

**ADDIS ABABA UNIVERSITY**  
**SCHOOL OF GRADUATE STUDIES**

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**SOFTWARE DEVELOPMENT FOR**  
**HYDROLOGIC ANALYSIS AND HYDRAULIC DESIGN OF**  
**MINOR HIGHWAY DRAINAGE STRUCTURES**

**BY**  
**AYDAGNE ZELLEKE**

**March 2007, Addis Ababa**

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MINOR HIGHWAY DRAINAGE STRUCTURES**

**A THESIS PRESENTED TO THE SCHOOL OF GRADUATE STUDIES,**

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**BY**

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When I was on my way to take my entrance exam for postgraduate studies at Amist Kilo on a cold, rainy morning, the Lord spoke to me through a verse, “Be strong, therefore, and prove yourself a man.” 1 Kings 2:2. That verse kept ringing in my spirit and inspiring me throughout my stay of about three years for the second time at this university. I would not have been here today, let alone this thesis work, had it not been for the compassion and love of Jesus Christ. My God, in whom I trusted, is good. Praise His Holy Name!

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## **A B S T R A C T**

In this thesis work, general watershed modeling concepts with relevance to road drainage, such as the rational method (IDF curve approach as well as rainfall frequency analysis approach), SCS Unit hydrograph, East African Flood Model etc. are reviewed. Their theoretical backgrounds as well as their limitations in contemporary highway engineering practice are discussed in adequate detail with special emphasis to practical application in the Ethiopian context. The most common methods of road drainage design in current practice are evaluated for their applicability in the Ethiopian context, the most practical ones are selected, and finally an algorithm is developed to be translated into a computer program which shall serve as an automated system for the selected procedures.

Accordingly, the advantages and pitfalls associated with each method are evaluated in light of contemporary Ethiopian road design practice. Improvements and new approaches are suggested where deemed necessary. Only after selection of the methods to be automated shall algorithms be established that will serve as a basis for the development of the computer code to be written in the Visual Basic programming language.

Although disproportionate importance has been attached to the hydrologic analysis part in the thesis, hydraulic design methods for nominal dimensioning of structures are also established through mathematical techniques in a view to prepare a simplified automation procedure.

As a result of this study, a user-friendly computer software has been developed in accordance with the procedures and methodology outlined in the previous paragraphs. The software is tested with a real project and the output compared with that from a manual design. Both hydrographs are included in this volume for comparison. Outcomes rely appreciably on a number of decisions that are at the exclusive discretion of the drainage designer, and it must be noted that the software will be primarily a vehicle of these decisions.

*Key words: hydrologic analysis; watershed modeling; road drainage design; minor drainage structures; culvert; hydraulic design; software development; object-oriented programming*

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## **ACRONYMS AND ABBREVIATIONS**

A1, A2, A3, A4, B, C, D – *Rainfall regions in Ethiopia shown in the map of fig.2.1.1*

AMC - *Antecedent Moisture Content*

ASCE - *American Society of Civil Engineers*

C - *Rainfall-runoff coefficient*

CN - *Curve Number*

ERA - *Ethiopia Roads Authority*

HEC - *Hydrologic Engineering Center (U.S.)*

IDF - *Intensity Duration Frequency*

KT - *Frequency Factor*

PFF - *Percentage of Full Flow*

POT - *Peaks – Over – a – Threshold*

Q – T - *Relationship between discharge and return period*

RD - *Remotest Distance*

SCS - *Soil Conservation Service (U.S.A)*

TCDE - *Transport and Construction Deign Authority (former name of TCDSCo)*

TRRL - *Transport and Road Research Laboratory (U.K)*

TC - *Time of Concentration*

USACE - *United States Army Corps of Engineers*

## **CHAPTER 1: BACKGROUND AND GENERAL ROAD DRAINAGE CONCEPTS**

### **1.1 BACKGROUND**

Minor drainage structures (Slab, box and pipe culverts spanning not more than 4.5 m with two spans) are a vital component of a highway alignment. What is more, the coordinates, invert level, and project level of drainage structures along that alignment are important inputs for a highway engineer engaged in geometric design. Vertical alignment can never be finalized until these data have been conclusively determined and supplied to the geometric designer.

Until recently, both geometric and drainage design for highways were carried out manually. The advent of various state-of-the-art software for geometric design in the last couple of years, however, has left drainage design behind, so that consulting offices confront difficulty in meeting project deadlines, mostly attributed to the technology gap between manual drainage design and computerized geometric design. This is because vertical alignment cannot be finalized before minor drainage structures are completed.

Moreover, no agreed-upon design standard existed in Ethiopia until ERA recently published a drainage manual with some novel ideas but still leaving much to be desired in terms of technical accuracy, and still not solving the manual method of drainage design that is the major problem in the delay of design projects.

From this student's own experience, the average designer takes about 1 hour to go through the design of a minor drainage structure. Considering the number of culverts one may come across even in a short stretch of highway alignment, that is quite a time-consuming task and worth being automated.

What is more, the existing software on the market have not been designed to take into account specific hydrologic and hydraulic problems in the Ethiopian practice and are too expensive for use by the average design office in the Ethiopian context.

Needless to say, no magic-bullet methodology exists for drainage design owing to the highly stochastic nature of hydrologic events.

*"The estimation of the Q-T relationship has motivated a multitude of journal articles, reports and conference proceedings, and these reflect a wide variety of approaches. Despite the volume of literature that has resulted from both theoretical and applied investigations, no consensus exists as to how best to proceed."* (Admasu, 1989)

## **1.2 OBJECTIVES**

The primary objective of this thesis is to facilitate the design of minor drainage structures in the Ethiopian practice by developing software for some selected design procedures that have found widespread use in the country.

The specific objective is that the thesis will result in a comprehensive software package for the hydrologic analysis and hydraulic design of minor highway drainage structures.

## **1.3 METHODOLOGY**

The methodology to be followed comprises four important stages:

### **a) Literature survey and examination of the practical aspects of drainage design:**

Different approaches to hydrologic analysis in drainage design are discussed from literature and the *ERA Drainage Manual* according to their applicability and frequency of use in Ethiopia. Alternative options that are not very common in the Ethiopian practice are also evaluated.

### **b) Evaluation of selected procedures for practical application in Ethiopia:**

Design procedures and hydrologic approaches are examined in view of their applicability in the Ethiopian practice. The most practical ones are selected for automation in consideration of their easy application, widespread use, and relatively less complexity of programming involved.

**c) Establishment of algorithms, flowcharts and development of the software:**

Once the procedures are selected and the theoretical principles for the development of the software are established, resulting algorithms are written, flowcharts are developed and interfaces for data entry and output are designed in consideration of user-friendliness and logical reliability.

**d) Writing of the code in Visual Basic programming language:**

The software will be developed using Visual Basic programming language as a multiple document interface (MDI) Windows application. It is intended that the software comprises several approaches to hydrologic analysis so that the designer can select a method of his choice and calculate his design discharge accordingly.

#### **1.4 GENERAL CONCEPTS - HYDROLOGIC ANALYSIS**

In order to optimize culvert design, many factors are considered vital. At many sites, designers have valid reasons for providing a safety factor in designs. These reasons include uncertainty in the design discharge estimate, potentially disastrous results in property damage or damage to the highway from headwater elevations which exceed the allowable, the potential for development upstream of the structure, and the chance that the design frequency flood will be exceeded during the life of the structure.

Quantitative evaluation of these variables would amount to a risk analysis, but for minor hydraulic structures, many of these factors must be evaluated intuitively. The purpose of this thesis work would be to come up with a system design that enables the designer to maximize structure performance and to optimize design in accordance with his/her evaluation of these site constraints, design parameters, and construction and maintenance costs.

### 1.4.1 RATIONAL METHOD

In road drainage design, the principal purpose of hydrologic analysis is to determine the maximum quantity of run-off (discharge) that can be accumulated at a certain drainage outlet (usually a gully) along a highway alignment section.

The Rational method, one of the most commonly used simplified models for road drainage, is primarily based on the concept that the maximum discharge from a watershed will always occur when the rain lasts long enough at its maximum intensity to enable all portions of the basin to contribute to the flow. The Rational formula in its familiar form as shown in Eq. 1.2.1 was presented by David Ernest Lloyd-Davies in 1906.

$$Q=0.00278 * C * I * A * F.S \quad (\text{Eq. 1.2.1})$$

Where

- Q – Discharge at outlet (m<sup>3</sup>/s)
- C – Rainfall-Runoff Coefficient
- I – Maximum probable rainfall Intensity (mm/hr)
- A– Catchment Area (hectares)
- F.S.- Factor of Safety (factor of ignorance)(Optional)

The drainage outlet usually has a delineated tributary catchment area (sometimes referred to as a watershed), designated with variable A in the above equation, that contributes runoff to it, the size of which can be easily determined from photogrammetry, and less accurately, by the help of maps (1:50,000 and above if available) or topographic survey.

The Rational method takes into account the following hydrologic characteristics or processes.

- Rainfall intensity
- Rainfall duration
- Rainfall frequency
- Catchment area
- Hydrologic abstractions
- Runoff Concentration
- Run-off Diffusion

Hence, the peak discharge is the product of

- Runoff coefficient
- Rainfall intensity
- Catchment area

with all the above processes lumped into these three parameters. (*Ponce, 1989*)

Although simplistic, the rational method, especially coupled with rainfall frequency analysis and a judicious fine-tuning of runoff coefficients, is generally considered to serve justice to the determination of runoff quantity for road drainage purposes with reasonable dependability.

