrt{40.8 \times 32.2/26.0} = 0.54 < 0.80$

Use V = 3.82 ft/s since it is less than 6.0 ft/s recommended as a maximum for a grass lined channel and results in a Froude Number less than 0.8.

Step 3: Find the time of translation using the velocity calculated in Step 2.

$t_{A-B} = L/(60V) = 5500/(60 \times 3.82) = 24.0$-minutes

Round off to the nearest time increment and use t_{A-B} = 25-minutes.

Step 4: Enter the hydrograph at A into Column 2 and the hydrograph at A, lagged by t_{A-B} = 25-minutes in Column 3 of Table-608.

Step 5: Enter in Column 4 the sum of local hydrographs (i.e., basin 2 at B and basin 3 at B) for reach A-B.

Step 6: Add to the hydrograph in Column 3 the total of local hydrographs (Column 4). Enter results in Column 5.

Step 7: Repeat Step 2 for reach B-C using criteria for natural waterway.

$(Qn/S^{1/2}b^{8/3}) = (272 \times 0.045)/(.008)^{1/2}(20)^{8/3}) = .0464$

From Table-701 at Z = 3,

y/b = .112

$Y = (.112 \times 20) = 2.24$

$\text{Area } (A) = (20 \times 2.24) + 3(2.24)^2 = 59.8 \text{ ft}^2$

$V = Q/A = 272/59.8 = 4.54$ ft/s

$T = (2 \times 3 \times 2.24) + 20 = 33.4$ ft

$F = 4.54/\sqrt{59 \times 32.3/33.4} = 0.60 < 0.95$

Use V = 4.5 fps since it is less than the recommended maximum of 8.0 ft/s and has a Froude Number less than 0.95.

Step 8: Repeat Step 3 for reach B-C.

t_{B-C} = 3000/(4.58 x 60) = 10.9-minutes

Round off to nearest time increments and use t_{B-C} = 10-minutes

Step 9: Repeat Step 4 for reach B-C

Step 10: Repeat Step 5 for reach B-C.

605.2 Convex Routing Method

The convex routing method provides for some of the effects of channel storage and, as a result, the storm hydrograph shape is modified in translation along a channel reach. Similar velocity limiting considerations are taken into account as discussed in Section 605.1. The recommended limits for calculating the translation velocity for the convex method are given in Table-606.

Equation 614 is the basic equation for the convex method as described by Soil Conservation Service (Reference-33).

$$O_2 = (1-C)O_1 + CI_1 \tag{614}$$

In which O_2 = Outflow from the reach at the end of the unit time increment (beginning of the next time increment).

O_1 = Outflow from the reach at the beginning of the time increment.

I_1 = Inflow into the reach at the beginning of the time increment.

C = A coefficient found using Equation 615

$$C = 1 - (1-C_1)^{\Delta t/B} \tag{615}$$

In which $C_1 = V/(V + 1.7)$

K = L/(3600V)

$B = KC_1$

L = Channel reach length in feet.

V = Translation velocity in fps

t = Unit time increment in hours; has to be less than or equal to one-fifth of T_R.

T_R = Time of rise of the storm hydrograph defined as the time from beginning to peak flow.

The use of the Convex method is illustrated through the following example:

Example No. 6: **Convex Routing Channel Routing**

Given: Same watershed and storm hydrographs given in Example No. 5.

Required: Composite storm hydrographs at design points B and C calculated using the Convex Routing method.

Solution:

Step 1: Determine flow to be used to calculate translation velocity V for reach A-B. Enter on Table-609 the values for Q_p and $3/4\ Q_p$:

$Q_p = 156\ ft^3/s$

$3/4\ Q_p = 3/4(156) = 117\ ft^3/s$

Step 2: Using the procedure outlined in Section 700 "Open Channel", calculate normal depth and velocity at $3/4Q_p$.

$(Qn)/(S^{1/2}b^{8/3}) = (117 \times .04)/(.006)^{1/2}(5)^{8/3}) = 0.82$

From Table-701 of "Major Drainage" at Z = 4;

$y/b = 0.461$

$y = 0.461b = (0.461 \times 5) = 2.31\ ft$

$A = (5)\ (2.31) + 4(2.31)^2 = 32.8\ ft^2$

$V = Q/A = 117/32.8 = 3.57\ ft/s$

$T = (2 \times 4 \times 2.31) + 5 = 23.5\ ft$

$F = V / \sqrt{Ag/T} = 3.57 / \sqrt{(32.8)(32.2) / (23.5)} = 0.53$

Use V = 3.57 ft/s since it results in a Froude Number of less than 0.8.

Step 3: Calculate C using Equation 615 and enter the resultant values in Table-609 for reach A-B.

$C_1 = V/(V + 1.7) = 3.57/(3.57 + 1.7) = 0.68$

$K = L/(3600V) = 5500/(3600 \times 3.57) = 0.43$

$B = KC_1 = (.43 \times .68) = .29$

$t = 5\text{-minutes}/60 = 0.083\ hours$

$C = 1-(1-C_1)^{\Delta t/B} = 0.28$

Step 4: Set up Equation 614 and enter coefficients in Table-609

$$O_2 = (1.-0.28)O_1 + (0.28)I_1 = (0.72)O_1 + (0.28)I_1$$

Step 5: Route the storm hydrograph from sub-basin 1 (point A) through reach A-B using Equation in Step 4:

a) Set up Table-609 by filling in the inflow Column 2.

b) Start with zero inflow at t = 15, namely $I_1 = 0.$

c) The starting routed outflow at t = 15 is also zero, namely $O_1 = 0.$

d) Calculation $O_2 = (0.72 \times 0.) + (0.28 \times 0.) = 0.$

e) Enter 0. in the routed outflow Column 3 for t = 20.

f) Calculate O_2; $O_2 = (0.72 \times 0.) + (0.28 \times 2) = 0.5.$

g) Enter 0.5 in the routed outflow Column 3 at t = 25.

h) Calculate O_2; $O_2 = (.72 \times .54) + (.28 \times 10) = 3.2.$

i) Enter 3.2 in the routed outflow Column 3 at t = 30.

j) Repeat the procedure until the entire hydrograph has been routed.

Step 6: Enter the sum of the local hydrographs from Basin 1 and 2 as Local Inflow for Reach A-B (Column 4).

Step 7: Add the Routed Flow (Column 3) and the Local Inflow (Column 4) for Reach A-B, round off to nearest three significant figures or a whole number and enter in the Total Outflow (Column 5) for Reach A-B. This becomes then the inflow hydrograph for Reach B-C.

Step 8: Repeat Step 1 for Reach B-C.

$Q_p = 295\ ft^3/s$

$3/4\ Q_p = 221\ ft^3/s$

Step 9: Repeat Step 2 for Reach B-C.

$(Qn)/(S^{1/2}b^{8/3}) = (221 \times 0.045)/(.008)^{1/2}(20)^{8/3}) = .0377$

$y/b = 0.10$

$y = (0.10 \times 20) = 2.0\ ft$

$A = (20 \times 2.0) + 3(2.0)^2 = 52\ ft^2$

$V = Q/A = 221/52 = 4.25\ ft/s$

$T = (2 \times 3 \times 2.0) + 20 = 32\ ft$

$F = V/\ \ A g/T = 4.25/\ \ (52)(32.2)/32 = 0.59$

Use V = 4.25 ft/s since it results in the natural channel Froude Number of less than 0.95.

Step 10: Repeat Step 3 for Reach B-C

$C_1 = V/(V + 1.7) = 4.25/5.95 = 0.71$

$K = L/(3600V) = 3000/(3600 \times 4.25) = 0.20$

$B = KC = (.2 \times .71) = 0.142$

$C = 1 - (1-C_1)^{\Delta t/B} = 1 - (1-.71)^{.083/.142} = 0.52$

Step 11: Set up Equation 4-7 and enter coefficients in Table 4-C for Reach B-C.

$O_2 = (1-.52)O_1 + (.52)I_1 = (0.48)O_1 + (.52)I_1$

Step 12: Repeat Step 5 for Reach B-C. Use Outflow from Reach A-B as Inflow into Reach B-C.

The routed outflow is the hydrograph at point C on the Watercourse.

605.3 Comparison of Routing Methods

Examining Tables-608 and -609 reveals the following data at point C:

Method	Peak Flow (cfs)	Time to Peak (minutes)	Volume (ac ft)
Direct	272	75	28.2
Convex	285	60	28.2

For the drainageways and hydrographs used in Examples 5 and 6 the Direct Translation Routing gave a 5 percent lesser peak flow at point C than the Convex Routing method. The time to peak was greater for the Direct Translation Routing example. The volumes were identical for both methods and indicate that the law of continuity applies to both. Although the peaks are not identical, they are within the accuracy of these type of calculations. However, under a different set of hydrographs and channel conditions, these two routing methods may have given different relative results and the resultant peak may have been more than 5 percent apart.

Routing calculations involve consideration judgement and are sensitive to hydrograph shapes and waterway conditions. Different routing methods will not always give identical results. However, only one routing method shall be used when analyzing a watershed. Routing techniques shall not be changed from reach to reach.

606 RESERVOIR ROUTING OF HYDROGRAPHS

A reservior is basically an enlargement of a river or drainage channel and storage in reservoirs may modify the shape of the incoming flood hydrograph or flood wave. If the reservoir does not have gates, the discharge (D) takes place over an uncontrolled weir or through an uncontrolled orifice in such a way that D is a function of the reservoir level.

Storm runoff detention is required for all new development (POLICY Section 303.5) and therefore detention reservoirs will be required (see Section 1200 "Detention"). In some instances, the sizing of the detention storage will be based upon hydrograph storage routing techniques rather than direct calculation of volume and discharge requirements. The methodology for manual computation of reservoir routing is presented in this section.

606.1 Modified Puls Method

The procedure for the original Puls Method was developed in 1928 by L.G. Puls of the U.S. Army Corps of Engineers. The method was modified in 1949 by the Bureau of Reclamation simplifying the computational and graphic requirements. The method is also referred to as the Storage-Indication or Goodrich Reservoir Routing Method. The differences, if any, are mainly in the form of the equation and means of initializing the routing. The procedures presented herein was obtained from References-8, -30, -33, -34, and -35.

