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Development of General Guidelines for the Planning of Stormwater Management Facilities: Application to Urban Watersheds in Kentucky

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RESEARCH REPORT NO. 157

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ABSTRACT

This report provides a planning methodology and a design tool to help determine the appropriate location and volume of detention basins required to control critical storm events. The technique involves using watershed characteristics including the SCS curve number, time of concentration, peak outflow rate, watershed area and the storage recurrence interval to help predict these detention volumes.

Historical rainfall records are used in a revised continuous simulation program (SYNOP, Hydroscience, Inc.) to determine the rainfall excess from which runoff hydrographs are produced. Various combinations of the watershed characteristics were input and computer analyses done to obtain the required data base. A statistical analysis is performed in each computer analysis to obtain the statistics on the required volume. Graphs were drawn from these statistical results as functions of the watershed characteristics and the release rate. Entering the graphs with the governing watershed characteristics, the designer can obtain a good estimate of the detention basin volume required.

DESCRIPTORS : Planning, Storm Water, Urban Watersheds, Urban Runoff, Detention Reservoirs

IDENTIFIERS : Detention Basin, Planning Methodology, Design Methodology

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ERRATA

Phrase on page 23 "The safe release rate" should read "The safe release rate ratio"

Phrase on page 39 "greater than soil moisture capacity" should read "greater than initial abstraction"

CHAPTER 1

INTRODUCTION

1.1 Problem Definition

In the past few decades, increased urbanization has decreased the amount of pervious areas in urban watersheds. This decrease in pervious areas has had the effect of increasing peak flowrates and volumes from the developed watersheds. This has resulted in the revision of stormwater management policies and the development of many tools to deal with the problem. Some of the tools include storing the stormwater in detention basins and tanks, underground storage tunnels and watershed improvements (i.e. roof gardens, grassed parking lots, etc.). The most popular alternative continues to be the detention basin.

Most designers are currently using the detention basin to control peak flows while other effects such as downstream flooding and timing considerations are not typically analyzed. The engineer is also faced with the problem of determining the location and size of the basins along with the design approach to be used. The design approaches currently being utilized are the design storm and continuous simulation techniques. The most widely used approach continues to be the design storm technique.

In dealing with urban flooding, the engineer is usually faced with the problem of designing a structure which will control the postdevelopment outflow from urban areas. The standard regulations normally require the postdevelopment peak flow of some return interval to be no greater than the predevelopment peak flow of the same return interval. To satisfy this requirement, detention structures are typically built to control these peak flows. As a result, the watershed is broken down into numerous small subsheds each possessing a detention structure. This design technique is phrased the on-site design approach and has become very popular in the last decade.

In recent years the on-site design method has come under increasing criticism. Various authors have studied the possibility of the on-site, piece-meal approach not reducing flooding and in some cases increasing the flooding downstream (McCuen, 1979 and Duru, 1981). These studies show that the timing characteristics of runoff hydrographs can be significantly changed by the construction of detention basins. In the past, these timing effects have generally not been modeled.

The regional stormwater management policies enacted by many municipalities have proven to be the correct way to solve this problem. Smiley and Haan (1976) showed regional planning can help reduce the number of detention basins while obtaining the same results. Regional planning gives the municipalities the ability to control the stormwater

runoff problems before they arise. This reduces expenses and unnecessary construction while making the watershed operate as a system rather than many small systems operating seperately.

In order to develop a regional stormwater management plan for a watershed, a comprehensive planning methodology is required. In general, the methodology will involve two phases, a planning phase and a design phase. The planning phase can be completed by the municipality or agency responsible for the development of the watershed. This phase of the methodology will involve the establishment of flowrate requirements throughout the watershed. The second phase of the methodology is the design phase. This phase can be completed by the actual developer or design engineer. The design phase involves the design and implementation of the structures necessary to meet the flowrate requirements which were established in the planning phase.

1.2 Review of Planning Techniques

The comprehensive planning for control of stormwater runoff is becoming a significant part of detention basin designs. Planning of the watershed as a stormwater management system is the safest, most economic way to determine the placement and sizing of detention basins. Not only is the placement and sizing important but also the resultant hydrograph timing interactions within a watershed are important. McCuen (1974) showed that the "individual

site" approach for designing detention basins can actually increase flooding downstream.

Several authors have examined the on-site, piece-meal approach. Smiley and Haan (1976) showed, the piece-meal approach can be more detrimental than the regional approach. The study consisted of a watershed composed of 800 acres which is divided into seven subsheds of 100 acres each while the remaining 100 acres is divided into four subsheds of 25 acres each. A number of alternatives were modeled with different combinations of one detention basin per large subshed (100 acres). The study showed, the use of one detention basin in four of the subsheds produced the same results as using one detention basin in each of the seven subsheds. The economical advantage of using only four basins opposed to seven basins warrants approval for this method. Further studies by McCuen (1979) and Duru (1981) support these conclusions.

The Penn State Runoff Model (Lakatos, 1976) was developed to analyze the timing effects of multiple detention basins on the entire watershed. The watershed is broken down into subsheds with runoff calculations being performed in each subshed. The model allows for both spatial and temporal rainfall variations to be used to account for a network of rain gauges or moving storms in the subsheds. Hydrographs from the subsheds are combined to form composite hydrographs from which stormwater management policies can be adopted.

Hawley et al. (1981) developed a planning methodology which does not require the actual design of a detention basin to determine the effects it will have on downstream areas. He showed, as long as a detention basin meets the requirement for peak flow discharge it will produce approximately the same outflow hydrograph as any other basin meeting the peak flow requirement. The method involves estimation of direct runoff hydrographs and detention basin outflow hydrographs. The SCS TR-55 (1975) graph method is used to estimate discharge rates while empirical timing equations are used to estimate the time coordinates. The interaction of the outflow hydrographs allows the engineer to determine if a detention basin is needed.

Zeigler and Lakatos (1982) made use of the Penn State Runoff Model to devise their planning technique. An analysis of both the existing watershed and the watershed with developments is required. The control of the critical design event and the more frequent 2-year and 10-year events along with the volume, placement, and timing considerations are all considered in determining the optimal plan.

Cherry et al. (1982) developed a planning technique to determine the allowable release rate of subbasins in a watershed. The "release rate percentage concept" can be used by municipalities when developing stormwater management policies and help predict problem areas for future development. The contribution of each subbasin to the peak flow of the watershed is divided by the peak flow of the

subbasin to obtain the release rate percentage. This percentage is used by the developers in each subbasin to determine the peak flow requirement which must be met. This method is advantageous because it considers the effects of development on the areas downstream in the watershed.

1.3 Review of Simple Design Techniques

As urban development has increased, the use of detention basins for flood control has become very popular. This has produced a variety of methods to estimate the volume of detention needed to control flooding. Generally, standards require the peak outflow rate after development to not exceed the peak outflow rate prior to development. Keeping this in mind, the engineer can design a detention basin which will not violate these standards. The design techniques, from a flood control standpoint, usually include:

1) The selection of a design storm.

2) The computation of the design inflow hydrograph resulting from the design storm.

3) The determination of the predevelopment peak outflow rate.

4) The determination of the minimum required volume of storage.

5) The sizing of the principal and emergency spillways in order to not violate the predevelopemnt peak outflow rate requirement.

In the past, several authors have presented various preliminary design procedures for detention basin design.

The Rational Formula has become a very popular method for use in detention basin sizing design. The four variables used in the rational method are the time of concentration (t_c), the runoff coefficient (C), the rainfall intensity (i) and the return period (T). The formula is represented by equation 1.1 and was originally developed to determine the peak flow resulting from a selected design storm.

$$Q = CiA \tag{1.1}$$

Q = peak flowrate (cfs)

A = area (acres)

Recently, many engineers have extended the rational method to detention basin design. The popular way to use the formula is to determine the change in runoff coefficient from predevelopment to postdevelopment. This change is then used in the formula to determine the increase in peak runoff. Hydrograph shapes are assumed and the peak flowrates are used to develop the hydrographs. The increase in runoff volume is then used for the detention volume.

The method makes four assumptions: 1) the recurrence interval of the peak flowrate is the same as that of the rainfall intensity, 2) the rainfall is uniform in space over the drainage area being considered, 3) the rainfall intensity is uniform throughout the duration of the storm and 4) the storm duration associated with the peak flowrate is equal to the time of concentration of the drainage area (Rossmiller, 1982).

Several problems arise when engineers try to expand the use of the method to detention basin sizing. Popular time of concentration formulas do not always include all the components of flow such as overland flow and channel flow and the C coefficient is difficult to determine accurately. Finally, this formula is a peak flowrate formula and is not intended to be used as a hydrograph generation formula.

The Soil Conservation Service (1975) developed a method to determine the required storage volume as a function of the watershed runoff volume and the release rate. Figure 1.1 applies to pipe drop inlets of 0 to 300 csm (cubic feet per second per square mile) release rate and weir flow structures of 0 to 150 csm release rate. Figure 1.2 applies to pipe drop inlets of over 300 csm release rate and weir flow structures of over 150 csm release rate. This method is limited because it applies only to 24 hour rainfall events with Type II rainfall distributions. The curves shown also limit the method because extrapolation can cause significant error.

Wycoff and Singh (1976) developed a method to determine the required storage for a given inflow hydrograph. The method gives an estimate of the storage required but does not require numeric reservoir routing computations. The method requires the peak inflow and outflow rates, time of base, time to peak and the volume of the inflow hydrograph be input. The relationship between these parameters is found through linear regression. Therefore, a new set of

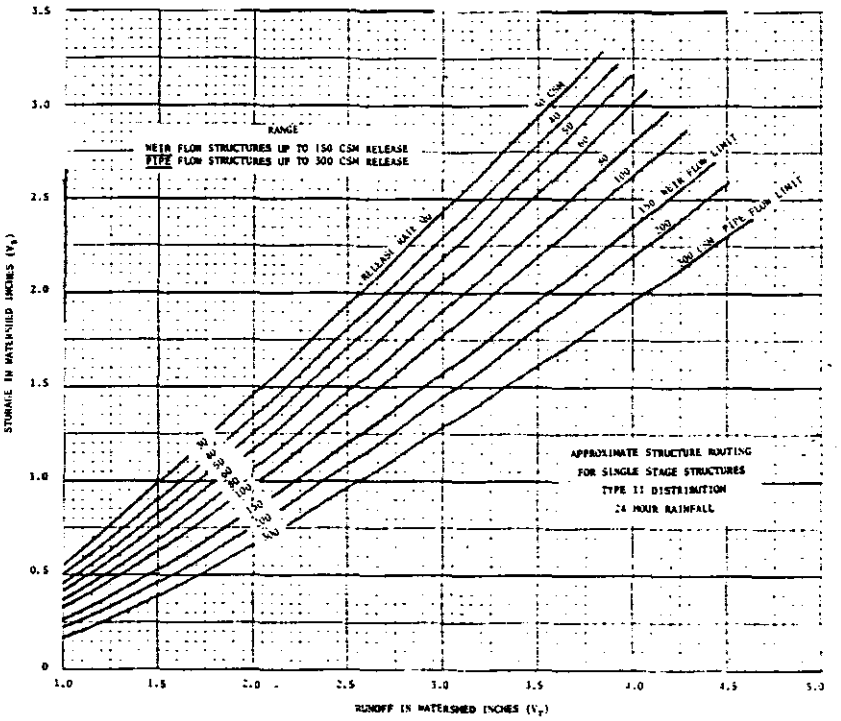

Figure 1.1 Single stage structure routing - low release.

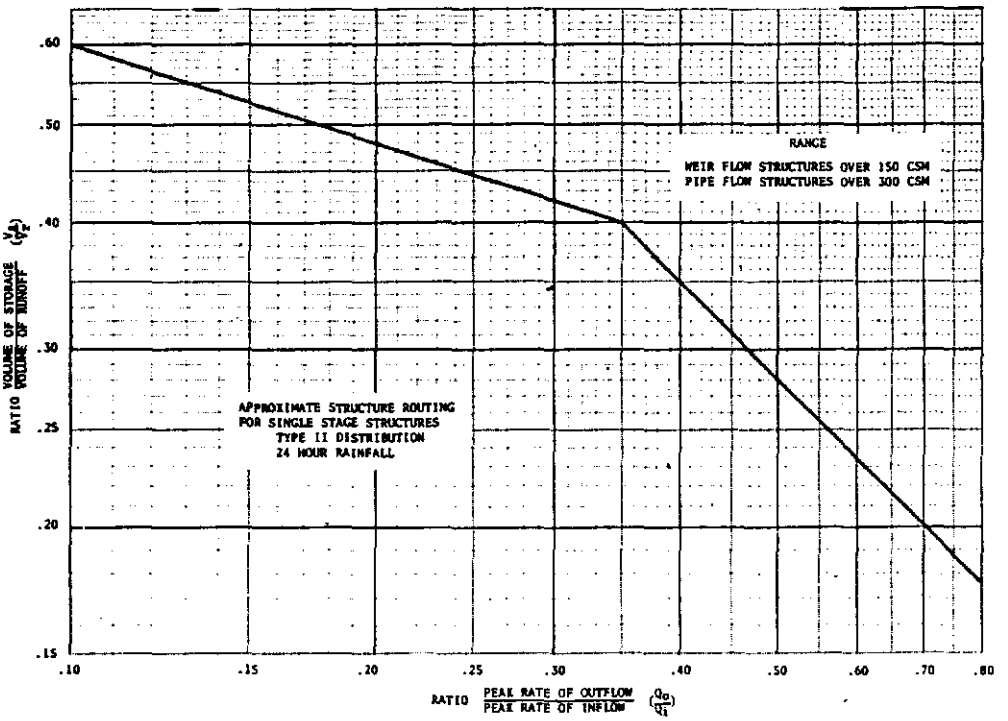

Figure 1.2 Single stage structure routing - high release.

parameters is required for each watershed.

Bouthillier and Peterson (1978) also developed a simple method to determine the storage volume required for stormwater runoff. The method assumed two differently shaped hydrographs, triangular (Figure 1.3) and cusp (Figure 1.4) shaped. Rainfall durations and frequencies were assumed while intensities were taken from the intensity-duration-frequency curves for the 5, 10 and 25 year events. Various runoff hydrographs were routed through a reservoir having a fixed base area, side slopes of 4 to 1 and a bottom pipe outlet. The principal spillway was assumed to be a circular pipe with the outflow rate proportional to the product of the area of the outlet and the square root of the head on the center of gravity of the pipe. Variables in the reservoir design included reservoir base area, reservoir height and reservoir outlet area while the peak inflow rate and rainfall duration could also be varied. The inflow hydrograph and the equation of continuity along with the reservoir and outlet dimensions were used to determine the outflow hydrograph.

Assuming the outflow rate at time step two, the storage at time step two can be found. The future outflow rates and storages were found using a converging technique. Various computer runs were done producing the required storage volume (RSV), total inlet volume (TIV), maximum allowable outlet rate (MAOR) and maximum inflow rate (MIR) for each run. A plot of MAOR/MIR verses RSV/TIV (Figure 1.5) is

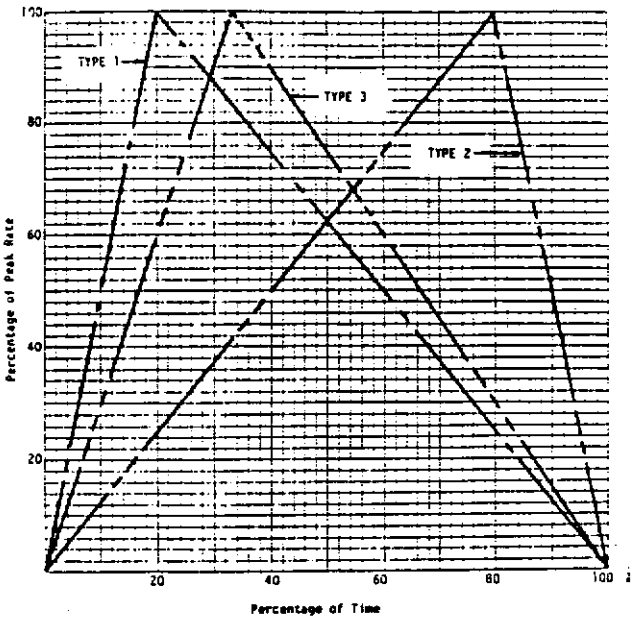

Figure 1.3 Triangular inflow hydrographs.

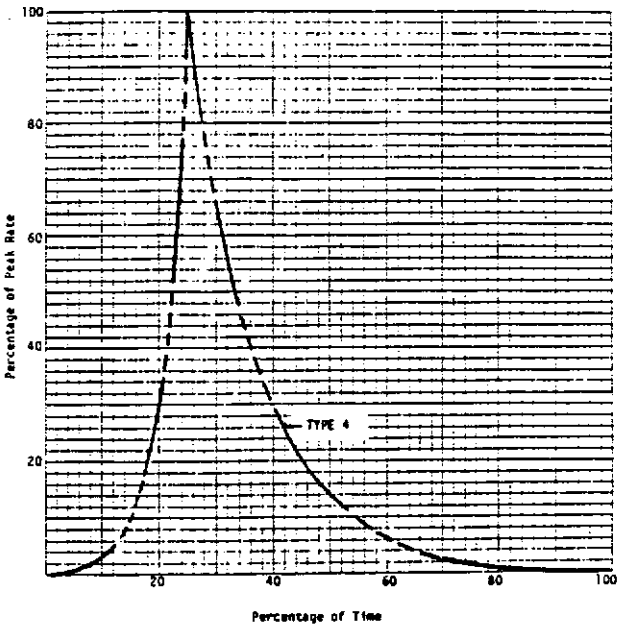

Figure 1.4 Cusp shaped inflow hydrographs.

obtained. For any design, MAOR, MIR and TIV are all known and RSV can be obtained. Results indicated the ratio RSV/TIV to MAOR/MIR is dependent on the inflow hydrograph shape and is independent of the peak inflow rate, storm duration and storm volume.

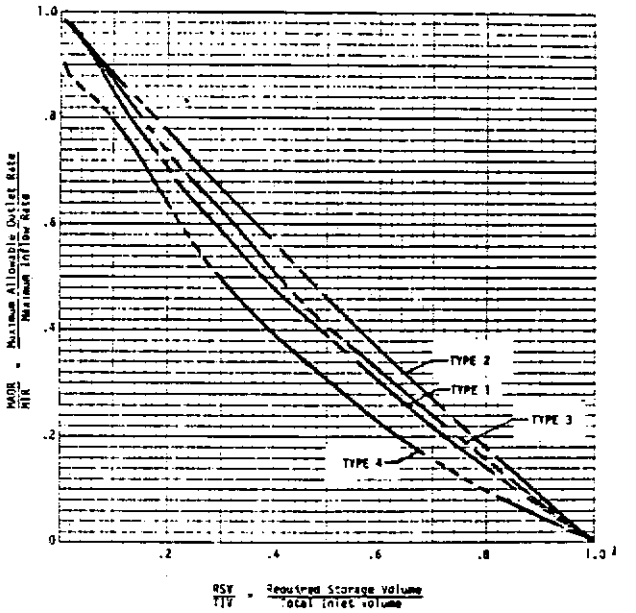

Figure 1.5 Composite plot of ratio of MAOR/MIR vs. RSV/TIV.

The major assumption in this technique is the time base of the hydrograph is assumed equal to the duration of the storm. Normally, the hydrograph extends much farther past the end of the storm. Typical of most design storm techniques, the approach does not consider antecedent conditions.

Paintal (1979) devised a technique for estimating detention storage as a function of drainage area and maximum allowable outflow rate. He assumed the rainfall duration to be a function of intensity but storms with durations longer

than the time of concentration were assumed to produce larger volumes even though the peak flows are smaller. The rising and receding limbs of the inflow hydrograph are set equal to the time of concentration and the peak flowrate is found using the rational formula. Using a trapezoidal inflow hydrograph and an assumed constant outflow rate or orifice cotrolled outflow rate, he found the required volumes and critical storm durations with simple calculus procedures.

The method uses a design storm approach without any consideration of the antecedent conditions. It also uses a relationship between the intensity, the rainfall duration and two constants. These constants are used to determine the time of concentration but require a regression analysis. The peak flow rate used in the method is computed with the rational method which is subject to some question.

Raasch (1979) developed a continuous simulation program to help design the detention basin size. The technique uses historical rainfall records along with a required time between events and a constant release rate. A continuous account of the records is kept and an event is detected when the rainfall rate is greater than the release rate. A statistical analysis is done on the largest annual storage volumes to determine the probabilities of occurrence for the various storage volumes. Graphs were obtained for four (10, 25, 50 and 100-year) recurrence intervals using a log Pearson Type III distribution. Each graph represents the

storage volume as a function of the recurrence interval, the outlet release rate and the time between events. The results showed significant variation between results obtained by other methods although indications show overall acceptability of the technique.

An approach developed by Ellis et al. (1981) combined the advantages of a continuous simulation approach but at the same time reduced the need for computer time. The method uses historical rainfall data and searches each year of record for the three rainfall events with the highest peak, the highest volume and the highest combination of the two. These events are then routed downstream to obtain the peak annual flows. Curves are developed which are functions of the percent impervious land, the recurrence interval and either the peak flow rate or the peak volume.

The results show that the runoff event causing the annual maximums often change as the percent imperviousness changes. This was shown to be caused by the prevailing antecedent moisture conditions. The authors showed that some events undergo dramatic changes due to an increase in imperviousness while others do not. In many instances the rainfall pattern producing the peak runoff rate did not produce the peak volume, supporting previous authors doubts. The disadvantage of this method is the use of only two or three rainfall events per year for the analysis, resulting in the reduction of computer time but yielding to the possibility of a multiple storm combination or a longer

storm duration of lower intensity not being considered.

Finally, Urbonas and Glidden (1982) developed simplified " on-site detention " storage volume and discharge requirement equations for controlling peak flows along major waterways. Their analysis focused on the Denver metropolitan area and considered control volumes and release rates from directly tributary subbasins.. The resultant 10-year and 100-year control volumes and release rates were then used to develop volume-discharge functions for each of 27 ponds in the area. Regression analysis produced simplified control volume and release rate equations for the area. The simplified equations were used to size detention basins on each of the areas where ponds already existed and the results were compared to the rigorous individual site sizing analysis. The results showed a smaller overall volume requirement using the simplified approach although some individual basins were larger using this approach. The method is confined to the 10-year and 100-year events and does not consider downstream timing effects from the individual basins.

1.4 Review of Design Approaches

In order to implement either a general planning or design technique some type of hydrologic input is required. The hydrologic input for any such technique is actually stochastic in nature. The random nature of this input may be considered by two different design approaches, the design

storm approach and the continuous simulation approach. These techniques are discussed in the following two sections.

1.4.1 The Design Storm Approach

The design storm approach is the technique most widely used today for hydrologic design. In using the design storm approach a structure is designed based on a synthetic hyetograph which is derived from a specified frequency and storm duration. In many cases the storm duration is assumed to be equal to the time of concentration of the watershed.

The total volume or average rainfall intensity associated with a particular design storm is usually obtained from tables or figures which were constructed from data obtained from a statistical analysis of historical rainfall records. One way which such data is typically presented is using an intensity-duration-frequency (IDF) curve. A typical IDF curve for Lexington, Kentucky is shown in Figure 1.6. The IDF curve can be used to predict the average intensity of a design storm by entering the graph with the desired duration and frequency.

In constructing a synthetic design storm, a uniform or variable rainfall distribution may be assumed. The rational method assumes a uniform distribution. Although this method is very simple and straight forward to use, some questions have been raised about the validity of the constant intensity assumption. Some authors have proposed a

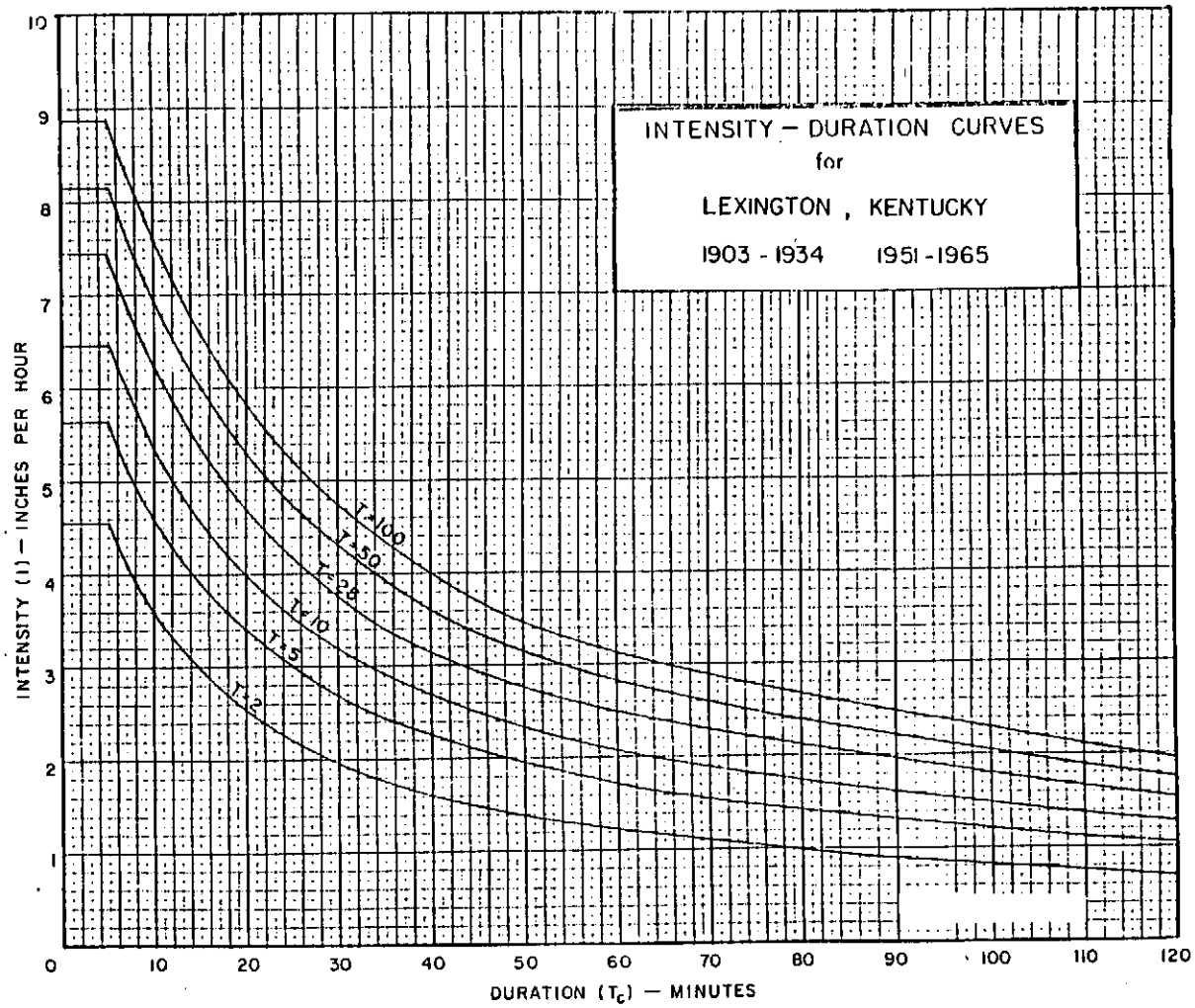

Figure 1.6 Intensity-Duration-Frequency Curve for Lexington, Kentucky.

synthetic distribution of intensities for rainfall events. Keifer and Chu (1957) developed a synthetic distribution for Chicago. Huff (1967) looked at a number of recorded storm distribution patterns from small watersheds in the central Illinois. He divided these patterns into four probability groups from the most intense (first quartile) to the least intense (fourth quartile). Given the quartile and volume of rainfall, the rainfall distribution can be determined. Finally, the U. S. Department of Agriculture, Soil Conservation Service (1979) has developed 24-hour and 6-hour rainfall distributions for use in developing runoff hydrographs.

