Section 2
Determination of Storm Runoff

2.01 General

This section will address proper procedures for the determination of flood hydrographs in the City of Lewisville.

Long-term records of rainfall and the resulting runoff in an area provide the best data source from which to base the design of storm drainage and control systems for that area. However, it is not possible to obtain these records in sufficient quantities in most watersheds. Therefore, numerous procedures have been developed to attempt to effectively relate a given amount of rainfall in a given physiographic area to a given pattern of runoff. This section of the manual contains a discussion of these procedures and the requirements for their use.

The determination of flood hydrographs in the City of Lewisville shall be accomplished through the use of one of two computer programs, HEC-1 or NUDALLAS. Both programs employ acceptable hydrologic methodologies and will be discussed in this section. The use of alternative means of flood hydrograph determination is acceptable subject to the approval of the Office of the City Engineer and provided full documentation of the alternative methodologies is presented for review.

For drainage area less than 100 acres, it is acceptable to utilize the Rational Method in cases where determination of the flood peak alone is required. A detailed discussion of the Rational Method is presented in this section.

2.02 Effect Of Urbanization

It is generally accepted that urban development has a pronounced effect on the rate and volume of runoff from a given rainfall. Urbanization generally alters the hydrology of a watershed by improving its hydraulic efficiency, reducing its surface infiltration and reducing its storage capacity. Figure 2-1 illustrates the effect of improving a watershed's hydraulic efficiency by presenting runoff rate versus time for the same storm with two different stages of watershed development.

The reduction of a watershed's storage capacity and surface infiltration results from the elimination of porous surfaces and ponding areas by grading and paving building sites, streets, parking lots, and sidewalks and by constructing buildings and other facilities characteristic of urban development.

Zoning maps, future land use maps, and watershed master plans should be used as aids in establishing the anticipated surface character following development. The selection of design runoff coefficients and/or percent impervious cover factors, which are explained in the following discussions of runoff calculation, must be based upon the appropriate degree of future urbanization.

2.03 Rainfall-Runoff Design Frequencies

All drainage structures or improvements in the City of Lewisville shall be designed to properly accomodate the runoff from a storm event of 100-year frequency.

2.04 Relating Rainfall To Runoff

The process of relating rainfall on a watershed to runoff at a given point in the watershed is generally accomplished in the following four discrete stages:

Determination of the design rainfall;

Calculation of abstractions (losses);

Generation of the runoff hydrograph for the subarea; and

Determination of the change in the shape of the hydrograph (termed routing) as the flood wave moves through the watershed.

Each of these four stages will be discussed in the following sections (A through D):

A. Design Storm Rainfall

1. Parameters of a Storm Event

A design storm rainfall event is described in terms of four parameters--frequency, total storm duration, distribution of intensity with time, and areal extent. The following considerations are pertinent to each:

A. Frequency - Storm frequency is a measure of the expected recurrence interval of a storm of a given magnitude. As an example, the 100-year frequency storm event (with a 100-year frequency magnitude) can be expected to recur on average once every 100 years. In other words, the 100-year storm event has a one percent chance of occuring in any given year.

In the building of drainage systems, a drainage structure's capacity is defined by the frequency of the storm event that it must be able to handle.

B. Total Storm Duration - Total storm duration is defined as the time from when rainfall begins to when the rainfall has ended.

The engineer's choice for design storm duration is generally dependent on the size of the pertinent watershed. For design purposes, a storm of 24-hour duration can be expected to adequately reflect all time-related effects on the runoff hydrograph. A storm duration of 96 hours is required for designation of the probable maximum flood (PMF) precipitation.

C. Distribution of Intensity with Time - In general, in convective and frontal storms the rainfall hyetograph peak tends to occur in the first third of the total storm duration while in cyclonic storms, the hyetograph peak tends to occur in the middle third of the storm. In reality, the ratio of the time-to-peak to total duration varies randomly at any location from storm to storm.

For design purposes in the City of Lewisville, the synthetic rainfall hyetograph peak shall occur at the midpoint of the total storm duration. Both HEC-1 and NUDALLAS have routines for generating acceptable design rainfall hyetographs.

D. Areal Extent - The intensity/duration/frequency relationships used to build rainfall hyetographs are based on rainfall amounts measured at a single location. Logically, the larger the watershed being studied, the less rainfall volume per unit area can be expected to fall uniformly over the watershed for a given frequency event.

Figure 15 in the National Weather Service's Technical Paper No. 40 presents a means of reducing rainfall totals for a given frequency event as drainage area size increases.

In addition, both the NUDALLAS and HEC-1 programs have the capability to modify runoff hydrographs to account for progressively smaller design storm volumes as areal coverages increases.

2. Intensity-Duration-Frequency Curves

National Weather Service Technical Paper 40 (TP-40), "Rainfall Frequency Atlas of the United States," May 1961, provides accumulated point rainfall amounts for storm durations from 30 minutes to 24 hours and event frequencies from 1 to 100 years. National Oceanic and Atmospheric Administration (NOAA) publication NOAA Hydro-35, "Five-to- 60- Minute Precipitation Frequency for the Eastern and Central United States," June 1977, provides accumulated point rainfall amounts for durations of 5 minutes, 15 minutes, and 1 hour for 1 and 100-year frequency events. NWS Technical Paper 49 (TP-49), "Two-to-Ten-Day Precipitation for Return Periods of 2 to 100 Years in the Contiguous United States," 1964, extends the duration range to 10 days.

TP-40 and Hydro-35 were used to generate Table 2-1 which presents point rainfall amounts for varying durations and frequencies in the Lewisville, Texas, area. Table 2-1 may be used for determining the rainfall volumes for various durations and frequencies to be input to HEC-1 or NUDALLAS. With this data, each program will generate an acceptable design rainfall hyetograph.

Figure 2-4 presents the rainfall frequency/duration/intensity curves applicable to the Lewisville, Texas area.

For design purposes in the City of Lewisville, the synthetic rainfall hyetograph peak shall occur sometime after the first quarter and sometime before the last quarter of the total storm duration. Both HEC-1 and NUDALLAS. have routines for generating acceptable design rainfall hyetographs.