The principle of mass continuity for a channel reach can be expressed by the equation:

$$(I-D)t = \Delta S$$

where I is the inflow rate, D is the discharge rate, t is the time interval, and ΔS is the change in storage. If the average rate of flow during a given time period is equal to the average of the flows at the beginning and end of the period, the equation can be expressed as follows:

$$(I_1 + I_2)\ t/2 - (D_1 + D_2)\ t/2 = S_2 - S_1$$

where the subscripts 1 and 2 refer to the beginning and end of period t. Rearranging the equation gives the form used for the Modified Puls Method (Equation 616).

$$I_1 + I_2 + (2S_1/t - D_1) = (2S_2/t + D_2) \tag{616}$$

The use of the equation is illustrated by the following example:

606.2 Example Calculation

Example No. 7: Reservoir Storage Routing

Given: Reservoir discharge - storage relationship (Table-610A)
Inflow hydrograph (Table-611)

Find: Reservoir outflow hydrograph

Solution:

Step 1: Select computational time interval. Value should be less than 40 percent of time to peak of hydrograph. An interval of t = 5-minutes was selected for this example.

Step 2: Convert storage values to discharge units. Since 5-minutes was the selected time interval then the discharge units are cfs minutes and the conversion for acre-feet is:

$1 \text{ AF} \times 43560 \text{ ft}^3/\text{AF} = \text{CFS-MIN} \times 60\text{s/min}$

or $\text{CFS-MIN} = 726 \times \text{AF}$

The storage values were converted and are presented in Column 4 Table-610A.

Step 3: Compute $2S/t + D$ values (see Table-610B) for t = 5-minutes.

For a discharge of 30 cfs then:

$2S/t - D = 2(13{,}070)/5 - 30 = 5200$ cfs

Step 4: Plot $2S/t \pm D$ values (Table-610B versus discharge). See Table-610C for example.

Step 5: Set up Table-611. The inflow hydrograph is placed in Column 3. The initial values are determined from Equation 616 as follows:

$I_1 = 0$

$I_2 = 13$

$D_1 = 0$

$S_1 = 5663$ (linearly interpolated from Table-610A for $I_2 = 13$ cfs

$2S_1/t = 2(5663)/5 = 2265$

Inserting these values into Equation 616 results in the following:

$0 + (13) + (2265) - (0) = (2S_2/t + D_2)$

or $(2S_2/t + D_2) = 2278$

Place this value in Column 5 at time interval 2.

Step 6: Using the value 2278 read the corresponding values for $2S_2/t - D_2 =$ 2278 and D-sub 2 = 2 from the curve in Table-610C. Enter the values in Column 4 row 2 and Column 6 row 2.

Step 7: Compute I-sub 1 + I-sub 2 + (2S/t - D-sub 1) using Columns 3 and 4. For example, I-sub 1 = 13, I-sub 2 = 52 and (2S/t - D-sub 1) = 2278 for a total of 2343. Enter this value in Column 5 row 3 since this value becomes the right hand portion of Equation 616.

Step 8: Using the value (2343) computed in Step 7, enter the figure in Table-610C and read the corresponding values for (2S-sub 2/t - D-sub 2) = 2343 and D-sub 2 = 3. Enter these values in Table-611.

Step 9: Repeat Steps 7 and 8 until the outflow hydrograph is determined.

Step 10: Using the maximum discharge value D = 333 cfs from Column 6 Table-611 and the corresponding value 2S/t + D = 11,888, determine the maximum storage by solving the equation:

$$(2S/5 + 333) = 11,888$$

$$S = 28,887.5 \text{ cfs-min}$$

or

$$S = 39.8 \text{ AF}$$

607 STATISTICAL AND REGIONAL ANALYSIS

For basins larger than 10 square miles, the preferred method to compute flood flows is to use actual records of discharges which have been recorded by gaged streams. The reliability of the statistical or regional approach is generally better than the Rational method, CUHP method, or other deterministic model, provided the period of record is sufficiently long (i.e., 20 years or greater).

Before proceeding with a statistical analysis, the analysis shall contact the Colorado Water Conservation Board, Boulder County, and/or the City of Longmont to obtain applicable data and criteria for evaluation.

607.1 Statistical Analysis

In urban hydrology, the preferred statistical approach is limited (1) by the almost total lack of adequate runoff records in urban areas, (2) by the effects of rapid urbanization, and (3) study areas having satisfactory gaging periods usually have records which represent undeveloped basins. Once urbanization occurs, the records representing natural conditions no longer apply to future conditions. Thus, use of the Rational method and the CUHP will generally be required for urban or urbanizing areas.

The statistical analysis has the greatest applicability to natural streams where the basins will remain in a natural state. Such streams include those with large basins where the urbanization effect on runoff will be negligible, and on small streams where the basin primarily consists of undevelopable land or land comprising green belt areas. Many of the streams in Boulder County falling in this category have already been evaluated and published in various reports by State and Federal agencies (see Section 200). These reports are to be utilized for flood information in the respective stream reaches.

In the statistical approach to determining the size of flood peaks, the logic involved is that nature over a period of years has defined a flood magnitude-frequency relationship that can be derived by study of actual occurrences. A period of record of a particular basin where the floods have been measured and recorded is considered to be a representative period. Floods that occurred during the period can be assumed to occur in a similar future period, that is, the period may be expected to repeat itself.

The purpose of statistical analysis is to use the recorded runoff events for a given period of record as a means of extrapolating to a longer period of time. For a 25 year period, the largest record flood is generally considered to have a recurrence interval of about 25 years. At the end of this 25 year period, because the period can be assumed to repeat itself, one could expect the largest flood of record to be equaled or exceeded once more during the next 25 years. For any given year the probability of a flood of any given frequency happening in that year is the same as the probability of it happening in any other year. Thus the 100-year flood has a 1 percent chance of being equaled or exceeded in any given year.

The statistical procedure acceptable for use in Boulder County is the one described by the U.S. Water Resources Council (Reference-36) that utilizes the Log Pearson Type III distribution. Any independent statistical analysis of records in Boulder County should allow the procedure outlined by the Water Resources Council.

607.2 Regional Analysis

The detailed evaluation of a gaged drainage basin can provide an indication of the flood frequency data for a similar but ungaged drainage basin within the same region. The concept is based on multiple regression analysis of the basins with geometric similarity such as area, shape, slope, and topography; hydrologic similarity such as rainfall, snowfall, soils, and valley storage, and geologic similarity with regard to those items which effect groundwater flow. Once these relationships have been developed for the gaged basin, they are transposed to the nearby ungaged watershed.

A regional analysis has been performed for the St. Vrain Creek basin of 92 square miles tributary to Lyons (Reference-67) and methodology developed to determine flood peaks. In accordance with the POLICY Section 304.4, the flood peaks in Zones II, III, and IV for basins greater than 10 square miles shall be determined using the following procedures.

The floods in the Colorado mountains are determined in two independent events, rain and snowmelt. The Log Pearson Type III distribution was used to analyze both type of events. Two basin parameters, drainage area and mean watershed elevation, were found to be the dominant variables in the regression analysis for the determination of flood peaks. Equations were developed for the mean, coefficient of variation (Standard Deviation) and skew for each type of event. These two frequency curves, one for rain and one for snowmelt, are then statistically combined to give a composite frequency curve.

The magnitude of flood flow at any exceedance probability or recurrence interval may be computed by the equation:

$$\log Q_t = m + K_T(S)\ (SIGMA) \qquad (617)$$

for each type of event, where

Q_T = flood flow with recurrence interval of T years, in cfs

m = mean of the logarithms of annual floods

SIGMA = skew of the logarithms of annual floods

$K_T(S)$ = frequency factor, which is a function of skew, for recurrence interval T and the composite probability for the flood event which equals or exceeds Q_T is

$$P(R\ S) = P(R) + P(S) - P(R\ S)$$

where

$$P(R) = P(Q_R \geq Q_T), \text{ the exceedance probability of rain event}$$

$$P(S) = P(Q_S \geq Q_T), \text{ the exceedance probability of snowmelt}$$

This equation becomes

$$P(R\ S) = p(R) + P\ (S) - P(R)P(S) \qquad (618)$$

if rain event and snowmelt event are statistically independent.

Final regression equations or regional relationships were developed using the gaged data for St. Vrain Creek and are as follows:

$$m_R = 0.703 \log A + 1.184 \qquad (619)$$
$$r = 0.586$$
$$se = 15\%$$

$$CV_R = SIGMA\ _R/m_R = 0.1399 \qquad (620)$$

$$se = 29\%$$

$$S_R = 0.36 \qquad (621)$$
$$se = 158\%$$

$$m_S = 1.042 \log A + 7.437 \log E - 6.653 \qquad (622)$$
$$r = 0.741$$
$$se = 8\%$$

$$CV_S = SIGMA_S/m = 0.0709 \qquad (623)$$

$$se = 29\%$$

$$S_s = -0.40 \qquad (624)$$
$$se = 90\%$$

where the subscripts R and S refer to rain and snowmelt events, respectively, and

A = drainage area, in square miles

E = mean watershed elevation, in 1,000-feet

CV = coefficient of variation

r = correlation coefficient

se = standard error of estimate

For any ungaged site in the region with given drainage area and mean watershed elevation, Equations 619 to 624 may be used to compute the statistical parameters which define the flood frequency curve for each type of event by Equation 617. These two frequency curves are then statistically combined using Equation 618 to give the final frequency curve for the ungaged site being considered.

608 COMPUTER PROGRAMS

Computer programs have been developed (1) to calculate the hydrographs using the CUHP method, (2) to calculate the channel routed hydrographs similar to the techniques presented in Section 605, and (3) to calculate the reservoir routed hydrographs using the technique presented in Section 606. The CUHP version D program can be obtained by contacting the UD&FCD at 2480 West 26th Avenue, Suite 156-B, Denver, Colorado 80211 (303) 455-6277. The channel and reservoir routing techniques have been programmed in the U.S. Army Corps of Engineers "HEC-1, Flood Hydrograph Package". Copies of the program can be obtained by contacting the Hydrologic Engineering Center, Corps of Engineers, U.S. Army at 609 Second Street, Davis, California 95616 (916)440-3285.

The recent (February, 1983) version of CUHP D program has been revised to write storm hydrographs to an output file for subsequent use with Multi-Plan River Routing Routine of HEC-1. Both programs are compatable with the FTN compiler at Boeing Computer Services Center, Englewood, Colorado. Modifications may be required for other computer compilers.