#### **1.4.1.1. RUNOFF COEFFICIENT**

In essence, the runoff coefficient is the ratio of the actual (i.e. calculated) peak runoff to the maximum possible runoff rate.

Runoff coefficients account for the processes of

- a. Hydrologic abstractions
- b. Runoff diffusion

*Hydrologic abstractions* include: interception, infiltration, surface storage, evaporation and evapo-transpiration.

*Runoff diffusion* is the catchment's ability to attenuate the flood peaks.

Some of the variables that influence the value of the runoff coefficient include:

- Soil characteristics
- Rainfall duration
- Rainfall intensity
- Shape of drainage area
- Soil moisture (AMC)
- Watershed slopes
- Design frequency
- Land use characteristics
- Depression storage
- Interception

In selecting/setting the runoff coefficient for a catchment the following considerations should be well noted.

- Less frequent storms (e.g. 50 years) require the use of higher coefficients because infiltration and other abstractions have a reduced role in runoff generation for larger storms.
- Coefficients usually given in tables represent average antecedent moisture conditions and are not designed to account for multiple storms or storms of very long duration.
- Special design cases usually warrant the use of higher runoff coefficients to simulate the existence of wet antecedent moisture conditions in the catchment.

- Experimental evidence has shown that runoff coefficients tend to increase from one storm to another occurring shortly thereafter, with runoff coefficients tending to increase with storm duration.
- Design values of runoff coefficients are usually a function of rainfall intensity and, therefore, of rainfall frequency.
- Higher values of runoff coefficients are applicable for higher values of rainfall intensity and return period.

The runoff coefficient is a function of the Terrain type (slope, to allow for runoff diffusion) and the Terrain cover (land use), which are parameters considered to be principal lumped indicators of catchment response to a storm event.

#### **1.4.1.2 RAINFALL INTENSITY AND FREQUENCY**

The rational method bases the calculation of peak flow on a chosen rainfall frequency. However, it must be emphasized here again that in nature, the frequencies of storms and floods are not necessarily synchronized largely due to the effect of antecedent moisture conditions, variability in channel transmission losses, over-bank storage, and the like.

When a watershed is saturated from recent rains, additional rainfall naturally causes substantial runoff. On the other hand, for example early in the rainy season in Ethiopia, rainfall may cause very limited runoff except in particularly rocky watersheds. If a given rainfall can cause different amounts of runoff, then rainfall frequency and runoff frequency cannot always be the same: a 100-year recurrence interval rainfall does not always cause a 100-year peak flow. It is in view of this incongruity that in practice runoff coefficients are usually adjusted, that is, to reflect postulated fluctuations in runoff frequency. This procedure, while empirical, has seemed to work well. (*Ponce, 1989*)

A rainfall frequency (Return period) applicable to the given design condition is selected, as frequency varies with the type of project and the degree of protection desired for that specific structure. The size and importance of the project, as well as design criteria issued by ERA and regional state design offices have a bearing in the selection of design frequency. The longer the return period i.e. the smaller the frequency, the greater the peak discharge calculated by the rational formula.

Different methods are used to determine the rainfall intensity to be employed in the rational method of catchment modeling. Discussed below are two of the most commonly used techniques for this purpose in the rational method.

- IDF (Intensity-Duration-Frequency) curve method
- Rainfall Frequency analysis

The following methods are also discussed due to their common application in the hydrologic analysis of catchments for road drainage.

- SCS Unit hydrograph
- East African Flood Model
- HEC-1 Flood model

## CHAPTER 2: APPROACHES TO HYDROLOGIC ANALYSIS

### 2.1 IDF CURVE METHOD

IDF curve method of Rainfall Intensity determination is employed for regions where observation and record of hydrologic data for a sufficient period of time have allowed the development of IDF curves, using mathematical methods, from isopluvial maps containing depth-duration-frequency data. Using these curves, one can determine the maximum rainfall intensity for the region applicable to a given frequency (return period) of occurrence.

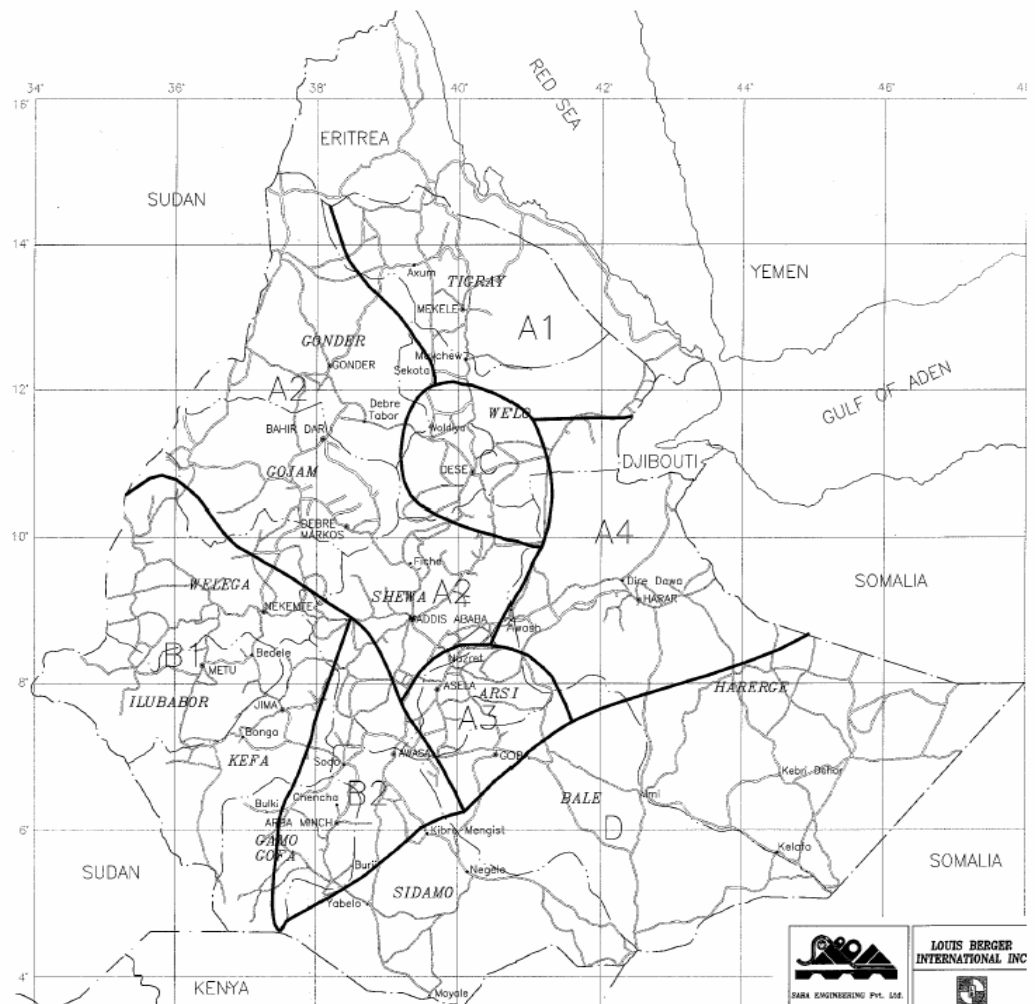


Fig. 2.1 Hydrologic/rainfall regions of Ethiopia (*ERA Drainage Manual (2002)*)

The IDF curves provided in the ERA manual have been regressed into equations for the purpose of this study. At first glance, it seems that the curves readily fit a logarithmic trend, but the logarithmic trend does not accurately represent the behavior of the curve for durations of more than 2-hours. Since the index  $r^2$  is not significantly different for the power equation, and the power equation represents more satisfactorily durations substantially in excess of 2 hours, it must be selected to represent the IDF curves. For the data used in the regression and the output equations used in the software, please refer to Appendix 1.

The procedure employs regressed models of the 4 IDF curves supplied by ERA for the eight hydrologic regions of Ethiopia to calculate the maximum rainfall intensity for a given region and frequency. The hydrologic or rainfall regions as shown in the map of fig. 2.1.1 have been classified and presented in the *ERA Drainage Manual (2002)*.

This map has been developed by applying statistical techniques to rainfall data from across the country and shows that the country can be divided into the above hydrological regions displaying similar rainfall patterns.

The following data are needed for the determination of rainfall intensity using IDF curves for a given design situation.

### **2.1.1 REMOTEST DISTANCE**

Hydraulic length of the longest runoff path measured from the furthest point in the watershed (with respect to the outlet) to the point of the outlet. It can be determined from photogrammetry and less accurately from a (1:50,000 or above) map of the catchment area.

### **2.1.2 AVERAGE VELOCITY OF RUNOFF**

Average velocity of flow on the catchment (a function of the catchment characteristics i.e. terrain type, terrain cover, catchment's hydrologic condition, soil type, etc). A designer can set up his own table of discharge coefficients after careful observation of

the catchment soil type, land use, terrain characteristics and its behavior in a storm event.

### 2.1.3 TIME OF CONCENTRATION

Time of concentration is the time that it takes a parcel of water to travel from the furthest point in the catchment to the outlet. Once the Remotest Distance and Average velocity of runoff are determined, the Time of Concentration is calculated from Eq. 2.1 below.

$$T_c = \frac{RD}{60 * Avg.V} \quad (\text{Eq. 2.1.1})$$

where  $T_c$ - Time of Concentration (min)

$RD$  – Remotest distance (m)

$Avg.V$  – Average velocity of runoff (m/s)

Since we seek to determine the peak runoff at the outlet, the  $T_C$  is made equal to the duration of the rainfall. Then the maximum possible rainfall intensity for a given probability of occurrence corresponding to that duration is determined from the IDF curve.