B. Design Storm Losses

Only a portion of the rainfall volume which falls on a watershed during a storm event actually ends up as stream runoff. The remainder is intercepted by infiltration, depression storage, evaporation and other mechanisms. The volumes of rainfall which becomes runoff is termed the "excess" rainfall. The difference between the observed total rainfall hyetograph and the excess rainfall hyetograph is termed abstractions or losses.

Numerous methodologies are available to calculate abstractions. They range from the straight forward uniform loss rate, which is simple to use but generally the least accurate, to a variety of approaches which are usually more accurate but are also more difficult to calibrate.

HEC-1 and NUDALLAS contain the following loss routines:

Uniform Loss Rate;

HEC Exponential Loss Rate;

SCS Curve Number Loss Rate; and

Holtan Loss Rate.

NUDALLAS contains, in addition, the Holtan Loss Rate methodology.

The default condition for losses as calculated in NUDALLAS makes use of the Block and uniform loss rate methodology. The specific default values contained in the program for the initial loss and average hourly losses for varying frequency events are contained in Table 2-3.

The use of these rates is acceptable in the City of Lewisville. The above loss rates in Table 2-3 should also be used as a guideline for application of the uniform loss methodology in HEC-1.

For discussion of all of the above methodologies, the engineer is referred to "HEC-1, Flood Hydrograph Package, User's Manual" and "NUDALLAS, Documentation and Supporting Appendixes."

C. Design Storm Runoff

Given the design storm excess rainfall, it is necessary to determine the storm runoff hydrograph for the subbasin of interest. In the City of Lewisville, flood hydrographs will be determined utilizing the Snyder's unit hydrograph methodology.

1. Snyder's Unit Hydrograph

In a study of watersheds of widely varying size, Snyder (1938) found consistent relationships for unit hydrographs in watersheds in which the time to peak (t_p) is 5.5 times longer than the duration of the unit hydrograph excess rainfall (t_r). In other words, for a "standard" unit hydrograph in which $t_p = 5.5t_r$, the following relationships were found to hold:

$$t_p = C_t(LL_{ca})^{0.3} \qquad (2\text{-}1)$$

$$q_p = 640\frac{C_p}{t_p} \qquad (2\text{-}2)$$

where:

t_r = duration of the unit hydrograph excess rainfall (hrs)

t_p = time from center of unit hydrograph excess rainfall duration to peak of unit hydrograph (hrs)

L = length of main stream (miles)

L_{ca} = distance from the point of interest to the point on the stream nearest the centroid of the watershed area (miles)

C_t = timing coefficient representing variation of sub-basin slope and storage

q_p = peak flow rate per unit of drainage area (cfs)

C_p = peaking coefficient dependent upon units and drainage basin characteristics

When the "standard" relationship $t_p = 5.5t_r$ does not hold, the time to peak (t_p) and the peak discharge per unit area (q_p) can be adjusted for the desired unit hydrograph computation interval t_r (required) as follows:

$$t_p(\text{adjusted}) = t_p + 0.25\left(t_r\ (\text{required})\ - \frac{t_p}{5.5}\right) \quad (2-3)$$

$$q_p\ (\text{adjusted}) = \frac{640(C_p)}{t_p} \quad (2-4)$$

2. Determination of Snyder's unit Hydrograph Shape

The Snyder's methodology is incapable of yielding a continuous and complete description of the unit hydrograph shape. This task is handled differently by the two programs.

In HEC-1, determination of the continuous shape of the unit hydrograph is carried out with an iterative procedure using the Clark unit hydrograph methodology. In NUDALLAS, determination of the shape is based on a method described in James A. Constant's paper, "A Mathematical Determination of the Ordinates of the Unit Hydrograph," 1970.

3. Application of Snyder's Unit Hydrograph in HEC-1 and NUDALLAS.

HEC-1 and NUDALLAS handle application of the Snyder's unit hydrograph slightly differently:

NUDALLAS was originally written to allow the engineer to apply basic hydrologic principles specifically in the Dallas-Fort Worth area with a minimum of input. Consequently, empirical percent urbanization curves, specific to the Dallas area, were developed and internalized in the program. These are used in the determination of the Snyder's parameter, t_p and are presented as Figures 2-2 and 2-3 at the conclusion of this section.

In NUDALLAS, the engineer is required to specify the following watershed parameters:

a. L, the length of the main stream in miles.

b. L_{ca}, the distance in miles from the point of interest to the point (on a stream) nearest the centroid of the watershed area.

c. The drainage area size in square miles.

d. S_{st}, the weighted slope of the main drainage course in feet per mile.

e. The percent urbanization, which is defined as the percent of the subarea which has been developed and improved with channelization or a storm collection network. The engineer is referred to Table 2-2 for a summary of land use types vs. percent urbanization.

f. The percent imperviousness, which is defined as the percent of the subarea that is covered with impervious material and is hydraulically connected to the subarea's drainage network. The engineer is referred to Table 2-2 for a summary of land use types vs. percent impervious cover. This parameter is not optional and requires a somewhat different input procedure.

g. The percent of sand (and consequently the percent of clay) which is to be determined from local soil surveys. This is not an optional parameter and requires a somewhat different input procedure.

HEC-1 requires two Snyder's parameters - t_p and C_p. These will be determined as follows:

a. C_p - It has been determined that in the Lewisville area C_p shall have a value of 0.719. A widely used alternative to the parameter C_p is termed C_p 640 and shall have a value of 460.0.

b. t_p - To maintain continuity with the NUDALLAS program, t_p will be determined using the percent urbanization curves developed by the Corps and presented at the conclusion of this section of the manual. This is carried out as follows:

Step 1 - Determine what percent of the watershed has clay soils and what percent has sand soils.

Step 2 - Determine the value of:

$$\frac{LL_{ca}}{(S_{st})^1/2}$$

where L, L_{ca}, and S_{st}, are as described above.

Step 3 - Using the curve for clay (Figure 2-2), determine the lag time (hours) for the given percent urbanization. Using the curve for sand (Figure 2-3), determine the lag time (hours) for the given percent urbanization. Multiply each value of the lag by its respective percent of occurrence and sum them to determine the weighted lag t_p.