The design storm approach has a number of limitations. The biggest limitation is that the method usually assumes that the probability of the peak runoff is the same as the probability of the input total precipitation (James and Robinson, 1982). In addition, the design storm does not yield any information about the conditions prior to the flood event, e.g. soil moisture, infiltration rate, water level in the pond, etc. and lacks the ability to model the effects of back to back storms.

1.4.2 Continuous Simulation Approach

The alternative to the design storm approach is the continuous simulation approach. The technique is called continuous simulation because it considers the time response of historical rainfall records. It is advantageous because

historical data can be used and the effects of antecedent moisture conditions can be modeled. This method can give statistical information on the entire historical record while the design storm technique cannot. The effects of smaller multiple storms can be modeled which may prove more critical than the single storm approach. However, the continuous simulation technique does have some disadvantages. The inability to obtain historical rainfall records and the cost of long computer runs can pose problems.

As a result, several authors have proposed techniques which improve on the design storm technique without the complete use of continuous simulation. Walesh et al. (1979) obtained the major rainfall events from historical rainfall data along with their respective hyetographs and antecedent moisture conditions. This data was used in an event model to determine runoff hydrographs. A statistical analysis was done on the resulting peak discharges and volumes to obtain discharge-probability and volume-probability relationships. This method has the advantage of strong statistically based results without the need to run a continuous simulation analysis.

Another study by Gofarth et al. (1981) applied a continuous simulation model to 26 years of rainfall data. The computer program SYNOP (Hydroscience, Inc., 1979) is applied to the data to obtain statistics for the events. The single year of record which most closely fits the

statistics of the 26 year statistics is then used as input for subsequent modeling.

1.5 Survey of Methods used in Kentucky

In order to determine the types of planning and design methodologies currently employed in the state of Kentucky, a survey was conducted of approximately 20 municipalities. This study indicated that the majority of the cities had no specific guidelines in relation to the planning and design of stormwater management facilities.

Of those cities that did possess some type of guidelines there was a wide variation in recommended techniques and standards. A brief summary of some of the different design standards is provided in Table 1.1.

1.6 Scope of the Present Study

In light of the results of the survey, this study has been conducted in an attempt to provide municipalities across the state with some general guidelines as well as design techniques for use in the planning and design of stormwater management facilities. This study recommends the use of a watershed or systems approach as opposed to an on-site approach. In order to accomplish this objective a comprehensive planning and design methodology has been developed.

Table 1.1 Typical design methods in Kentucky

City	Design Standard
Louisville	100 year/ 1 hour Surface Grav. System 100 year/ 3 hour Pump/Force main/sink-hole
Lexington	Postdevelopment peak runoff = predevelopment peak runoff 10 year/ 1 hour, 25 year/ 24 hour, and 100 year/ 1 hour storm
Frankfort	Estimate of quantity of stormwater entering the new development using the 10 year storm
Hopkinsville	Rate of stormwater leaving site shall not be significantly different than if the sight had remained undeveloped 100 year/ 3 hour storm
Owensboro	At least 100 year storm for emergency spillway design
Winchester	Postdevelopment peak runoff = Predevelopment peak runoff 100 year/ 24 hour storm

CHAPTER 2

PLANNING METHODOLOGY

2.1 Development of Proposed Methodology

The proposed methodology utilizes the release rate percentage concept developed by Cherry et al. (1982) for its foundation. The concept is used because it is easy to apply and considers the effects of development on the downstream portions of a watershed. The proposed methodology requires the municipalities to analyze the subbasins in the watershed as a system and develop comprehensive plans for the entire watershed. The comprehensive plans should provide the developer with the exact standards he must meet when developing a subbasin. The technique relies on the concept that it is not the peak runoff which is important but the runoff contributing to the watershed peak prior to development.

Before an analysis can be performed, control points in the watershed must be selected by the municipality. Control points are culverts, bridges, subbasin outlet points, or the watershed outlet where flooding must be controlled. After the control points are selected, the municipality determines predevelopment hydrographs for each subbasin and routes them through the watershed in order to generate hydrographs at the selected control points. Figure

2.1 shows a typical watershed hydrograph along with the contributing subbasin hydrographs. The figure shows that each subbasin hydrograph contributes an amount of runoff to the peak runoff rate of the watershed. The contribution of the hydrograph from subbasin 3 is shown in Figure 2.2. In this figure, A represents the peak flowrate from subbasin 3 and B represents the contribution to the watershed peak from subbasin 3. The release rate percentage concept recommends using this contributing amount for the design of detention facilities and not the peak runoff rate for the subbasin.

The safe release rate for a subbasin can be determined by calculating the ratio of the subbasin runoff contributing to the watershed peak (Point B) to the predevelopment peak runoff rate (Point A). The subbasins with the lower ratios represent more hydraulically sensitive areas. These ratios are used by the municipalities to set the required release rates for each subbasin in the watershed.

After the preliminary analysis is made by the municipality, the developer must route postdevelopment hydrographs through the watershed. If the postdevelopment peak exceeds the predetermined release rate set by the municipality, a detention facility is required. The complete planning methodology may be summarized in the following five steps. Steps 1 and 2 apply to the municipality while steps 3 through 5 apply to the developer.

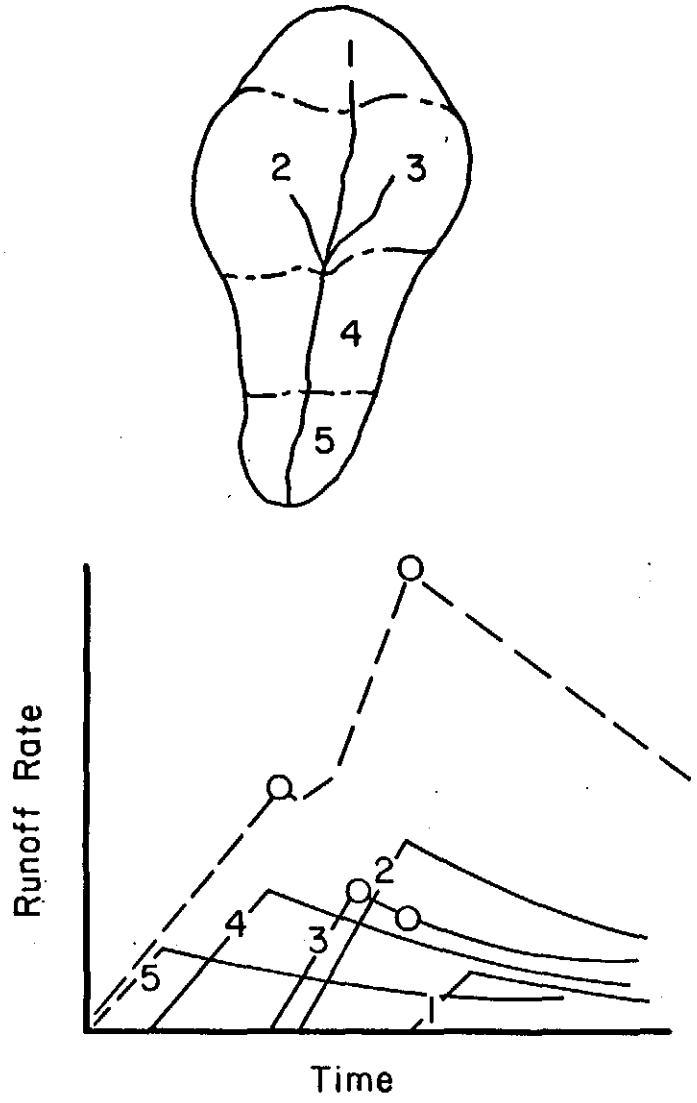

Figure 2.1 Typical watershed hydrograph

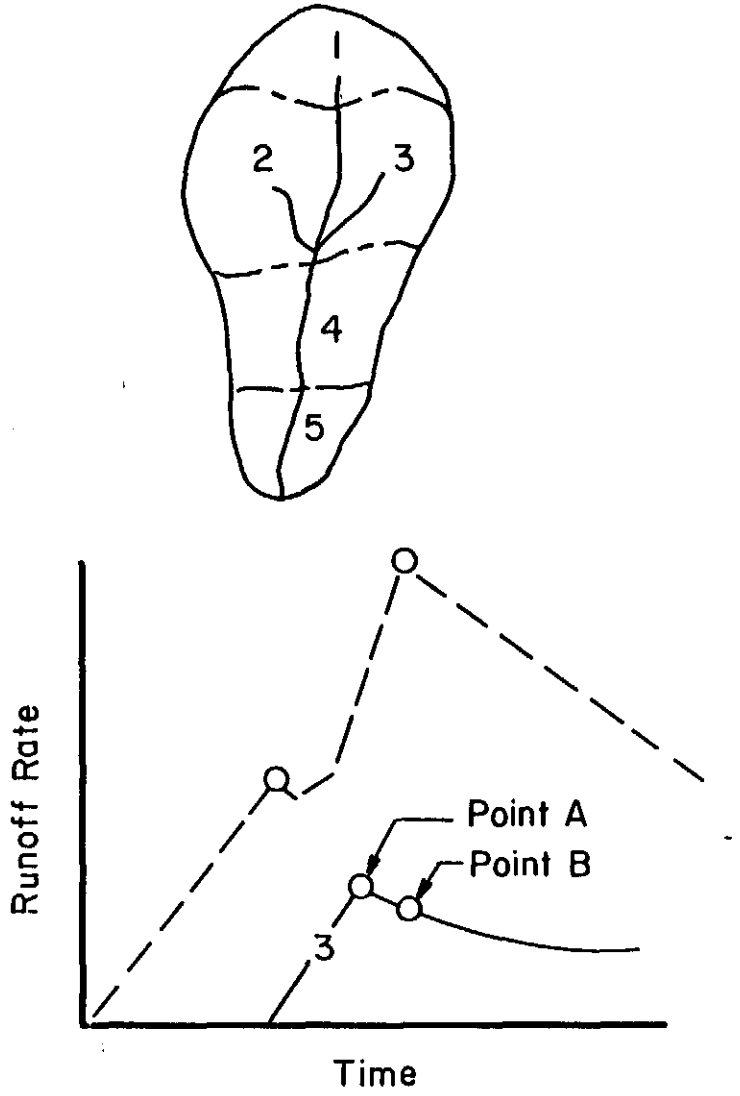

Figure 2.2 Contribution of subbasin to watershed peak

MUNICIPALITY

1) Determine points of interest in the watershed (culverts, bridges, problem areas which flooding has previously occurred, etc.).

2) Determine predevelopment hydrographs for the subbasins and route them to the control points. Determine the release rates for each subbasin using the previously defined method.

DEVELOPER

3) Compute and route postdevelopment hydrographs to the control points and determine the peak flows. If the peak flows exceed the release rates set by the municipality continue to step 4.

4) Apply on-site management techniques to increase infiltration and reduce impervious surfaces. Recompute postdevelopment discharges and if they are still greater than predevelopment discharges, detention facilities are required.

5) Use the subbasin release rates and the other hydrologic characteristics of the subbasin to design a detention facility.

This methodology is an advantage to the municipality because they know prior to development what peak flowrate must not be exceeded. It is an advantage to the developer because it makes it easier on them to determine the size of the detention basin required. Finally, it is an advantage to the residents because the watershed is treated as a system. This reduces the possibility of flooding downstream of development.

2.2 Development of Hydrographs

Several methods exist for generating hydrographs. These include unit hydrograph methods, synthetic unit

hydrograph methods, such as the SCS method and Snyder's method and linearized subhydrograph methods. This report recommends using the linearized subhydrograph method (Sarikelle et al., 1978). This method uses the watershed area, the time of concentration, the rainfall duration and the excess rainfall intensity to calculate the hydrograph. The rainfall excess is considered to be continuous and evenly distributed over the entire watershed. The three possibilities which may exist are 1) the time of rain is less than the time of concentration (Figure 2.3), 2) the time of rain is equal to the time of concentration (Figure 2.4) and 3) the time of rain is greater than the time of concentration (Figure 2.5). The three possible cases are illustrated by the following equations which provide the general shape for each condition.

Case I: $t_r < t_c$

$$q_p = i_e A[2t_r/(t_r + t_c)] \quad (2.1)$$

$$t_b = t_r + t_c \quad (2.2)$$

$$V = i_e t_r A \quad (2.3)$$

Case II: $t_r = t_c$

$$q_p = i_e A \quad (2.4)$$

$$t_b = 2t_r \quad (2.5)$$

$$V = i_e t_r A \quad (2.6)$$

Case III: $t_r > t_c$

$$q_p = i_e A \quad (2.7)$$

$$t_b = t_r + t_c \quad (2.8)$$

$$V = i_e t_r A \quad (2.9)$$

Typical Impulse Hydrographs

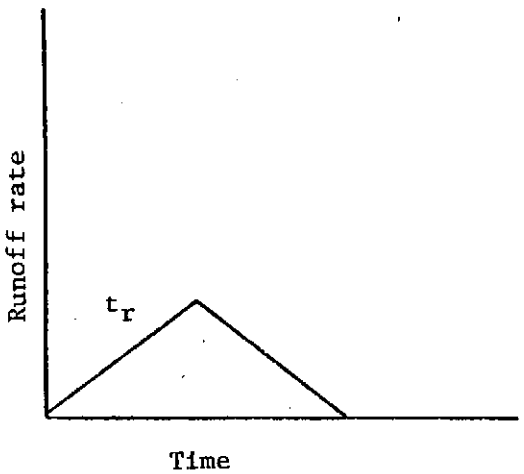

Figure 2.3 $t_r<t_c$

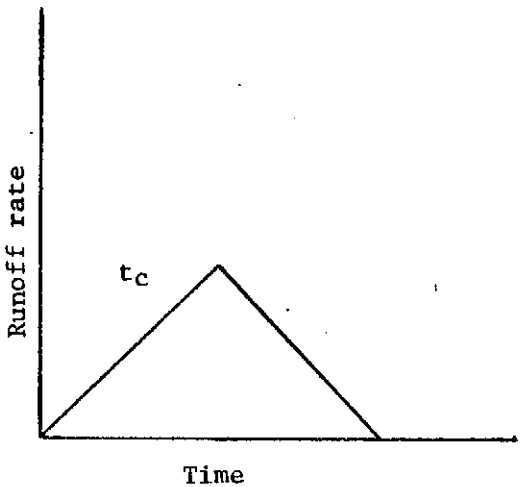

Figure 2.4 $t_r=t_c$

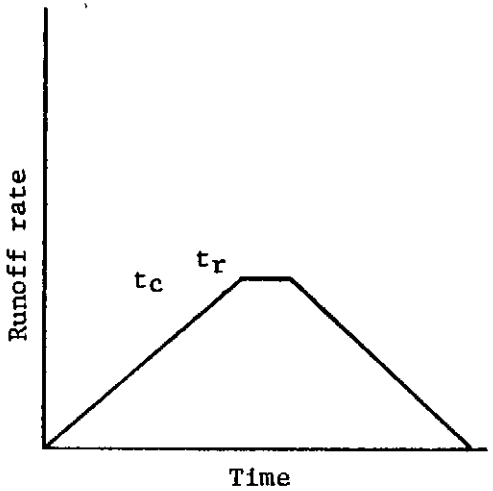

Figure 2.5 $t_r>t_c$

q_p = peak outflow rate (cfs)

i_e = effective rainfall intensity (inches/hour)

A = area of watershed (acres)

t_r = time of rain (minutes)

t_c = time of concentration (minutes)

V = volume of runoff (acre-inches)

The subbasin hydrographs can be computed using equations 2.10 through 2.14. The required data is the subbasin area, the time of concentration, the rainfall duration and the excess rainfall hyetograph.

$t_r < t_c$:

For $t < t_r$ $Y_t = Ai[2t_r/(t_r + t_c)]t/t_r$ (2.10)

For $t > t_r$ $Y_t = Ai[2t_r/(t_r + t_c)](t_r+t_c-t)/t_c$ (2.11)

$t_r > t_c$:

For $t < t_c$ $Y_t = Ai(t/t_c)$ (2.12)

For $t_c < t < t_r$ $Y_t = Ai$ (2.13)

For $t > t_r$ $Y_t = Ai(t_r + t_c - t)/t_c$ (2.14)

where

t = time after start of storm (minutes)

Y_t = ordinate of hydrograph at time t(acre-inches/hour)

t_r = duration of rainfall (minutes)

t_c = time of concentration (minutes)

i = excess rainfall intensity (inches/hour)

A = area of the watershed (acres)

2.2.1 Methods for Determining the Time of Concentration

The time of concentration is the amount of time it takes a raindrop to travel from the most remote area of the watershed to the outlet of the watershed. This variable is critical and must be calculated as accurately as possible. Two accepted methods for determining the time of concentration are the Kerby-Kirpich method (1940) and the Izzard method (1946). These two techniques are certainly not the only accepted methods, however, they are recommended because they consider the different flow processes such as overland flow, shallow channel flow and open channel flow. The techniques and their applicability are discussed below.

2.2.1.1 Kerby-Kirpich Equation

The Kerby-Kirpich equation is a good equation to use to approximate the time of concentration. This equation considers both the overland flow component and the channel flow component. These two components usually compose the majority of the runoff process. This method is popular because it is easy to use and the watershed characteristics are easy to determine. The time of concentration is represented by equation 2.15. The roughness value to be used in equation 2.15 can be found in Table 2.1.

$$t_c = 0.0078L_c^{0.77}S_c^{-0.385} + (2nL_o/3\ S_o)^{0.467} \quad (2.15)$$

where

L_c = length of channel (ft.)

S_c = slope of channel (decimal)

n = roughness coefficient (shown in Table 2.1)

L_o = length of overland flow (ft.)

S_o = slope of overland flow (decimal)

Table 2.1 Typical roughness values

Typical Surface	Roughness Factor
smooth impervious surface	0.02
smooth bare packed surface	0.10
pasture or average grass	0.40
forests	0.80

2.2.1.2 Izzard Method

Izzard developed a technique which estimates the flow velocity for overland flow and shallow channel flow. Flow velocities for open channels can be estimated using Manning's equation by assuming full channel flow. The time of concentration can be determined by summing the flow times in each segment. This is represented by equation 2.16.

$$t_c = \Sigma L_i/V_i \qquad (2.16)$$

where

t_c = time of concentration

n = number of segments

L_i = length of flow segment

V_i = average velocity in the flow segment

Figure 2.6 can be used for the estimation of flow velocities in the overland and shallow channel flow segments. This figure should not be used for the open

channel velocity determination. This method is popular because it is easy to use although the average open channel flow velocity determination may be difficult to determine. This is because a typical channel geometry must be determined which will require an assumption or direct measurement from a topographic map.

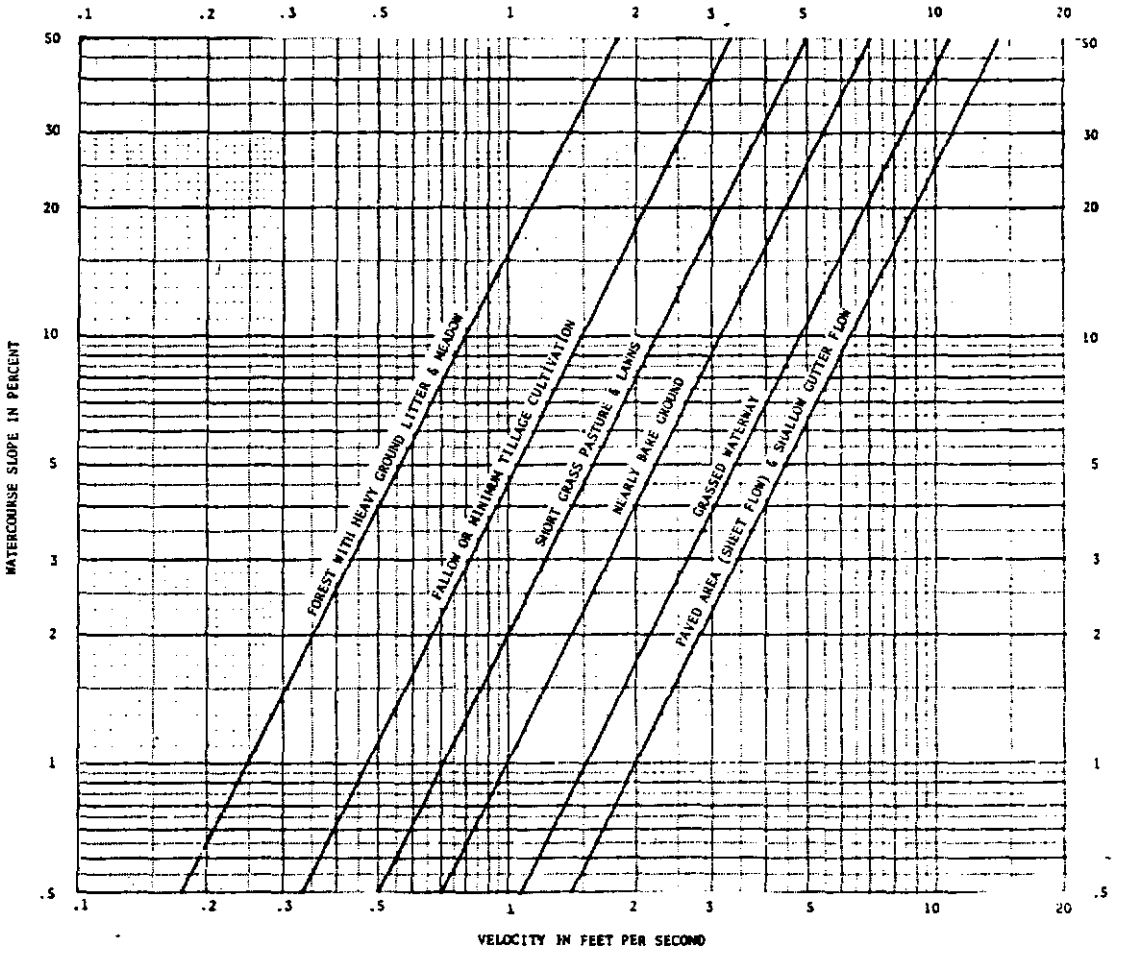

Figure 2.6 Average velocities for estimating travel time for overland flow.

2.2.2 Determination of Rainfall Hyetographs

In order to apply the methodology to Kentucky a series of rainfall hyetographs were obtained. The hyetographs were

obtained by determining the rainfall events from the historical data which produced the maximum flowrates. It was determined that two storms were responsible for the peak flowrates produced with a variety of watershed characteristic combinations. These storms had durations of three and six hours respecively. The dimensionless graphs of the cumulative percent of storm duration verses the cumulative percent of storm volume for these two storms and the typical storm volumes associated with the storm frequency and antecedent condition is shown in Appendix A

The volumes were taken from the Rainfall Frequency Values for Kentucky, 1979 while the antecedent conditions were obtained from the historical data. The user should enter the graph to obtain the rainfall volume associated with the desired frequency. The rainfall volume can then be distributed by using the graphs in Appendix A. The excess hyetograph should be obtained by applying the SCS method which will be discussed in the following section. The excess hyetograph can then be applied to the appropriate planning or design methodology to obtain the subbasin hydrographs.

2.2.3 Determination of the Effective Rainfall Intensities

The effective rainfall intensities in equations 2.1 through 2.14 are the rainfall intensities after accounting for all losses such as infiltration, depression storage, and evaporation. Because of its wide spread use and readily available parameters it is recommended that the SCS method

be used to calculate the effective rainfall intensities.

The SCS method uses curve numbers to represent the characteristics of a watershed. Values of the curve numbers range from 0 to 100. The higher curve number represents the more impervious subbasin. In applying the SCS method, a weighted curve number is obtained for the entire watershed. The equation used by the SCS to compute the excess rainfall is represented by equation 2.17.

$$Q = (P - IA)^2/(P - IA + S) \qquad (2.17)$$

where

Q = accumulated runoff in inches

P = accumulated precipitation in inches

S = total soil moisture capacity for storage of water in inches

IA= all losses subject to the characteristics of the undeveloped areas in inches

The IA term represents all losses due to depression storage, interception, evaporation and infiltration prior to the beginning of runoff. The generally accepted value is 0.2S for agricultural watersheds. However, 0.1S was used in the analysis to better account for the urban conditions which prevail. The storage term, S, is a function of the curve number, CN, and can be represented by the relationship in equation 2.18.

$$S = (1000/CN) - 10 \qquad (2.18)$$

Curve numbers can be found in a number of references (Soil Conservation Service, 1972). Table 2.2 shows curve

Figure 2.2 Runoff curve numbers for selected land uses.

LAND USE DESCRIPTION	HYDROLOGIC SOIL GROUP			
	A	B	C	D
Cultivated land[1]: without conservation treatment	72	81	88	91
: with conservation treatment	62	71	78	81
Pasture or range land: poor condition	68	79	86	89
good condition	39	61	74	80
Meadow: good condition	30	58	71	78
Wood or Forest land: thin stand, poor cover, no mulch	45	66	77	83
good cover[2]	25	55	70	77
Open Spaces, lawns, parks, golf courses, cemeteries, etc.				
good condition: grass cover on 75% or more of the area	39	61	74	80
fair condition: grass cover on 50% to 75% of the area	49	69	79	84
Commercial and business areas (85% impervious)	89	92	94	95
Industrial districts (72% impervious).	81	88	91	93
Residential:[3]				
Average lot size — Average % Impervious[4]				
1/8 acre or less — 65	77	85	90	92
1/4 acre — 38	61	75	83	87
1/3 acre — 30	57	72	81	86
1/2 acre — 25	54	70	80	85
1 acre — 20	51	68	79	84
Paved parking lots, roofs, driveways, etc.[5]	98	98	98	98
Streets and roads:				
paved with curbs and storm sewers[5]	98	98	98	98
gravel	76	85	89	91
dirt	72	82	87	89

1/ For a more detailed description of agricultural land use curve numbers refer to National Engineering Handbook, Section 4, Hydrology, Chapter 9, Aug. 1972.

2/ Good cover is protected from grazing and litter and brush cover soil.

3/ Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.

4/ The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

5/ In some warmer climates of the country a curve number of 95 may be used.

numbers for a variety of land uses. The curve numbers in the table are for the antecendent moisture condition of II. The definitions of the antecedent moisture conditions are shown in Table 2.3. Table 2.4 shows the curve numbers for different antecedent moisture conditions of I and III.

Table 2.3 Definition of Antecedent Condition

Condition	General Description	5-Day Antecedent Rainfall in inches: Dormant Season	5-Day Antecedent Rainfall in inches: Growing Season
I	Dry	0.5	1.4
II	Average	0.5-1.1	1.4-2.1
III	Wet	1.1	2.1

Table 2.4 Antecedent Condition Conversion Factors for CN's

Curve Number for Condition II	Curve Number Converted from Condition II to: Condition I	Condition III
10	4.0	22.2
20	9.0	37.0
30	15.0	50.1
40	22.0	60.0
50	31.0	70.0
60	40.2	78.0
70	51.1	84.7
80	63.2	91.2
90	78.3	96.3
100	100.0	100.0

2.3 Routing of the Hydrographs

Once the composite hydrographs for the particular subbasins are obtained they must be routed to the point in question. Several methods exist for routing a hydrograph through a watershed. These include the Muskingum Method (Viessman et al., 1977), the Convex Method (U. S. Department of Agriculture, 1971) and the time lag method. The time lag method uses Manning's equation and the peak flows from the subbasins to route the hydrographs through the watershed. Using the typical geometry of the channel, the channel roughness and the channel slope, the crossectional area of flow can be computed. The flow velocity can then be computed and is divided into the channel length to obtain the time lag. Once the time lag is obtained, the entire hydrograph is tanslated the length of the channel by the amount of the time lag.