It is relevant to mention here that the process of runoff concentration can lead to three distinct types of catchment response.

- I. *Concentrated catchment flow*: Effective rainfall duration = Concentration Time

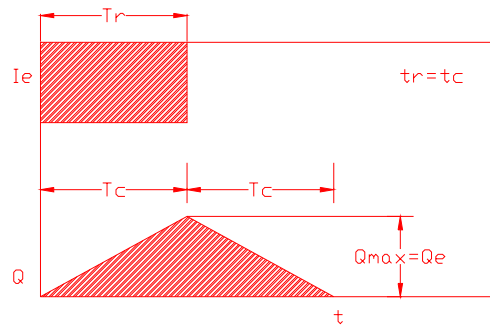


Fig. 2.2: Concentrated catchment flow

II. Super-concentrated catchment flow: Effective Rainfall duration > concentration time

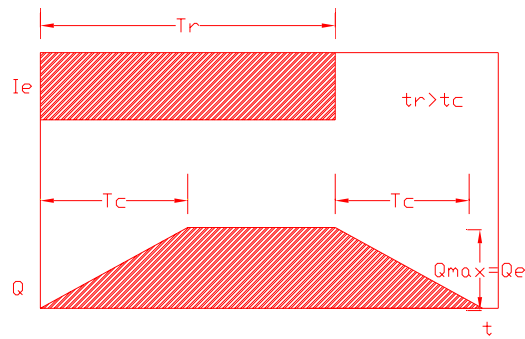


Fig. 2.3: Super-concentrated catchment flow

III. Sub-concentrated catchment flow: Effective Rainfall Duration < concentration time

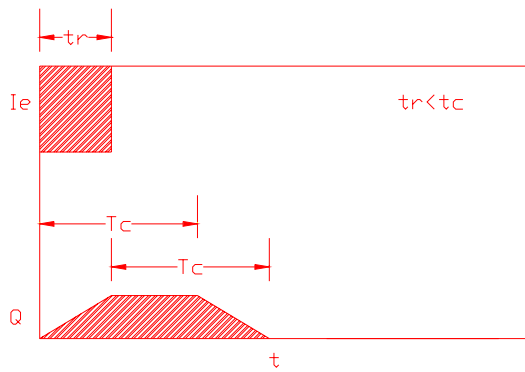


Fig.2.4: Sub-concentrated catchment flow

In the first type and the second type, the flow reaches equilibrium. In the third type of response, the flow at the outlet does not reach equilibrium. After rainfall stops, the flow recedes back to zero. The requirement that volume be conserved and recession time be equal to the concentration time lead to the idealized flattop response shown in the third figure.

In practice, concentrated and super-concentrated flows are typical of small catchments, i.e. those likely to have short concentration times. On the other hand, sub-concentrated flows are typical of midsize and large catchments i.e. those with large concentration times.

#### **2.1.4 RUNOFF COEFFICIENT**

The runoff coefficient is one of the inputs in the IDF curve method. It has been discussed in adequate detail in the previous section on the rational method.

#### **2.1.5. LIMITATIONS OF THE IDF CURVE METHOD**

The IDF curve method does not take into account the following characteristics or processes.

- Spatial or temporal variations in either total or effective rainfall
- Concentration time much greater than rainfall duration
- A significant portion of runoff occurring in the form of stream flow
- Doesn't explicitly account for the catchment's antecedent moisture although it may be implicitly accounted for by varying the runoff coefficient.
- Its output is significantly affected by catchment shape. It is not suited to catchments where drainage area doesn't increase more or less linearly with catchment length (i.e, slender catchments)

In view of the above considerations, the IDF curve method tends to be more realistic and practical when used for small and midsize catchments (usually less than 500 Ha or

5 km<sup>2</sup>). For larger catchments, statistical analysis of rainfall data for the region is a more realistic option of rainfall intensity determination. Since it is impractical and beyond the scope of this thesis to exhaust evaluating all statistical distributions, only a few will be considered based on their widespread use in Ethiopia.

## **2.2. RAINFALL FREQUENCY ANALYSIS**

Although sophisticated models can be developed that quantify hydrological behavior, often the detailed data such as the spatial and temporal distribution of rainfall that describe basic hydrological processes in space and time are not available. Thus, the uncertainties caused by lack of data can often only be addressed statistically.

Frequency Analysis in general, with reference to hydrology, refers to the techniques whose objective is to analyze the occurrence of hydrologic variables within a statistical framework i.e. by using measured data and basing predictions on statistical laws.

Rainfall Frequency Analysis in particular refers to the application of frequency analysis to study the occurrence of rainfall intensity, with a view to compensate for the incomplete knowledge of the physical processes involved in the relevant phases of the hydrologic cycle.

The procedure is relatively straightforward and only the record length limits its effectiveness, as its predictive capability decreases sharply when used to evaluate rainfalls substantially in excess of the record length. Consequently, it is used predominantly for situations where there is a record of rainfall intensity sufficient to warrant predictions within the desired return periods.

There are different types of statistical distribution functions that are used to describe the pattern of hydrologic phenomena and predict future behavior from past records of the events in that particular area or watershed. The most commonly used statistical distributions for frequency analysis are:

- Normal Distribution
- Log-Normal Distribution
- Gumbel's Extreme Value Type I Distribution
- Log-Pearson III Distribution

The first two have not found recurrent use in frequency analysis of extreme flood or rainfall intensity events. (ILCA, December 1983]. So the second two distributions, which have been established to be more representative of rainfall intensity and flood frequency behavior in Tropical Africa shall be considered with a view to application in rainfall intensity frequency analysis.

The underlying assumption of this approach is the synchronization of rainfall frequency and flood frequency on a given catchment, which is not always the case. It is an assumption that can only be carried further at the expense of accuracy, since the accuracy of the estimate will increasingly rely on the accuracy of the estimation of discharge coefficient as the size of the catchment increases. This is because the synchronization of the two events is affected among others by antecedent moisture content, variability in losses and overbank storage, which can only be represented as a lumped parameter by the discharge coefficient in the rational method.

The need to use rainfall instead of direct streamflow data arises from the lack of streamflow data characteristic of almost every small watershed in Ethiopia. The following study attests to this fact.

*"Most of the data on rainfall and streamflow in Ethiopia is quite recent. Also the gauging stations are few in number, and they are distributed unevenly.... More than 70% of the rainfall and streamflow gauging stations are located near the headwaters of big river basins...About 65% of the [streamflow gauging] stations have been installed since 1972...Up to the beginning of 1987 about 25% of the installed stations had been abandoned or were non-operational."*(Admasu, 1989)

The following statement further reinforces the fact that this downside is not specific to Ethiopia.

*"In most countries, there are usually plenty of rainfall records, but the more elaborate and expensive streamflow measurements, which are what the engineer needs for the assessment of water resources or of damaging flood peaks, are often limited and are rarely available for a specific river under investigation."*(Shaw, 1988)

### 2.2.1 LOG-PEARSON TYPE III DISTRIBUTION

The log-pearson Type III distribution is a statistical technique for fitting frequency distribution data to predict the design of flood for a river. That concept has been extended here to predict the design rainfall intensity for a catchment. Once the statistical information is computed for the catchment, a frequency distribution can be constructed. The probabilities of rainfalls of various sizes can be extracted from the curve. The advantage of the log-Pearson III distribution is that extrapolation can be made of the values for events with return periods well beyond the observed events. This technique is the standard technique used by Federal Agencies in the United States. [*Oregon State University, 2002-2005*]

The Log-Pearson Type III distribution is represented by the general equation:

$$\log y = \sum_{i=1}^n y_i + kS_y \quad (\text{Eq. 2.2.1})$$

where the standard deviation  $S_y = \sqrt{\frac{1}{n-1} \sum_{i=1}^n (y_i - y)^2}$  (Eq. 2.2.2)

and  $k$  is the frequency factor. Figure 2.5 shows the graphical details and the mathematical meaning of the parameters in the log-pearson III probability distribution.

In quite a few literatures (*Chow & Maidment, 1981, Ponce, 1989*), the frequency factor for the log-Pearson Type III analysis is found tabulated versus the probability level  $P_j$  and the skew coefficient  $C_s$ .