D. Routing the Flood Hydrograph

As a flood wave passes downstream through a channel or detention facility, its shape is altered due to the effects of storage. The procedure for determining how the shape of the flood hydrograph changes is termed flood routing.

1. Stream Routing vs. Reservoir Routing - Flood routing can be classified into two broad but related categories: open channel stream routing and reservoir routing. Reservoir routing is often used to determine the effect on a runoff hydrograph of stormwater detention. Open channel stream routing is used to determine the changing shape of the runoff hydrograph as a function of the channel geometry or storage capacity.

2. Available Methodologies - HEC-1 and NUDALLAS contain several acceptable flood routing routines. For work in the City of Lewisville, it is recommended that the engineer employ either the Muskingum method or the Modified Puls method.

a. Muskingum Method - The Muskingum method models the storage volume as a combination of wedge storage caused by a non-level water surface along the routing reach and prism storage formed by a volume of constant cross section along the length of the prismatic channel. Unlike the Modified Puls method, it is not limited by the assumption of a level

Coefficients for specific surface types can be used to develop a composite runoff coefficient based in part on 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.

Table 2-5 presents recommended ranges for "C" values for various residential districts and specific surface types for the 5-year frequency storm. Adjustment of the "C" value for use with larger (less frequent) storms can be made by multiplying the right side of the Rational Formula by a frequency factor C_f

which is used to account for antecedent precipitation conditions. The Rational Formula now becomes:

$$Q = CiAC_f \qquad (2\text{-}8)$$

Table 2-4 presents recommended values of C_f. The product of C times C_f should not exceed 1.0.

E. Rainfall Intensity

Rainfall intensity (i) is the average rainfall rate in inches per hour which is considered for a particular basin or sub-basin and is selected on the basis of design rainfall duration and design frequency of occurrence. The design duration is equal to the critical time of concentration for all portions of the drainage area under consideration that contribute flow to the design point during the critical time of concentration. The frequency of occurrence is a statistical variable which is established by design standards or chosen by the engineer as a design parameter. All drainage structures in the City of Lewisville will be designed to accomodate the 100-year design frequency. The design rainfall intensity to be used in the rational equation is determined for a given duration and frequency from the frequency/duration/intensity curves presented in Figure 2-4.

The time of concentration used in the rational equation is the critical time of concentration for the point of interest. The critical time of concentration is the time associated with the peak runoff from all or part of the upstream drainage area to the point of interest. Runoff from a watershed usually reaches a peak at the time when the entire drainage area is contributing, in which case, the time of concentration is the time for water to flow from the most remote point in the watershed to the point of interest. However, the runoff rate may reach a peak prior to the time the entire upstream drainage area is contributing. In this instance, only the portions of the drainage area able to contribute flow to the design point during the critical time of concentration should be used in determining the peak discharge. A trial-and-error procedure can be used to determine the critical time of concentration.

The time of concentration to any point in a storm drainage system is a combination of the "inlet time" and the "time of flow in the conduit."

The inlet time is the time for water to flow over the surface to the storm sewer inlet. Inlet time decreases as the slope and the imperviousness of the surface increases, and it increases as the distance over which the water has to travel increases and as retention by the contact surfaces increases. Average velocities for estimating travel time for overland flow can be calculated using Figure 2-5.

The inlet time shall be determined by direct computation using the following formula:

$$T = \frac{D_f}{60V} \qquad (2\text{-}9)$$

where

T = overland flow time (minutes).

D_f = flow distance (feet).

V = average velocity of runoff flow (ft./sec).

Overland flow distances will rarely exceed 400' in developed areas. If the overland flow time is calculated to be in excess of 20 minutes, the designer should verify that the time is reasonable considering the projected ultimate development of the area.

The time of flow in the conduit is the quotient of the length of the conduit and the velocity of flow as computed using the hydraulic characteristics of the conduit. The time of concentration within a conduit is usually less than the actual time for the flood crest to reach a given point by an amount equal to the time required to fill the conduit. The time required to fill the conduit is defined as the time of storage. The time of storage shall be neglected in the design of storm runoff conduits even though it may represent an appreciable percentage of the total time of concentration in some instances. This procedure will not substantially affect the precision of the calculations and will contribute to a conservative design.

F. Drainage Area (A)

The size and shape of the drainage area must be determined. The area may be determined through the use of topographic maps, supplemented by field surveys where topographic data has changed or where the contour interval is too great to distinguish the direction of flow. A drainage area map will be provided for each project. The drainage area contributing to the system being designed and the drainage subarea contributing to each inlet point shall be identified. The outlines of the drainage divides must follow actual lines rather than the artificial land divisions as used in the design of sanitary sewers. The drainage divide lines are determined by the pavement slopes, locations of downspouts, paved and unpaved yards, grading of lawns and many other features that are introduced by the urbanization process.

As mentioned previously, the drainage area used in determining peak discharges is the portion of the area that contributes flow to the design point within the critical time of concentration.

TABLE 2-1

POINT RAINFALL AMOUNT FOR
VARYING DURATIONS AND FREQUENCIES IN
LEWISVILLE, TEXAS

Duration	1-Yr	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	500-Yr	SPF
5-min		.490	.572	.634	.726	.798	.870		
10-min		.809	.948	1.06	1.21	1.34	1.46		
15-min		1.03	1.21	1.35	1.55	1.71	1.87		
30-min	1.18	1.43	1.75	1.99	2.32	2.59	2.85	4.10	
1-hr	1.53	1.84	2.31	2.65	3.13	3.51	3.88	5.18	
2-hr	1.77	2.21	2.96	3.49	4.13	4.63	5.15	6.30	
3-hr	1.95	2.45	3.23	3.82	4.50	5.09	5.65	7.10	
6-hr	2.36	2.89	3.87	4.60	5.41	6.06	6.80	8.50	
12-hr	2.78	3.40	4.60	5.45	6.29	7.23	8.20	10.48	
24-hr	3.17	3.95	5.32	6.26	7.42	8.42	9.42	12.05	
48-hr		4.50	5.95	7.05	8.50	9.50	10.80		
72-hr									14.70
96-hr		5.25	6.95	8.10	9.70	11.00	12.30		

Sources: National Weather Service. 1961. National Weather Service Technical Paper No. 40, "Rainfall Frequency Atlas of the United States."