2.4 Example Application of Methodology

The following example is taken from a hypothetical watershed shown in Figure 2.7. The watershed is composed of three subbasins. Subbasin 1 has an area of 100 acres, subbasin 2 has an area of 75 acres and subbasin 3 has an area of 125 acres. Table 2.5 gives the subbasin parameters needed to compute the hydrograph ordinates. Subbasin 1 and 3 are assumed pasture or average grass while subbasin 2 is assumed to be forest. This is used to determine the curve

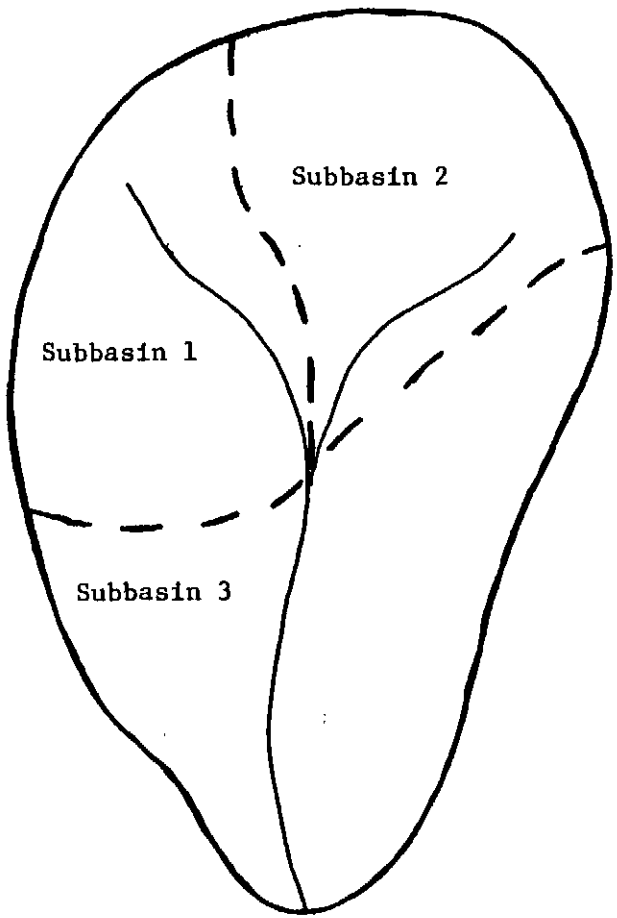

Figure 2.7 Hypothetical watershed

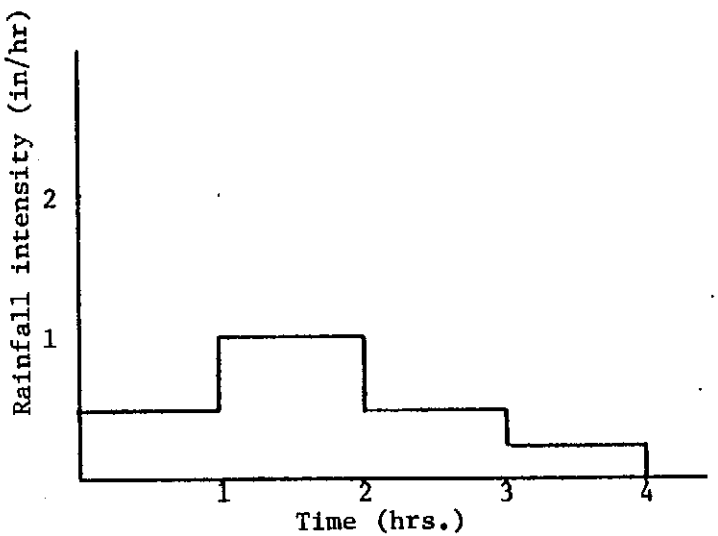

Figure 2.8 Rainfall hyetograph

numbers and the roughness parameters to be used to calculate the time of concentration.

Table 2.5 Specific Parameters for subbasin 1, 2 and 3

Parameter	1	2	3
Length of channel (ft.)	1350	1000	1650
Slope of channel (decimal)	0.06	0.05	0.032
Length of overland flow (ft.)	200	150	200
Slope of overland flow (decimal)	0.03	0.025	0.019
Channel base width (ft.)	NA	NA	10.0
Channel side slope (ft./ft.)	NA	NA	3.0
Roughness (overland flow)	0.40	0.80	0.40
Roughness (Manning's)	NA	NA	0.65
Curve Number	75	70	75
Area (acres)	100	75	125

Using the Kerby-Kirpich method we can easily compute the times of concentration for each subbasin. Note that the time of concentration for subbasin 1 and 2 is the time it takes a raindrop to travel from the most remote point in the subbasin to the point where subbasin 1 and 2 converge. The time lag for these subbasins will be the time it takes a drop to travel from this convergence point to the outlet of the watershed. Computing the times of concentration we get 38.9 minutes for subbasin 1, 48.4 minutes for subbasin 2 and 49.6 minutes for subbasin 3.

Figure 2.8 shows the hypothetical rainfall hyetograph from which the rainfall excess hyetograph and the runoff hydrographs will be computed. The SCS method for computing excess rainfall is used to compute the rainfall excess hyetograph. Using equation 2.18 and the curve numbers given in Table 2.5, the soil moisture capacity for the three subbasins can be calculated. Subbasin 1 and 3 have a soil

moisture capacity of 3.33 inches while subbasin 2 has a soil moisture capacity of 4.29 inches. This is computed assuming an antecedent moisture condition of II. Table 2.6 shows the resulting rainfall excess hyetographs. Notice that runoff begins during the second hour of rain but not at the beginning of the rainfall impulse. The rainfall becomes excess rainfall when the accumulated rainfall becomes equal to or greater than the soil moisture capacity. This occurs in subbasin 1 and 3 at approximately ten minutes after the start of the second impulse and in subbasin 2 at approximately 20 minutes after the start of the second impulse. Therefore, the time of the effective rain for the second impulse of rain is not one hour but 50 minutes for subbasins 1 and 3 and 40 minutes for subbasin 2.

Table 2.6 Rainfall excess hyetograph produced

Time (hrs)	P (accum. rain)	Q (sub. 1 and 3)	dQ	Q (sub. 2)	dQ
0	0.00	0.00	0.00	0.00	0.00
1	0.50	0.00	0.00	0.00	0.00
2	1.50	0.17	0.17	0.08	0.08
3	2.00	0.38	0.21	0.24	0.16
4	2.25	0.51	0.13	0.34	0.10

All of the effective rainfall impulses in this example are longer than the times of concentration of the subbasins. Therefore, equations 2.12 through 2.14 will be used to generate the subbasin hydrographs. The typical shapes of the impulse hydrographs produced in this example are shown in Figure 2.5. If the time of rain had been less than or equal to the time of concentration, equations 2.10 and 2.11 and

Figures 2.3 and 2.4 would have been used, respectively.

Table 2.7 shows the impulse hydrographs produced in subbasin 1 using equations 2.12 through 2.14. A ten minute increment is chosen to obtain a more accurate hydrograph. As the impulses begin to overlap each other they are added together. This is shown in Table 2.7 by adding hydrograph ordinates across the page at the same time step. The composite hydrograph schematic for subbasins 1 and 2 are shown in Figures 2.9 and 2.10, respectively.

Table 2.7 Hydrographs produced by Subbasin 1

Time after start of storm (min.)	Impulse 1	Impulse 2	Impulse 3	Composite
0.0	0.0			0.0
10.0	0.0			0.0
20.0	5.1			5.1
30.0	10.3			10.3
40.0	15.4			15.4
50.0	20.0			20.0
60.0	20.0	0.0		20.0
70.0	14.9	5.4		20.3
80.0	9.7	10.8		20.5
90.0	4.6	16.2		20.8
100.0	0.0	21.0		21.0
110.0		21.0		21.0
120.0		21.0	0.0	21.0
130.0		15.6	3.3	18.9
140.0		10.2	6.7	16.9
150.0		4.8	10.0	14.8
160.0		0.0	13.0	13.0
170.0			13.0	13.0
180.0			13.0	13.0
190.0			9.7	9.7
200.0			6.3	6.3
210.0			3.0	3.0
220.0			0.0	0.0

The hydrographs from subbasins 1 and 2 are combined at the convergence point to obtain the composite hydrograph

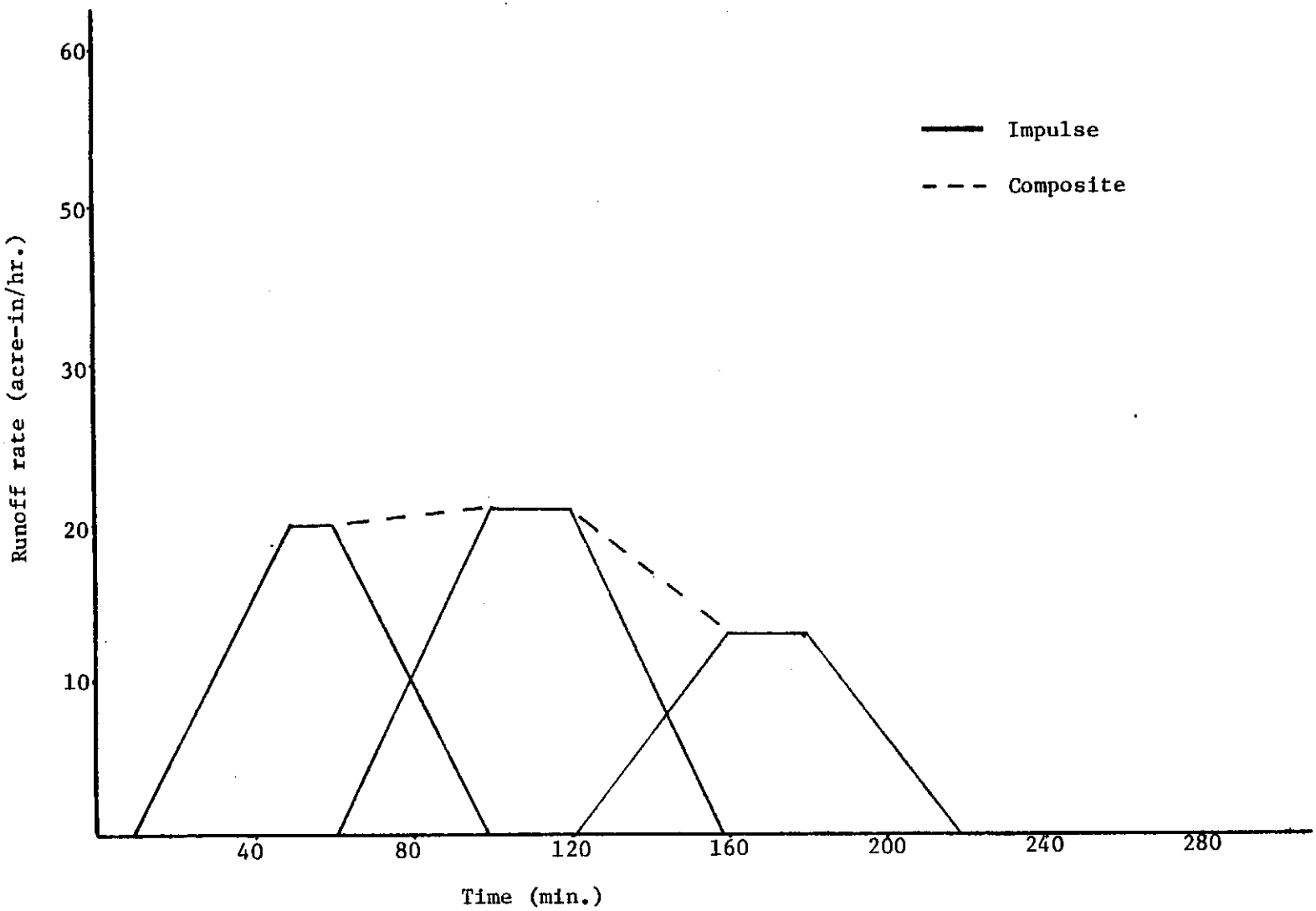

Figure 2.9 Hydrograph produced from Subbasin 1

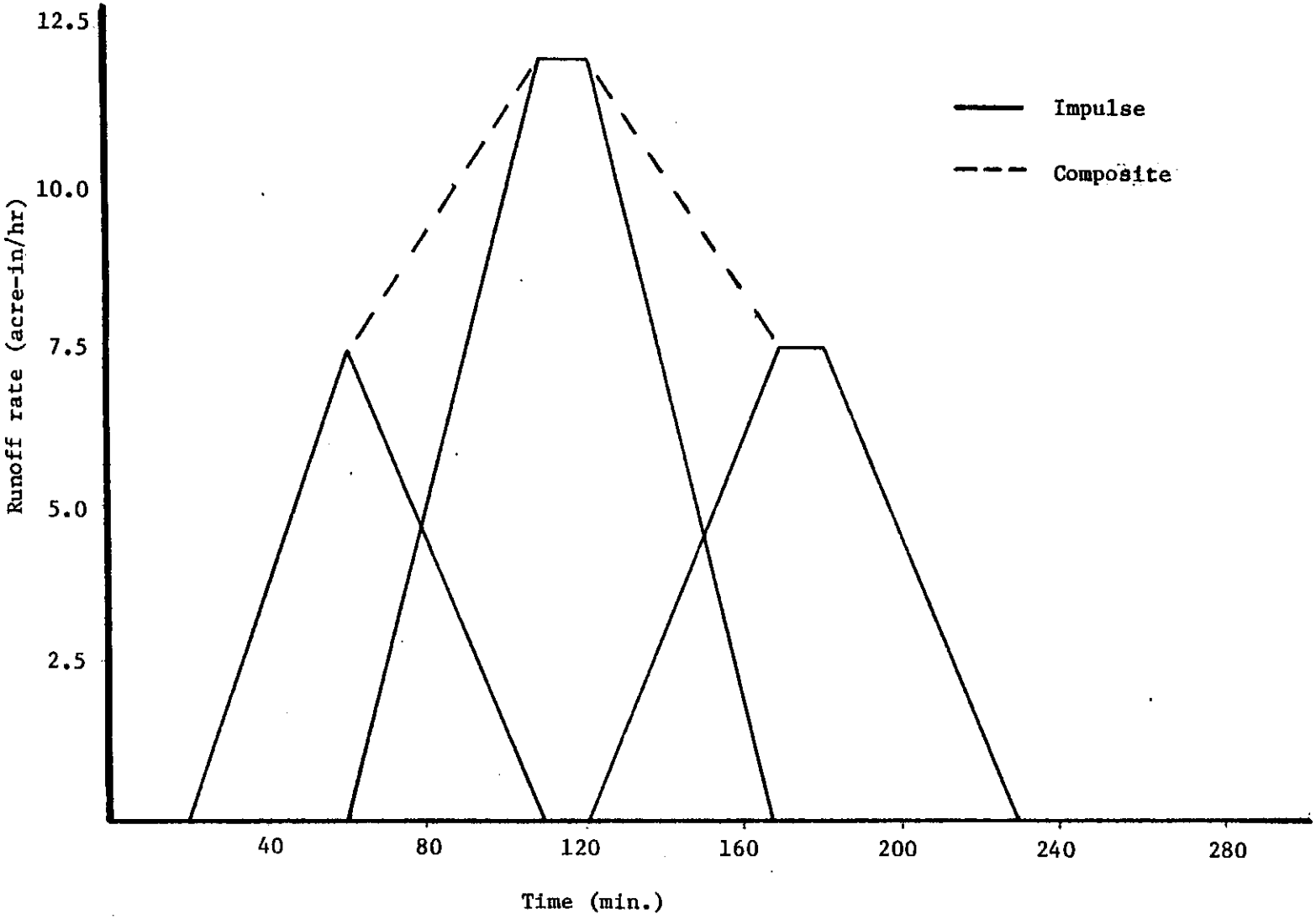

Figure 2.10 Hydrograph produced from Subbasin 2

shown in Table 2.8. The peak flow is 33.0 acre-feet per hour (33.3 cfs) and is used in Manning's equation along with the channel geometry to obtain the lag time to the watershed outlet. The channel is flowing at a depth of 3.93 feet with an area of 72.6 square feet at the peak flowrate. The channel velocity is 0.46 feet per second. The channel length from Table 2.5 is 1650 feet resulting in a lag time of 60.0 minutes.

Table 2.8 Composite hydrographs from Subbasins 1 and 2

Time after start of storm (min.)	Subbasin 1	Subbasin 2	Composite
0.0	0.0	0.0	0.0
10.0	0.0	0.0	0.0
20.0	5.1	0.0	5.1
30.0	10.3	1.9	12.2
40.0	15.4	3.7	19.1
50.0	20.0	5.6	25.6
60.0	20.0	7.4	27.4
70.0	20.3	9.6	29.9
80.0	20.5	10.3	30.8
90.0	20.8	10.8	31.6
100.0	21.0	11.5	32.5
110.0	21.0	12.0	33.0
120.0	21.0	12.0	33.0
130.0	18.9	11.0	29.9
140.0	16.9	10.1	27.0
150.0	14.8	9.2	24.0
160.0	13.0	8.3	21.3
170.0	13.0	7.5	20.5
180.0	13.0	7.5	20.5
190.0	9.7	6.0	15.7
200.0	6.3	4.4	10.9
210.0	3.0	2.9	5.9
220.0	0.0	1.3	1.3
230.0		0.0	0.0

The hydrographs from subbasin 1 and 2 are routed to the watershed outlet using the calculated time lag. The hydrograph from subbasin 3 (Figure 2.11) is combined with

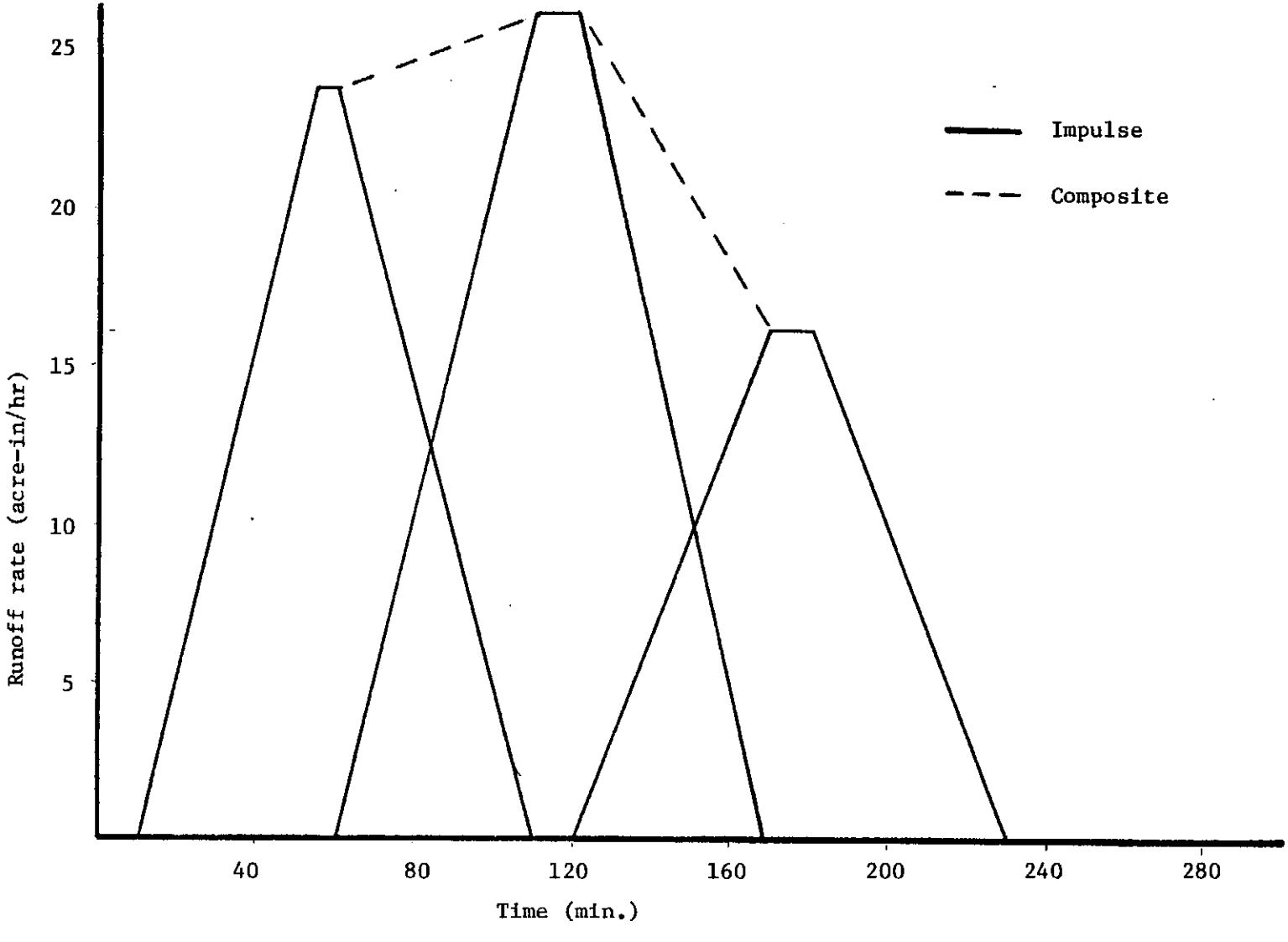

Figure 2.11 Hydrograph produced from Subbasin 3

the routed hydrographs to obtain the composite hydrograph for the watershed. This calculation is shown in Table 2.9 and illustrated in Figure 2.12.

Table 2.9 Watershed hydrograph

Time after start of storm (min.)	Subbasin 1	Subbasin 2	Subbasin 3	Composite
0.0				
10.0			0.0	0.0
20.0			5.0	5.0
30.0			10.0	10.0
40.0			15.1	15.1
50.0			20.1	20.1
60.0			25.0	25.0
70.0	0.0		25.3	25.3
80.0	5.1	0.0	25.5	30.6
90.0	10.3	1.9	25.7	37.9
100.0	15.4	3.7	26.0	45.1
110.0	20.0	5.6	26.3	51.9
120.0	20.0	7.4	26.3	53.7
130.0	20.3	9.6	24.3	54.2
140.0	20.5	10.3	22.2	53.0
150.0	20.8	10.8	20.2	51.8
160.0	21.0	11.5	18.3	50.8
170.0	21.0	12.0	16.3	49.3
180.0	21.0	12.0	16.3	49.3
190.0	18.9	11.0	13.0	42.9
200.0	16.9	10.1	9.7	36.7
210.0	14.8	9.2	6.5	30.5
220.0	13.0	8.3	3.2	24.5
230.0	13.0	7.5	0.0	20.5
240.0	13.0	7.5		20.5
250.0	9.7	6.0		15.7
260.0	6.3	4.4		10.7
270.0	3.0	2.9		5.9
280.0	0.0	1.3		1.3

The release rate percentages for each subbasin are then calculated using the ratio of the contributing flow to the peak flow. This is illustrated in Figure 2.12. Table 2.10 shows the contributing peak flows, the peak flows and the accompanying release rate percentage for each subbasin.

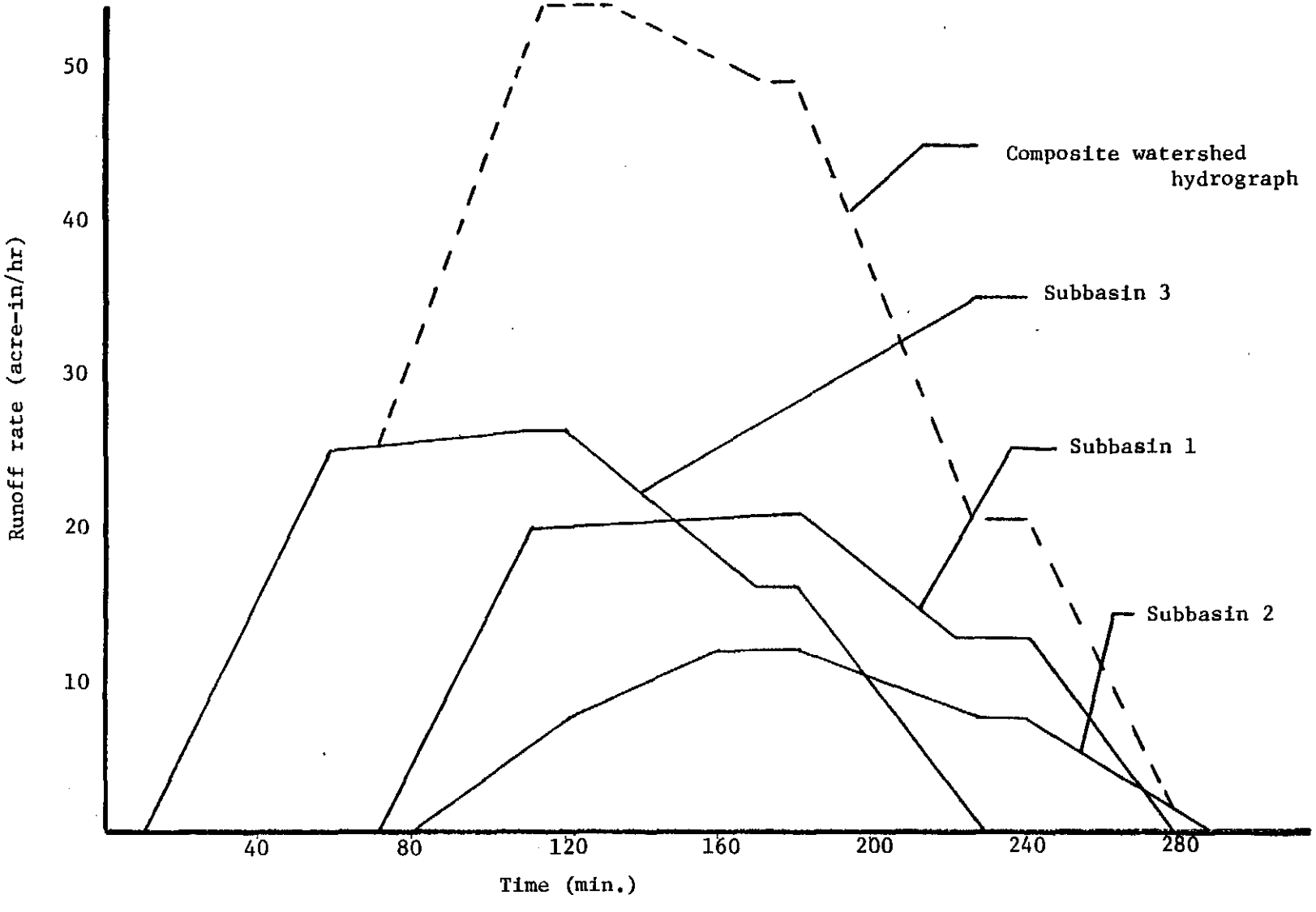

Figure 2.12 Watershed hydrograph

Table 2.10 Subbasin realease rate percentages

Subbasin	Contributing Flow (CFS)	Peak Flow (CFS)	Release Rate %
1	20.3	21.0	96.7
2	9.6	12.0	80.0
3	24.3	26.3	92.4

From Table 2.10, subbasins 1 and 2 have release rate percentages of 96.7 and 80.0, respectively, while subbasin 3 has a release rate percentage of 92.4. The flowrates can be used by the developer to determine if a detention facility is required after development of a subbasin. If the requirement that the postdevelopment peak not exceed the predevelopment contributing flow to the watershed peak is enforced here, the developer of subbasin 1 would have to meet a release rate of 96.7 percent or a maximum discharge of20.3 cubic feet per second. Likewise the developer of subbasin2 would have to meet a release rate of 80.0 percent or a flowrate of 9.6 cubic feet per second. Finally, the developer of subbasin 3 would be required to limit any discharge to 24.3 cubic feet per second.