RUNOFF COEFFICIENTS AND PERCENT IMPERVIOUS

LAND USE OR SURFACE CHARACTERISTICS	PERCENT IMPERVIOUS	RUNOFF COEFFICIENTS FREQUENCY 2	5	10	100
Business:					
Commercial Areas	95	.87	.87	.88	.89
Neighborhood Areas	70	.60	.65	.70	.80
Residential:					
Single-Family	Figure-603	.40	.45	.50	.60
Multi-Unit (detached)	50	.45	.50	.60	.70
Multi-Unit (attached)	70	.60	.65	.70	.80
1/2 Acre Lot or Larger	Figure-603	.30	.35	.40	.60
Apartments	70	.65	.70	.70	.80
Industrial:					
Light Areas	80	.71	.72	.76	.82
Heavy Areas	90	.80	.80	.85	.90
Parks, Cemetaries	7	.10	.10	.35	.60
Playgrounds	13	.15	.25	.35	.65
Schools	50	.45	.50	.60	.70
Railroad Yard Areas	40	.40	.45	.50	.60
Undeveloped Areas:					
Historic Flow Analysis Greenbelts, Agricultural	2	(see "Lawns")			
Offsite Flow Analysis (when land use not defined)	45	.43	.47	.55	.65
Streets:					
Paved	100	.87	.88	.90	.93
Gravel	13	.15	.25	.35	.65
Drives and Walks	96	.87	.87	.88	.89
Roofs	90	.80	.85	.90	.90
Lawns, Sandy Soil	0	.00	.01	.05	.20
Lawns, Clayey Soil	0	.05	.10	.20	.40

NOTE: The Rational Formula coefficients do not apply for larger basins where the time-of-concentration exceeds 60 minutes.

EFFECTIVE RAINFALL PARAMETERS

DEPRESSION AND RETENTION LOSSES

LAND COVER	DEPRESSION & RETENTION (INCHES)	RECOMMENDED (INCHES)
Impervious:		
Large Paved Areas	0.05 - 0.15	0.1
Roofs - Flat	0.1 - 0.3	0.1
Roofs - Sloped	0.05 - 0.1	0.05
Pervious:		
Lawn Grass	0.2 - 0.5	0.5
Wooded Area and Open Fields	0.2 - 0.6	0.4

NOTE: Values are valid for CUHP method only

INFILTRATION PARAMETERS FOR HORTON'S EQUATION

SCS HYDROLOGIC SOIL GROUP *	INFILTRATION (IN/HR) INITIAL (f_i)	INFILTRATION (IN/HR) FINAL (fo)	DECAY COEFFICIENT (a)
A	5.0	1.0	0.0007
B	4.5	0.6	0.0018
C	3.0	0.5	0.0018
D	3.0	0.5	0.0018

* SCS - Soil Conservation Service

INCREMENTAL INFILTRATION DEPTHS

TIME MINUTES**	DEPTH (INCHES*) SCS HYDROLOGIC SOIL GROUP A	B	C & D
5	.384	.298	.201
10	.329	.195	.134
15	.284	.134	.096
20	.248	.099	.073
25	.218	.079	.060
30	.194	.067	.052
35	.175	.060	.048
40	.159	.056	.045
45	.146	.053	.044
50	.136	.052	.043
55	.127	.051	.042
60	.121	.051	.042
65	.115	.050	.042
70	.111	.050	.042
75	.107	.050	.042
80	.104	.050	.042
85	.102	.050	.042
90	.100	.050	.042
95	.098	.050	.042
100	.097	.050	.042
105	.096	.050	.042
110	.095	.050	.042
115	.095	.050	.042
120	.094	.050	.042

* Based on central value of each time increment in Hortons Equation.

** Time at end of the time increment

STANDARD FORM SF-2

DETERMINATION OF EFFECTIVE RAINFALL

LOCATION: SECTION 10 , **TOWNSHIP** 2N , **RANGE** 69W

DESIGN STORM: 10 YR Recurrence ZONE I

EXAMPLE 3

		PERVIOUS AREA 56 %				IMPERVIOUS AREA 44 %				
Time (min.) 1	Incremental Precipitation (in.) 2	Maximum Infiltration (in.) 3	Depression Storage (in.) 4	Effective Precipitation (in.) 5	56 % Effective Precipitation (in.) 6	Depression Storage (in.) 7	Loss (in.) 8	Effective Precipitation (in.) 9	44 % Effective Precipitation (in.) 10	Total Effective Precipitation (in.) 11
5	0.03	0.20	0	0	0	0.03	0	0	0	0
10	0.06	0.13	0	0	0	0.06	0	0	0	0
15	0.14	0.10	0.04	0	0	0.01	0	0.13	0.06	0.06
20	0.26	0.07	0.19	0	0	0	0.01	0.25	0.11	0.11
25	0.43	0.06	0.07	0.30	0.17	0	0.02	0.41	0.18	0.35
30	0.21	0.05	0	0.16	0.09	0	0.01	0.20	0.09	0.18
35	0.10	0.05	0	0.05	0.03	0	0.01	0.09	0.04	0.07
40	0.07	0.04	0	0.03	0.02	0	0	0.07	0.03	0.05
45	0.07	0.04	0	0.03	0.02	0	0	0.07	0.03	0.05
50	0.06	0.04	0	0.02	0.01	0	0	0.06	0.03	0.04
55	0.06	0.04	0	0.02	0.01	0	0	0.06	0.03	0.04
60	0.06	0.04	0	0.02	0.01	0	0	0.06	0.03	0.04
65	0.06	0.04	0	0.02	0.01	0	0	0.06	0.03	0.04
70	0.06	0.04	0	0.02	0.01	0	0	0.06	0.03	0.04
75	0.06	0.04	0	0.02	0.01	0	0	0.06	0.03	0.04
80	0.04	0.04	0	0	0	0	0	0.04	0.02	0.02
85	0.03	0.04	0	0	0	0	0	0.03	0.01	0.01
90	0.03	0.04	0	0	0	0	0	0.03	0.01	0.01
95	0.03	0.04	0	0	0	0	0	0.03	0.01	0.01
100	0.03	0.04	0	0	0	0	0	0.03	0.01	0.01
105	0.03	0.04	0	0	0	0	0	0.03	0.01	0.01
110	0.03	0.04	0	0	0	0	0	0.03	0.01	0.01
115	0.03	0.04	0	0	0	0	0	0.03	0.01	0.01
120	0.02	0.04	0	0	0	0	0	0.02	0.01	0.01
TOTAL	2.00	—	0.30	0.69	0.39	0.10	0.05	1.85	0.82	1.21

TABLE 605

STANDARD FORM SF-3
DETERMINATION OF STORM HYDROGRAPH
EXAMPLE 4

Time (min)	Unit Hydrograph (cfs)	Effective Precipitation in Inches																					Storm Hydrograph (cfs)
		.06	.11	.35	.18	.07	.05	.05	.04	.04	.04	.04	.04	.04	.02	.01	.01	.01	.01	01	.01	.01	.01	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
5	115	7																						7
10	345	21	13																					34
15	528	32	38	40																				110
20	463	28	58	121	21																			228
25	350	21	51	185	62	8																		327
30	260	16	39	162	95	24	6																	342
35	210	13	29	123	83	37	17	6																308
40	168	10	23	91	63	32	26	17	5															267
45	138	8	18	74	47	25	23	26	14	5														240
50	110	7	15	59	38	18	18	23	21	14	5													218
55	88	5	12	48	30	15	13	18	18	21	14	5												199
60	70	4	10	39	25	12	10	13	14	18	21	14	5											185
65	55	3	8	31	20	10	8	10	10	14	18	21	14	5										172
70	40	2	6	25	16	8	7	8	8	10	14	18	21	14	2									159
75	30	2	4	19	13	6	6	7	7	8	10	14	18	21	7	1								143
80	20	1	3	14	10	5	4	6	6	7	8	10	14	18	11	3	1							121
85	15	1	2	11	7	4	4	4	4	6	7	8	10	14	9	5	3	1						100
90	8	0	2	7	5	3	3	4	4	4	6	7	8	10	7	5	5	3	1					84
95	2	0	1	5	4	2	2	3	3	4	4	6	7	8	5	4	5	5	3	1				72
100	0	0	0	3	3	1	2	2	2	3	4	4	6	7	4	3	4	5	5	3	1			62
105			0	1	1	1	1	2	2	2	3	4	4	6	3	2	3	4	5	5	3	1		53
110				0	0	1	1	1	1	2	2	3	4	4	3	2	2	3	4	5	5	3	1	47
115					0	0	0	1	1	1	2	2	3	4	2	1	2	2	3	4	5	5	3	41
120						0	0	0	1	1	1	2	2	3	2	1	1	2	2	3	4	5	5	35
125							0	0	0	1	1	1	2	2	1	1	1	1	2	2	3	4	5	27
130								0	0	0	1	1	1	2	1	1	1	1	1	2	2	3	4	21
135									0	0	0	1	1	1	1	1	1	1	1	1	2	2	3	16
140										0	0	0	1	1	1	0	1	1	1	1	1	2	2	12
145											0	0	0	1	0	0	0	1	1	1	1	1	2	8
150												0	0	0	0	0	0	0	1	1	1	1	1	5
155													0	0	0	0	0	0	0	1	1	1	1	4
160														0	0	0	0	0	0	0	1	1	1	3
165															0	0	0	0	0	0	0	1	1	2
170																0	0	0	0	0	0	0	1	1
175																	0	0	0	0	0	0	0	0
																		0	0	0	0	0	0	0
																			0	0	0	0	0	0
																				0	0	0	0	0
																					0	0	0	0
																						0	0	0
																							0	0

UDFCD

NOVEMBER 1983

CHANNEL ROUTING PARAMETERS

RECOMMENDED LIMITS FOR CALCULATING
DIRECT TRANSLATION VELOCITY

TYPE OF CHANNEL	*MAXIMUM VELOCITY	*MAXIMUM FROUDE NO.
Natural Waterway	8.0 fps	0.95
Man Made:		
Grass Lined	6.0 fps	0.8 **
Riprap Lined	8.0 fps	0.8 **
Concrete Lined	12.0 fps	No Limit

* Use whichever results in lower velocity.
** Applies only to channels built using the velocity limitations, presented in this MANAUL otherwise use F $\leq$ 0.95.

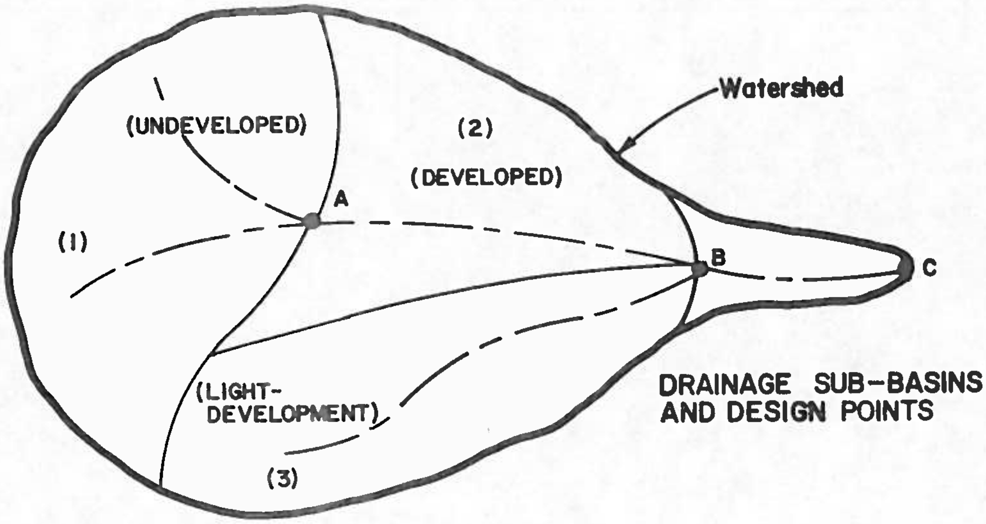
RECOMMENDED LIMITS FOR CALCULATING TRANSLATION
VELOCITY USING CONVEX ROUTING METHOD

TYPE OF CHANNEL	*MAXIMUM FLOW	*MAXIMUM FROUDE NO.
Natural Waterway	3/4 Q peak	0.95
Man Made:		
Grass Lined	3/4 Q peak	0.80 **
Riprap Lined	3/4 Q peak	0.80 **
Concrete Lined	3/4 Q peak	No Limit

* Use whichever results in lower velocity.
** Applies only to channels built using the velocity limits presented in this MANUAL, otherwise use F $\leq$ 0.95.