National Oceanic and Atmospheric Administration. 1977. NOAA Hydro-35, "Five- to 60-Minute Precipitation Frequency for the Eastern and Central United States" and;

National Weather Service. 1964. NWS Technical Paper No. 49, "Two- to Ten-Day Precipitation for Return Periods of 2 to 100 Years in the Contiguous United States."

Note: This table to be used with either the "HEC-1" or "NUDALLAS" methods.

TABLE 2-2
PERCENT URBANIZATION AND IMPERVIOUSNESS
SUMMARY WITH ASSOCIATED LAND USE CATEGORIES
RECOMMENDED VALUES

Title	Percent Imperviousness	Percent Urbanization
Low Density Residential	38	70
Medium Density Residential	56	80
High Density Residential	70	90
Multi-family Residential	70	95
Mobile Home Parks	20	40
Central Business District	95	95
Strip Commercial	90	90
Shopping Centers	95	95
Institutional-School, Churches	40	50
Industrial	90	95
Transportation, Major Highways	35	80
Parks and Developed Open Space	6	10
Cropland	3	5
Grassland	0	0
Woodlands, Forest	0	0
Water Bodies	100	100
Barren Land, Gravel Pits	0	0

Sources: Determination of Percent Urbanization/Imperviousness in Watersheds, May 1, 1986, U.S. Army Corps of Engineers and;

Urban Hydrology for Small Watersheds, Soil Conservation Service Technical Release No. 55.

TABLE 2-3

NUDALLAS BLOCK AND UNIFORM LOSS RATE

RECOMMENDED VALUES FOR

LEWISVILLE, TEXAS

Event Frequency	Sand		Clay	
	Initial Loss (in.)	Avg. Hourly Loss (in.)	Initial Loss (in.)	Avg. Hourly Loss (in.)
1-Year	2.1	0.26	1.5	0.2
2-Year	2.1	0.26	1.5	0.2
5-Year	1.8	0.21	1.3	0.16
10-Year	1.5	0.18	1.12	0.14
25-Year	1.3	0.15	0.95	0.12
50-Year	1.1	0.13	0.84	0.1
100-Year	0.9	0.10	0.75	0.07
500-Year	0.6	0.08	0.5	0.05
SPF	0.6	0.08	0.5	0.05

Source: NUDALLAS, Documentation and Supporting Appendixes, U.S. Army Corps of Engineers, For Worth District.

TABLE 2-4

FREQUENCY FACTOR COEFFICIENTS FOR
ADJUSTMENT OF THE RATIONAL METHOD C VALUE

Frequency of Storm	Frequency Factor C_f
5	1.00
25	1.10
50	1.20
100	1.25

Source: Urban Storm Drainage Criteria Manual-Volume 1, Denver Regional Council of Governments, Wright-McLaughlin Engineers, March 1969.

TABLE 2-5

RATIONAL METHOD RUNOFF COEFFICIENTS
FOR 5-YEAR FREQUENCY STORM

Description of Area	Runoff Coefficents for Basin Slopes Less than 1%	1%-3.5%	3.5%-5.5%
Residential Districts			
Single Family Areas (Lots greater than ½ acre)	0.25	0.35	0.40
Single Family Areas (Lots ¼-½ acre)	0.35	0.40	0.45
Single Family Areas (Lots less than ½ acre)	0.40	0.45	0.50
Multi Family Areas	0.60	0.65	0.70
Apartment Dwelling Areas	0.75	0.80	0.85
Business Districts			
Downtown Areas	0.85	0.87	0.90
Neighborhood Areas	0.75	0.80	0.85
Industrial Districts			
Light Areas	0.50	0.65	0.80
Heavy Areas	0.60	0.75	0.90
Railroad Yard Areas	0.20	0.30	0.40
Parks, Cemetaries	0.10	0.18	0.25
Playgrounds	0.20	0.28	0.35
Streets			
Asphaltic	0.80	0.80	0.80
Concrete	0.85	0.85	0.85
Drives and Walks (Concrete)	0.85	0.85	0.85
Roofs	0.85	0.85	0.85
Lawn Areas			
Sandy Soil	0.05	0.08	0.12
Clay Soil	0.15	0.18	0.22
Undeveloped Areas			
Sandy Soil			
Woodlands	0.15	0.18	0.25
Pasture	0.25	0.35	0.40
Cultivated	0.30	0.55	0.70
Clay Soil			
Woodlands	0.18	0.20	0.30
Pasture	0.30	0.40	0.50
Cultivated	0.35	0.60	0.80

pool in the routing reach. The Muskingum parameters K and X are required in HEC-1 and shall be determined as defined in this section of the manual.

The Muskingum model solves the following linear version of the continuity equation:

$$S = K\ [XI + (I-X)\ 0] \qquad (2\text{-}5)$$

where:

S = storage in the routing reach

I = inflow rate

O = outflow rate

K = a proportionality parameter which equals the travel time through the routing reach of an incremental flood wave. K may be estimated for use in HEC-1 as follows:

- in wide rectangular channels, K in seconds = (length of the reach in feet) divided by (1.67 x (avg. velocity of flow in feet per second)). Note that in HEC-1, K must be converted to hours.

- in wide parabolic channels, K in seconds = (length of the reach in feet) divided by (1.44 x (avgerage velocity of flow in feet per second)). Note that in HEC-1, K must be converted to hours.

- in triangular channels, K in seconds = (length of the reach in feet) divided by (1.33 x (average velocity of flow in feet per second)). Note that in HEC-1, K must be converted to hours.

(Note: In HEC-1, "Users Manual," see RM card description for limitations on size of K)

X = a storage parameter which varies between 0 and 0.5. When an incremental change in the inflow to the routing reach has virtually no effect on the outflow (as in a reservoir), X should equal 0. When an incremental change in the inflow will effectuate a virtually identical immediate change in the outflow (as in a pipe flowing full), X should equal 0.5. For most natural channels, X should range between 0.1 and 0.3.