The methodology should be revised if the release rate percentage is less than 50 percent. The subbasin or subbasins which have release rate percentages greater than 50 percent should absorb the contributing flow to the watershed peak from the subbasins with less than 50 percent release rates. The subbasins with an original release rate of greater than 50 percent would then be required to release a flow no greater than their contributing flow less an

appropriate percentage of the contributing flow from the subbasins with a release rate percentage of less than 50. This would then require the subbasins with a release rate percentage of less than 50 percent to release no more than double their original release rate.

Table 2.11 shows an example of this technique. Subbasin 1 and 2 contribute flows of 23 and 25 cubic feet per second to the watershed peak while subbasin 3 contributes 10 cubic feet per second. The peak flows for subbasins 1, 2 and 3 are 25, 30 and 30 cubic feet per second respectively. Therefore, 10 cubic feet per second has to be distributed between subbasin 1 and 2. Subbasin 1 should absorb the proportionate amount of its peak divided by the sum of its peak and the peak of subbasin 2. Subbasin 2 absorbs the proportionate amount of its peak divided by the sum of its peak and the peak of subbasin 1. This results in a distribution of 4.5 cubic feet per second for subbasin 1 and 5.5 cubic feet per second for subbasin 2. Subbasin 1 now has a new release rate constraint of 18.5 cubic feet per second while subbasin 2 has a new release rate constraint of 19.5 cubic feet per second. Subbasin 3 has a new release rate constraint of double its original constraint or 20 cubic feet per second.

Table 2.11 Example with release rates less than 50 percent

Subbasin	Contributing Flow (CFS)	Peak Flow (CFS)	Release Rate %	New Release Rate
1	23.0	25.0	92.0	18.5
2	25.0	30.0	83.3	19.5
3	10.0	30.0	33.3	20.0

CHAPTER 3

DESIGN METHODOLOGY

3.1 Development of Proposed Methodology

The proposed design methodology is a quick, simple way to help the designer estimate the volume needed to detain a proposed critical storm. The detention volume ultimately required is assumed to be a function of the watershed characteristics and the rainfall distribution. Historical rainfall data is used in a continuous simulation program to obtain a required storage volume as a function of watershed characteristics, return interval, and a specific release rate. These relationships are presented in graphical form for use by the designer in obtaining the required storage volume.

The Synoptic Rainfall Analysis Program (SYNOP) was chosen as the simulation program to analyze the historical rainfall data. The program is set up to read National Weather Service rainfall tapes making data preparation much easier. Originally, the program was written to read hourly rainfall records and obtain statistics on the specific storm characteristics of duration, intensity, volume, and the time between storms. A storm is represented by the detection of any amount of measureable precipitation. The end of the storm is detected by some consecutive number of dry hours.

In this regard, a storm can be represented as a period of rainfall, a number of dry hours and another period of rainfall, etc. In this study, an eight hour duration of no rainfall was used to define the end of the storm. This variable is important because a smaller duration means many smaller storms of shorter duration will be analyzed with higher average intensities while a larger duration will result in a smaller number of storms with longer durations and smaller average intensities.

However, the nature of this study is to determine the required detention volume for each storm. Because antecedent moisture conditions are accounted for, the same peak detention volumes will be obtained regardless of the number of consecutive dry hours used to represent a break in a rainfall event. The changes which will occur are the lengths and average intensities of the rainfall events. This will affect the characteristics of the critical rainfall events but not the basin storage requirements which result from these events.

3.2 Description of the Analysis Program

In order to determine the required storage volume as a function of the watershed characteristics, a specified release rate, and a return interval, SYNOP was modified to do a statistical analysis on the required detention volume needed for each storm event. The required detention volume was determined using a simple hydrograph generation and

routing technique which was imbedded in SYNOP. For each individual storm event a composite hydrograph was obtained for specified values of curve number, time of concentration, release rate and watershed area.

A spread of curve numbers was used ranging from 70 to 100. These curve numbers are representative of the weighted curve number of the entire watershed after development. The SCS method for computing runoff, which uses the curve number, was used in the analysis and is discussed in section 2.2.3.

The revised program uses a simple technique to adjust the curve number when a rainfall event is detected. The adjustment is done according to the season, growing or dormant, and the 5-day antecedent moisture condition. The season is determined according to the typical seasons representing the area being analyzed. Illustrating, a typical growing season for central Kentucky includes the months of April through October while the remaining months are considered the dormant season. The program also kept a continuous account of the 5-day antecedent moisture condition. On the basis of the season and the 5-day antecedent moisture condition, the antecedent moisture condition class was determined. Once the antecedent moisture condition class was determined the curve number was adjusted appropriately for each storm. The specifics of the technique were discussed in section 2.2.3 and further information can be obtained from Barfield et al. (1981).

After the excess rainfall hyetograph is computed for a rainfall event, the runoff hydrograph can then be computed. The method used is the linearized subhydrograph method (Sarikelle et al., 1978) which was discussed in section 2.2. This method uses the time of concentration, the rainfall duration, the rainfall intensity and the watershed area to calculate the runoff. The rainfall excess is considered to be continuous and evenly distributed over the entire watershed.

Once the composite runoff hydrograph is obtained, it is routed through the detention basin using the previously set release rate. The release rate for each computer analysis is an assumed value representative of typical peak flows. The actual units of the release rate is inches per hour per acre. The program uses the release rate to determine the volume of detention required for the combination of watershed characteristics used.

The program uses a constant release rate concept to determine the required volume. The concept assumes all inflow is released until the inflow rate is greater than the the release rate. At this time, some storage is required (Figure 3.1). A continual account of the required storage volume is kept for each rainfall event. The maximum storage required for each storm event is also kept and later used in a statistical analysis.

In cases where storage is required after a rainfall event has ceased, a constant outflow rate is still assumed

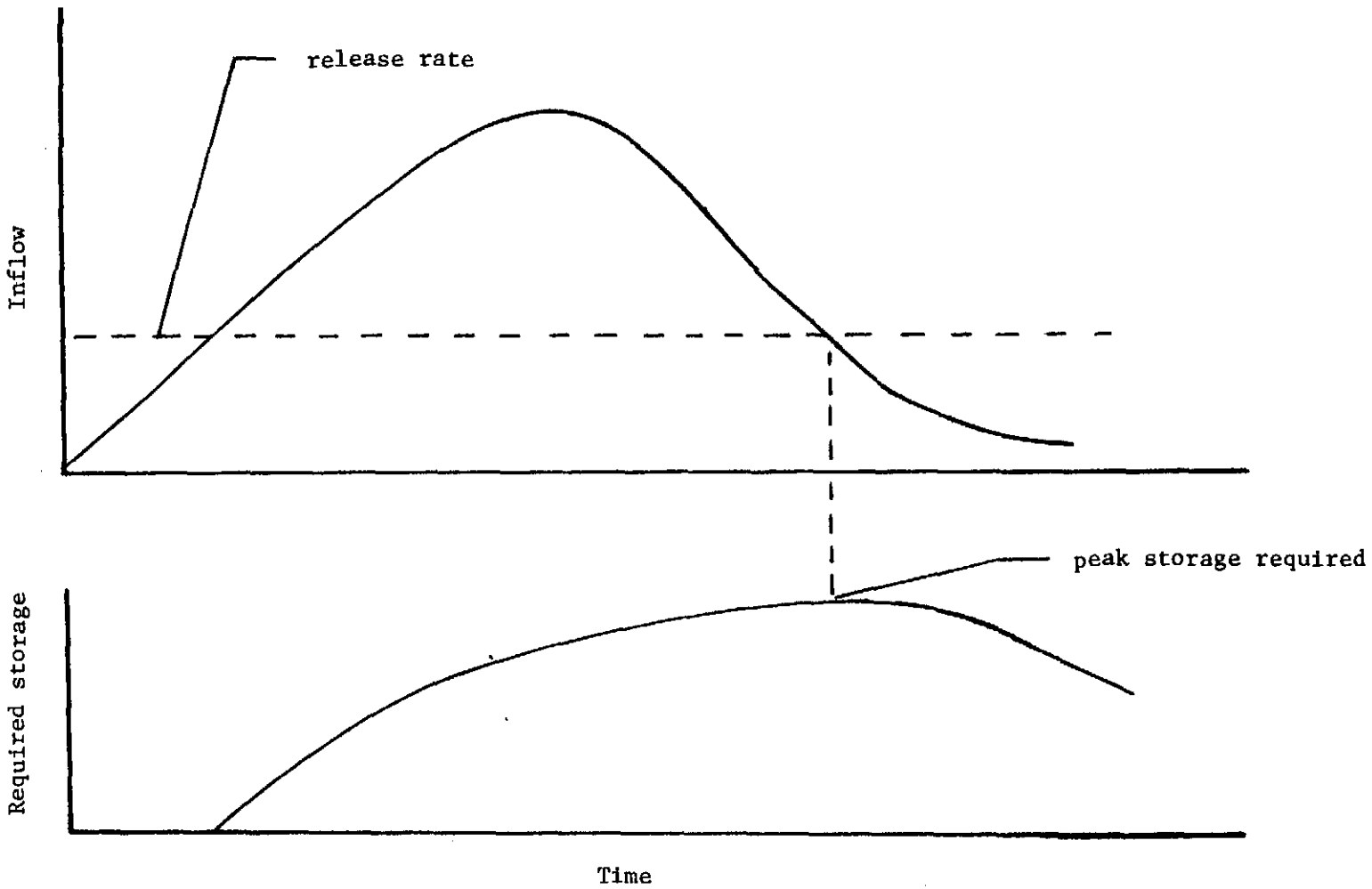

Figure 3.1 Storage accounting

and accounting of the storage requirement continues (Figure 3.2). This is done to account for the possibility of a storage overlap at the beginning of the next rainfall event. When a storage overlap is encountered, the overlap is assumed to be the inital storage requirement for the next rainfall event (Figure 3.2). This assures that both the antecedent moisture condition and storage status are accounted for in the analysis. Naturally, if the basin drains completely before the next rainfall event occurs, the initial storage is zero for that event (Figure 3.3).

3.3 Application of Methodology to Kentucky

Once the analysis program was developed it was applied to Kentucky using a 25 year hourly rainfall record for Lexington, Kentucky. Several hundred computer runs were performed for different values of curve number, area, time of concentration and release rate. As a result of these computer runs a series of graphs were produced.

After the watershed characteristics were chosen the model was used to predict required storage volumes for each combination desired. Twenty five years of data was used which resulted in storage volume recurrence intervals of 25 years and less. Typical municipal standards require that the detention basin be designed for the 50 or 100 year storm. Therefore, some type of regression analysis had to be performed to obtain the storage volume requirements for the 50 and 100 year events.

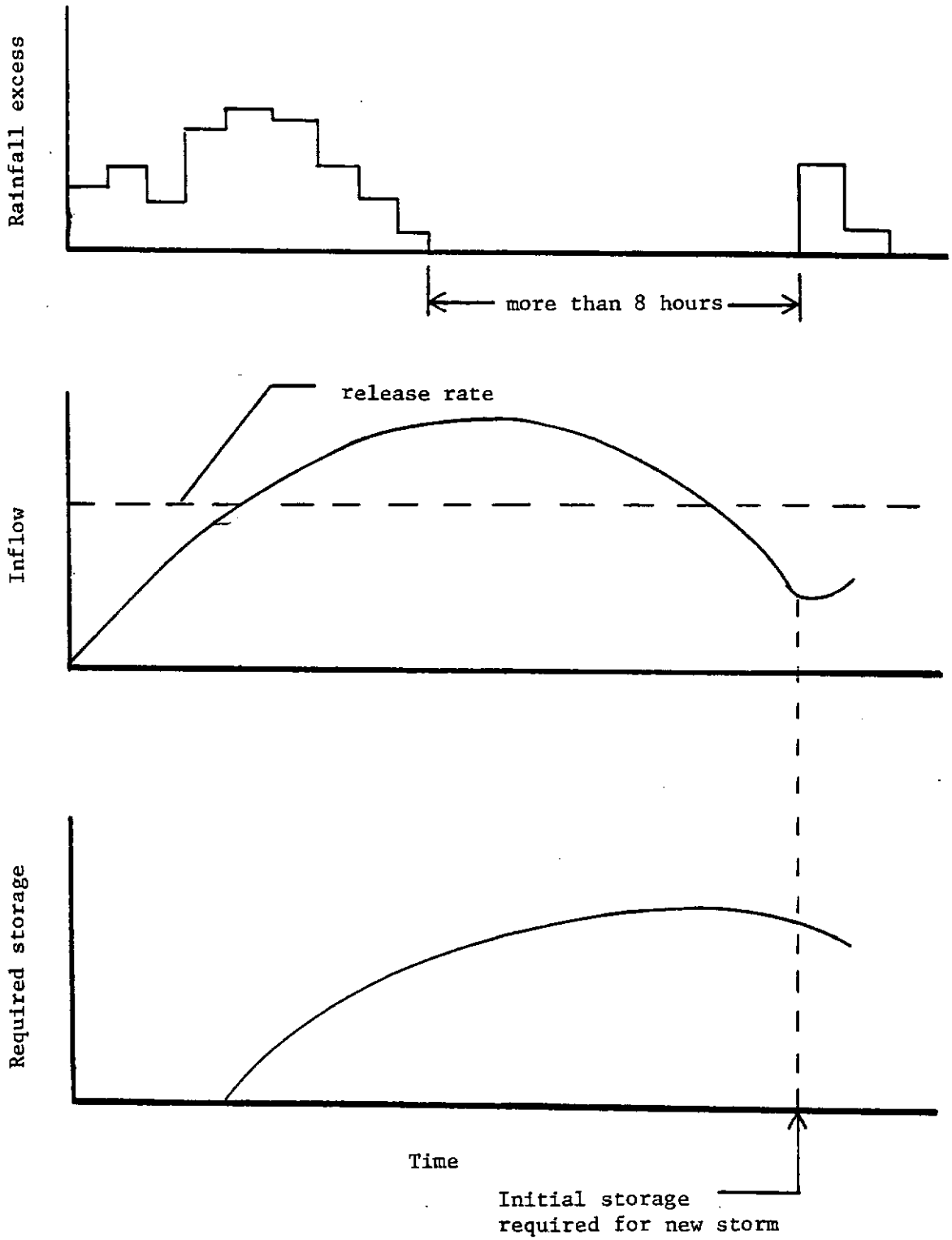

Figure 3.2 Overlap of storage

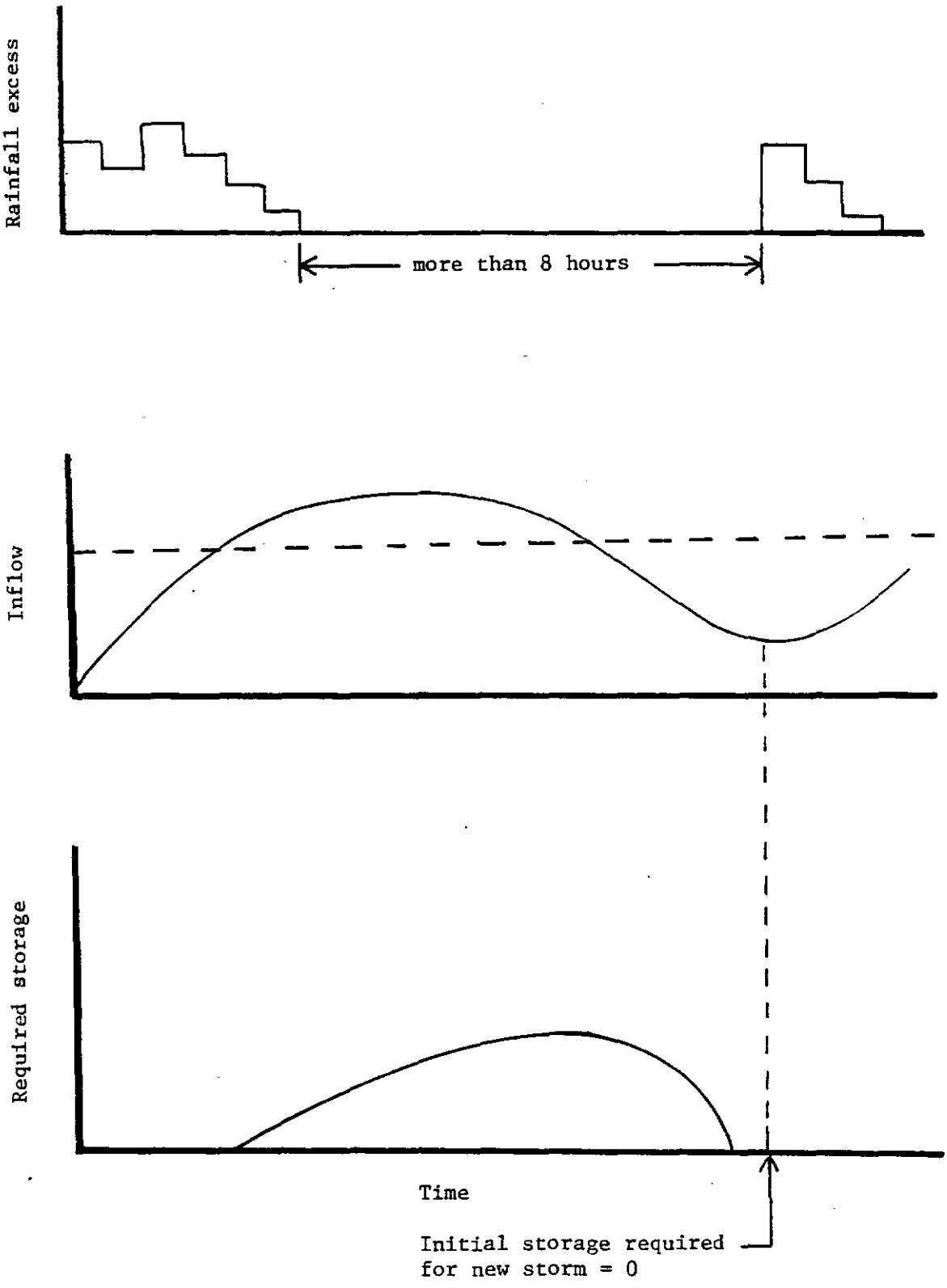

Figure 3.3 No overlap of storage

Graphs of the results showed an exponential relationship between the inverse of the return period and the required storage volume. This is represented by equation 3.1. A typical graph is shown in Figure 3.4.

$$1/T = e^{m} \times V + e^{b} \qquad (3.1)$$

T = return period (years)

V = required storage volume (acre-ft.)

m,b = coefficients

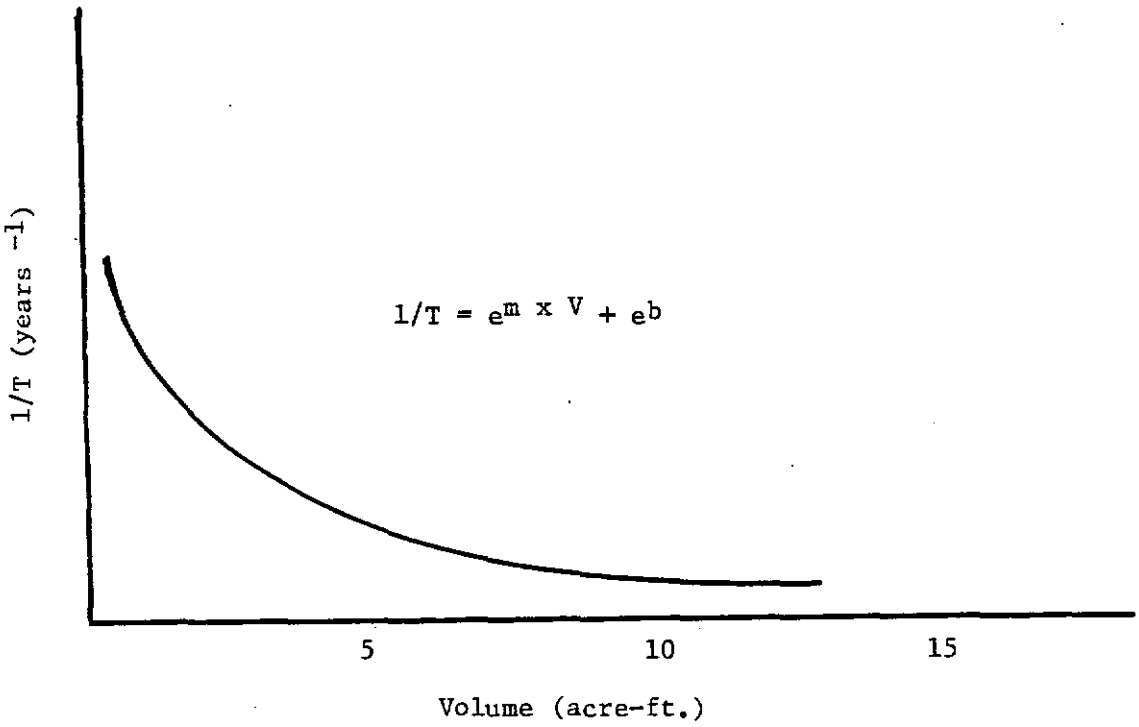

Figure 3.4 Typical graph of exponential relationship

The above relationship can be reduced to a linear relationship by taking the natural logarithm of both sides. This reduces equation 3.1 to equation 3.2. All variables

are the same as previously defined.

$$\ln (1/T) = m \times V + b \qquad (3.2)$$

A simple regression analysis was done on each computer analysis to obtain the best fit linear function representing the results from the analysis. The 50 and 100 year storage volume requirements were obtained using these functions. An application of the methodology is shown in Section 3.5.

3.4 Description of Design Graphs

The graphs produced from the statistical analysis are presented in Appendix B. A typical graph is reproduced in Figure 3.5. As can be seen from the figure, the required storage volume is represented on the y-axis and the subbasin area is represented on the x-axis. The time of concentration of the subbasin, the weighted curve number of the subbasin and the frequency of the storm are shown at the top of the figure. The release rate in the units of cubic feet per second per acre is represented on the appropriated lines on the figure. Appendix B seperates these figures by frequency of the storm (i.e. 10, 25, 50 and 100-year frequencies).

The design graphs were developed with urban watersheds in mind. Watershed characteristics were used which were most typical of urban watersheds. This was done to predict the detention storage volumes which were required for the most typical urban watersheds.

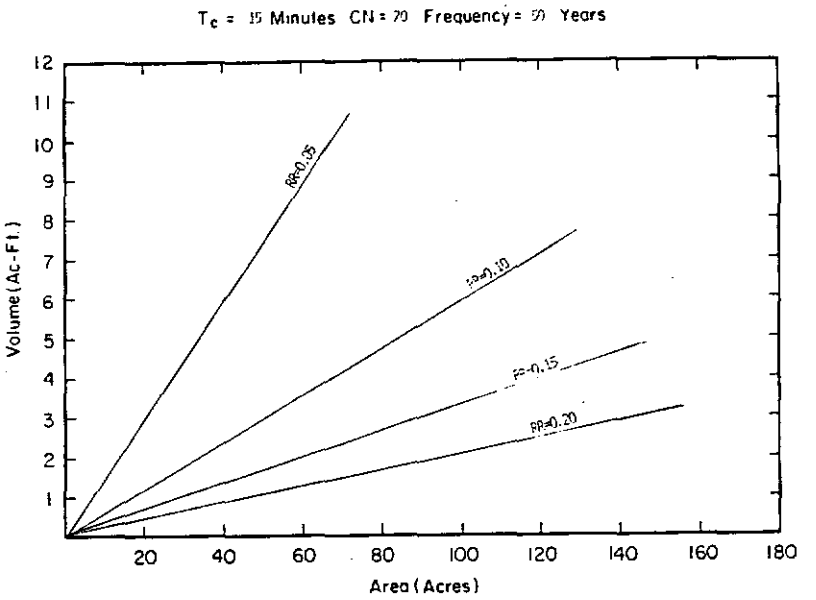

Figure 3.5 Typical design graph from Appendix B

The development of the graphs was done in such a way that the user can extrapolate where needed. The graphs are linear, making it very easy to extend them to the desired need. A large spread of release rates ranging from 0.05 to 0.40 inches per acre per hour were chosen to provide the user with more ease for extrapolation. Times of concentration were chosen ranging from 15 minutes to two hours. Curve numbers were chosen ranging from 70 to 95. Areas up to 400 acres were used but the linear graphs allow the user to use any area desired.

3.5 Example Application of Design Graphs

Table 3.1 shows typical subbasin hydrograph peaks and contributing peaks along with the subbasin characteristics for a typical watershed. This hypothetical example will be used to show the user how the design graphs in Appendix B

are applied.

Table 3.1 Example subbasin characteristics

Subbas. No.	Area (acres)	Peak Flow (cfs)	Contrib. Flow (cfs)	T_c (min)	CN	Rel. Rate per acre (cfs/ac)	Required Storage (ac-ft.)
1	50	15	10	15	70	0.20	1.10
2	60	21	18	24	80	0.30	1.61
3	75	22	22	30	80	0.29	2.30
4	100	40	35	37	90	0.35	5.72

The requirements set by the municipality in this example are that the subbasins not produce a postdevelopment peak flow greater than the predevelopment contributing flow. The design should be made to control a storm with a return period of 50 years.

The required storage volume for subbasin 1 is 1.10 acre-feet and can be found directly from the graphs using the hydrologic characteristics shown in the table. However, subbasin 2 has a time of concentration of 24 minutes while the graphs were computed using times of concentration of 15 and 30 minutes. Interpolation between these two graphs is required to obtain the storage volume.

Entering the graphs, a storage volume of 1.40 acre-feet is found for a time of concentration of 15 minutes while 1.75 acre-feet is required for a time of concentration of 30 minutes. Using a straight line interpolation between these values we find the required storage volume to be 1.61 acre-feet. Using this method, the required storage volumes for subbasins 3 and 4 can be found to be 2.30 and 5.72 acre-feet, respecively. Results are shown in Table 3.1.

CHAPTER 4

SUMMARY AND CONCLUSION

4.1 Summary

This study has identified a need for a comprehensive planning methodology for developing watersheds in the state of Kentucky. The authors have shown that this methodology should include the two phases of planning and design. These two phases should encompass all aspects of planning and design and should treat the subbasins of the watershed as an interacting system.

With this in mind, the authors have developed a planning and design methodology which can be applied to watersheds in the state of Kentucky. The data used in the analysis is representative only of central Kentucky but further computer analysis on the other regions in the state will yield the required results to obtain design graphs for these regions.

The planning methodology recommends the use of the SCS infiltration equation, the linear subhydrograph method and a time lag routing procedure for generating and routing subshed hydrographs. Despite some of the theoretical limitations of the SCS equation, the method was used because of the usual availability of the necessary soil and land use data, the widespread use and familiarity of the method by

the engineering community and the capability of the method to model antecedent moisture conditions in a fast and efficient manner.