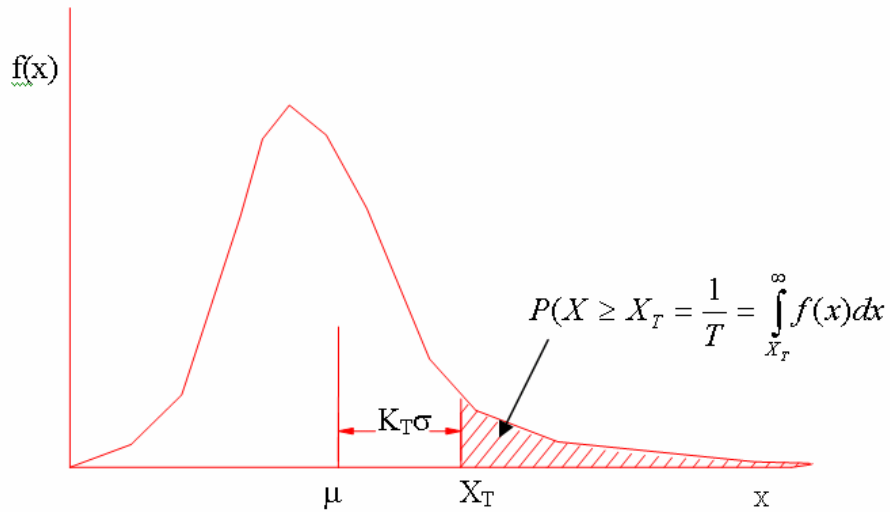


Fig 2.5. Probability distribution as described by the log-Pearson III function

After sifting through available research material and conducting some literature review, the parametric equation 2.2.3 approximated by *Kite (1977)* has been found to be a better estimate for application in a computer program. (*Kite, 1977*). When  $C_s=0$ , the frequency factor is equal to the standard normal variable  $Z$ . When  $C_s \neq 0$ ,

$$K_T = Z + (Z^2 - 1)k + \frac{1}{3}(Z^3 - 6Z)k^2 - (Z^2 - 1)k^3 + Zk^4 + \frac{1}{3}k^3 \quad (\text{Eq. 2.2.3})$$

$$\text{where } k = \frac{C_s}{6}$$

The value of  $Z$  corresponding to an exceedence probability of  $P$  ( $P=1/T$ ) can be calculated by finding the value of an intermediate variable  $w$ :

$$w = \ln\left[\left(\frac{1}{P^2}\right)\right]^{\frac{1}{2}} \quad (0 < P \leq 0.5) \quad (\text{Eq. 2.2.4})$$

then calculating  $Z$  using the approximation:

$$Z = w - \frac{2.515517 + 0.802853w + 0.010328w^2}{1 + 1.432788w + 0.189269w^2 + 0.001308w^3} \quad (\text{Eq. 2.2.5})$$

When  $P > 0.5$ ,  $(1 - P)$  is substituted for  $P$  equation 2.2.4 and the value of  $Z$  computed by equation 2.2.5 is given a negative sign. Though it may seem a fictitiously high

precision from a practical standpoint considering the abstraction of hydrologic input variables for Ethiopia, it would be appropriate, even if only for purely academic reasons, to mention that the error in Eq. 2.2.3 is less than 0.00045 in  $Z$  (*Abramowitz and Stegen, 1965.*)

### 2.2.2 GUMBEL'S EXTEME VALUE TYPE I DISTRIBUTION

Extreme Value Type I or Gumbel's probability distribution is a commonly used double exponential distribution of extreme values (annual series) which was originally applied by Gumbel. The easy to use approach of the Gumbel distribution and its tested predictive capability have made it another popular statistical tool in drainage design to model catchment behavior after a storm event.

As in the case of the log-Pearson III distribution, the concept will be extended to predict a design rainfall intensity from statistical analysis of a series of rainfall intensity records for a given return period and duration.

Gumbel has fitted the EVI distribution to long records of river flows from many countries. The cumulative density function (CDF) of the Gumbel distribution is the following double exponential function:

$$F(x) = [e^{-e}]^{-y} \quad (\text{Eq. 2.2.6})$$

in which  $y = \left[ \frac{x-u}{\alpha} \right]$

is the Gumbel (reduced) variate and  $F(x)$  is the probability of non-exceedence. In rainfall or flood frequency analysis, the probability of interest is the probability of exceedence i.e the complementary probability to  $F(x)$

$$G(x) = 1 - F(X)$$

The return period  $T$  is the reciprocal of the probability of exceedence. Therefore:

$$\frac{1}{T} = 1 - [e^{-e}]^{-y}$$

$$y = -\ln \ln \left[ \frac{T}{T-1} \right] \quad (\text{Eq. 2.2.6})$$

In the Gumbel method, values of flood discharge/rainfall intensity are obtained from the frequency formula  $x = \bar{x} + ks$ . The frequency factor  $k$  is evaluated with

$$y = \bar{y}_n + k\sigma_n \quad (\text{Eq. 2.2.7})$$

in which  $y$  = Gumbel (reduced) variate, a function of the return period (Eq. 2.2.6)

The mean  $\bar{y}_n$  and the standard deviation  $\sigma_n$  of the Gumbel variate are functions of the record length  $n$ . Values of  $\bar{y}_n$  and  $\sigma_n$  as a function of record length  $n$  are usually tabulated. Substituting for  $k$  and rearranging, the Gumbel equation can be written as:

$$x = \bar{x} + \frac{y - \bar{y}_n}{\sigma_n} S$$

$$x = \bar{x} - \frac{\ln \ln \frac{T}{T-1} + \bar{y}_n}{\sigma_n} S \quad (\text{Eq. 2.2.8})$$

When the record length  $n$  approaches  $\infty$ , the mean  $\bar{y}_n$  approaches the value of the Euler constant (0.5772), and the standard deviation approaches the value  $\frac{\pi}{\sqrt{6}}$  (Ponce, 1989).

Based on this mathematical behavior of the Gumbel variate, Lattenmaier and Burger have suggested a modification to the Gumbel method (Ponce, 1989). According to this modification, better flood estimates are obtained by using the limiting values of the mean and standard deviation of the Gumbel variate ( i.e those corresponding to  $n = \infty$ ,

in  $y = \bar{y}_n + k\sigma_n$ ), instead of basing these values on the record length. In this case, taking  $\bar{y}_n = 0.5772$  and  $\sigma_n = \pi/\sqrt{6} = 1.2825$ , the general Extreme Value Type I distribution can be expressed as:

$$x = \bar{x} - (0.78 \ln \ln \left[ \frac{T}{T-1} + 0.45 \right]) * S \quad (\text{Eq. 2.2.9})$$

where S – is the standard deviation of the data series.

### 2.2.3 SELECTION OF DATA SERIES

The complete record of rainfall at a given gauging station is called the complete duration series. To perform rainfall frequency analysis, it is necessary to select a rainfall series i.e. a sample of events extracted from the complete duration series.

There are two types of series:

- The Partial Duration Series
- The Extreme Value series

The partial duration or Peaks-Over-a-Threshold (POT) series consists of rainfalls whose magnitude is greater than a certain base value. When the base value is such that the number of events in the series is equal to the number of years of record, the series is called an annual exceedence series. The annual exceedence series takes into account all extreme events above a certain value, regardless of when they occurred.

In the extreme value series, every year of record contributes one value to the extreme value series. This is called the annual maxima series, which considers only one extreme event per yearly period.

The difference between the two series is likely to be more marked for short records in which the second largest annual events may strongly influence the character of the

annual exceedence series. As conventional design wisdom dictates, when in doubt, it always pays to err on the side of caution.

In practice, the annual exceedence series is used for frequency analyses involving short return periods, ranging from 2 to 10 years. For larger return periods the difference between the annual exceedence and annual maxima series is small. Although the annual exceedence series is useful for some purposes, it is limited by the fact that all the observations are not independent, that is to say, the occurrence of a large flood could well be related to saturated soil conditions produced during another larger flood occurring a short time earlier. As a result, it is usually safer to use the annual maxima series for analysis, especially for return periods ranging from 10 to 100 years and more.

### **2.3 SCS CURVE NUMBER METHOD**

For drainage basins where no runoff has been measured, the SCS Curve Number is another way to estimate the depth of direct runoff from the rainfall depth, given an index describing runoff response characteristics. The SCS Method was originally developed by the Soil Conservation Service (*Soil Conservation Service 1964; 1972*) for conditions prevailing in the United States.

Since then, it has been adapted to conditions in other parts of the world. Some regional research centers have developed additional criteria; but the basic concept is still widely used all over the world. Although this method was in use in Ethiopia for quite some time for both minor and major drainage design, it gained currency in the Ethiopian practice especially after it has been introduced into the *ERA Drainage Manual (2002)*.

For individual storm events, the model determines peak discharges, their time of occurrence and water surface elevations. It can also provide complete hydrographs when desired. The SCS curve number method can be classified as a complete, event-type model. It is a general, measured-parameter model, being easily applicable to most agricultural and urban ungauged watersheds, using only readily obtainable measured or estimated input parameters.

The advantage of this method is that it is straightforward to apply and the physical parameters are easily determined.

The major pitfall in its application is that the method is limited to watersheds with a drainage area of approximately 1000 km<sup>2</sup> or less. [Chow & Maidment, 1981] One of the reasons for this limitation is the Unit Hydrograph theory assumes uniform rainfall and runoff from the entire drainage basin. This assumption is less reliable if the drainage area becomes much larger than recommended; however, it is justifiably suitable for application in minor drainage structures as they typically serve as outlets to small drainage basins.

### **2.3.1 DERIVATION OF EMPIRICAL RELATIONSHIPS**

The SCS model starts with total precipitation from a storm event and translates that rainfall, according to empirical relationships obtained from multiple correlation analyses, into a runoff volume. The method attempts to account for the effects of a) total precipitation b) an initial rainfall abstraction c) a time-variable infiltration rate and d) antecedent soil moisture.

When the data of accumulated rainfall and runoff for long-duration, high-intensity rainfalls over small drainage basins are plotted, they show that runoff only starts after some rainfall has accumulated, and that the curves asymptotically approach a straight line with a 45-degree slope.

The SCS Curve Number Method is based on these two phenomena. The initial accumulation of rainfall represents interception, depression storage, and infiltration before the start of runoff and is called initial abstraction. After runoff has started, some of the additional rainfall is lost, mainly in the form of infiltration; this is called actual retention. With increasing rainfall, the actual retention also increases up to some maximum value: the potential maximum retention.