REFERENCE: USDCM DRCOG Rev. May 1, 1984

DATA FOR CHANNEL ROUTING EXAMPLES



SAMPLE SUB-BASIN HYDROGRAPHS
FLOW BY SUB-BASIN IN FT^3/SEC.

TIME (MIN)	BASIN 1 AT A	BASIN 2 AT B	BASIN 3 AT B
10	0	0	0
15	0	2	0
20	2	12	1
25	10	48	8
30	30	108	20
35	90	157	70
40	120	169	75
45	140	155	80
50	150	136	78
55	156	121	70
60	150	108	60
65	145	97	55
70	138	86	40
75	130	70	28
80	115	60	20
85	100	50	14
90	88	42	8
95	78	33	5
100	68	24	3
105	58	16	1
110	45	9	0
115	40	5	0
120	32	4	0
125	24	2	0
130	16	1	0
135	9	0	0
140	4	0	0
145	3	0	0
150	0	0	0

STANDARD FORM SF-4

TABLE 608

DIRECT TRANSLATION CHANNEL ROUTING

EXAMPLE 5

JOB NAME & LOCATION: Direct Transition Routing

Job No. ______

By ______

Date ______

Chkd By ______

Date Chkd ______

Storm Recurrence: 5 -yrs. Unit Time Δt= 5.0 min. (0.083 hr.)

Reach	Hydrograph Pt. Inflow	Hydrograph Pt. Outflow	Q_p (cfs)	V (fps)	F	L (ft.)	$t = \frac{L}{60V}$ (min)
A-B	A	B	156	3.82	0.54	5500	24.0 : Use 25
B-C	B	C	272	4.54	0.60	3000	10.9 : Use 10
							: Use ______

Time (min.) (1)	Inflow (cfs) (2)	Reach ______ Routed Outflow (cfs) (3)	Local Inflow (cfs) (4)	Tot. Outflow New Inflow (cfs) (5)	Reach ______ Routed Outflow (cfs) (6)	Local Inflow (cfs) (7)	Tot. Outflow New Inflow (cfs) (8)	Reach ______ Routed Outflow (cfs) (9)	Local Inflow (cfs) (10)	Tot. Outflow New Inflow (cfs) (11)
5	0	0	0	0	0	NONE	Same As Routed Outflow- Reach B-C			
10	0	0	0	0	0					
15	0	0	2	2	0					
20	2	0	13	13	0					
25	10	0	56	56	2					
30	30	0	128	128	13					
35	90	0	227	227	56					
40	120	0	244	244	128					
45	140	2	235	237	227					
50	150	10	214	224	244					
55	156	30	191	221	237					
60	150	90	168	258	224					
65	145	120	152	272	221					
70	138	140	126	266	258					
75	130	150	98	248	272					
80	115	156	80	236	266					
85	100	150	64	214	248					
90	88	145	50	195	236					
95	78	138	38	176	214					
100	68	130	27	157	195					
105	58	115	17	132	176					
110	48	100	10	110	157					
115	40	88	5	93	132					
120	32	78	4	82	110					
125	24	68	2	70	93					
130	16	58	1	59	82					
135	9	48	0	48	70					
140	4	40	0	40	79					
145	3	32	0	32	48					
150	0	24	0	24	40					
155		16	0	16	32					
160		9	0	9	24					
165		4	0	4	16					
170		3	0	3	9					
175		0	0	0	4					
180					3					
185					0					

STANDARD FORM SF-5

CONVEX CHANNEL ROUTING

TABLE 609

EXAMPLE 6

JOB NAME & LOCATION: Convex Channel Routing

Storm Recurrence: 100 -yrs. Unit Time Δt= 5 min. (0.083 hr.)

Job No. ______

By ______

Date ______

Chkd By ______

Date Chkd ______

Reach	Hydrograph Pt. Inflow	Hydrograph Pt. Outflow	Q_p (cfs)	$\frac{3}{4}Q_p$ (cfs)	C_1 $\frac{V}{V+1.7}$	K $\frac{L}{3600V}$	B KC_1	C $1-(1-C_1)^{\Delta t/B}$	Routing equation
									$O_2=(1-C)O_1+(C)I_1$
A-B	A	B	156	117	0.68	0.43	0.29	0.28	$O_2=(0.72)O_1+(0.28)I_1$
B-C	B	C	295	221	0.71	0.20	0.142	0.52	$O_2=(0.48)O_1+(0.52)I_1$
									$O_2=(\ \ )O_1+(\ \ )I_1$

Time (min.) (1)	Inflow (cfs) (2)	Reach A-B Routed Outflow (cfs) (3)	Local Inflow (cfs) (4)	Tot. Outflow New Inflow (cfs) (5)	Reach B-C Routed Outflow (cfs) (6)	Local Inflow (cfs) (7)	Tot. Outflow New Inflow (cfs) (8)	Reach ____ Routed Outflow (cfs) (9)	Local Inflow (cfs) (10)	Tot. Outflow New Inflow (cfs) (11)
5	0	0	0	0	0		0			
10	0	0	0	0	0		0			
15	0	0	2	2	0		0			
20	2	0	13	13	1		1			
25	10	0.5	56	57	7.3		7			
30	30	3.2	128	131	33.1		33			
35	90	10.7	227	238	84.0		84			
40	120	32.9	244	277	164.1		164			
45	140	57.3	235	292	222.8		223			
50	150	80.5	214	295	258.8		259			
55	156	99.9	191	291	277.6		278			
60	150	115.6	168	284	284.6		285			
65	145	125.2	152	277	284.3		284			
70	138	130.8	126	257	280.5		280			
75	130	132.8	98	231	268.3		268			
80	115	132.0	80	212	248.9		249			
85	100	127.2	64	191	229.7		230			
90	88	119.6	50	170	209.6		210			
95	78	110.8	38	149	189.0		189			
100	68	101.6	27	129	168.2		168			
105	58	92.2	17	109	147.8		178			
110	48	82.6	10	93	127.6	NONE	128			
115	40	72.9	5	78	109.6		110			
120	32	63.7	4	68	93.2		93			
125	24	54.8	2	57	80.1		80			
130	16	46.2	1	47	68.1		68			
135	9	37.7	0	38	57.1		57			
140	4	29.7	0	30	47.2		47			
145	3	22.5	0	22	38.2		38			
150	0	17.0	0	17	29.8		30			
155		12.3		12	23.1		23			
160		8.8		9	17.4		17			
165		6.4		6	13.0		13			
170		4.6		5	9.4		9			
175		3.3		3	7.1		7			
180		2.4		2	5.0		5			
185		1.7		2	3.4		3			
190		1.2		1	2.7		3			
195		.9		1	1.8		2			
200		.6		1	1.4		1			
205		.5		1	1.2		1			
210		.3		0	1.1		1			
215		.2		0	.5		1			
220		.2		0	.2		0			
225		.1		0	.1		0			

DATA FOR RESERVOIR ROUTING EXAMPLE

A. STORAGE-DISCHARGE RELATIONSHIP

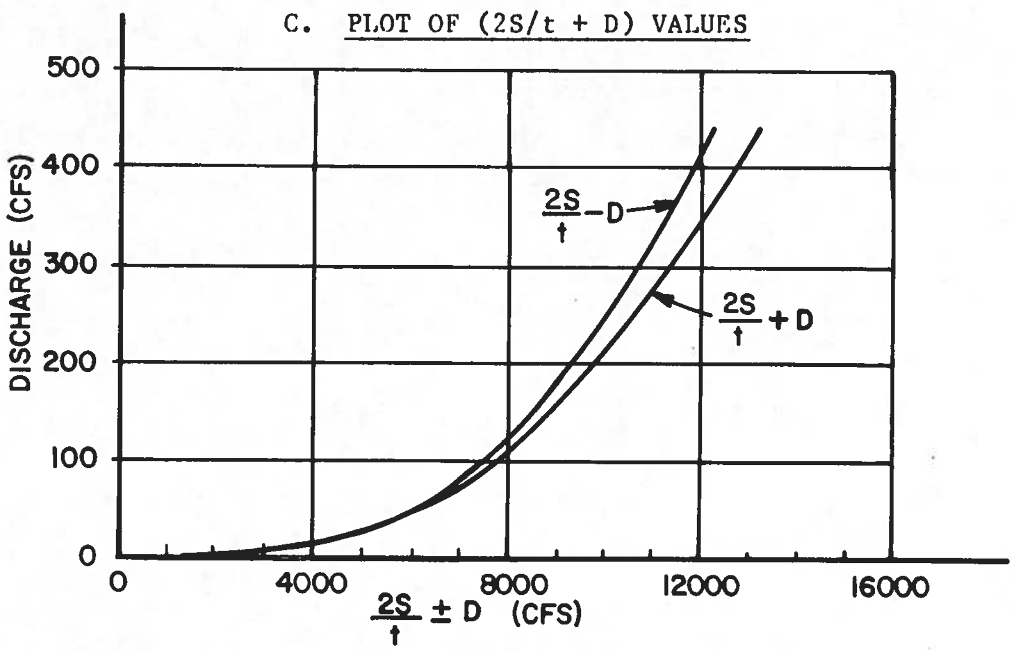
ELEVATION FT (1)	DISCHARGE CFS (2)	STORAGE (AF) (3)	STORAGE * (CFS-MIN) (4)
5042.5	0	0	0
5043.2	30	18	13,070
5043.5	101	26	18,880
5044.0	332	39	28,310
5044.2	440	44	31,940

* Storage in acre-feet converted to discharge units for selected time interval of minutes (see Text Section 606.2)