The linearized subhydrograph method has been shown to produce very good results for a number of actual watersheds. The main limitation of the method is the assumption of a linear response and a constant time of concentration for the watershed. For the small watersheds considered in this study these assumptions should be acceptable.

Two different methods were presented for determining the time of concentration of a watershed. In general, Izzard's method is more accurate. If the Kerby-Kirpich equation is used care should be taken to insure that the channel geometry and roughness remain the same before and after development since changes in these parameters are not accounted for in the equation.

The general planning methodology recommends using a simple time lag method for routing the subshed hydrographs through the watershed. It should be noted that this method only considers the translation of the hydrograph. Any hydrograph attenuation due to channel storage is neglected. If the designer determines that storage effects are significant then a more sophisticated routing technique such as the Muskingum method or the kinematic wave method should be used. However, for preliminary design studies on small watersheds the time lag method should be sufficient.

Despite these minor limitations the planning

methodology developed in this study is advantageous in many ways. It gives the user the knowledge of the interactions of the subbasins in the watershed and how development will affect these interactions. It allows the municipality to control the outflow of subbasins as they develop giving them confidence that flooding downstream will be kept at a minimum. Most importantly, the watershed and the subbasins in the watershed are treated as a system resulting in an economic savings due to fewer detention basins and less flooding.

The design methodology is based on a series of design charts which may be used to obtain a preliminary estimate of the storage as a function of the time of concentration, curve number, return interval, subshed area and release rate. The actual outflow hydrograph from a detention basin will be dependent upon the geometry of the basin and the type of outlet control. Since this information is not available in a preliminary analysis a constant release rate was assumed. As a result, the storages obtained from the design charts will tend to be slightly underestimated. This problem could possibly be overcome by using a more realistic outflow hydrograph shape. The exact storage required for the basin may be obtained by routing the critical design hydrograph through the basin once the actual basin geometry and outlet structures have been selected.

Despite the limitation of the constant release rate the design methodology is advantageous because it uses

historical rainfall data and considers antecedent conditions in a continuous simulation analysis. This assures the user that the critical events were selected for the methodology. It also yields quick results and most importantly it can be applied to the specific area of the state for which the design is needed.

4.2 Conclusion

A sound planning and design methodology which treats the watershed as a system is a positive move toward better stormwater management. Despite the minor limitations imposed by the fundamental assumptions in the study the proposed methodologies should provide a very good tool for the preliminary planning of stormwater management facilities. The application of these methodologies will create a better understanding of the effects of development on watersheds and what can be done to illeviate the problems caused by these developments.

APPENDIX A

Storm volumes and antecedent conditions for the critical storms with durations of 3 and 6 hours is shown below. Also shown is a dimensionless figure which can be used to distribute the storm volume throughout the length of the storm.

Duration (hrs.)	Storm Type	AMC	Frequency (years) 10	25	50	100
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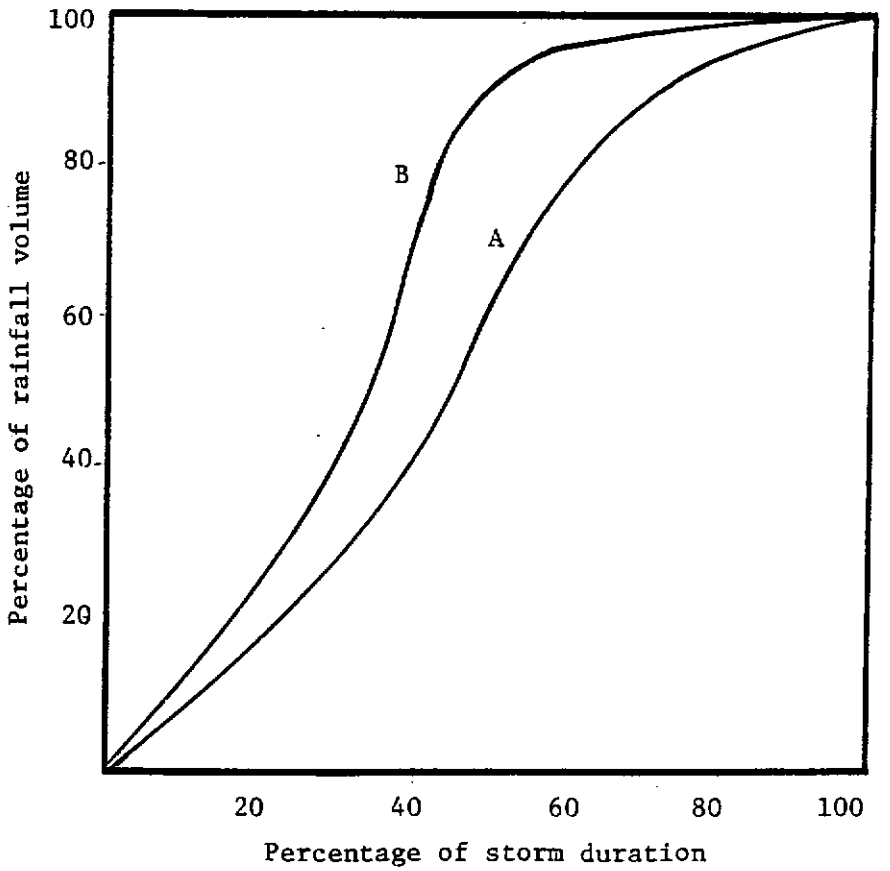
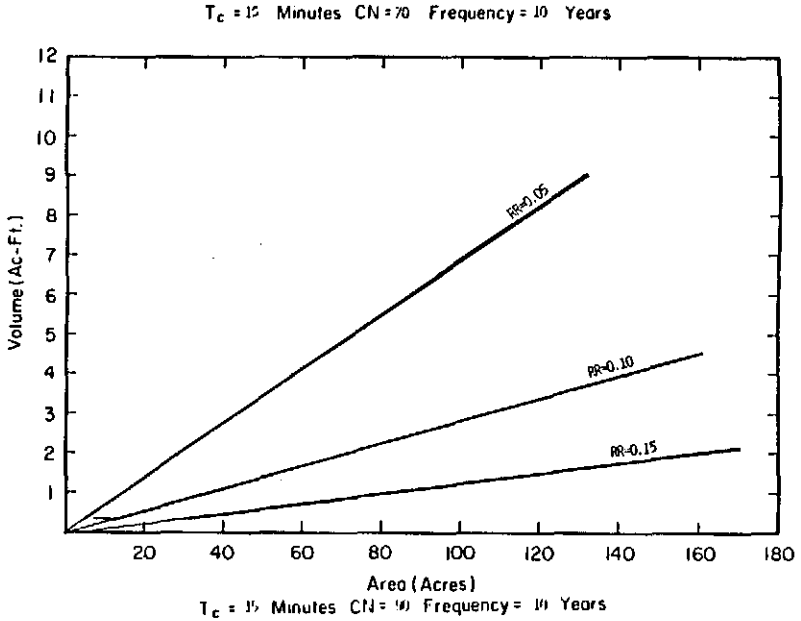
6	B	II	3.2	3.7	4.1	4.4



APPENDIX B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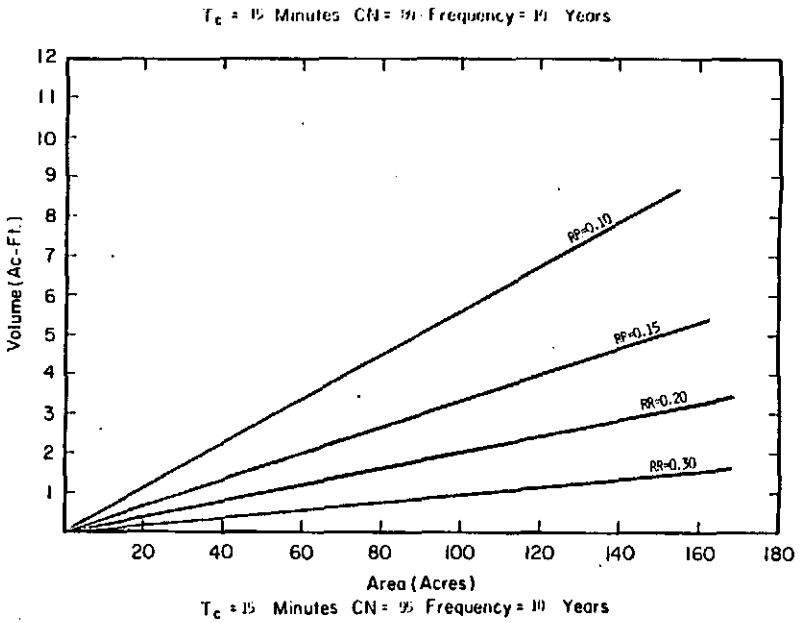
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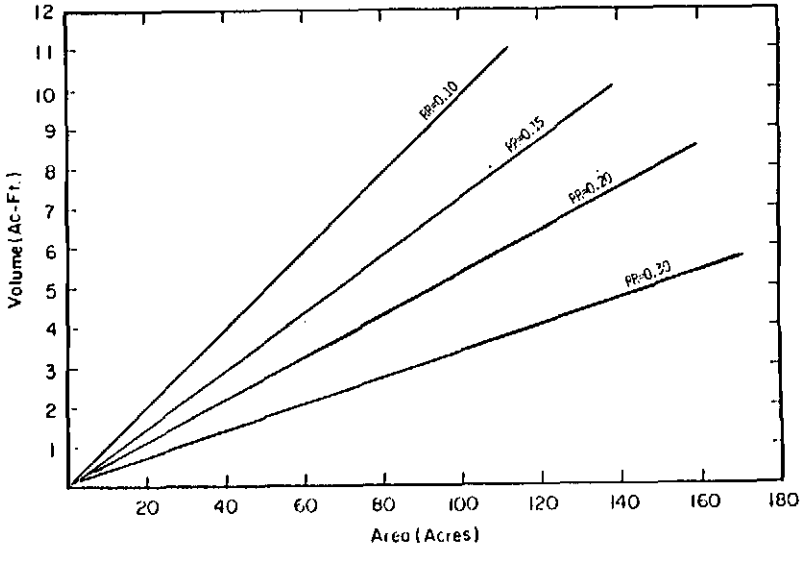
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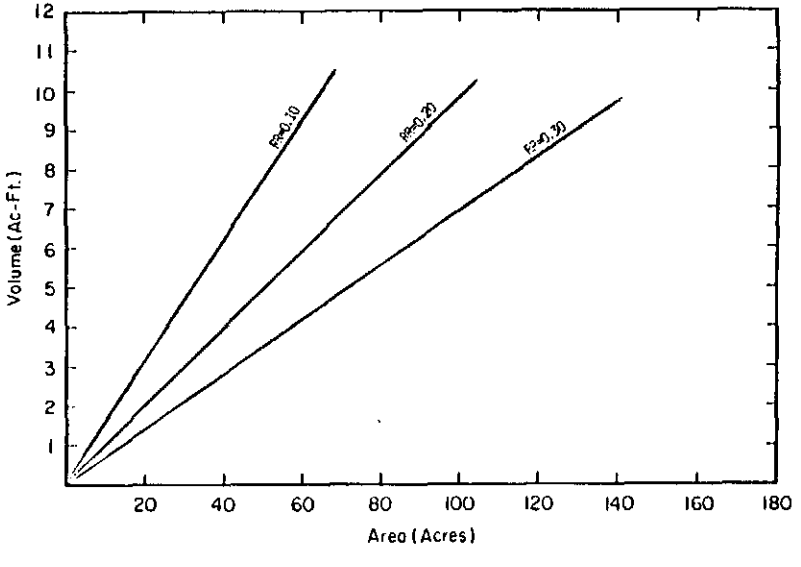
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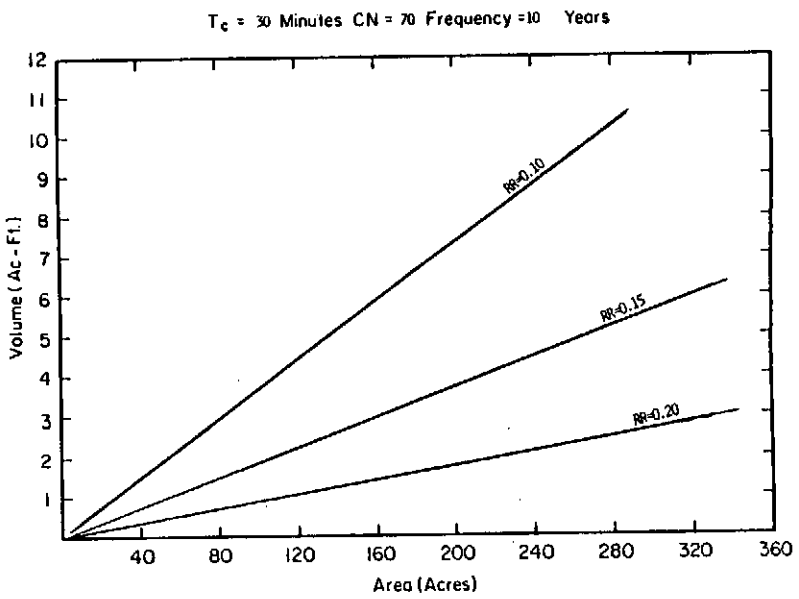
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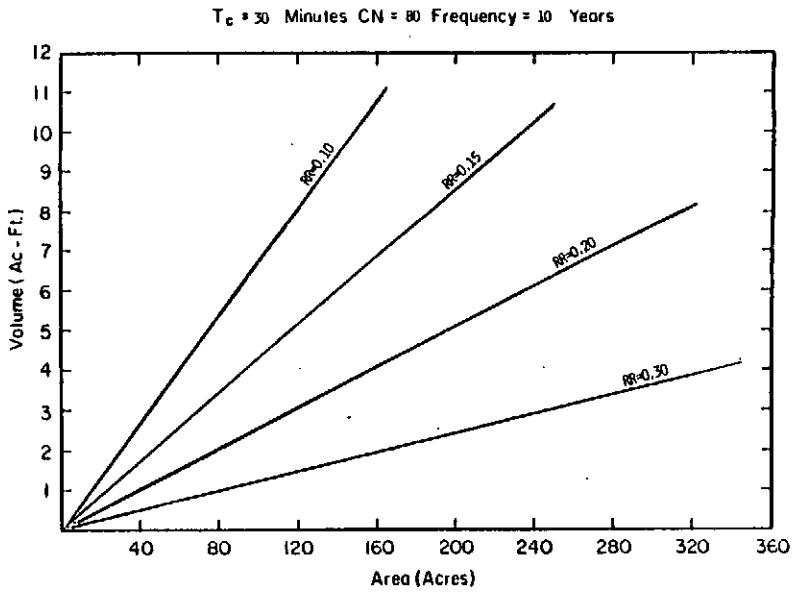
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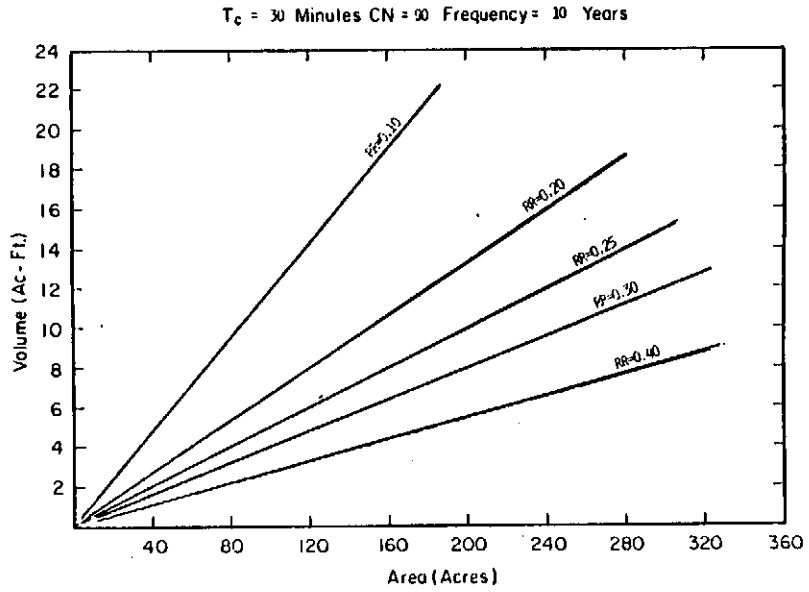
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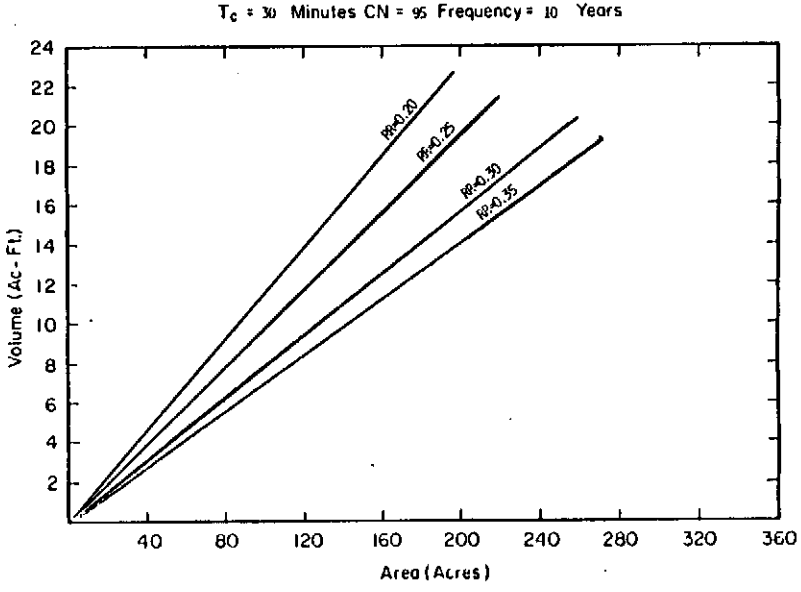
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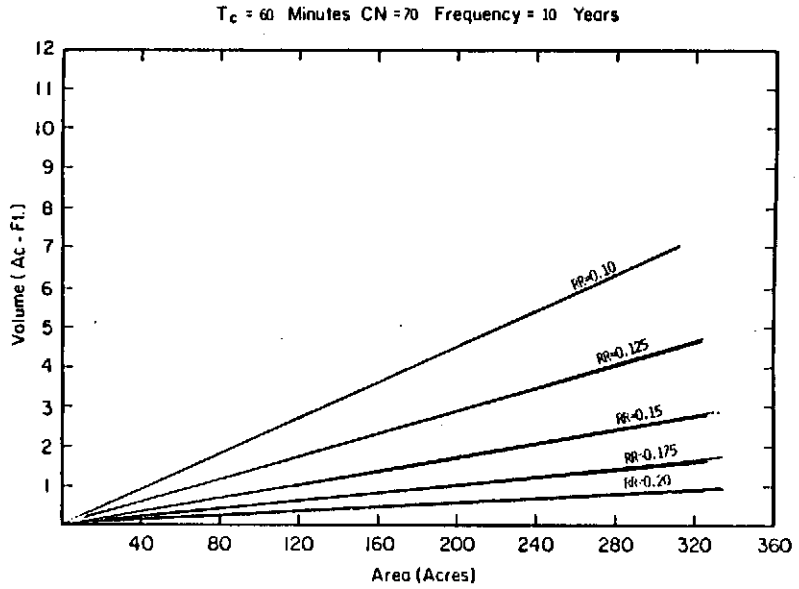
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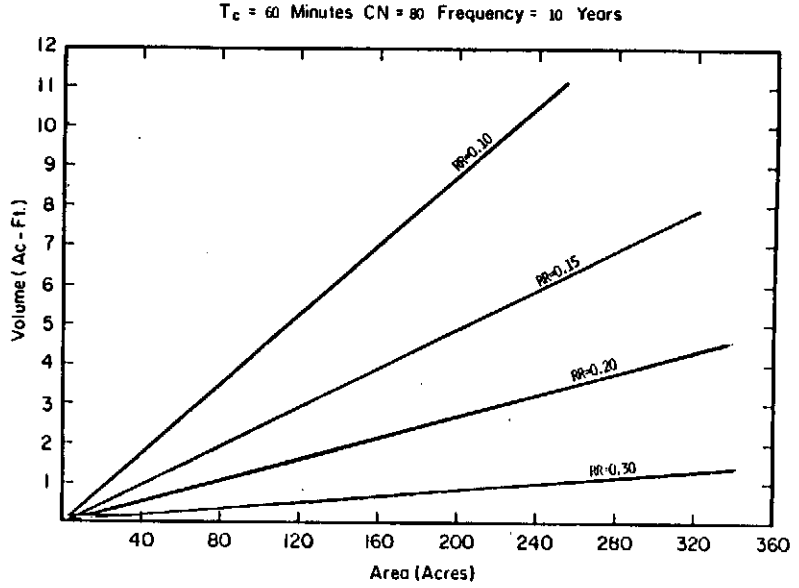
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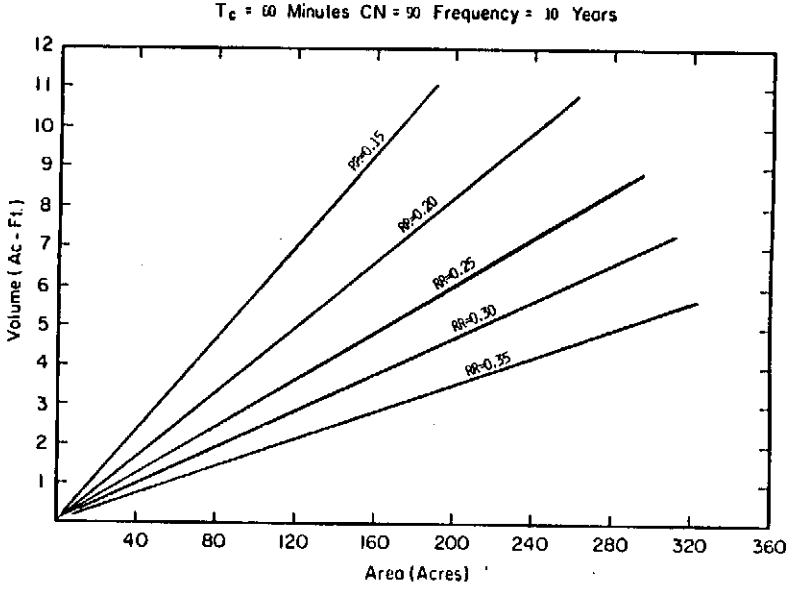
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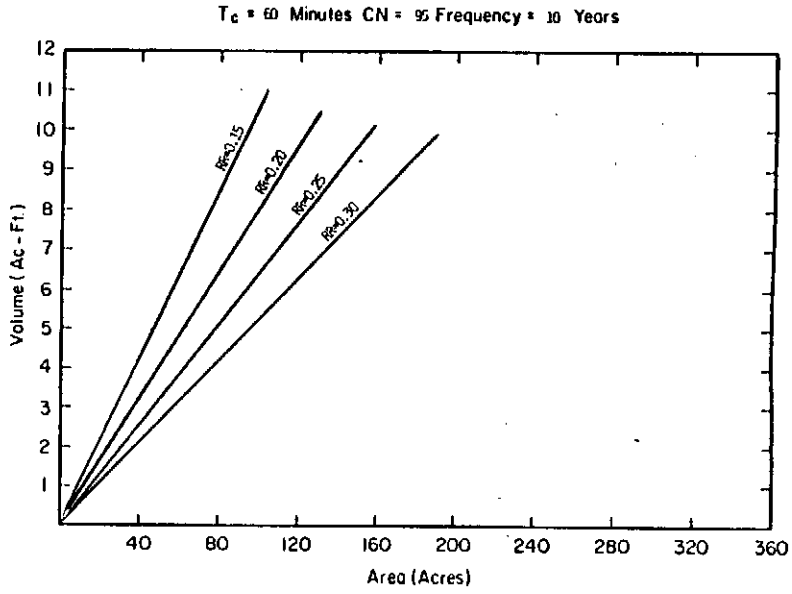
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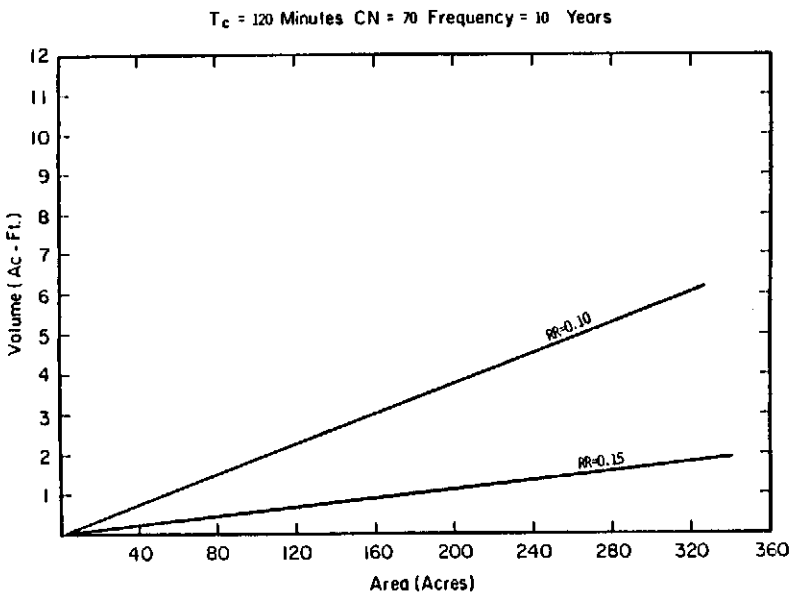
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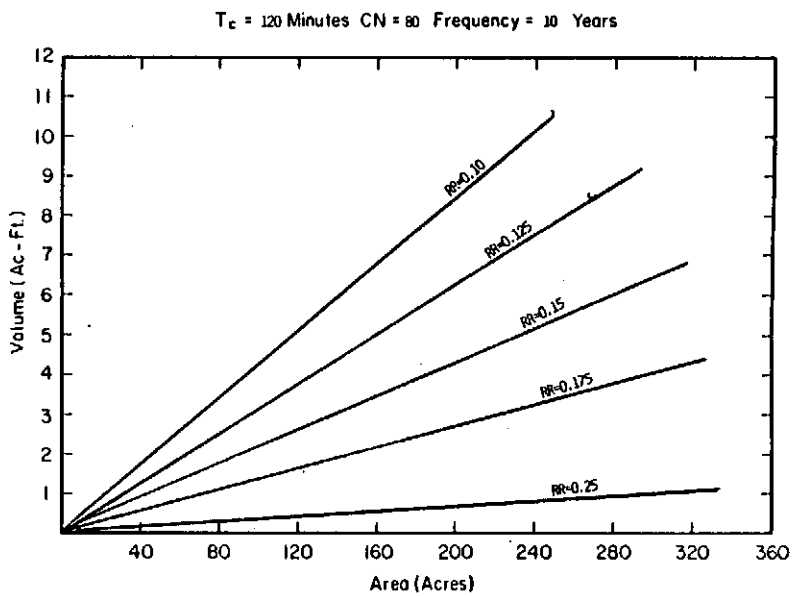
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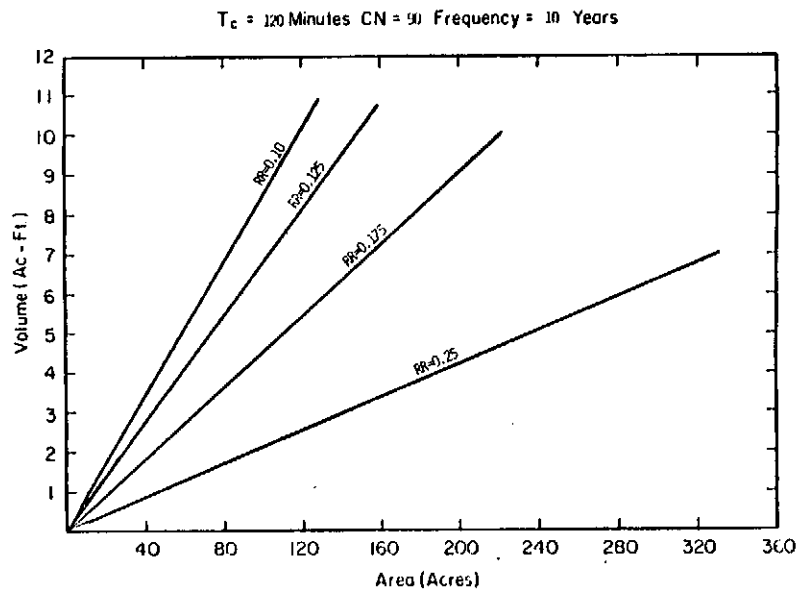
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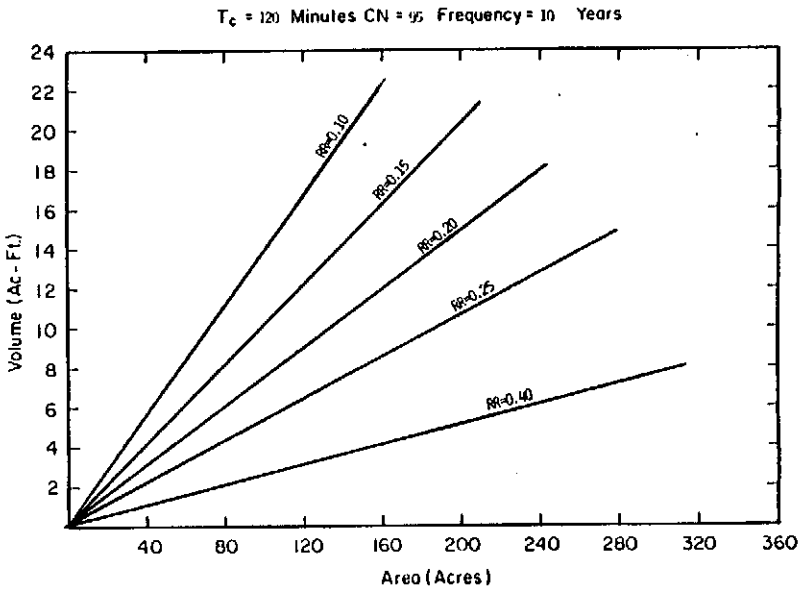
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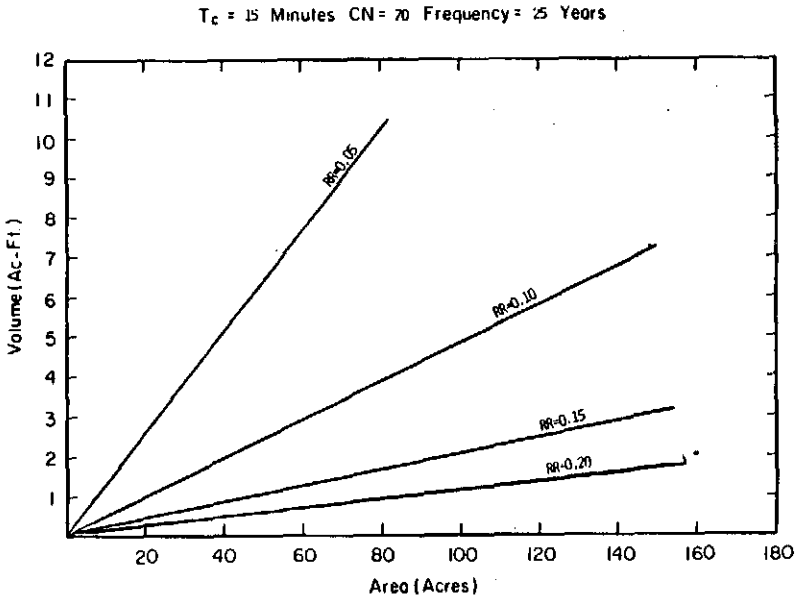
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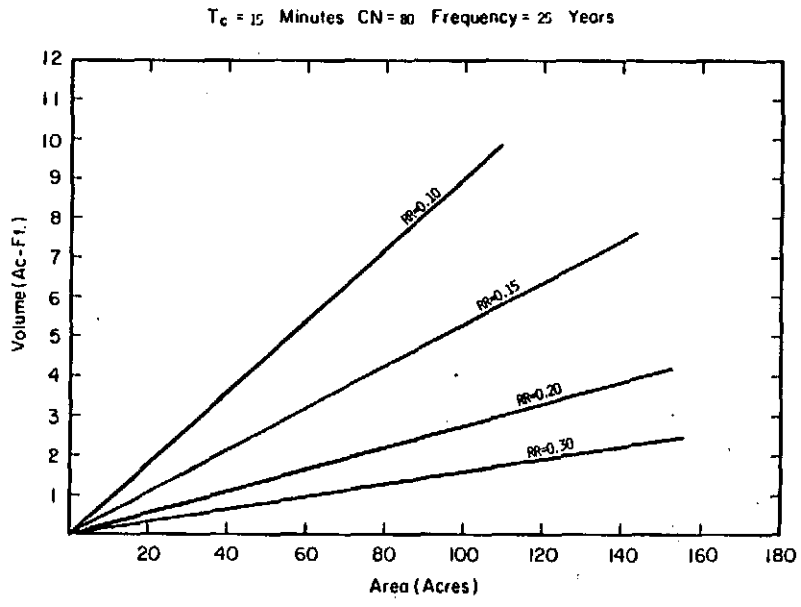
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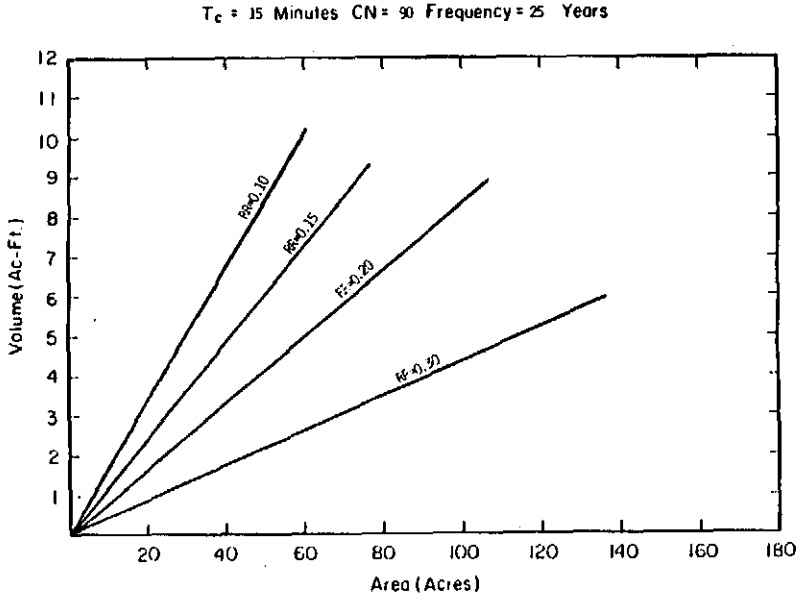
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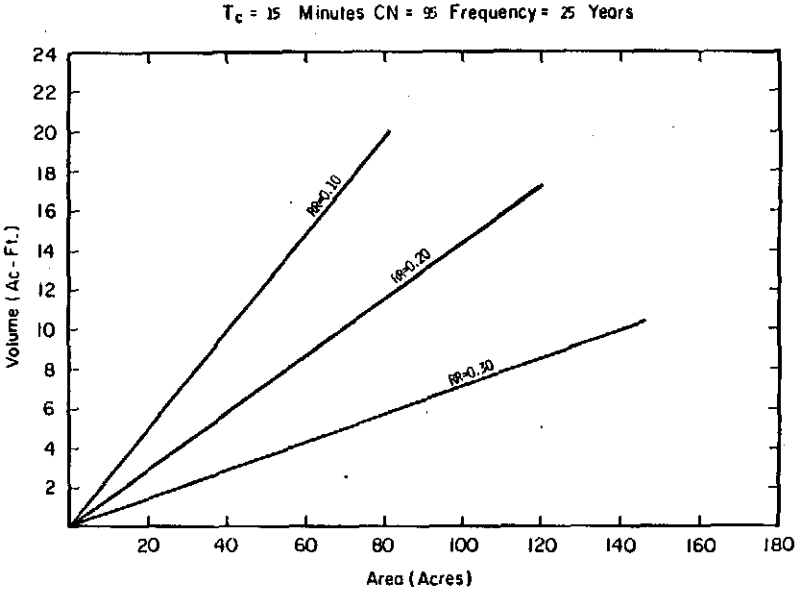
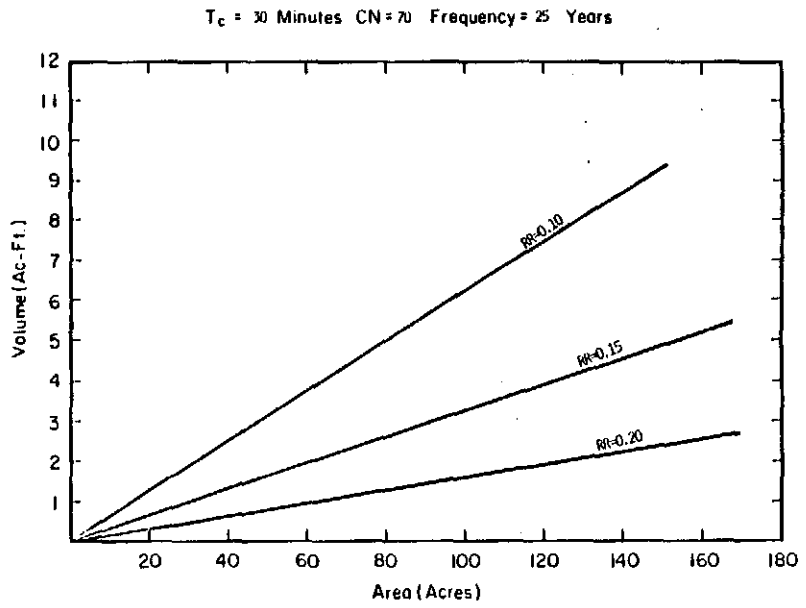
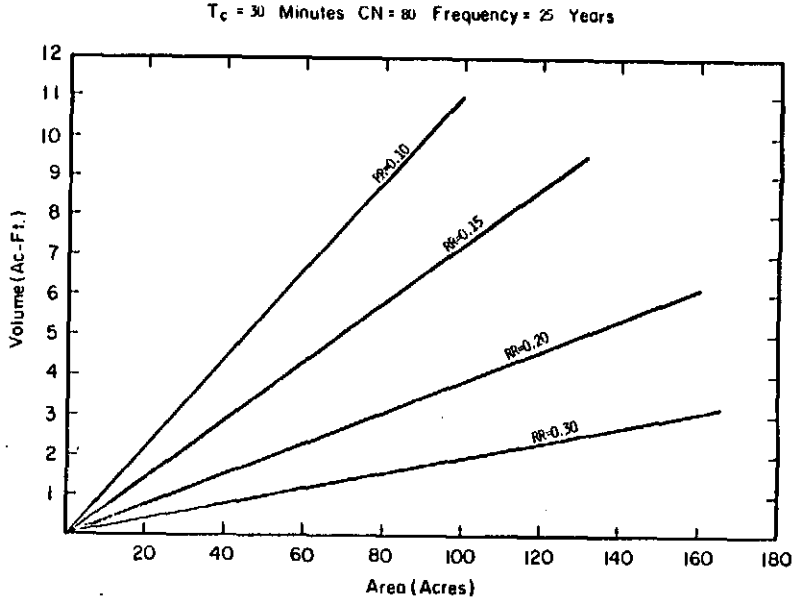
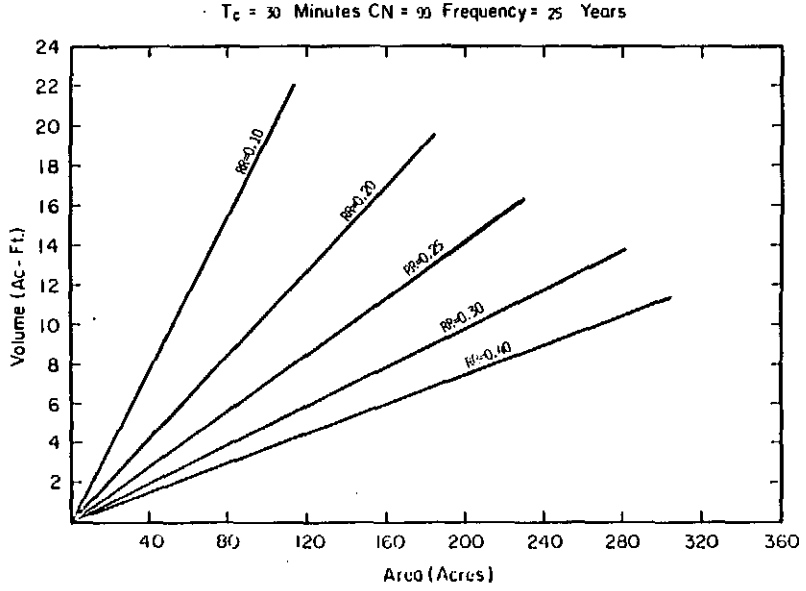
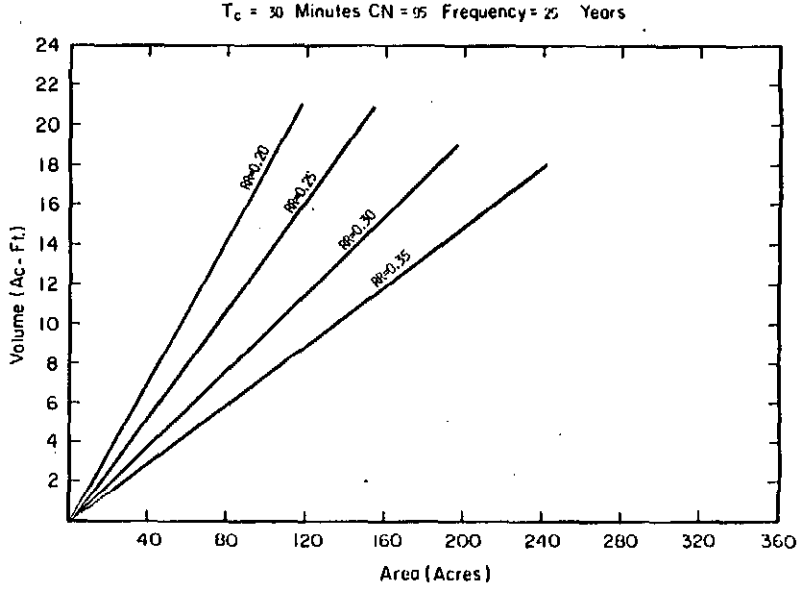
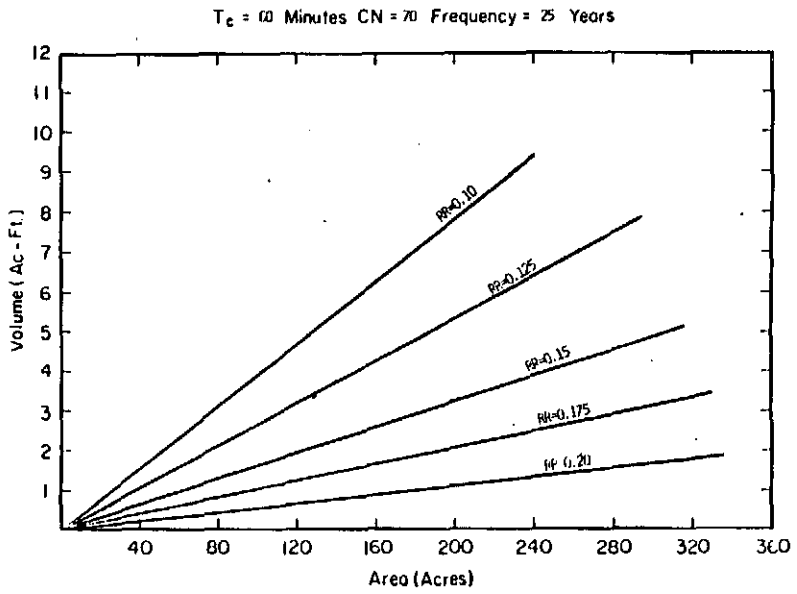