The overall routing reach may require partitioning into subreaches when channel geometry changes significantly or if the limits of K (see HEC-2, "User's Manual," description of RM card) cannot be satisfied.

For a discussion of the Muskingum routing methodology, the engineer is referred to the "HEC-1, User's Manual."

b. Modified Puls Method - The Modified Puls method is based upon a solution of the simple continuity equation:

I - O = change in S (2-6)

where:

I = inflow

O = outflow

S = storage

The engineer must provide as input an estimate of starting conditions and storage vs. discharge values as calculated either by a backwater model such as HEC-2, Water Surface Profiles, or as in the case of a reservoir, by the characteristics of the outlet works.

The overall routing reach should be partitioned into smaller subreaches when significant changes in channel geometry warrant it.

For a detailed discussion of the Modified Puls methodology, the engineer is referred to the "Handbook of Applied Hydrology," Ven Te Chow, 1964 or "NUDALLAS., Documentation and Supporting Appendixes."

3. Distributed Models for Flood Routing - The above-mentioned routing methodologies are usually adequate for general hydrologic design. However, there are available a number of "distributed" flow routing models which make fewer simplifying assumptions about the physical nature of the flow and are therefore able to more accurately simulate the transition of a flood wave through a channel. These range from the simple kinematic wave approach (available in HEC-1) to dynamic wave models such as the National Weather Service programs DAMBRK and DWOPER.

The Texas Water Commission currently requires the use of NWS DAMBRK for the routing of simulated dam break flood waves necessary in required dam failure analyses. In situations where wide floodplains exist, the slope of the channel invert is less than 0.5 ft./mi., and/or significant effects from downstream disturbances exist, a more sophisticated approach to flood routing may be in order.

2.05 Rational Method

A. General

The Rational Method represents an accepted method for determining peak storm runoff rates for small watersheds that have a drainage system unaffected by complex hydrologic situations such as ponding areas, storage basins and watershed transfers (overflows) of storm runoff. The widely used method provides satisfactory results if understood and applied correctly. It is generally recommended that the Rational Method be used for areas less than 100 acres in the City of Lewisville.

B. Definition of Rational Formula

The Rational Method is based on the direct relationship between rainfall and runoff, and is expressed by the following equation:

$$Q = CiA \qquad (2\text{-}7)$$

where:

Q is defined as the peak rate of runoff in cubic feet per second. Actually, Q is in units of inches per hour per acre. Since this rate of in/hr/ac differs from cubic feet/second by less than one percent, the more common cfs is used.

C is the dimensionless coefficient of runoff representing the ratio of peak discharge to rainfall intensity (i).

i is the average intensity of rainfall in inches per hour for a period of time equal to the critical time of flow concentration for the drainage area to the point under consideration.

A is the area in acres contributing runoff to the point of design during the critical time of concentration.

C. Assumptions of Rational Method

Basic assumptions associated with the Rational Method are:

1. The computed peak rate of runoff at the design point is a function of the average rainfall rate during the time of concentration to that point.
2. The frequency or recurrence interval of the peak discharge is equal to the frequency of the average (uniform) rainfall intensity associated with the critical time of concentration (duration).

3. The time of concentration is the critical time of concentration and is discussed in part E of this section.

4. The ratio of runoff to rainfall, C, is uniform during the storm duration.

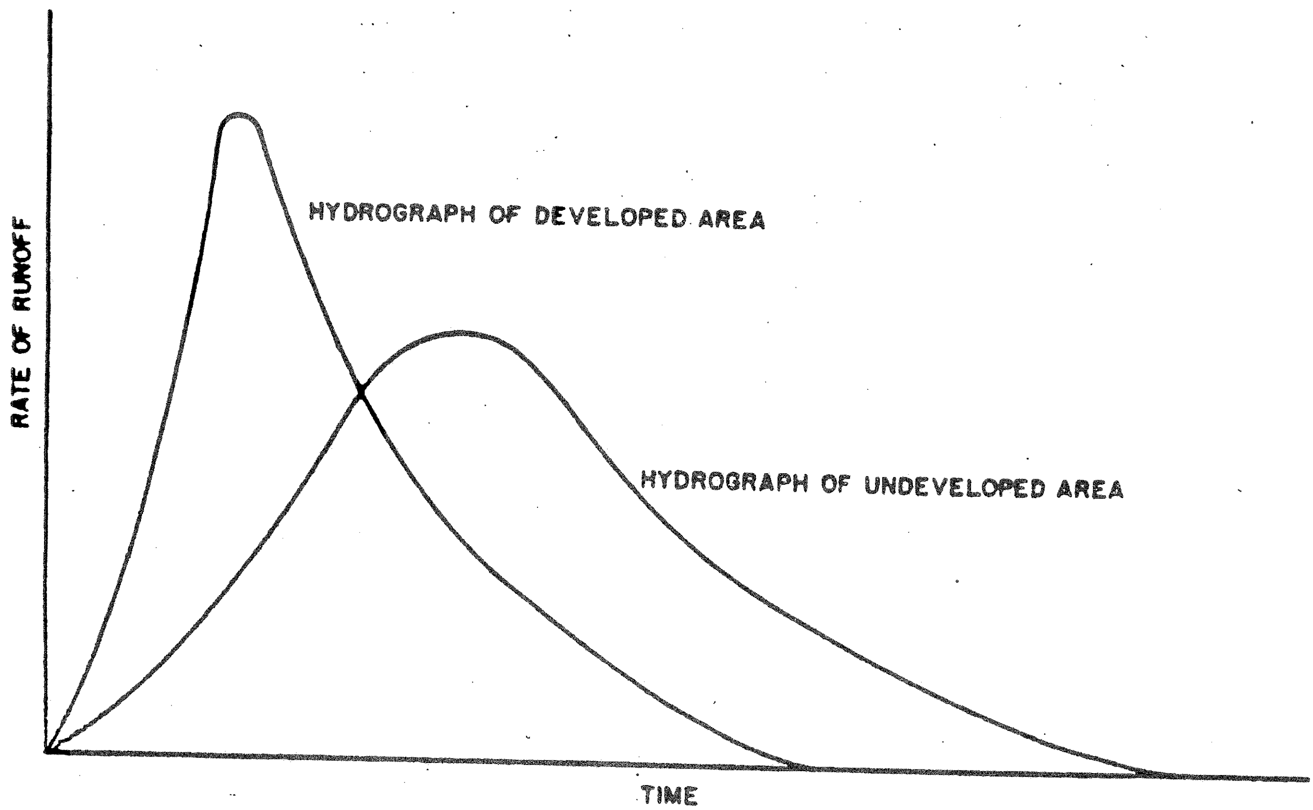
5. Rainfall intensity is uniform during the storm duration.