B. COMPUTATION OF (2S/t + D) VALUES

DISCHARGE CFS	STORAGE (CFS-MIN)	$\frac{2S}{t}$ (CFS)	$\frac{2S}{t}-D$ (CFS)	$\frac{2S}{t}+D$ (CFS)
0	0	0	0	0
30	13,070	5,230	5,200	5,260
101	18,880	7,550	7,450	7,650
332	28,310	11,320	10,990	11,650
440	31,940	12,780	12,340	13,220

C. PLOT OF (2S/t + D) VALUES



WRC ENG. REFERENCE: WRC Computational Procedures

STANDARD FORM SF-6

TABLE 611

RESERVOIR STORAGE ROUTING

(MODIFIED- PULS METHOD)

EXAMPLE 7

JOB NAME & LOCATION: Reservoir Storage

Job No. ______
By ______
Date ______
Chkd By ______
Date Chkd ______

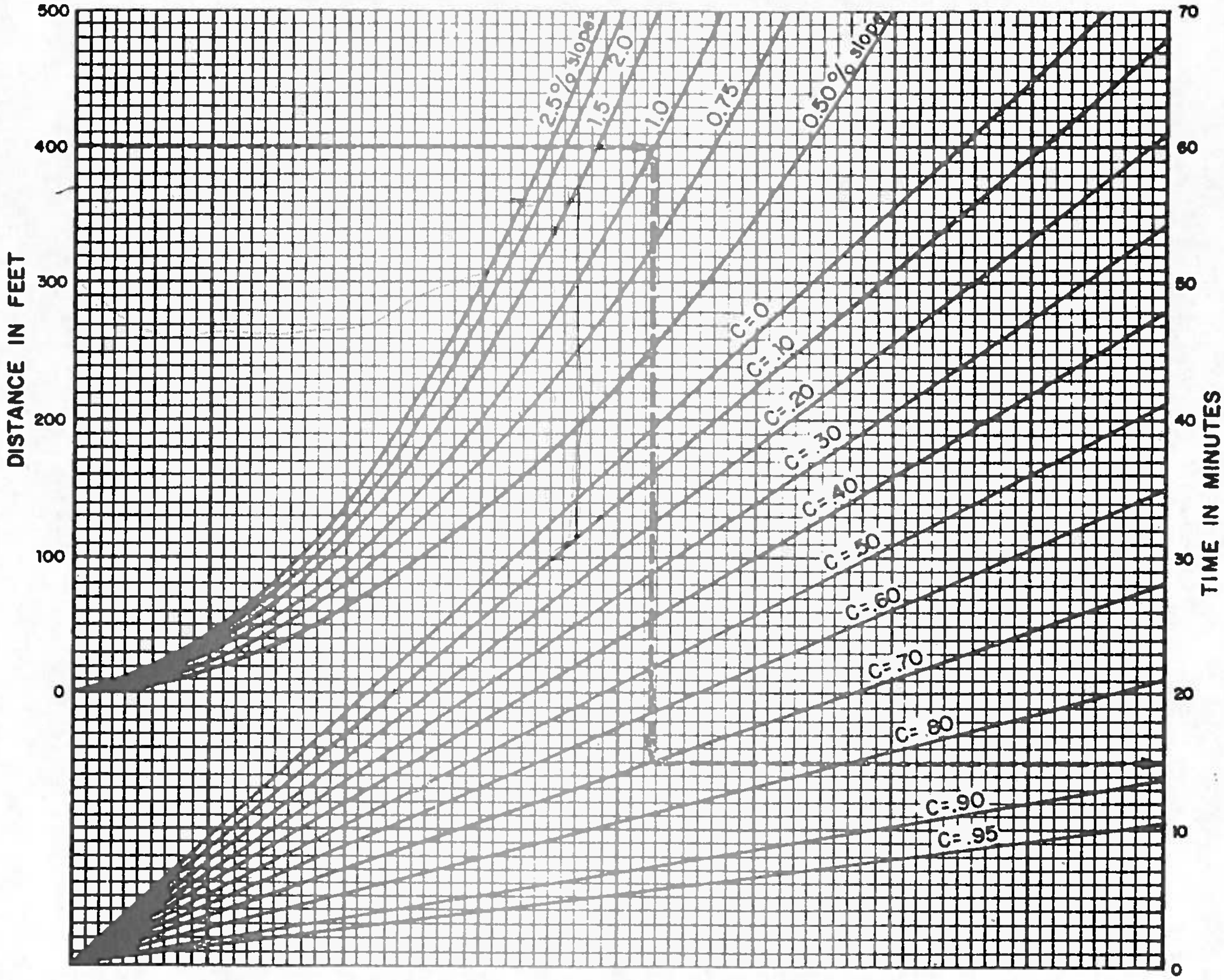
Storm Recurrence 100 yrs   Unit Time 5 min

Time Interval (1)	Time (min) (2)	Inflow I (cfs) (3)	$\frac{2S}{t} - D$ (cfs) (4)	$\frac{2S}{t} + D$ (cfs) (5)	Discharge D (cfs) (6)
1	5	0			0
2	10	13	2278	2278	2
3	15	52	2343	2343	3
4	20	160	2555	2555	4
5	25	454	3169	3169	10
6	30	803	4426	4426	20
7	35	952	6050	6181	50
8	40	909	7550	7911	105
9	45	817	8650	9276	165
10	50	717	9500	10184	220
11	55	626	10200	10843	262
12	60	553	10700	11380	330
13	65	483	11000	11736	332
14	70	405	11100	11888	333
15	75	331	11000	11836	332
16	80	267	10800	11598	315
17	85	216	10550	11283	290
18	90	177	10350	10943	270
19	95	144	10050	10671	240
20	100	121	9800	10315	230
21	105	104	9500	10025	215
22	110	91	9050	9695	190
23	115	81	8850	9222	168
24	120	70	8450	9001	155
25					
26					
27					
28					
29					
30					

(7) Caculate Maximum Storage(S_{max}) for maximum discharge(D)

$$S_{max} = \left(\left(\frac{2S}{t} + D\right) - D\right)\left(\frac{t}{2}\right) = ((11888) - (333))(2.5) = 28,887.5 \text{ cfs-min} = 39.8 \text{ AF}$$

OVERLAND TIME OF FLOW



THE ABOVE CURVES ARE A SOLUTION OF THE FOLLOWING EQUATION:

$$t_i = \frac{1.8\,(1.1 - C_5)\sqrt{L}}{\sqrt[3]{S}}$$

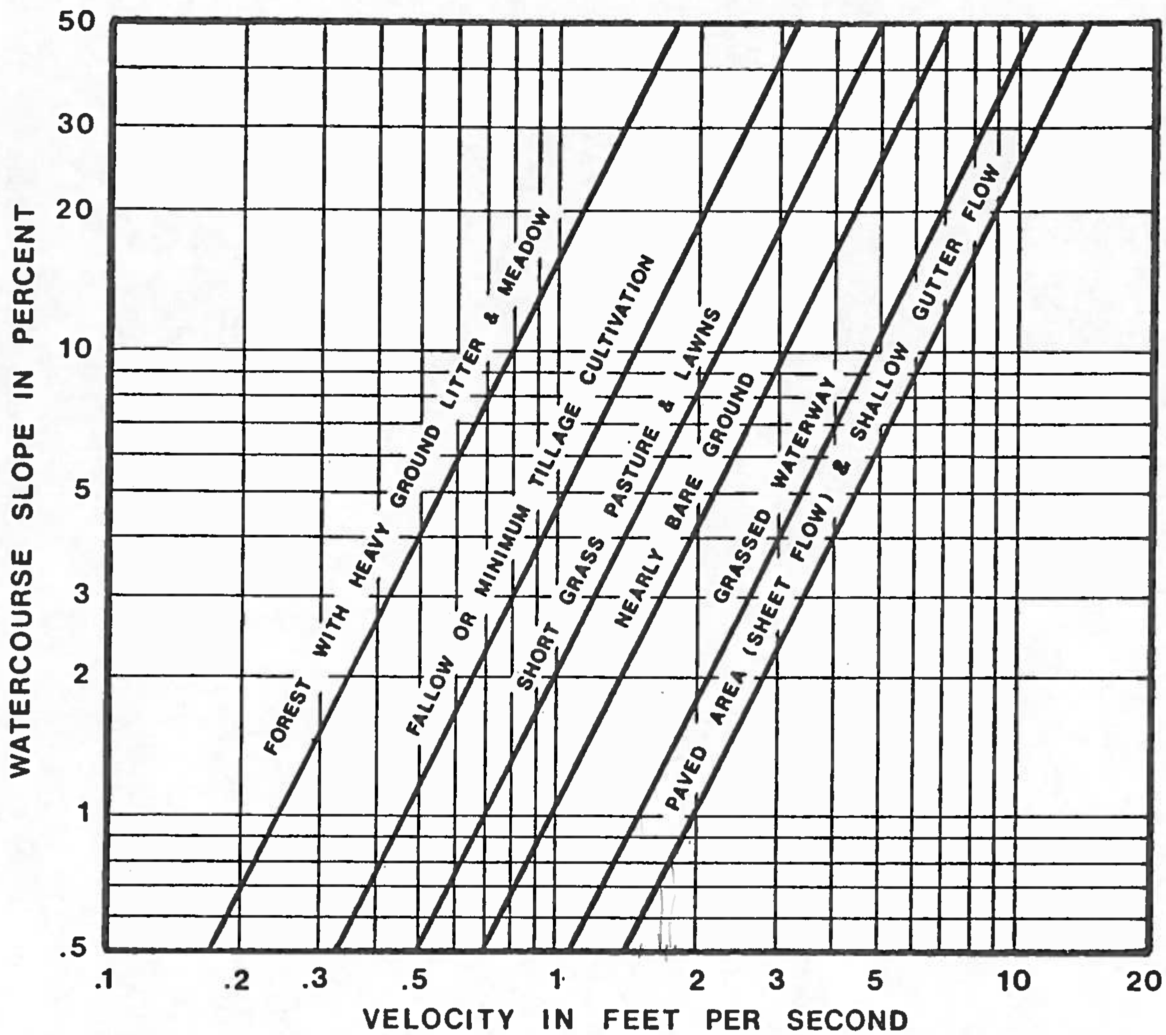
where: t_i = initial flow time (min.)
S = slope of basin (%)
C_5 = runoff coeficient for 5 year frequency **(Table 601)**
L = length of basin (ft.)

Notes: 1. The curves are for use with the Rational Method, see Text Section 602.