T_c = 15 Minutes CN = 95 Frequency = 25 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.20
RR=0.30

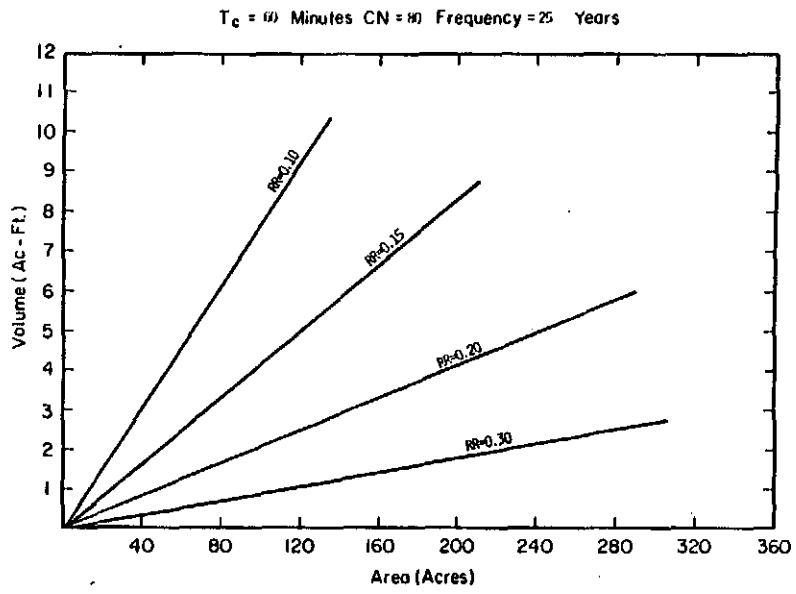

T_c = 30 Minutes CN = 70 Frequency = 25 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.15
RR=0.20

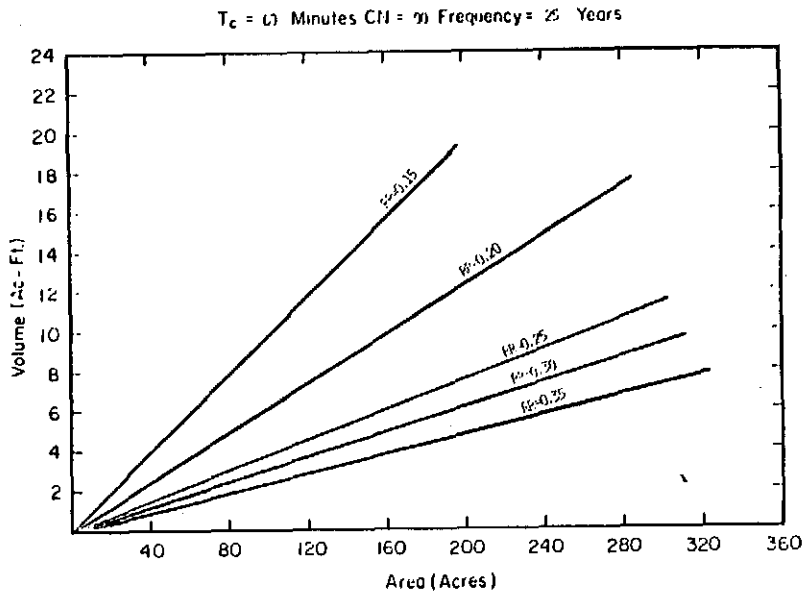

T_c = 30 Minutes CN = 80 Frequency = 25 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.15
RR=0.20
RR=0.30

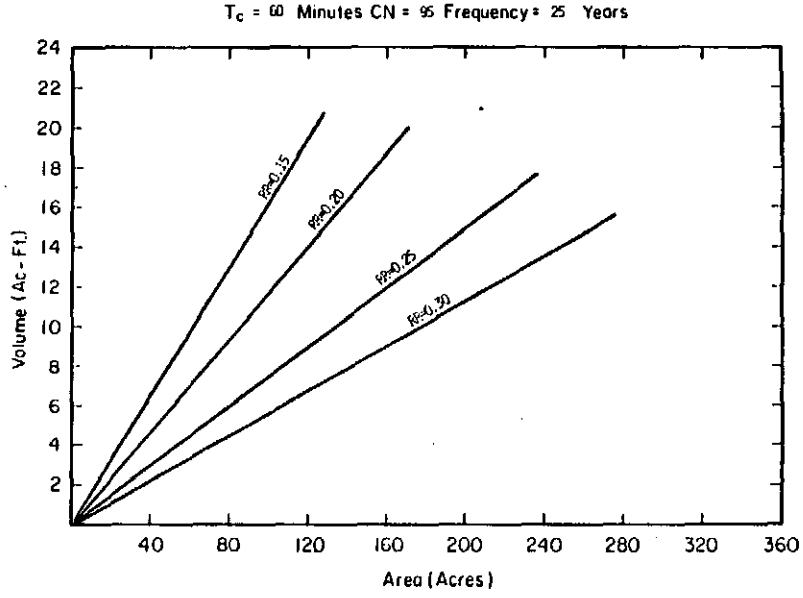

T_c = 30 Minutes CN = 90 Frequency = 25 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.20
RR=0.25
RR=0.30
RR=0.40

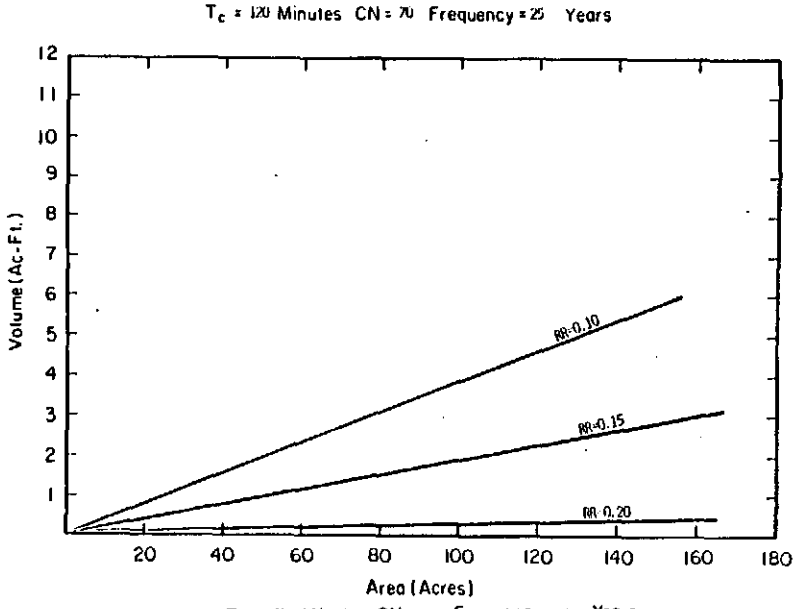

T_c = 30 Minutes CN = 95 Frequency = 25 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.20
RR=0.25
RR=0.30
RR=0.35

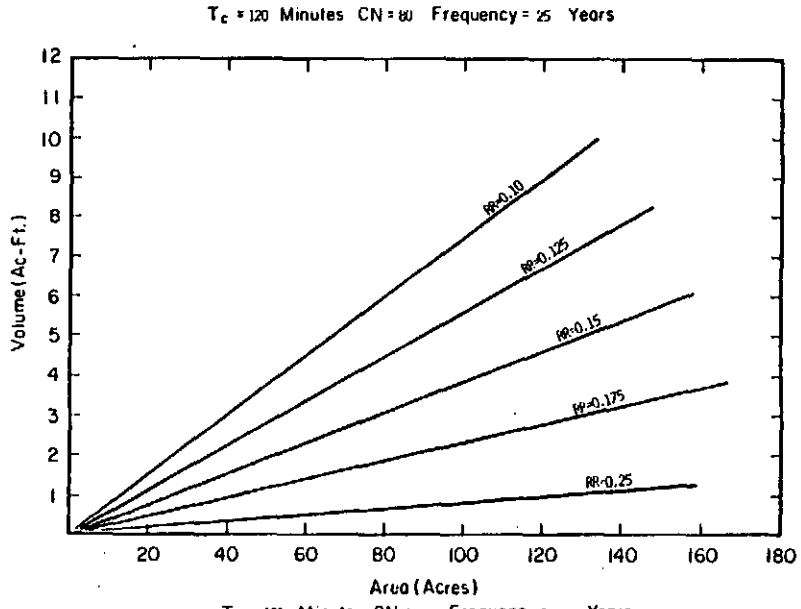

T_c = 60 Minutes CN = 70 Frequency = 25 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.125
RR=0.15
RR=0.175
RR 0.20

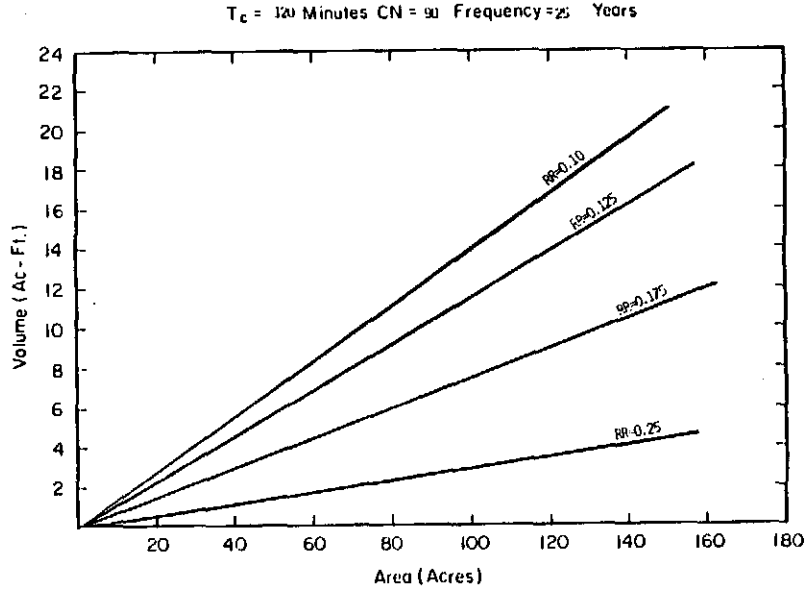

T_c = 60 Minutes CN = 80 Frequency = 25 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.15
RR=0.20
RR=0.30