6. The contributing area is that area that drains to the design point within the critical time of concentration.

D. Runoff Coefficient (C)

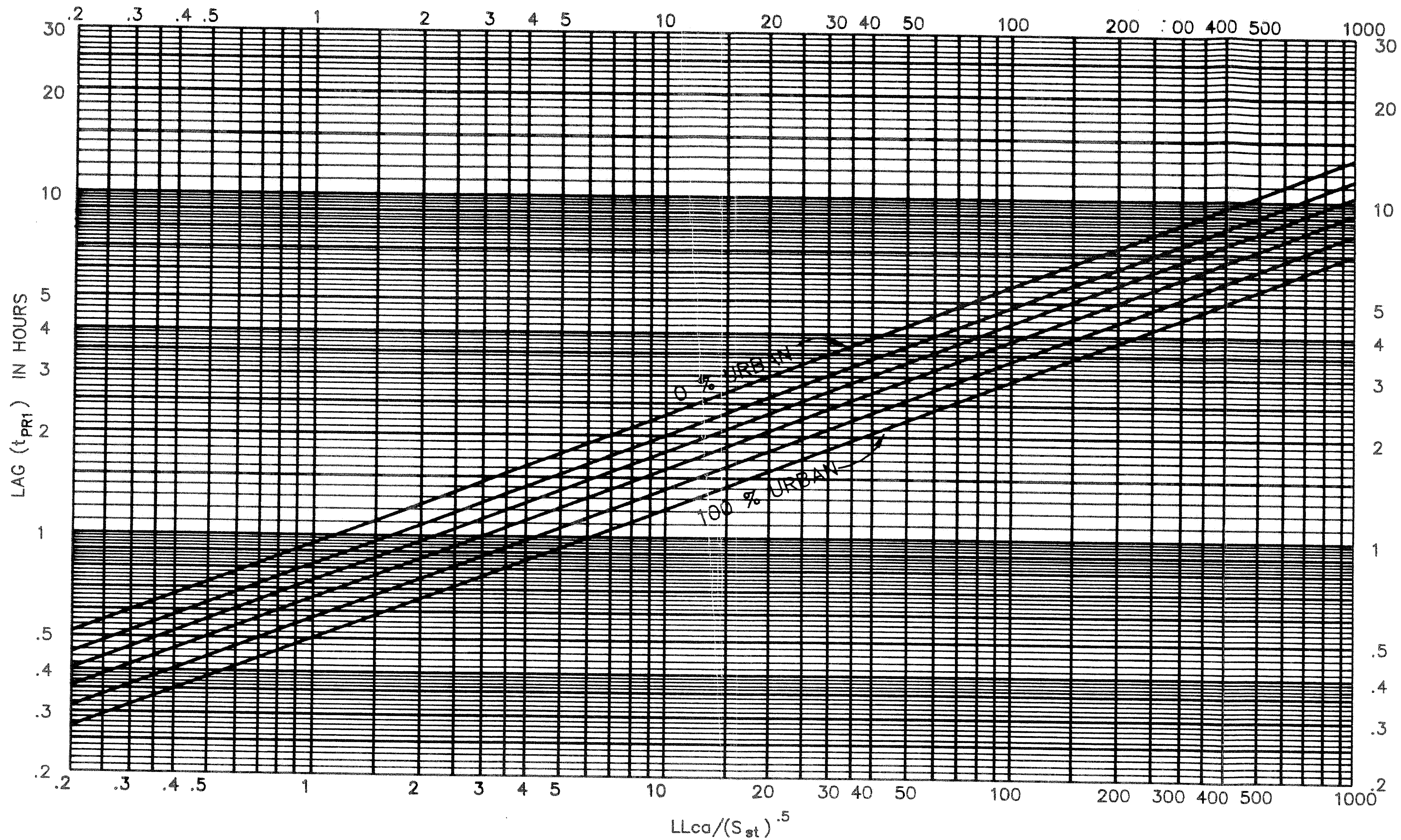
In relating peak rainfall rates to peak discharges, the runoff coefficient "C" in the Rational Formula is dependent on the character of the drainage area's surface. The rate and volume of a storm's rainfall that reaches an area's storm sewer system depends on the relative porosity (imperviousness), ponding character, slope and conveyance properties of the surface. Soil types, vegetation conditions and impervious surfaces, such as asphalt pavements and roofs of buildings are the major determining factors in selecting an area's "C" factor. The type and condition of the surface determines its ability to absorb precipitation and transport runoff. The rate at which a soil absorbs precipitation generally decreases as and if the rainfall continues for an extended period of time. The soil absorption or infiltration rate is also influenced by the presence of soil moisture before a rain (antecedent precipitation), the rainfall intensity, the proximity of the groundwater table, the degree of soil compaction, the porosity of subsoil, vegetation, ground slopes, depressions, and storage. Onsite inspections and aerial photographs may prove valuable in estimating the nature of the surface within the drainage area.

It should be noted that the runoff coefficient "C" is the variable of the Rational Method which is least susceptible to precise determination. Proper use requires judgement and experience on the part of the engineer, and its use in the formula implies a fixed ratio for any given drainage area, which in reality is not the case. A reasonable coefficient must be chosen to represent the integrated effects of infiltration, detention storage, evaporation, retention, flow routing, and interception, all of which affect the time distribution and peak rate of runoff.