To describe these curves mathematically, SCS assumed that the ratio of actual retention to potential maximum retention is equal to the ratio of actual runoff to

potential maximum runoff, the latter being rainfall minus initial abstraction. In mathematical form, this empirical relationship is

$$\frac{F}{S} = \frac{Q}{P - I_a} \quad (\text{Eq. 2.3.1})$$

where

F = actual retention (mm)

S = potential maximum retention (mm)

Q = accumulated runoff depth (mm)

P = accumulated rainfall depth (mm)

$I_a$  = initial abstraction (mm)

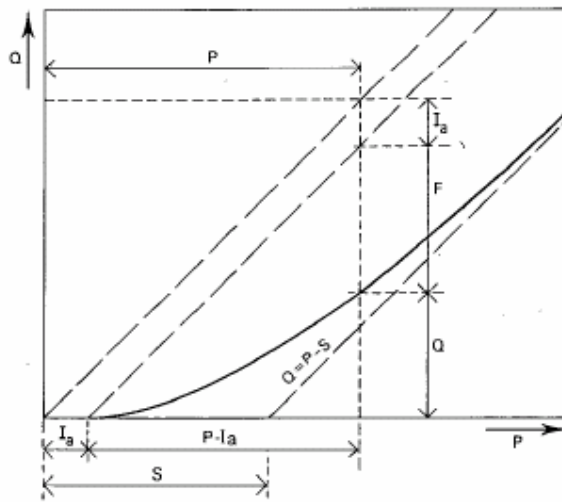


Fig. 2.6 Accumulated runoff  $Q$  versus accumulated rainfall  $P$  according to the Curve Number Method

Figure 2.6 shows the above relationship for certain values of the initial abstraction and potential maximum retention. After runoff has started, all additional rainfall becomes either runoff or actual retention (i.e. the actual retention is the difference between rainfall minus initial abstraction and runoff).

$$F = P - I_a - Q \quad (\text{Eq. 2.3.2})$$

Combining Equations 2.3.1 and 2.3.2 yields

$$Q = \frac{(P - I_a)^2}{P - I_a + S} \quad (\text{Eq. 2.3.3})$$

To eliminate the need to estimate the two variables  $I_a$  and  $S$  in Equation 2.3.3, a regression analysis was made on the basis of recorded rainfall and runoff data from small drainage basins. The data showed a large amount of scatter (Soil Conservation Service 1972).

In the Curve Number Method as presented by *Soil Conservation Service (1964;1972)*, the initial abstraction  $I_a$ , was found to be 20% of the potential maximum retention  $S$ . This value represents an average because the data plots showed a large degree of scatter. Nevertheless, various authors (*Aron et al. 1977, Fogel et al. 1980, and Springer et al. 1980*) have reported that the initial abstraction is less than 20% of the potential maximum retention; percentages of 15, 10, and even lower have been reported.

As described above, for all practical purposes, the following average relationship was found:

$$I_a = 0.2 S \quad (\text{Eq. 2.3.4})$$

Combining Equations 2.3 and 2.4 yields:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \quad (\text{Eq. 2.3.5})$$

Equation 2.3.5 is the rainfall-runoff relationship used in the Curve Number Method. It allows the runoff depth to be estimated from rainfall depth, given the value of the potential maximum retention  $S$ . This potential maximum retention mainly represents infiltration occurring after runoff has started. This infiltration is controlled by the rate of infiltration at the soil surface, or by the rate of transmission in the soil profile, or by the water-storage capacity of the profile, whichever is the limiting factor.

The potential maximum retention  $S$  has been converted to the Curve Number  $CN$  in order to make the operations of interpolating, averaging, and weighting more nearly linear. This relationship is:

$$CN = \frac{25400}{254 + S} \quad \text{or} \quad S = \frac{25400}{CN} - 254 \quad (\text{Eq. 2.3.6})$$

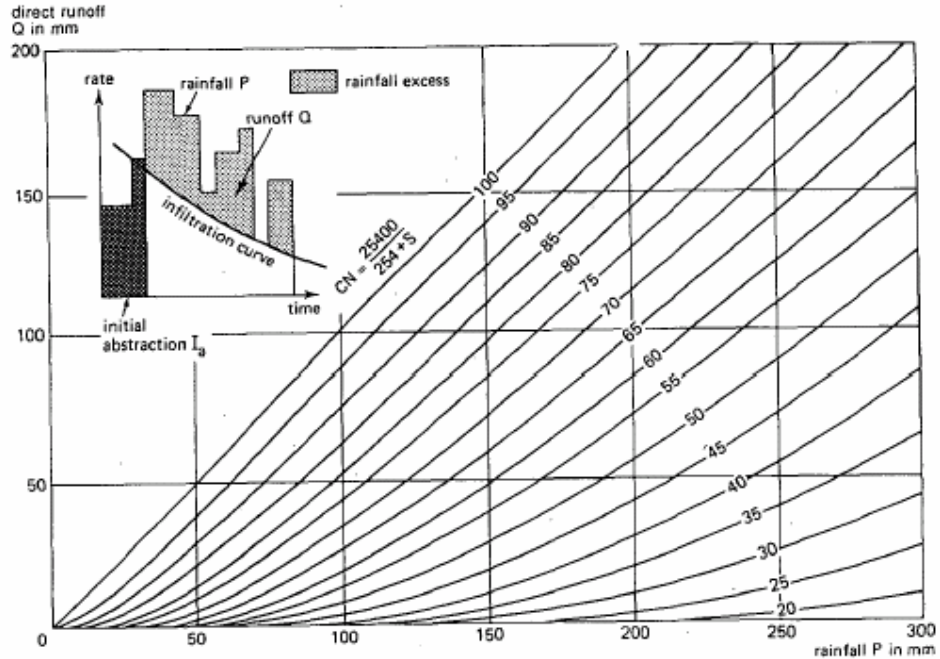


Fig 2.7. Graphical solution of Equation 2.3.6 showing runoff depth  $Q$  as a function of rainfall depth  $P$  and curve number  $CN$  (after Soil Conservation Service 1972)

As the potential maximum retention  $S$  can theoretically vary between zero and infinity. Equation 2.3.6 shows that the Curve Number  $CN$  can range from 100 to 0. Figure 2.7 shows the graphical solution of Equation 2.3.5, indicating values of runoff depth  $Q$  as a function of rainfall depth  $P$  for selected values of Curve Numbers. For paved areas, for example,  $S$  will be zero and  $CN$  will be 100; all rainfall will become runoff. For highly permeable, flat-lying soils,  $S$  will go to infinity and  $CN$  will be zero; all rainfall will infiltrate and there will be no runoff. In drainage basins, the reality will be somewhere in between.

It should be well noted here that the Curve Number Method was developed to be used with daily rainfall data measured with non-recording rain gauges. The relationship

therefore excludes time as an explicit variable (i.e. rainfall intensity is not included in the estimate of runoff depth).

## **2.3.2 FACTORS DETERMINING THE CURVE NUMBER VALUE**

The Curve Number is a dimensionless parameter indicating the runoff response characteristic of a drainage basin. In the Curve Number Method, this parameter is related to land use, land treatment, hydrological condition, hydrological soil group, and antecedent soil moisture condition in the drainage basin.

### **2.3.2.1 LAND USE OR COVER**

Land use represents the surface conditions in a drainage basin and is related to the degree of cover. In the SCS method, the following categories are distinguished:

- *Fallow* is the agricultural land use with the highest potential for runoff because the land is kept bare;
- *Row crops* are field crops planted in rows far enough apart that most of the soil surface is directly exposed to rainfall;
- *Small grain* is planted in rows close enough that the soil surface is not directly exposed to rainfall;
- *Close-seeded legumes* or *rotational meadow* are either planted in close rows or broadcasted. This kind of cover usually protects the soil throughout the year;
- *Pasture range* is native grassland used for grazing, whereas meadow is grassland protected from grazing and generally mown for hay;
- *Woodlands* are usually small isolated groves of trees being raised for farm use.

### **2.3.2.2 TREATMENT OR PRACTICE IN RELATION TO HYDROLOGICAL CONDITON**

Land treatment applies mainly to agricultural land uses; it includes mechanical practices such as contouring or terracing, and management practices such as rotation of crops, grazing control, or burning.

Rotations are planned sequences of crops (*row crops, small grain, and close-seeded legumes* or *rotational meadow*). Hydrologically, rotations range from *poor* to *good*. Poor rotations are generally one-crop land uses (monoculture) or combinations of row crops, small grains, and fallow. Good rotations generally contain close-seeded legumes or grass.

For grazing control and burning (*pasture range* and *woodlands*), the hydrological condition is classified as *poor, fair, or good*.

*Pasture range* is classified as *poor* when heavily grazed and less than half the area is covered; as *fair* when not heavily grazed and between one-half to three-quarters of the area is covered; and as *good* when lightly grazed and more than three-quarters of the area is covered.

*Woodlands* are classified as *poor* when heavily grazed or regularly burned; as *fair* when grazed but not burned; and as *good* when protected from grazing.