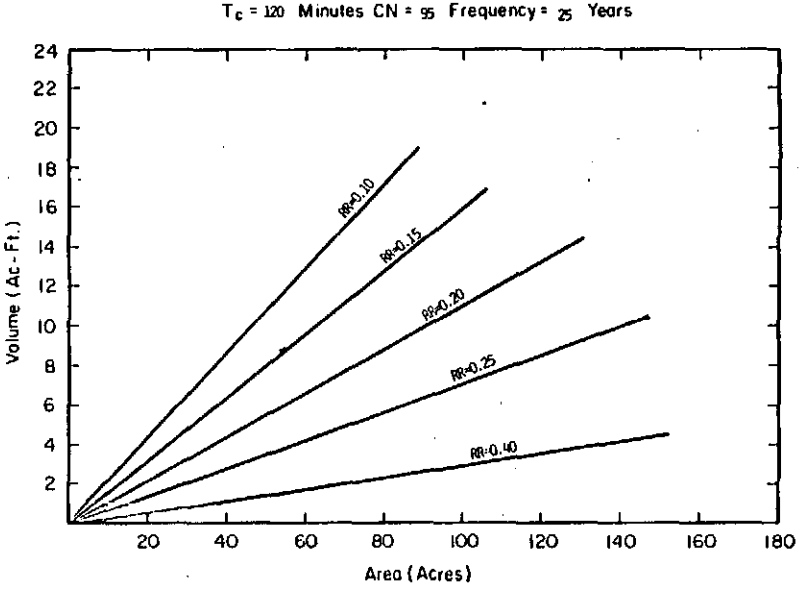

T_c = 60 Minutes CN = 90 Frequency = 25 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.15
RR=0.20
RR=0.25
RR=0.30
RR=0.35

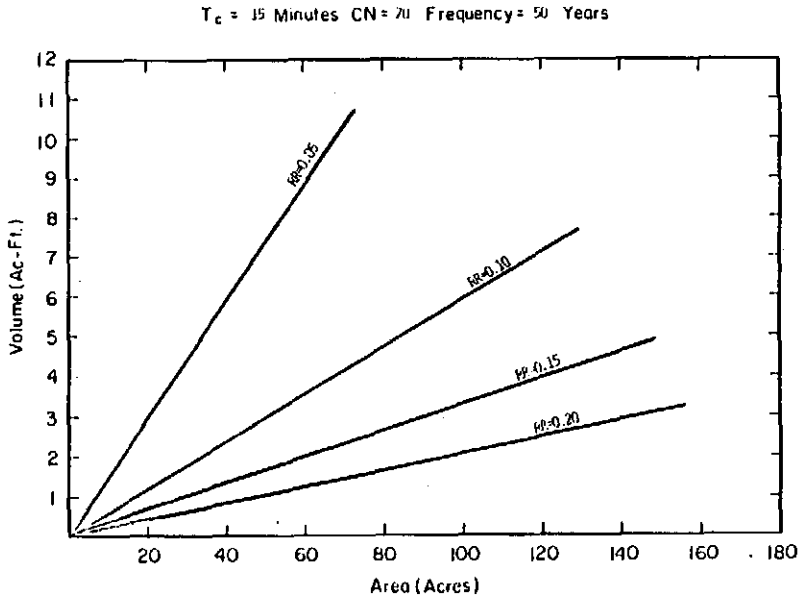

T_c = 60 Minutes CN = 95 Frequency = 25 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.15
RR=0.20
RR=0.25
RR=0.30

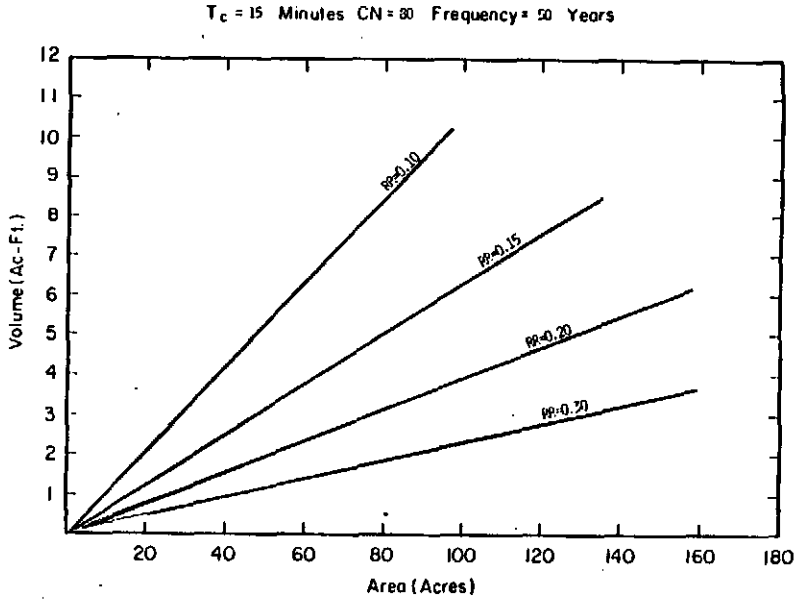

T_c = 120 Minutes CN = 70 Frequency = 25 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.15
RR=0.20

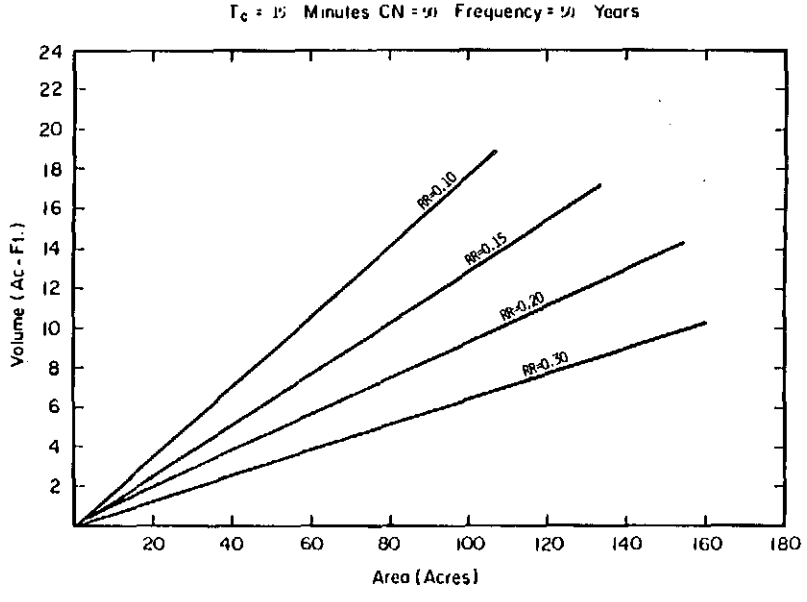

T_c = 120 Minutes CN = 80 Frequency = 25 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.125
RR=0.15
RR=0.175
RR=0.25

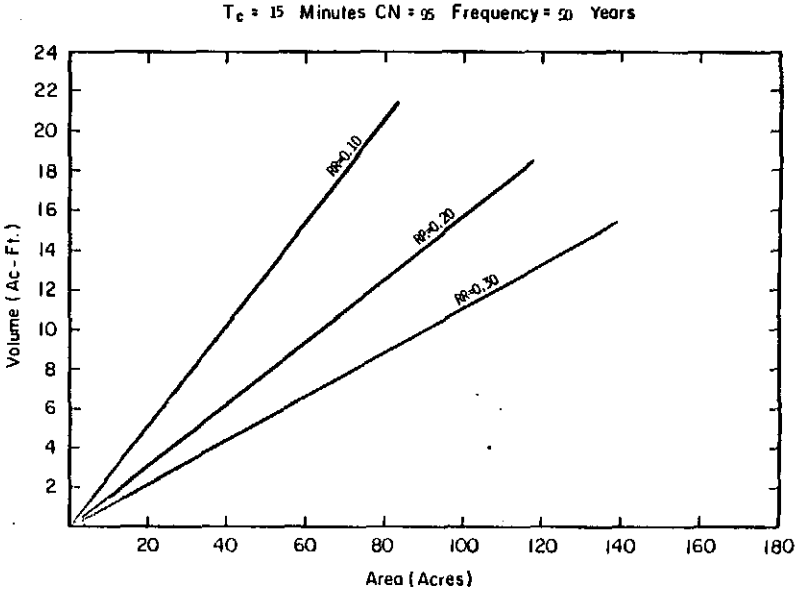

T_c = 120 Minutes CN = 90 Frequency = 25 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.125
RR=0.175
RR=0.25

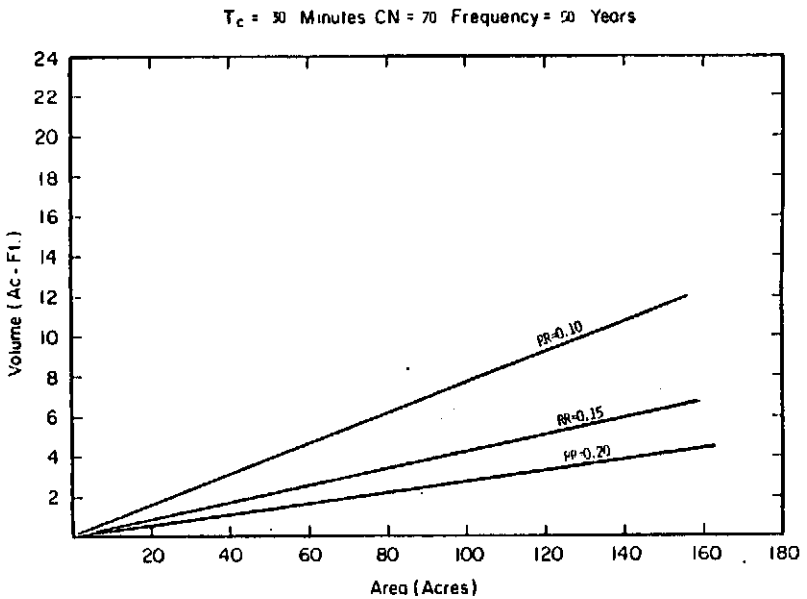

T_c = 120 Minutes CN = 95 Frequency = 25 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.15
RR=0.20
RR=0.25
RR=0.40

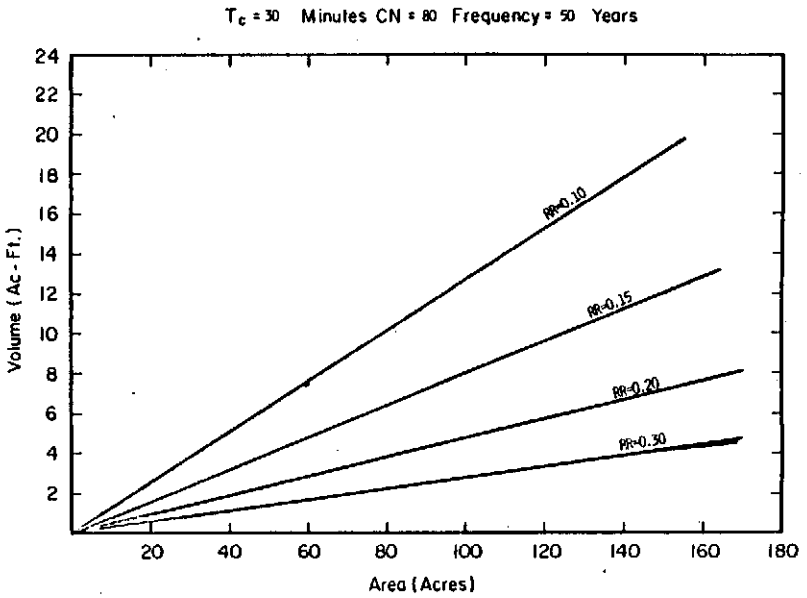

T_c = 15 Minutes CN = 70 Frequency = 50 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.35
RR=0.10
RR=0.15
RR=0.20

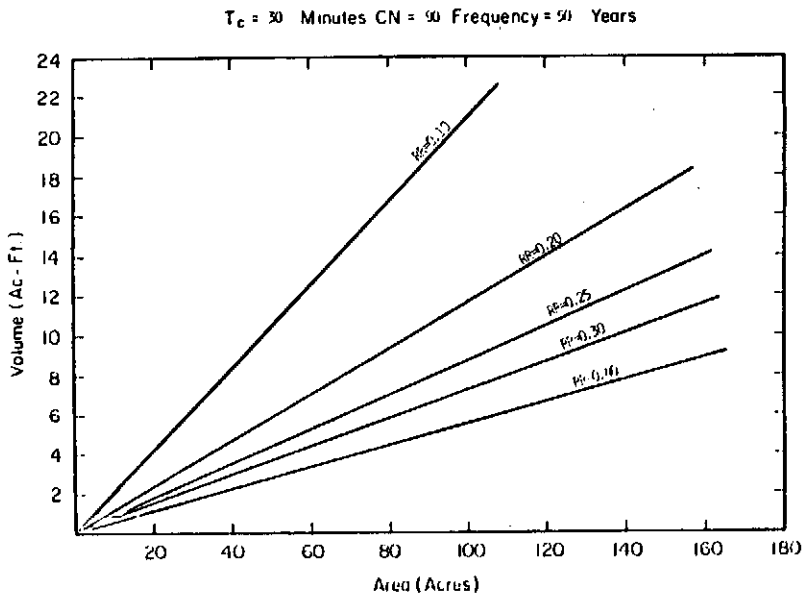

T_c = 15 Minutes CN = 80 Frequency = 50 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.15
RR=0.20
RR=0.30

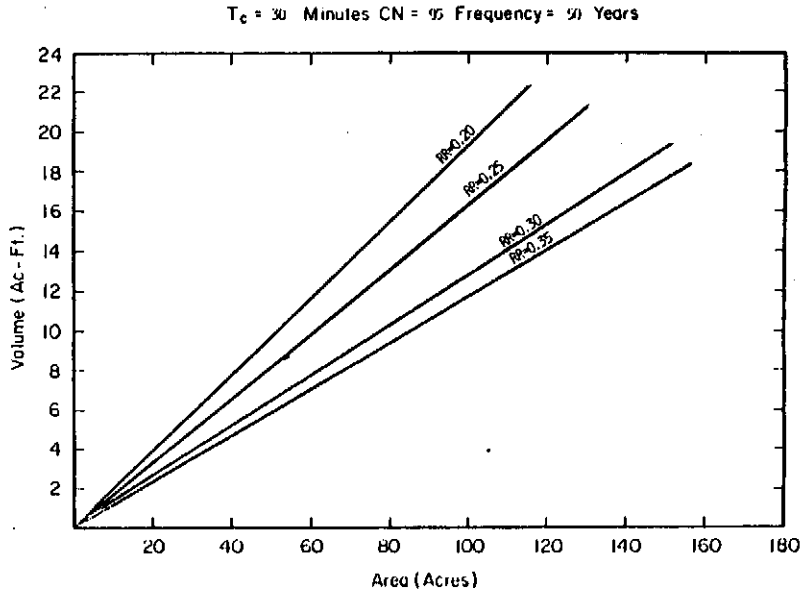

T_c = 15 Minutes CN = 90 Frequency = 50 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.15
RR=0.20
RR=0.30

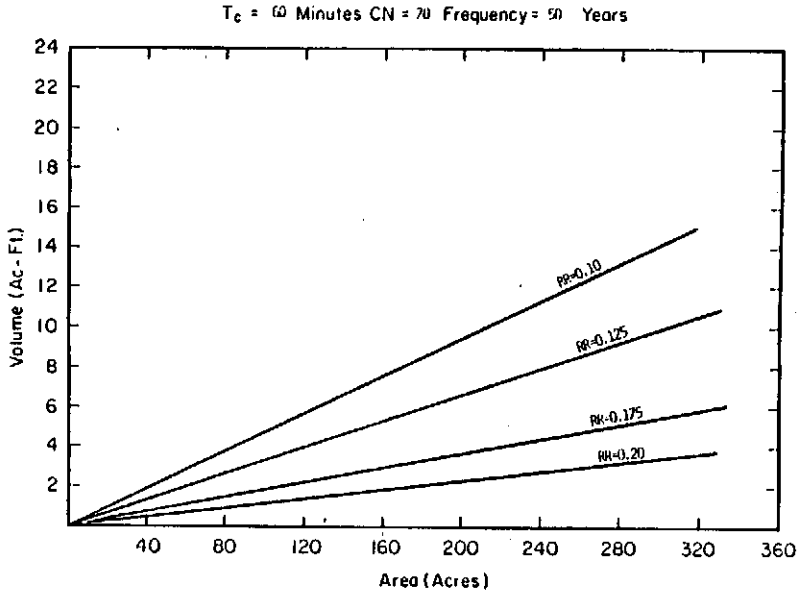

T_c = 15 Minutes CN = 95 Frequency = 50 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.20
RR=0.30

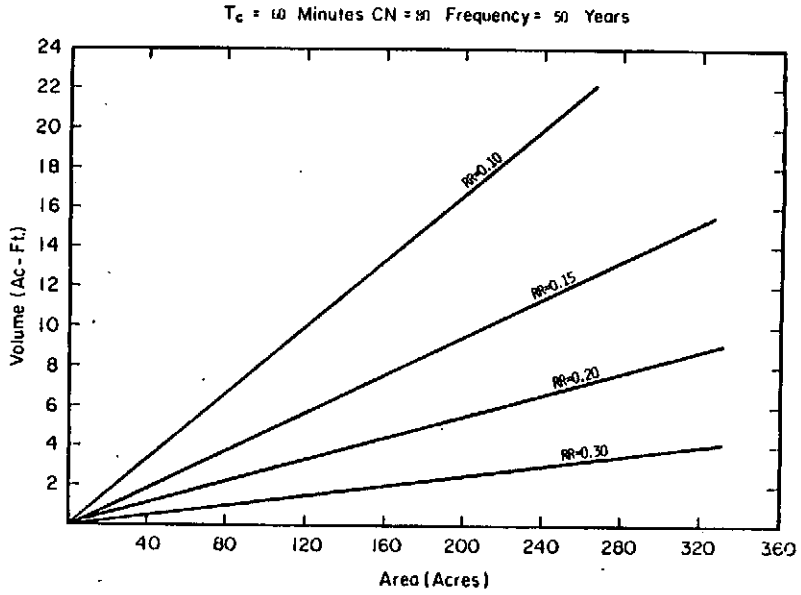

T_c = 30 Minutes CN = 70 Frequency = 50 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.15
RR=0.20

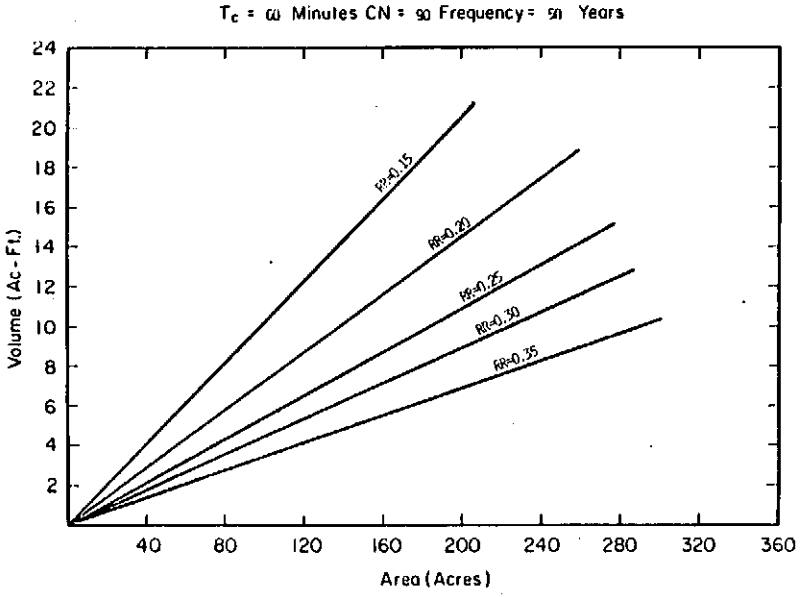

T_c = 30 Minutes CN = 80 Frequency = 50 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.15
RR=0.20
RR=0.30

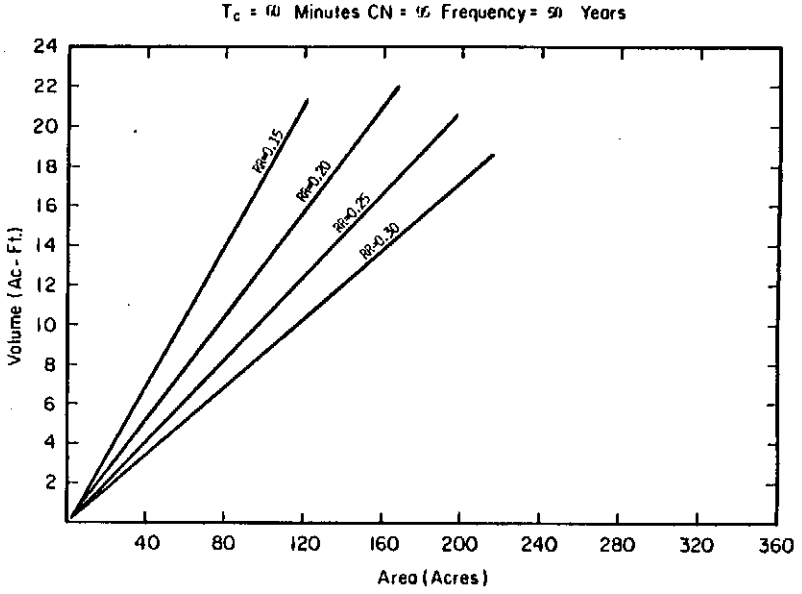

T_c = 30 Minutes CN = 90 Frequency = 50 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.20
RR=0.25
RR=0.30

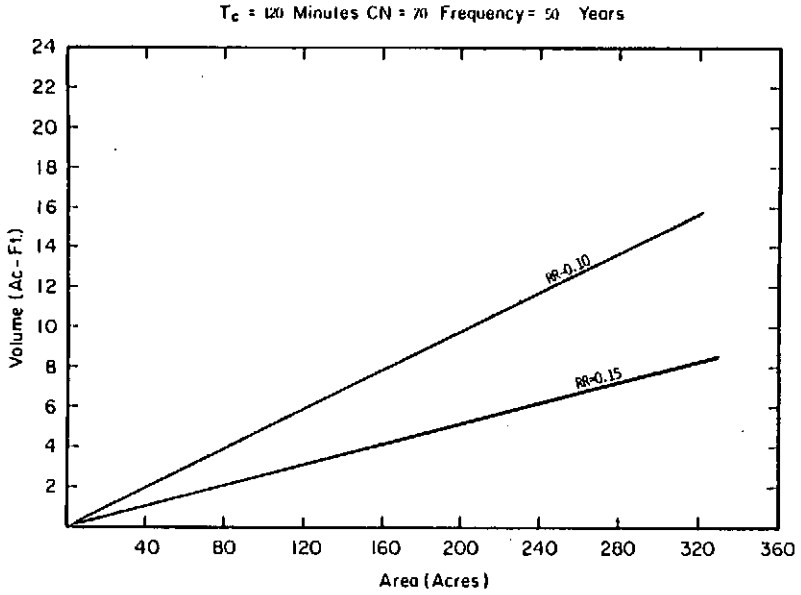

T_c = 30 Minutes CN = 95 Frequency = 50 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.20
RR=0.25
RR=0.30
RR=0.35

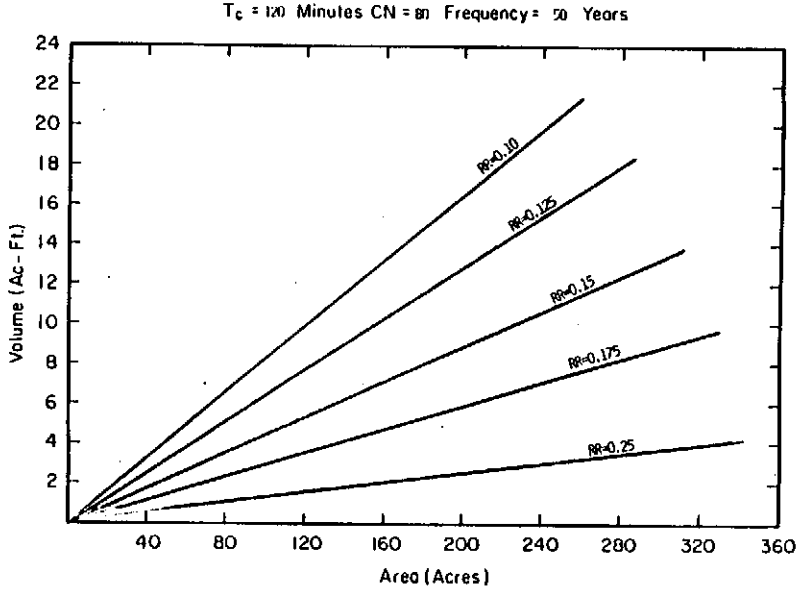

T_c = 60 Minutes CN = 70 Frequency = 50 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.125
RR=0.175
RR=0.20

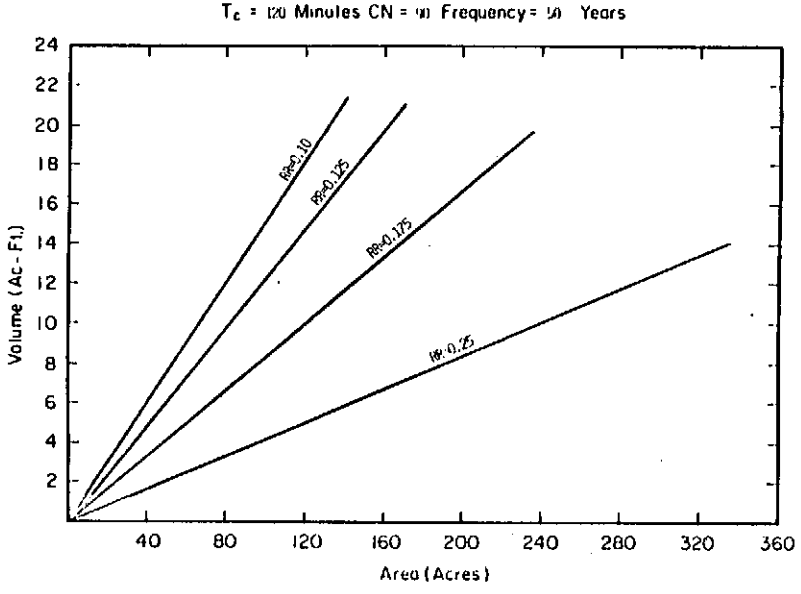

T_c = 60 Minutes CN = 80 Frequency = 50 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.15
RR=0.20
RR=0.30

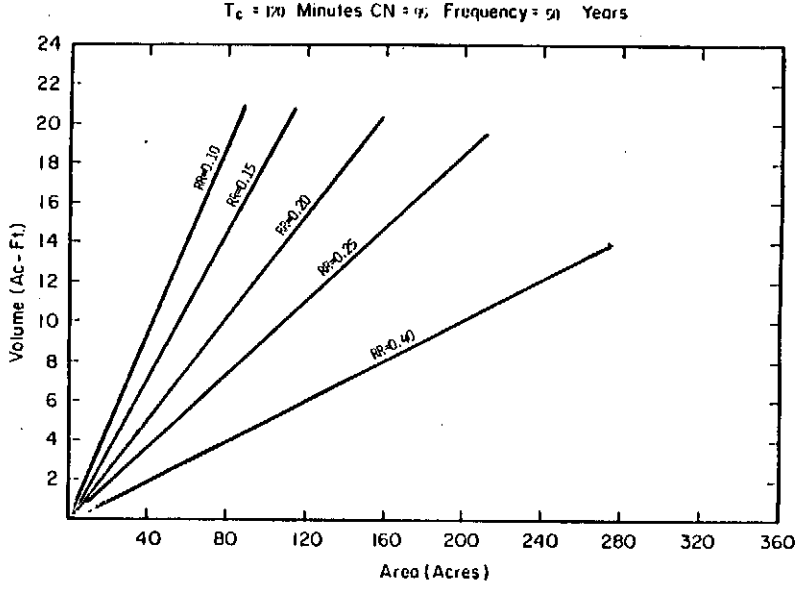

T_c = 60 Minutes CN = 90 Frequency = 50 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.15
RR=0.20
RR=0.25
RR=0.30
RR=0.35