EFFECT OF WATERSHED DEVELOPMENT
ON STORM HYDROGRAPH

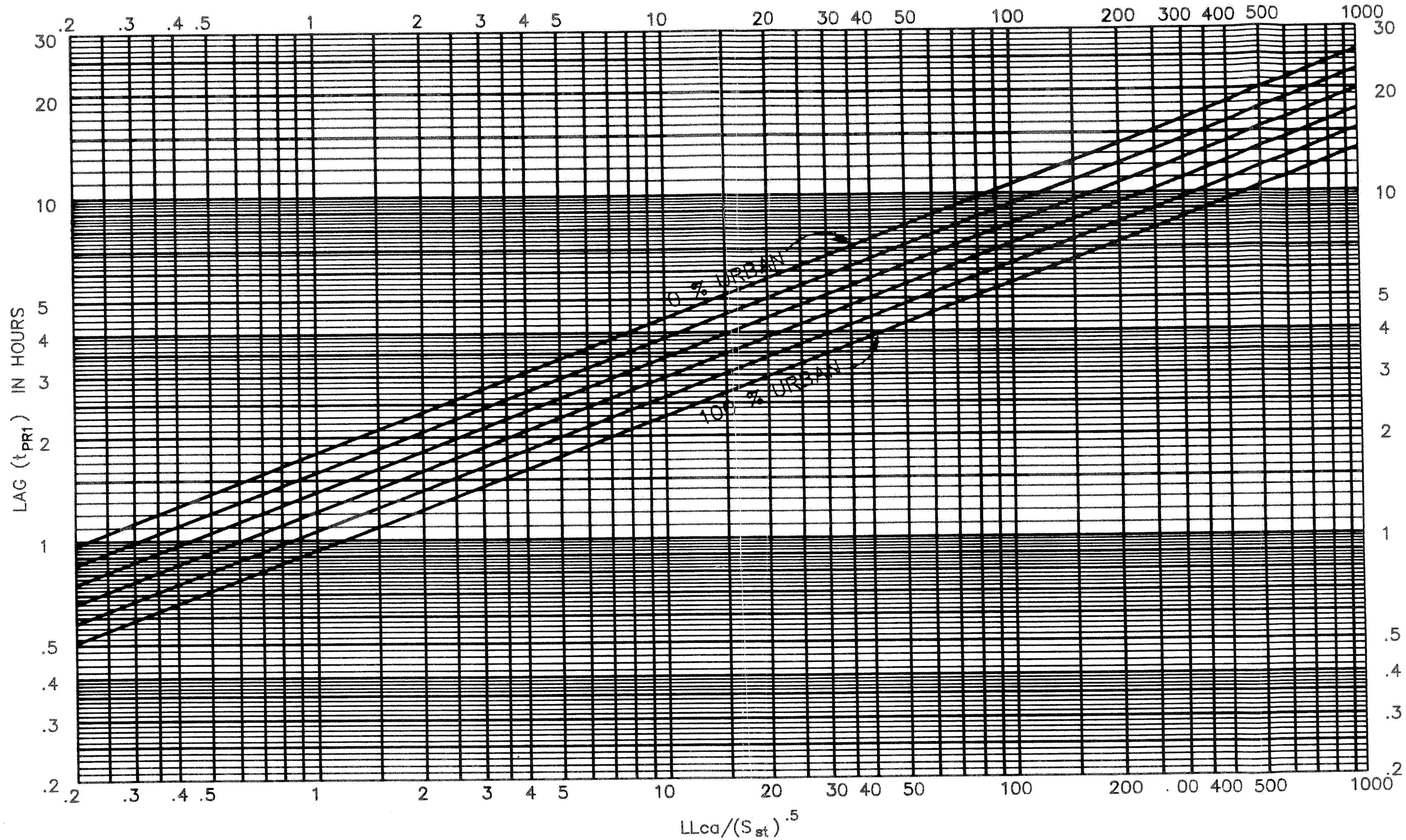
FIG. 2-1



ESPEY, HUSTON & ASSOCIATES, INC.
Engineering & Environmental Consultants

NUDALLAS
CLAY URBANIZATION
CURVE
FIG. 2-2

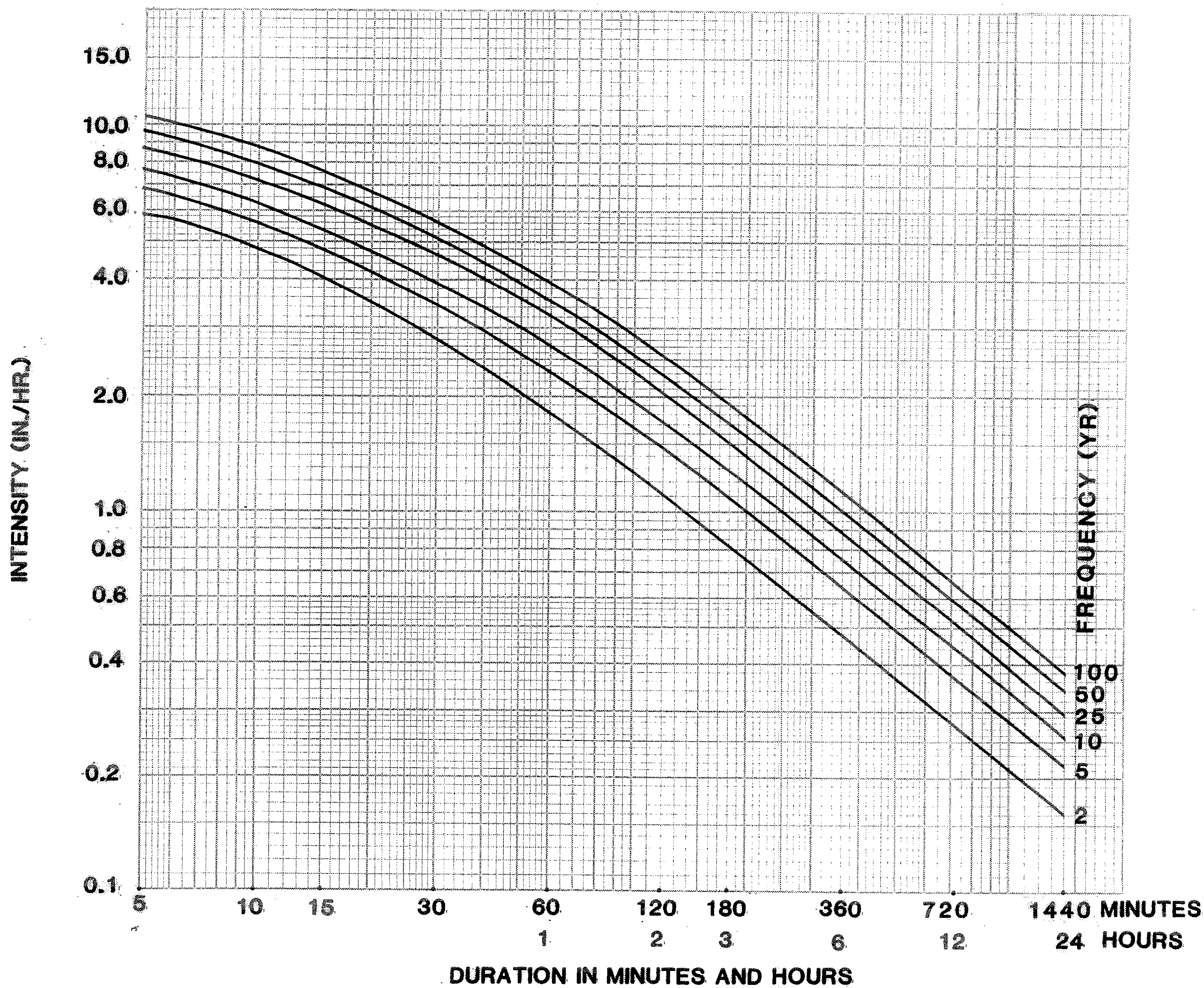
SOURCE: U.S. ARMY ENGINEER DISTRICT
FT. WORTH