### **2.3.2.3 HYDROLOGICAL SOIL GROUP**

Soil properties greatly influence the amount of runoff. In the SCS method, these properties are represented by a hydrological parameter: the minimum rate of infiltration obtained for a bare soil after prolonged wetting. The influence of both the soil's surface condition (infiltration rate) and its horizon (transmission rate) are thereby included. This parameter, which indicates a soil's runoff potential, is the qualitative basis of the classification of all soils into four groups. The Hydrological Soil Groups, as defined by the SCS soil scientists, are:

*Group A:* Soils having high infiltration rates even when thoroughly wetted and a high rate of water transmission. Examples are deep, well to excessively drained sands or gravels.

*Group B:* Soils having moderate infiltration rates when thoroughly wetted and a moderate rate of water transmission. Examples are moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

*Group C:* Soils having low infiltration rates when thoroughly wetted and a low rate of water transmission. Examples are soils with a layer that impedes the downward movement of water or soils of moderately fine to fine texture.

*Group D:* Soils having very low infiltration rates when thoroughly wetted and a very low rate of water transmission. Examples are clay soils with a high swelling potential, soils with a permanently high water table, soils with a clay pan or clay layer at or near the surface, or shallow soils over nearly impervious material.

Land use or cover	Treatment or practice	Hydrological condition	Hydrological soil group			
			A	B	C	D
Fallow	Straight row	Poor	77	86	91	94
Row crops	Straight row	Poor	72	81	88	91
	Straight row	Good	67	78	85	89
	Contoured	Poor	70	79	81	88
	Contoured	Good	65	75	82	86
	Contoured/terraced	Poor	66	74	80	82
	Contoured/terraced	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
	Straight row	Good	63	75	83	87
	Contoured	Poor	63	74	82	85
	Contoured	Good	61	73	81	84
	Contoured/terraced	Poor	61	72	79	82
	Contoured/terraced	Good	59	70	78	81
Close-seeded legumes or rotational meadow	Straight row	Poor	66	77	85	89
	Straight row	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	Contoured	Good	55	69	78	83
	Contoured/terraced	Poor	63	73	80	83
	Contoured/terraced	Good	51	67	76	80
Pasture range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	Contoured	Fair	25	59	75	83
	Contoured	Good	6	35	70	79
Meadow (permanent)		Good	30	58	71	78
Woodlands (farm woodlots)		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads			59	74	82	86
Roads, dirt			72	82	87	89
Roads, hard-surface			74	84	90	92

Table 2.1. Curve Numbers for Hydrological Soil-Cover Complexes for Antecedent Moisture Conditions Class II and  $I_a = 0.2S$  (after Soil Conservation Service, 1972)

#### 2.3.2.4 ANTECEDENT MOISTURE CONDITION

The soil moisture condition in the drainage basin before runoff occurs is another important factor influencing the final CN value. In the Curve Number Method, the soil

moisture condition is classified in three Antecedent Moisture Condition (AMC) Classes:

*AMC I:* The soils in the drainage basin are practically dry (i.e. the soil moisture content is at wilting point).

*AMC II:* Average condition.

*AMC III:* The soils in the drainage basin are practically saturated from antecedent rainfalls (i.e. the soil moisture content is at field capacity).

These classes are based on the 5-day antecedent rainfall (i.e. the accumulated total rainfall preceding the runoff under consideration). In the original SCS method, a distinction was made between the dormant and the growing season to allow for differences in evapo-transpiration.

*The ERA Drainage Manual (2002)* recommends for antecedent moisture conditions in Ethiopia to use *dry* for region D1, *wet* for Region B1, and *average* AMC for all other regions. The portion of Region A2 in the vicinity of Bahirdar should also be treated as *wet*.

#### **2.3.2.5 ESTIMATING THE CURVE NUMBER VALUE**

To determine the appropriate CN value, various tables can be used. Firstly, there are tables relating the value of CN to land use or cover, to treatment or practice, to hydrological condition, and to hydrological soil group. Together, these four categories are called the Hydrological Soil-Cover Complex. The relationship between the CN value and the various Hydrological Soil-Cover Complexes is usually given for average conditions, i.e. *Antecedent Soil Moisture Condition Class II*. Secondly, there is a conversion table for the CN value when on the basis of 5-day antecedent rainfall data the Antecedent Moisture Condition should be classified as either *Class I* or *Class III*.

### 2.3.2.6 ESTIMATING THE DEPTH OF THE DIRECT RUNOFF

Once the final CN value has been determined, the direct runoff depth can be calculated.

This can be done in two ways:

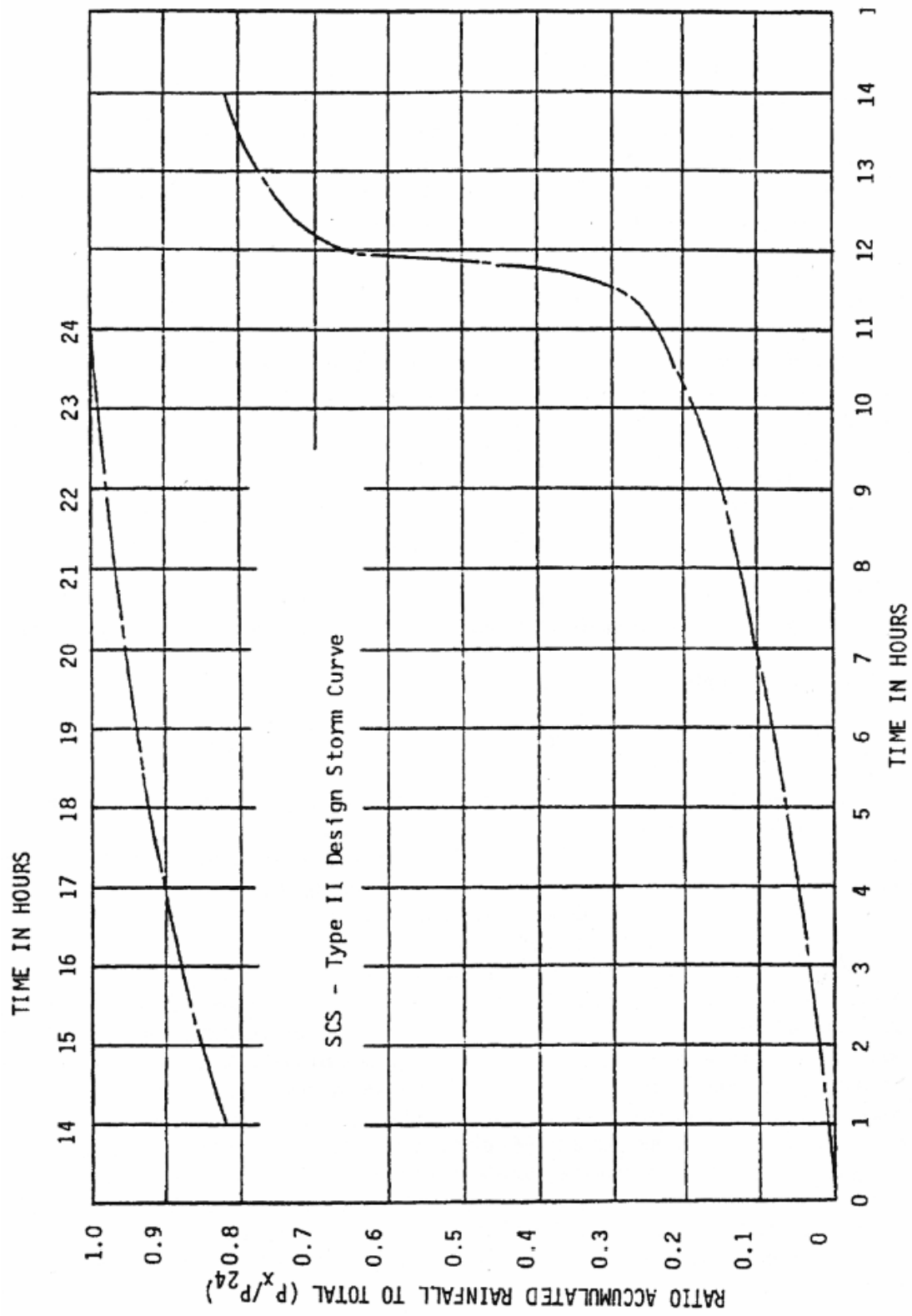
- Graphically, by using the design rainfall depth in Figure 2.7, and reading the intercept with the final CN value;
- Numerically, by using Equation 2.3.6 to determine the potential maximum retention  $S$  and substituting this  $S$  value and the design rainfall depth into Equation 2.3.5.

The latter analytic method is obviously preferred in writing a computer code to calculate the direct runoff for the selected duration and return period.

Next, the appropriate rainfall data is determined. From the regressed equations of the IDF curves provided by ERA, the depth of design rainfall as a function of its duration for the given return period is calculated.

The depth values of the direct runoff can now be calculated by substituting into Equation 2.3.5 the above  $S$  value and the rainfall depth data found from the regressed equations. These direct-runoff-depth data as a function of the duration of the design rainfall are the basis on which to determine the capacity of surface drainage systems in flat areas.

If no information is available on how the amount of design rainfall is distributed over the selected duration, the usual assumption is that the intensity will be uniformly distributed. In our case, the *SCS Type II rainfall distribution*, as recommended to be appropriate by *ERA Drainage Manual* for Ethiopia, is used to carry out the rainfall intensity distribution throughout the selected duration.