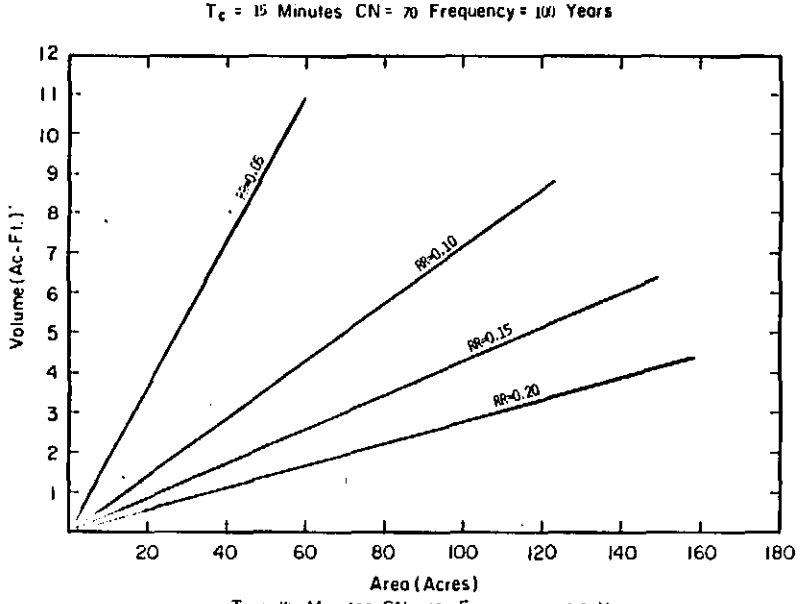

T_c = 60 Minutes CN = 95 Frequency = 50 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.15
RR=0.20
RR=0.25
RR=0.30


T_c = 120 Minutes CN = 70 Frequency = 50 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.15


T_c = 120 Minutes CN = 80 Frequency = 50 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.125
RR=0.15
RR=0.175
RR=0.25


T_c = 120 Minutes CN = 90 Frequency = 50 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.125
RR=0.175
RR=0.25


T_c = 120 Minutes CN = 95 Frequency = 50 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.15
RR=0.20
RR=0.25
RR=0.40


T_c = 15 Minutes CN = 70 Frequency = 100 Years
Volume (Ac-Ft.)
Area (Acres)
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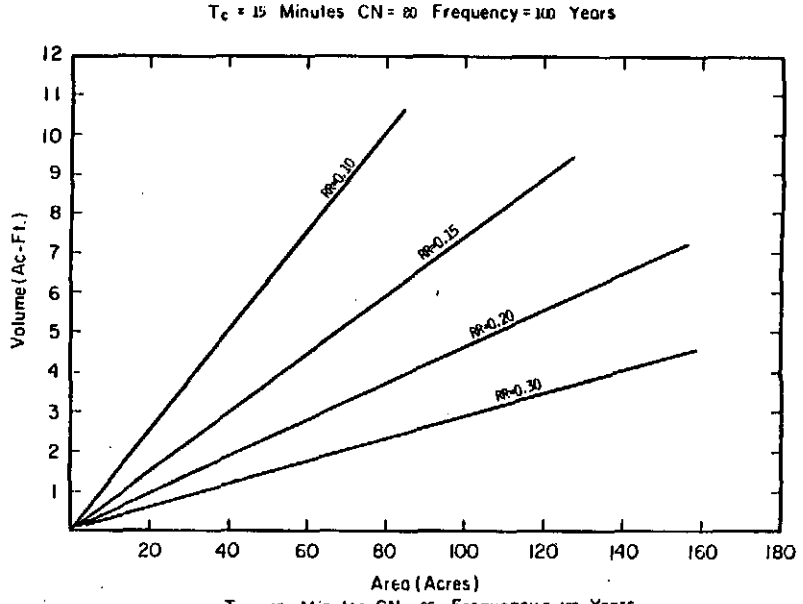
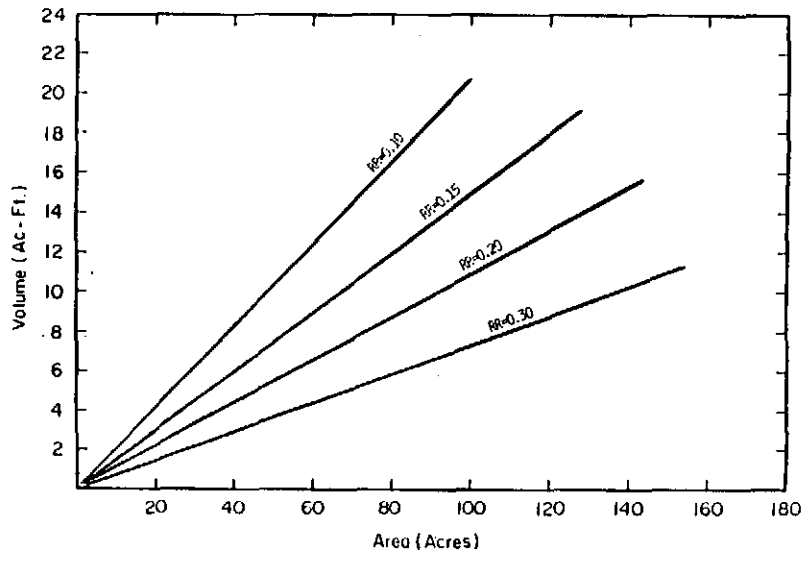
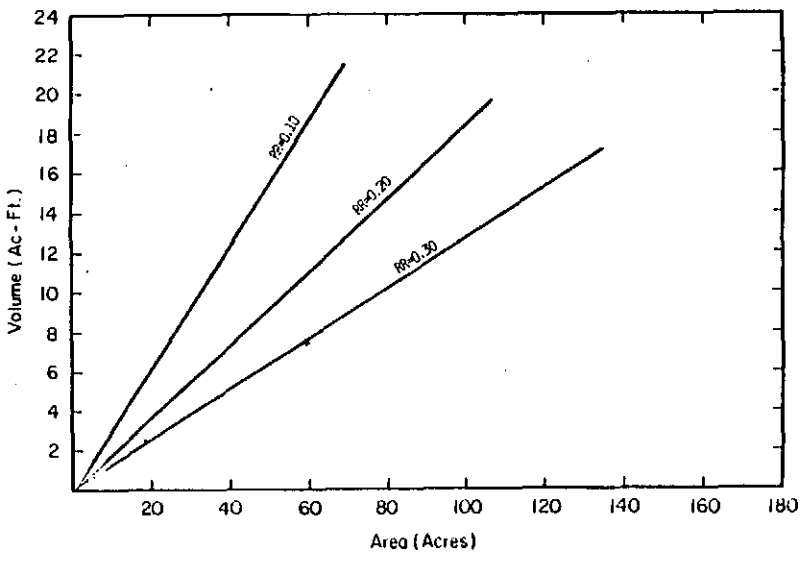
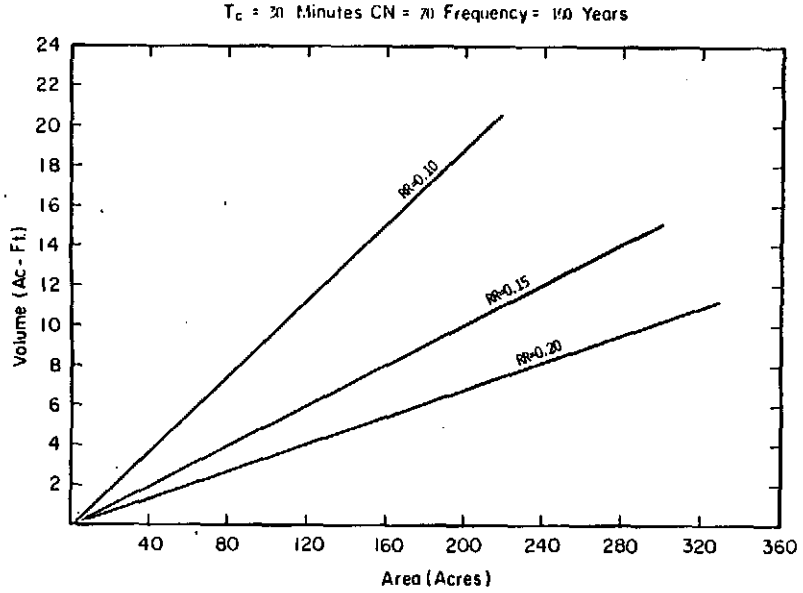
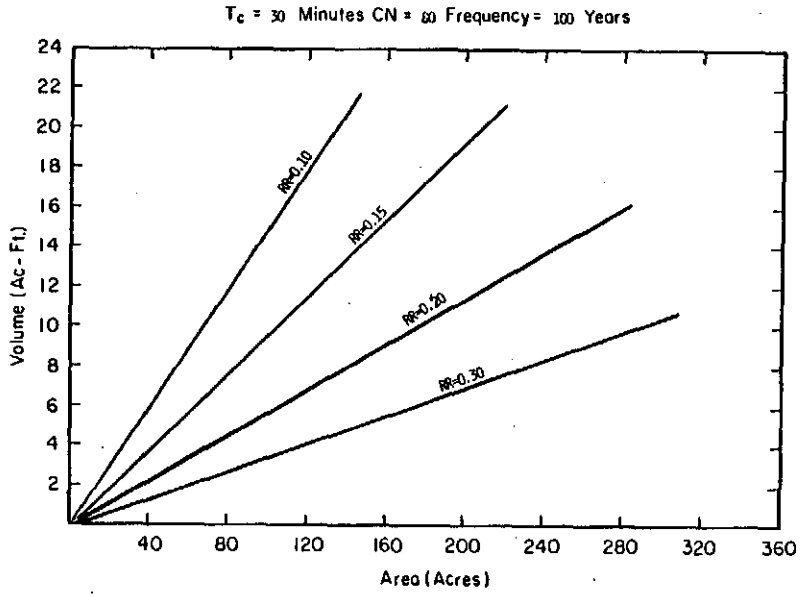
RR=0.10
RR=0.15
RR=0.20

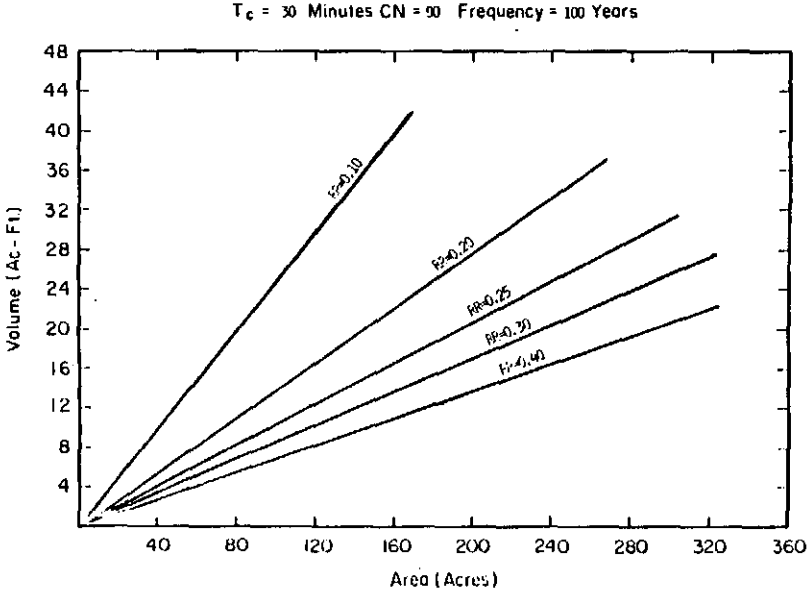

T_c = 15 Minutes CN = 80 Frequency = 100 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.15
RR=0.20
RR=0.30

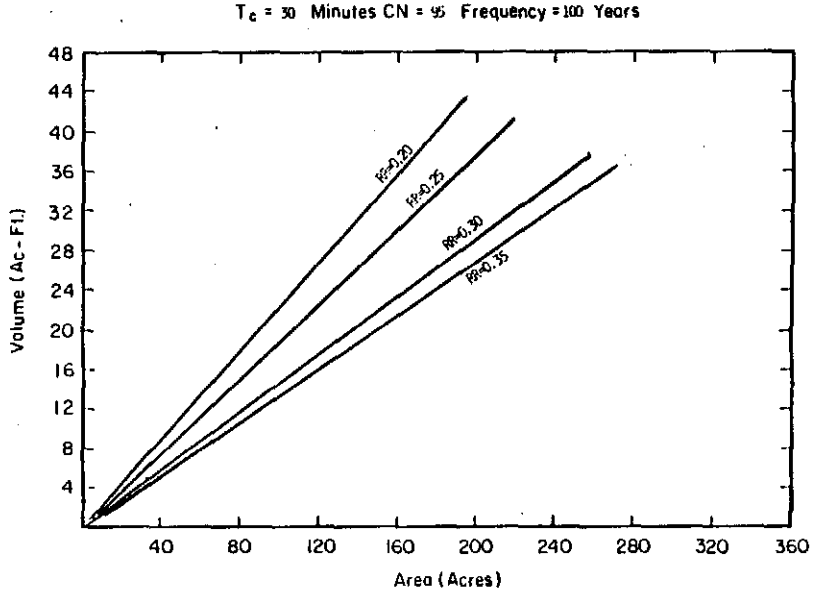

T_c = 15 Minutes CN = 90 Frequency = 100 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.15
RR=0.20
RR=0.30

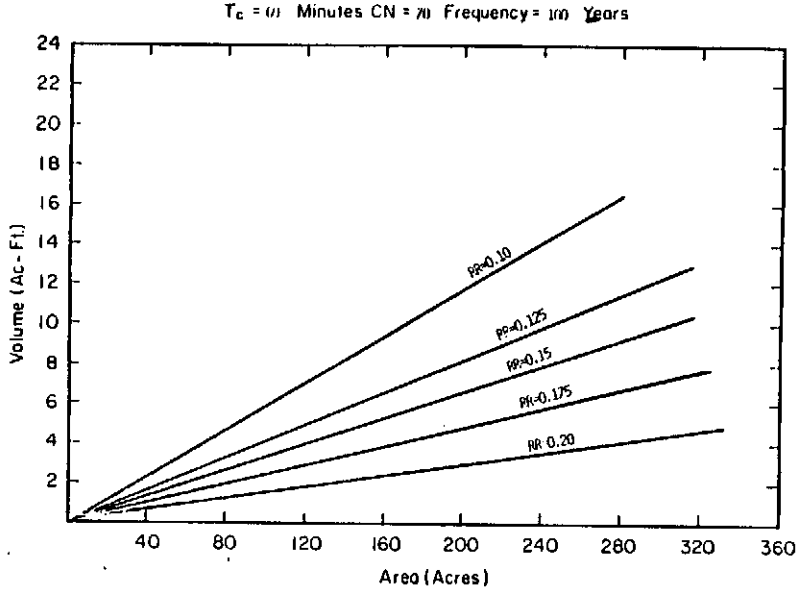

T_c = 15 Minutes CN = 95 Frequency = 100 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.20
RR=0.30

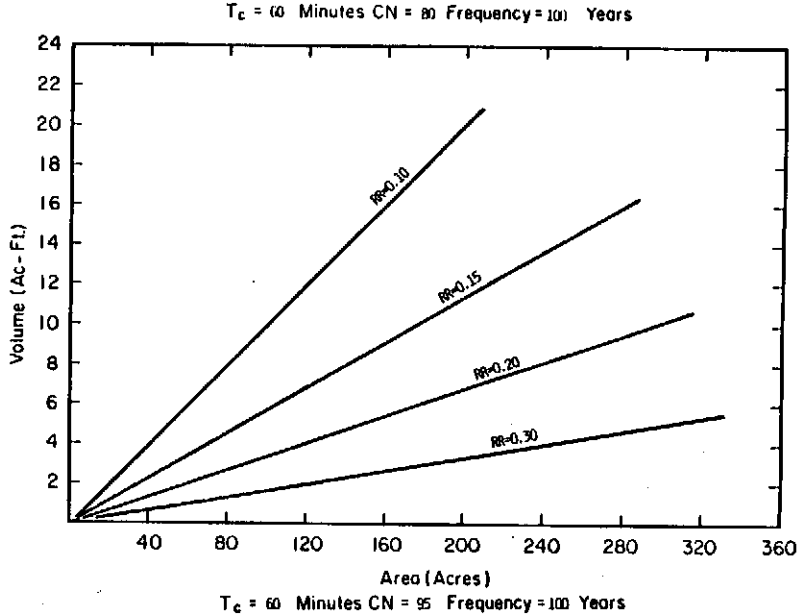

T_c = 30 Minutes CN = 70 Frequency = 100 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.15
RR=0.20

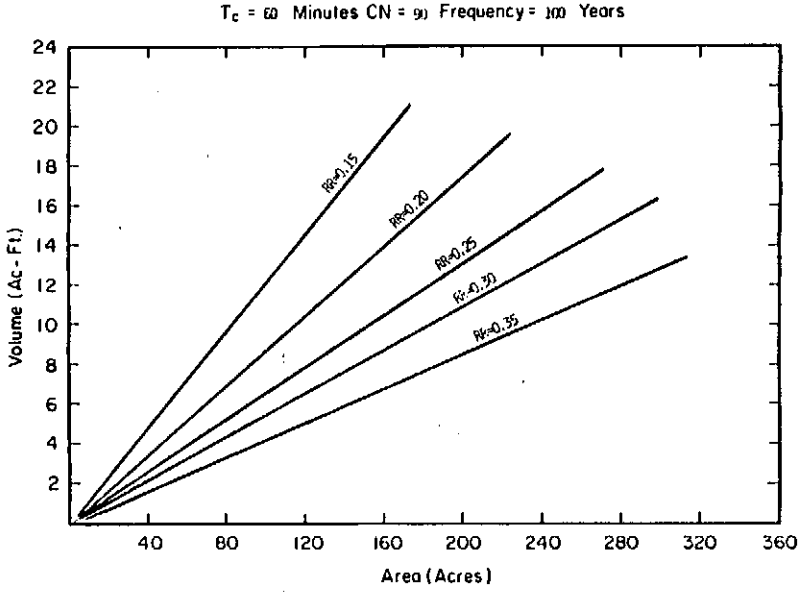

T_c = 30 Minutes CN = 80 Frequency = 100 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.15
RR=0.20
RR=0.30

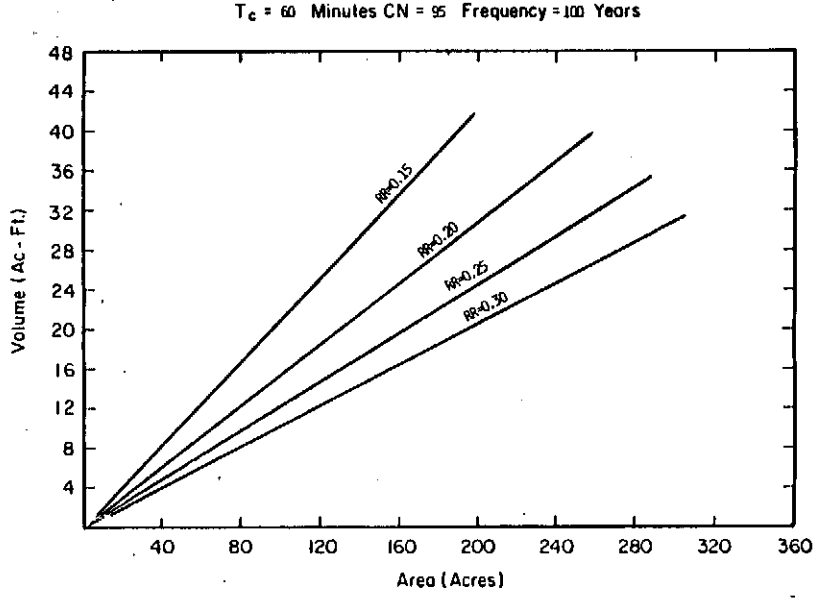

T_c = 30 Minutes CN = 90 Frequency = 100 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.20
RR=0.25
RR=0.30
RR=0.40


T_c = 30 Minutes CN = 95 Frequency = 100 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.20
RR=0.25
RR=0.30
RR=0.35


T_c = 60 Minutes CN = 70 Frequency = 100 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.125
RR=0.15
RR=0.175
RR 0.20


T_c = 60 Minutes CN = 80 Frequency = 100 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.15
RR=0.20
RR=0.30


T_c = 60 Minutes CN = 90 Frequency = 100 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.15
RR=0.20
RR=0.25
RR=0.30
RR=0.35


T_c = 60 Minutes CN = 95 Frequency = 100 Years
Volume (Ac-Ft.)
Area (Acres)
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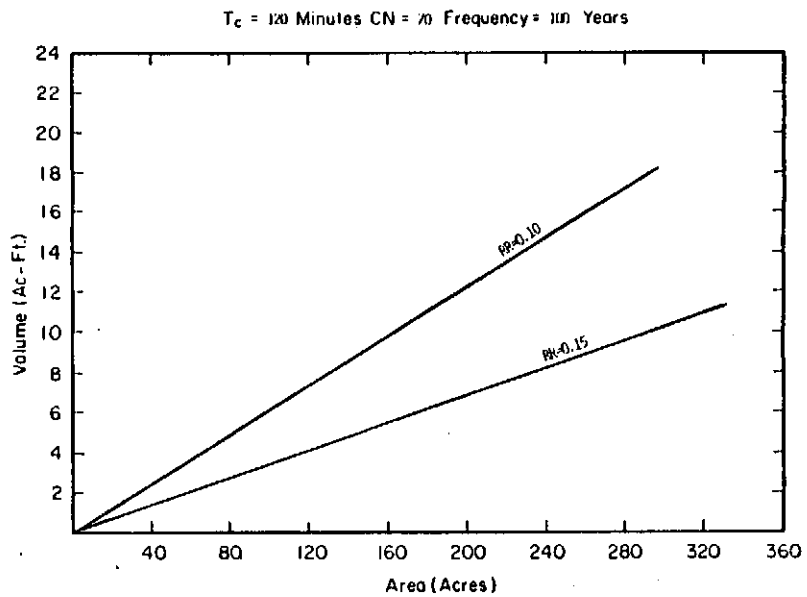
RR=0.20
RR=0.25
RR=0.30


T_c = 120 Minutes CN = 70 Frequency = 100 Years
Volume (Ac-Ft.)
Area (Acres)
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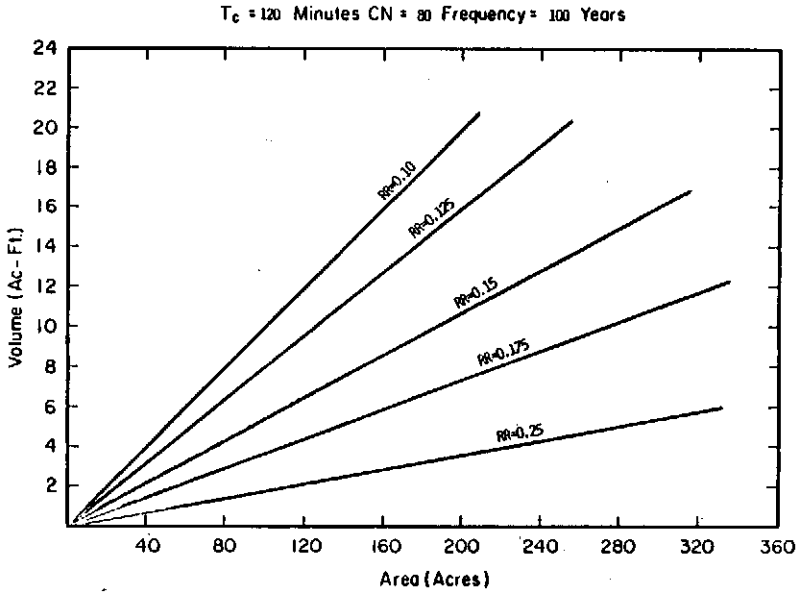
T_c = 120 Minutes CN = 80 Frequency = 100 Years
Volume (Ac-Ft.)
Area (Acres)
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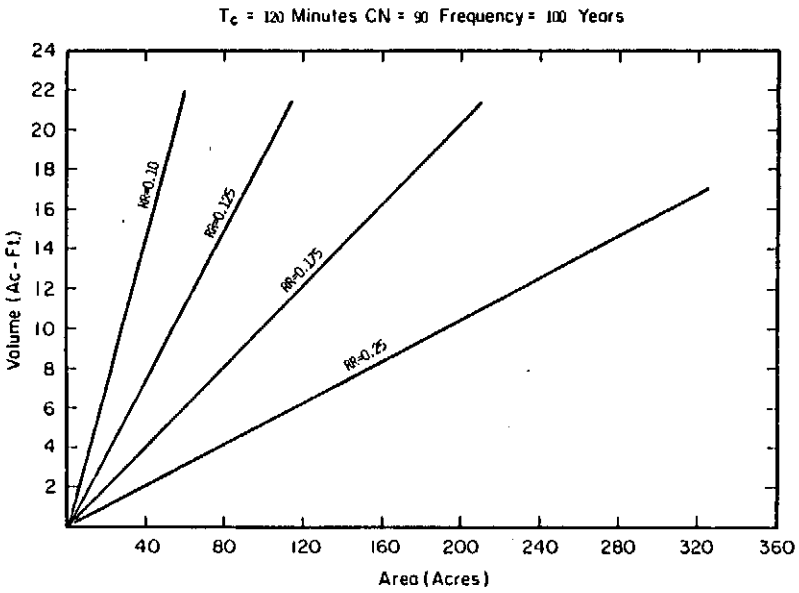
RR=0.15
RR=0.175
RR=0.25


T_c = 120 Minutes CN = 90 Frequency = 100 Years
Volume (Ac-Ft.)
Area (Acres)
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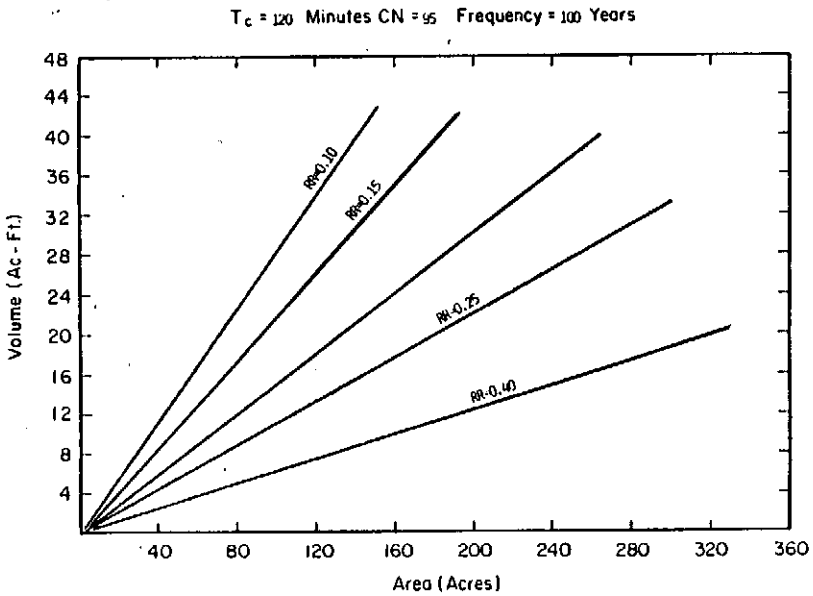
RR=0.175
RR=0.25


T_c = 120 Minutes CN = 95 Frequency = 100 Years
Volume (Ac-Ft.)
Area (Acres)
RR=0.10
RR=0.15
RR=0.25
RR=0.40

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