ESPEY, HUSTON & ASSOCIATES, INC.
Engineering & Environmental Consultants

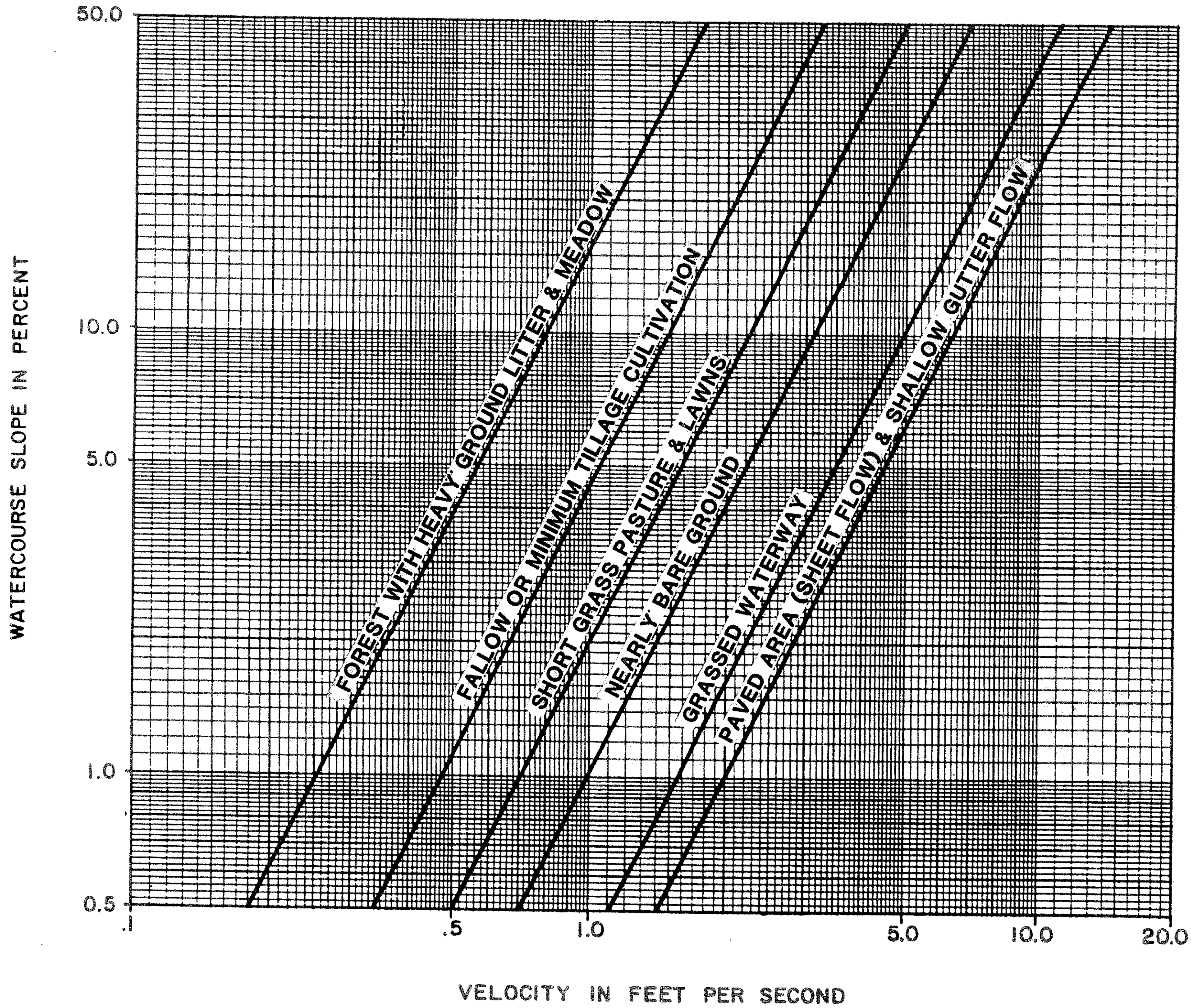
NUDALLAS
SAND URBANIZATION
CURVE
FIG. 2–3

SOURCE: U.S. ARMY ENGINEER DISTRICT, FT. WORTH



RAINFALL CURVES FOR LEWISVILLE, TEXAS

FIG. 2-4



AVERAGE VELOCITIES FOR ESTIMATING TRAVEL TIME
FOR OVERLAND FLOW

FOR LEWISVILLE, TEXAS

FIG. 2-5