Source: SCS-TP-149

Fig. 2.8. SCS Type II Design Storm Curve

The choice of the *Type II* rainfall intensity distribution is justified by the fact that this distribution has been developed from rainfall records in interior rather than coastal regions and would be appropriate for Ethiopia. (*ERA Drainage Manual, 2002*)

#### **2.3.2.7 ESTIMATING THE TIME DISTRIBUTION OF DIRECT RUNOFF**

Since the physical characteristics of a basin (shape, size, slope, etc.) remain relatively constant, one can expect considerable similarity in the shape of hydrographs resulting from similar high-intensity rainfalls. This is the essence of Sherman's unit hydrograph theory.

Sherman, after analyzing a great number of time-intensity graphs (hyetographs) of excess rainfall with duration equal to or smaller than the unit storm period, concluded that the resulting hydrographs for a particular drainage basin closely fit the following properties:

- The base length of the hydrograph of direct runoff is essentially constant, regardless of the total depth of excess rainfall;
- If two high-intensity rainfalls produce different depths of excess rainfall, the rate of direct runoff at corresponding times after the beginning of each rainfall are in the same proportion to each other as the total depths of excess rainfall;
- The time distribution of direct runoff from a given excess rainfall is independent of concurrent runoff from antecedent periods of excess rainfall.

The principle involved in the first and second of these statements is known as *the principle of proportionality*, by which the ordinates of the hydrograph of direct runoff are proportional to the depth of excess rainfall. The third statement implies that the hydrograph of direct runoff from a drainage basin due to a given pattern of excess rainfall at whatever time it may occur is invariable. This is known as *the principle of time invariance*.

These fundamental principles of *proportionality* and *time invariance* make the unit hydrograph an extremely flexible tool for developing composite hydrographs. The total hydrograph of direct runoff resulting from any pattern of excess rainfall can be built up by superimposing the unit hydrographs resulting from the separate depths of excess rainfall occurring in successive unit time periods. In this way, a unit hydrograph for a relatively short duration of excess rainfall can be used to develop composite hydrographs for high-intensity rainfalls of longer duration.

If we know the shape of the unit hydrograph, we can convert any historical or statistical rainfall into a composite hydrograph of direct runoff by using the Curve Number Method to calculate the excess rainfall depths and the Unit Hydrograph Method to calculate the direct runoff rates as a function of time.

### 2.3.2.8 PARAMETRIC UNIT HYDROGRAPH

Numerous procedures to construct a unit hydrograph for ungaged basins have been developed. The dimensionless unit hydrograph used by the *Soil Conservation Service* (1972) was developed by *Mockus* (1957).

$t/T_p$	$q_t/q_p$	$t/T_p$	$q_t/q_p$	$t/T_p$	$q_t/q_p$
0	0	1.75	0.45	3.50	0.036
0.25	0.12	2.00	0.32	3.75	0.026
0.50	0.43	2.25	0.22	4.00	0.018
0.75	0.83	2.50	0.15	4.25	0.012
1.00	1.00	2.75	0.105	4.50	0.009
1.25	0.88	3.00	0.075	4.75	0.006
1.50	0.66	3.25	0.053	5.00	0.004

Table 2.2. Dimensionless time and runoff ratios of the SCS parametric unit hydrograph (SCS, 1972)

The shape of this dimensionless unit hydrograph predetermines the time distribution of the runoff; time is expressed in units of time to peak  $T_p$ , and runoff rates are expressed in units of peak runoff rate  $q_p$ . Table 2.2 shows these time and runoff ratios numerically and Fig. 2.9 (solid line) shows them graphically.

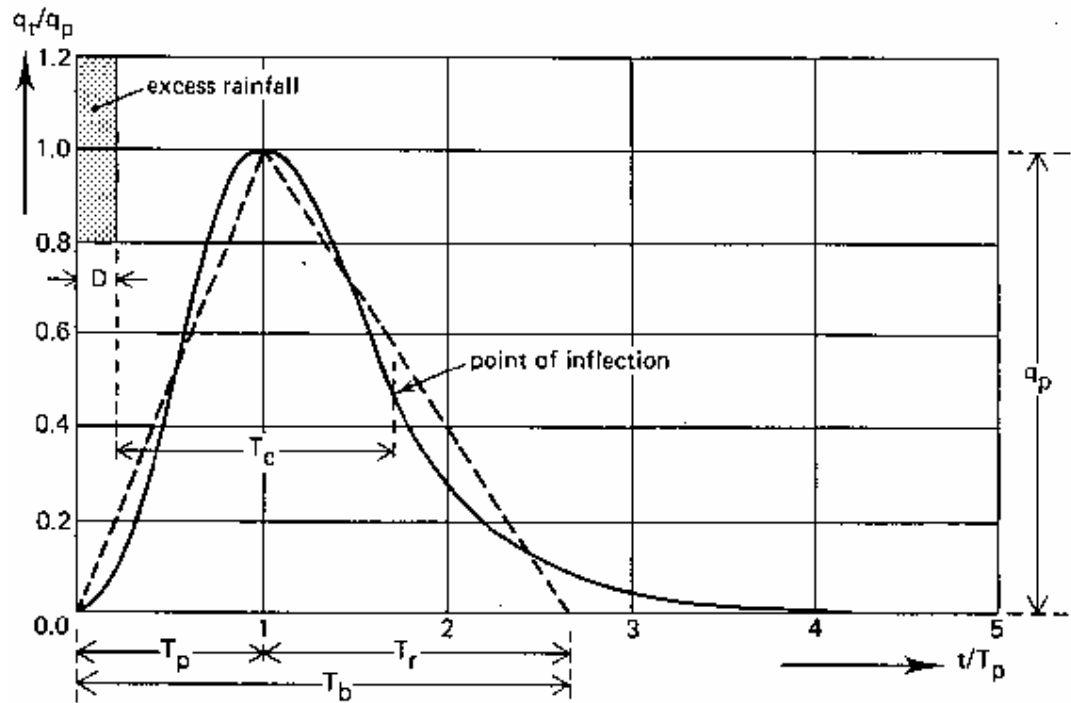


Fig. 2.9. Dimensionless curvilinear unit hydrographs (solid line) and equivalent triangular unit hydrograph (dashed line) (SCS, 1972)

Since the area under the rising limb of the curvilinear unit hydrograph represents 37.5 percent of the total area, the time base  $T_b$  of the triangular unit hydrograph equals  $1/0.375=2.67$  in order to have also the same total areas under both hydrographs, representing 1mm of excess rainfall.

Mockus, using an equivalent triangular unit hydrograph with the same units of time and runoff as the curvilinear unit hydrograph, and using the equation of the area of a triangle and expressing the volumes in  $m^3$ , showed that one can obtain for the dimensional triangular unit hydrograph

$$10^6 A \times 10^{-3} Q = (1/2)(3600 \times q_p) \times 2.67 T_p \quad (\text{Eq. 2.3.7})$$

where

$A$ = area of drainage basin (km<sup>2</sup>)

$Q$ = excess rainfall (mm)

$Q_p$ = peak runoff rate unit hydrograph (m<sup>3</sup>/s)

$T_p$  = time to peak runoff unit hydrograph (h)

Rearranging equation Eq. 2.8

$$q_p = 0.208 \frac{AQ}{T_p} \quad (\text{Eq. 2.3.8})$$

In equation 2.9, the only unknown parameter is time to peak  $T_p$ . This can be estimated in terms of time of concentration  $T_c$ .

$$T_p = 0.7T_c \quad (\text{Eq. 2.3.9})$$

For small drainage basins, as has been assumed in the Rational Method, the time to peak is regarded as being equal to the time of concentration.

In the case of SCS, time of concentration  $T_c$  is based on another empirical formula given by Kirpich (1940)

$$T_c = 0.02L^{0.77}S^{-0.385} \quad (\text{Eq. 2.3.10})$$

where

$T_c$  = time of concentration (min)

$L$  = maximum length of travel (m)

$S$  = Slope, equal to  $H/L$  where  $H$  is the difference in elevation between the most remote point in the basin and the outlet.

The parameters to estimate the time of concentration can be derived from a topographic map. So, by estimating  $T_c$ , we can calculate the time to peak  $T_p$  and consequently the peak runoff rate  $q_p$ . Thus, a dimensional unit hydrograph for a particular basin can be derived from the dimensionless curvilinear unit hydrograph.

## **2.4. EAST AFRICAN FLOOD MODEL**

The East African Flood Model was developed by the Department of Environment, Transport and Road Research Laboratory and published in TRRL Laboratory Report 706, 1976. The model has been developed by taking four years of data from 13 small catchments in Kenya and Uganda and analyzing them to come up with improved methods of flood estimation for highway bridges and culverts. The model was then used to predict the 10-year flood using a 10-year design storm. A simple technique is then developed for predicting the peak flow and base time of design hydrographs for ungauged catchments.

This Model has also found use with some consulting firms in Ethiopia on a limited number of occasions. It will be attempted to discuss the model in brief and why its use is not widespread in Ethiopia.

### **2.4.1. A BRIEF DESCRIPTION OF THE MODEL**

The East African Flood Model is made up of two parts. A linear reservoir is used for the period between the rain hitting the ground surface and the flood flow entering the stream system (the “land phase”) and a finite-difference routing technique is used for the passage of the flood wave down the river to the catchment outlet.