2. The curves shall not be used for distances in excess of 500'.

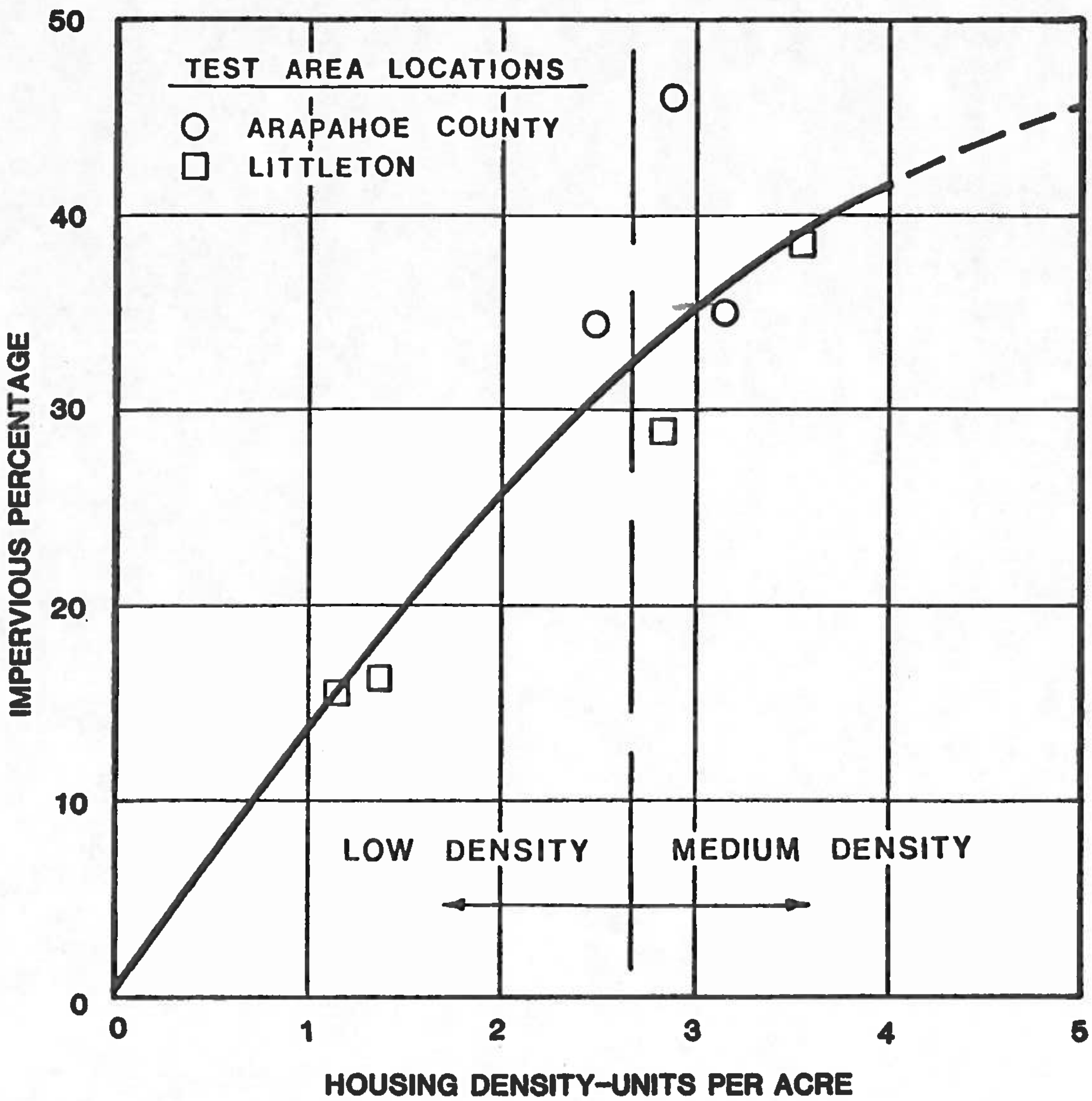
WRC ENG. | REFERENCE: "Urban Storm Drainage Criteria Manual" DRCOG, Denver, Colorado 1969

TRAVEL TIME VELOCITY FOR RATIONAL METHOD

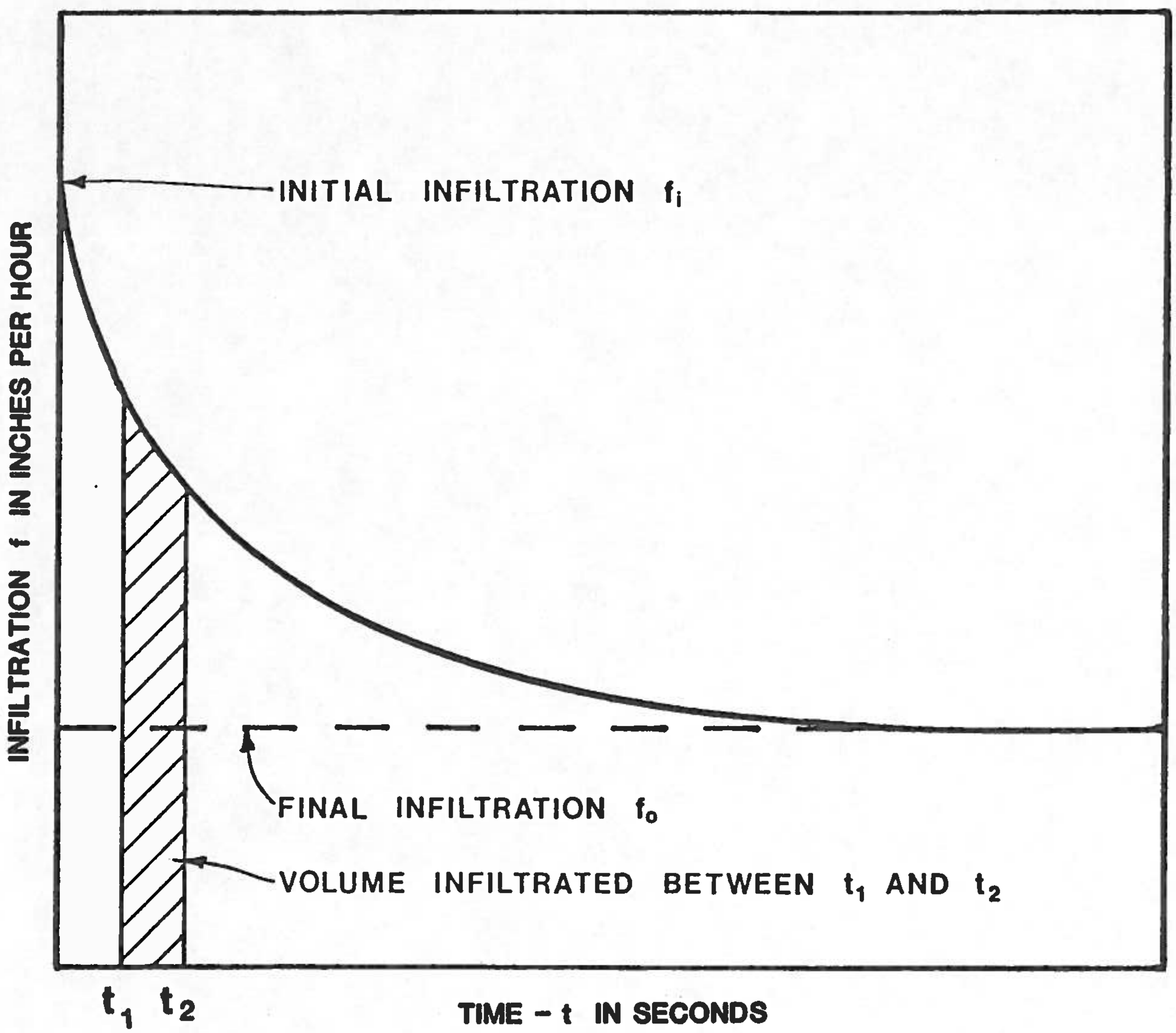


WRC ENG.	REFERENCE: "Urban Hydrology For Small Watersheds" Technical Release No. 55, USDA, SCS Jan. 1975.

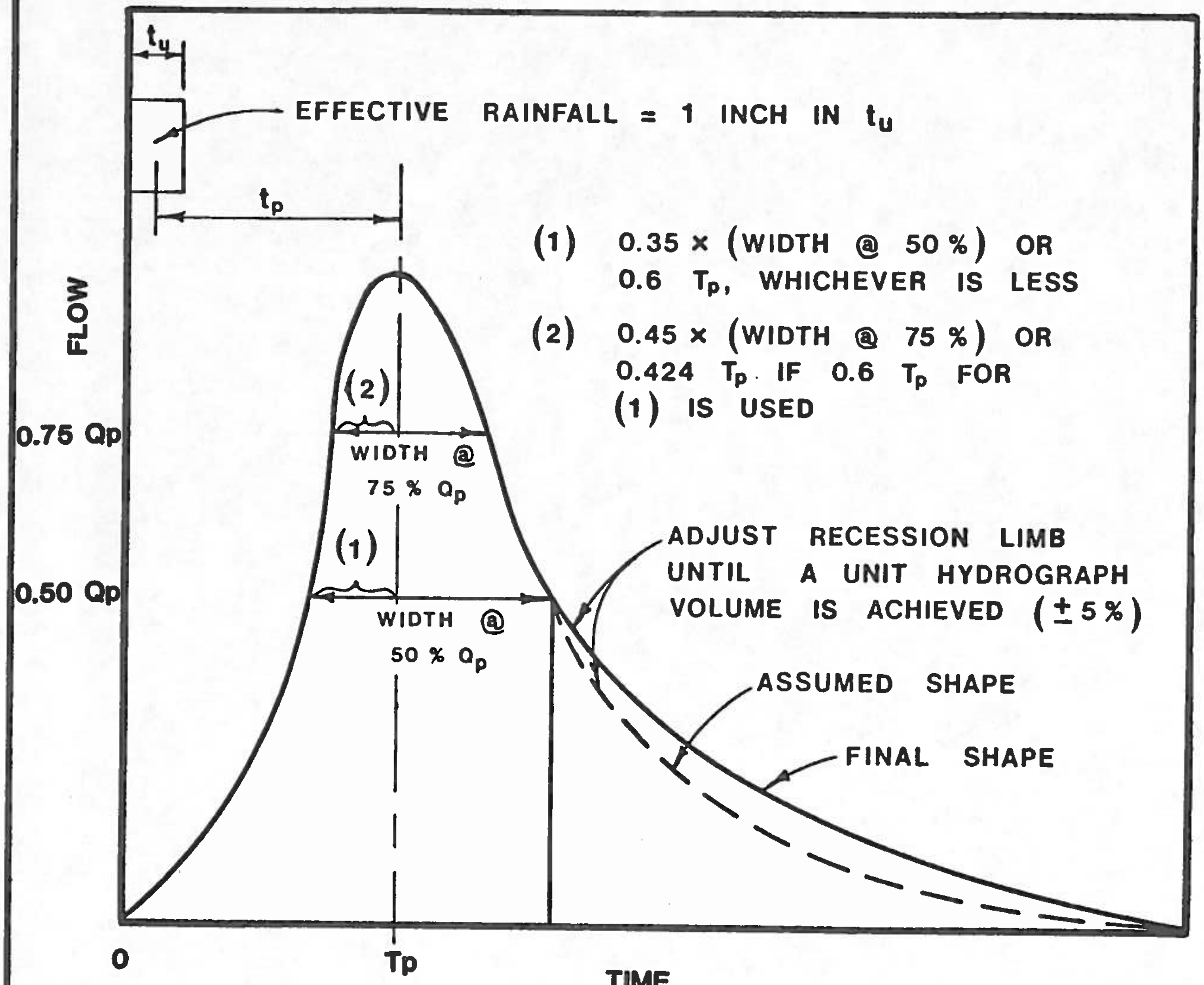
RESIDENTIAL HOUSING DENSITY VS IMPERVIOUS AREA



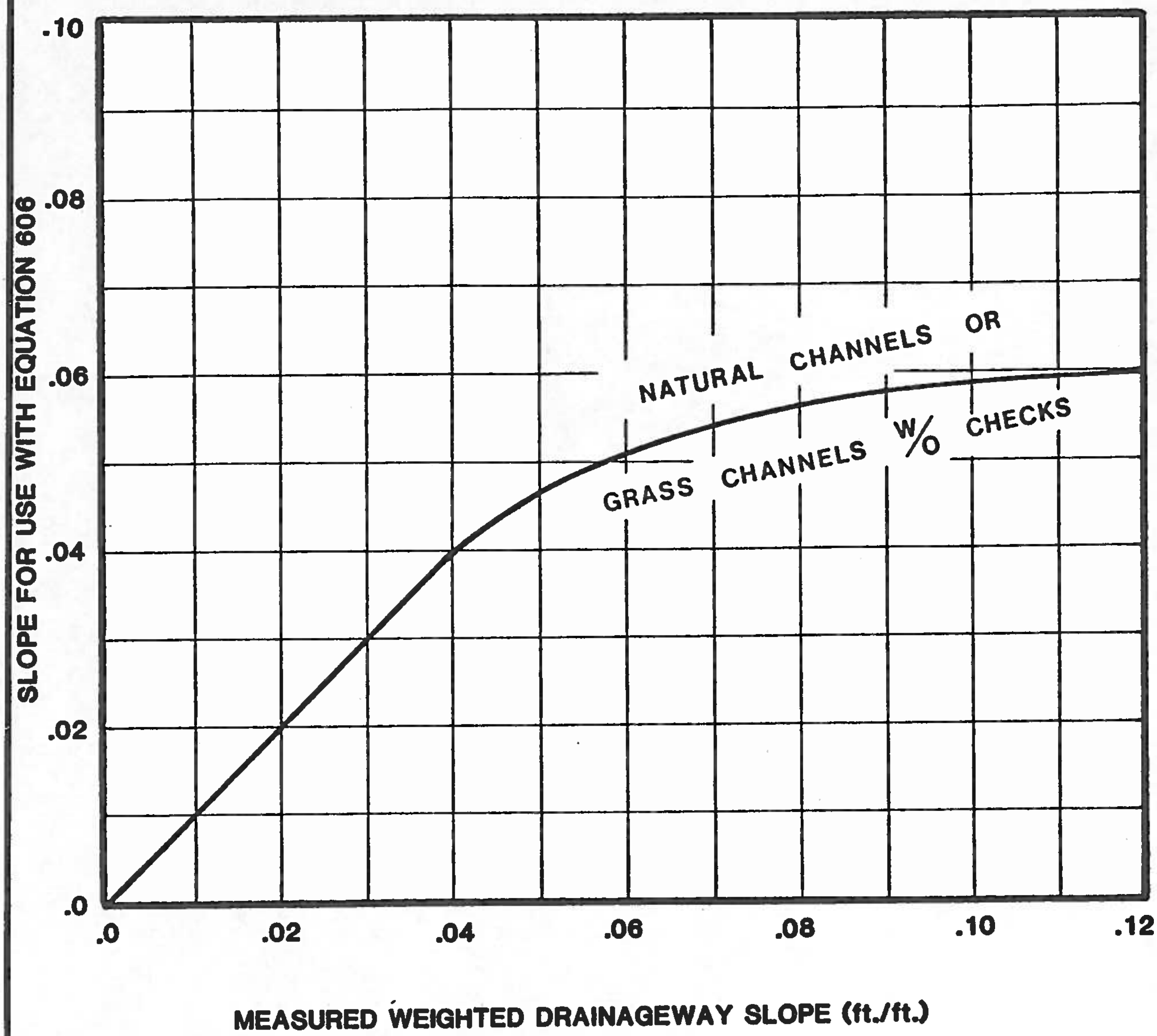
REPRESENTATION OF INFILTRATION EQUATION



UNIT HYDROGRAPH

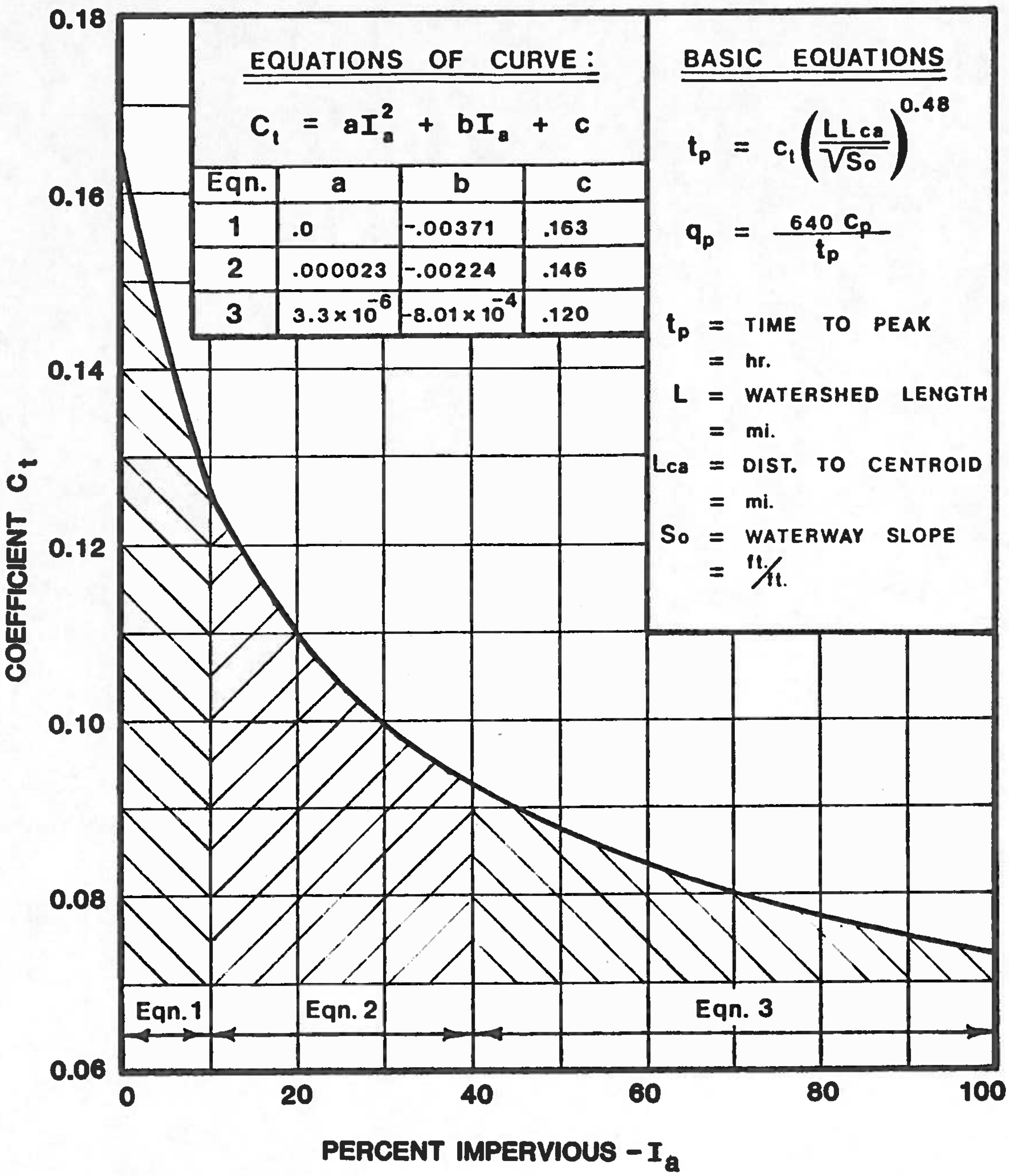


SLOPE ADJUSTMENT FOR STEEP DRAINAGEWAYS



WRC ENG. | REFERENCE: <u>USDCM</u>, DRCOG Revised 5-1-84

RELATIONSHIP BETWEEN C_t & IMPERVIOUSNESS



EQUATIONS OF CURVE:

$$C_t = aI_a^2 + bI_a + c$$

Eqn.	a	b	c
1	.0	-.00371	.163
2	.000023	-.00224	.146
3	3.3×10^{-6}	-8.01×10^{-4}	.120

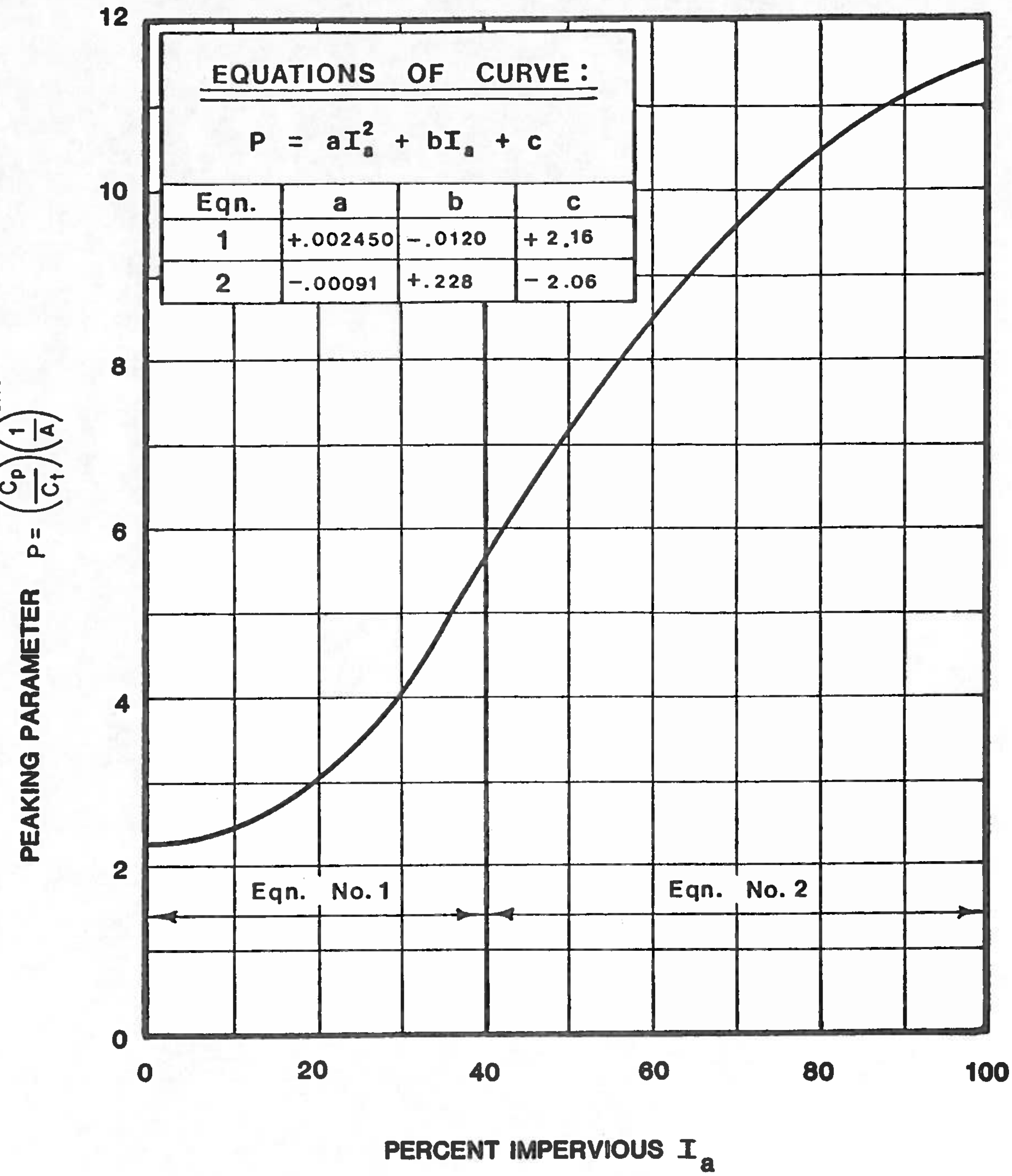
BASIC EQUATIONS

$$t_p = C_t \left(\frac{L L_{ca}}{\sqrt{S_o}} \right)^{0.48}$$

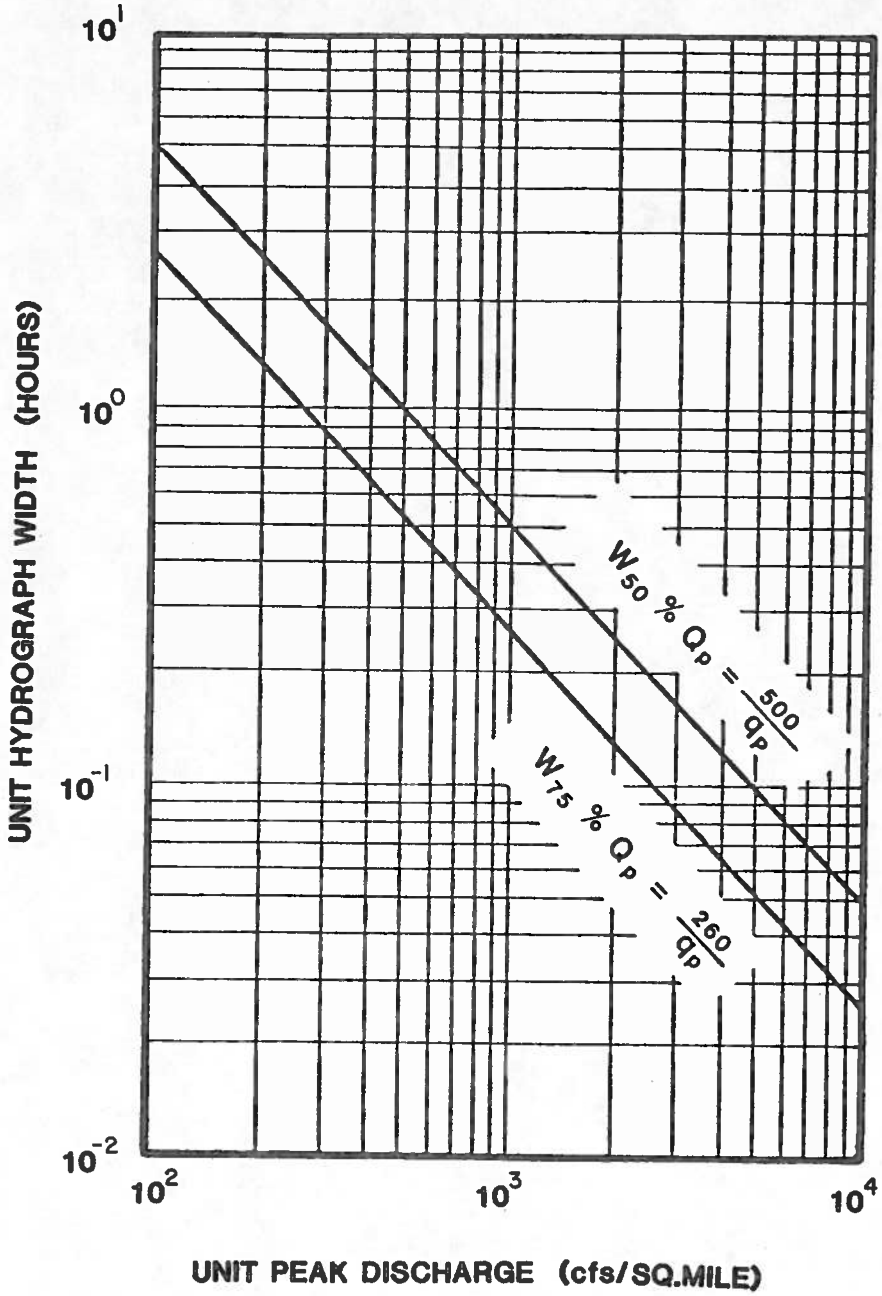
$$q_p = \frac{640\, C_p}{t_p}$$

t_p = TIME TO PEAK
= hr.
L = WATERSHED LENGTH
= mi.
Lca = DIST. TO CENTROID
= mi.
So = WATERWAY SLOPE
= ft./ft.

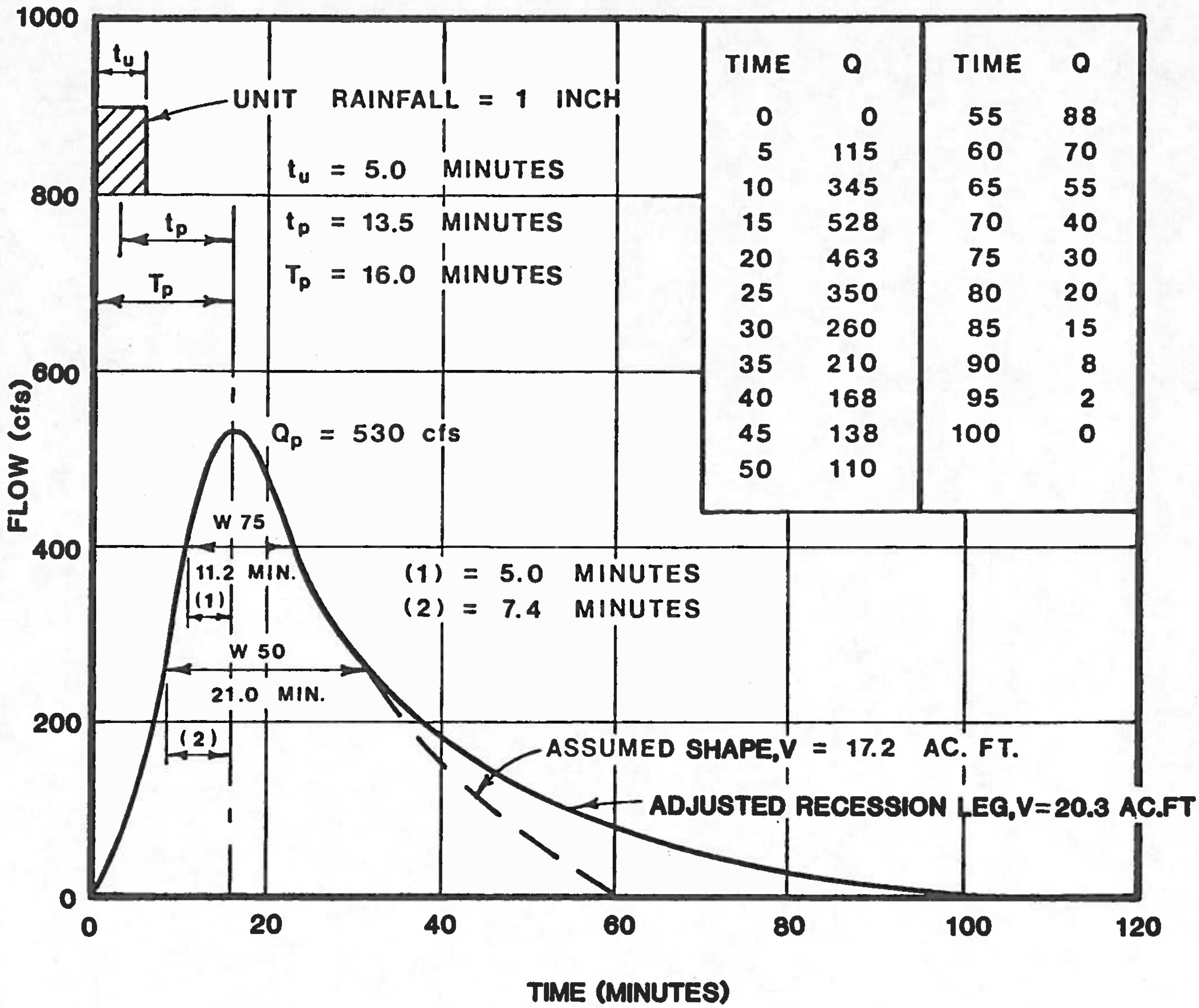
RELATIONSHIP BETWEEN PEAKING PARAMETER AND IMPERVIOUSNESS



UNIT HYDROGRAPH WIDTHS



EXAMPLE OF UNIT HYDROGRAPH SHAPING



TIME	Q	TIME	Q
0	0	55	88
5	115	60	70
10	345	65	55
15	528	70	40
20	463	75	30
25	350	80	20
30	260	85	15
35	210	90	8
40	168	95	2
45	138	100	0
50	110		