The simple land model may be summarized as:

- a) Early rain fills the initial retention (Y). Runoff at this stage is zero.
- b) Subsequent rain falling on the parts of the catchment from which runoff will occur ( $C_A$ ) enters the reservoir stage.
- c) Runoff is given by the linear reservoir relation:

$$q = \frac{1}{k} S$$

where S – the reservoir storage

k – is the reservoir lag time

This simple model, and derivations from it, were compared with the unit hydrograph using data from a small catchment, and the results were promising. With small catchments, the determination of the flood hydrograph during travel down the stream system is negligible. For larger catchments, it can be considerable. But these translation effects can be allowed for by varying  $k$  (reservoir lag time). This was attempted when data from the larger catchments were analyzed but poor results were obtained. In addition, if the value of the lag time is dependent on catchment size, values obtained from small catchments are difficult to apply to large catchments.

The approach finally adopted was to divide the catchment into a number of sub-catchments, the runoff from which was simulated using the land-phase model. The translation of this runoff down the stream system to the catchment outlet was modeled using a modification of the finite-difference technique developed by *Morgali and Linsley, (1965)*

This is the main drawback of this method as the finite-difference equations are only approximations, the equations will become unstable, and errors will be introduced unless the following equation is satisfied.

$$V + \sqrt{gy} \leq \frac{\Delta x}{\Delta t}$$

where  $V$  = velocity

$y$  = depth of flow

$g$  = acceleration due to gravity

$\Delta x$  and  $\Delta t$  are the chosen distance and time increments. Typical values for the initial incremental distance and time were 200m and 30s respectively.

The second and more obvious drawback with this model is that although representative catchments have been taken from Kenya and Uganda, Ethiopian catchments were not included in the initial study and unless further statistical regionalization techniques are applied to the data, the model in its original form would

not be very suitable for Ethiopian conditions. Hence, this model has not been selected for automation in this thesis work on the basis of these two major drawbacks.

## **2.5. HEC-1 FLOOD MODEL**

HEC-1 is the first of a series of hydrological programs developed by the *US Army Corps of Engineers (1972)*. It simulates the rainfall-runoff relation in a catchment including reservoirs and may also simulate the effects of dam brakes. The program allows the choice of different formulations of the hydrologic processes involved.

As HEC-1 as a road drainage tool has not found much popularity, it does not merit detailed discussion within the scope of this work. It suffices to mention that the main drawback with this model is that one of the methods that it employs is *Muskingum Routing* technique and the *Muskingum coefficients*  $K$  and  $X$  and the routing constant  $R$  are found after calibration of the catchment to compute these parameters. In minor drainage structures, we mostly deal with ungaged catchments, as a result of which no calibration is possible, and hence HEC-1 has not found widespread practical application in minor drainage design. There is also the question of whether a detailed flood plain analysis for each structure using hydrologic and hydraulic calibration methods would be justified considering the number of minor drainage structures needed even in a short highway alignment section.

The other obvious drawback with the HEC-1 Flood model, of course, is that its DOS-based interface makes it very cumbersome for a speedy output of data with today's Windows user interface.

## **2.6 APPROPRIATE METHODS FOR PRACTICAL APPLICATION IN ETHIOPIA**

On the basis of the discussions and the conclusions drawn from the same above, the first three methods namely:

- Rational method with IDF curve approach
- Rational method with rainfall frequency analysis approach
- SCS Unit hydrograph method

have been selected, partly due to their popularity and acceptance in the Ethiopian practice and partly because of the relatively less complex programming effort required to automate them, as the most appropriate and most practical in the context of Ethiopian conditions.

In a practical application, it always helps to compare outputs from these methods prior to making design decisions. That is also one of the purposes of automating the procedures involved in each of the approaches. The theoretical backgrounds as well as the subsequent establishment of algorithms for them and their automation procedure are discussed in detail in the next chapter.

## CHAPTER 3: SOFTWARE DEVELOPMENT

The previous chapter described the most commonly applied hydrologic analysis methods in practical application in Ethiopia and elsewhere, discussed their advantages and drawbacks, and selected the most appropriate ones for practical application in Ethiopia. This chapter will discuss the mathematical formulation of these selected methods and discuss the idea behind the computer code used to write the proposed easy-to-use and practically-applicable software. The system design developed in the following pages would finally be set down in the form of a computer code in the Visual Basic programming language.

### 3.1. HYDROLOGIC ANALYSIS: RATIONAL METHOD USING IDF CURVES

Data on remotest distance, average velocity of runoff, time of concentration and runoff coefficient are needed for the determination of rainfall intensity using IDF curves for a given design situation using the relation:

$$Q=0.00278 * C * I * A * F.S \quad (\text{Eq. 3.1.1})$$

Where

- Q – Discharge at outlet (m<sup>3</sup>/s)
- C – Rainfall-Runoff Coefficient
- I – Maximum probable rainfall Intensity (mm/hr)
- A– Catchment Area (hectares)
- F.S.- Factor of Safety (factor of ignorance)(Optional)

#### 3.1.1 REMOTEST DISTANCE

Hydraulic length of the longest runoff path measured from the furthest point in the watershed (with respect to the outlet) to the point of the outlet. It can be determined

from photogrammetry and less accurately from a (1:50,000 or above) map of the catchment area.

### 3.1.2. AVERAGE VELOCITY OF RUNOFF

Average velocity of flow on the catchment (a function of the catchment characteristics i.e. Terrain type and Terrain Cover). A designer can set up his own table of discharge coefficients after careful observation of the catchment soil type, land use, terrain characteristics and its behavior in a storm event. This will be facilitated in the software as will be discussed in Section 3.1.4.

### 3.1.3. TIME OF CONCENTRATION

Time of concentration is the time that it takes a parcel of water to travel from the furthest point in the catchment to the outlet. Once the Remotest Distance and Average Velocity of Runoff are determined, the Time of Concentration is calculated from the formula below.

$$T_c = \frac{RD}{60 * Avg.V} \quad (\text{Eq. 3.1.2})$$

where  $T_c$ - Time of Concentration (min)

$RD$  – Remotest distance (m)

$Avg.V$  – Average velocity of runoff (m/s)

Since we seek to determine the peak runoff at the outlet, the  $T_c$  is made equal to the duration of the rainfall. Then the maximum possible rainfall intensity for a given probability of occurrence corresponding to that duration is determined from the IDF curve.

### **3.1.4 RUNOFF COEFFICIENTS**

Despite the fact that runoff coefficient and average velocity of runoff are extremely site-specific parameters and should be determined only after a close and careful observation of the catchment response to hydrologic events (refer to considerations above), the *ERA/TCDE (Transport Construction Design Enterprise)* design standard provides drainage designers with only a single Table of Runoff Coefficients and average velocities for the whole of Ethiopia. It must be categorically stated here that, given Ethiopia's diverse assortment of hydrologic characteristics, the use of runoff coefficients from a single official table would not even be close to making runoff frequency equal to rainfall frequency.

Despite the table's limitations, it is intended that the software to be developed supports this table as default to comply with established national practice; however, in consideration of this deficiency it also supports a feature whereby the designer can set up and set as default his own custom site-specific Table Template for runoff coefficients and average velocities.

## **3.2. HYDROLOGIC ANALYSIS: RATIONAL METHOD USING RAINFALL**

### **FREQUENCY ANALYSIS**

The software's leverage as a simplified hydrologic modeling tool with respect to road drainage is intended to lie primarily in the fact that it should support two of the most commonly used probability distribution functions for statistical analysis of rainfall data to determine peak rainfall intensity for a given frequency of occurrence.

- Log-Pearson Type III
- Gumbel's Extreme Value Type I

The algorithms developed to be used in the coding of the software are outlined as follows.

### 3.2.1 ALGORITHM FOR LOG-PEARSON TYPE III ANALYSIS

1. Assemble the rainfall series  $X_i$ .
2. Calculate the logarithms  $Y_i$  of  $X_i$ .

$$Y_i = \log X_i$$

3. Calculate the mean  $Y$ , standard deviation  $S_y$  and skew coefficient  $C_{sy}$ .

$$Y = \frac{1}{n} \sum_{i=1}^n Y_i$$

$$S_y = \sqrt{\frac{1}{n-1} \sum_{i=1}^n (Y_i - Y)^2}$$

$$C_{sy} = \frac{a_y}{S_y^3}$$

where 
$$a_y = \frac{n}{(n-1)(n-2)} \sum_{i=1}^n (Y_i - Y)^3 \text{ or skewness}$$

$n$ -is the number of members of the series

4. Calculate the logarithms of the rainfalls for each of several chosen probability levels  $P_j$  using the frequency formula:

$\text{Log RF}_j = Y + K_j * S_y$  where  $K_j$  is the frequency factor (a function of the probability level desired and the skew coefficient. Calculation of frequency factors is shown below.)

When  $C_s=0$ , the frequency factor is equal to the standard normal variable  $Z$ .

When  $C_s \neq 0$ ,

$$K_T = Z + (Z^2 - 1)k + \frac{1}{3}(Z^3 - 6Z)k^2 - (Z^2 - 1)k^3 + Zk^4 + \frac{1}{3}k^3 \quad (\text{Eq. 3.2.1})$$

$$\text{where } k = \frac{Cs}{6}$$

The value of Z corresponding to an exceedence probability of P (P=1/T) can be calculated by finding the value of an intermediate variable w:

$$w = \ln\left[\left(\frac{1}{P^2}\right)\right]^{\frac{1}{2}} \quad (0 < P \leq 0.5)$$

then calculate Z using the approximation:

$$Z = w - \frac{2.515517 + 0.802853w + 0.010328w^2}{1 + 1.432788w + 0.189269w^2 + 0.001308w^3} \quad (\text{Eq. 3.2.2})$$

When P > 0.5, (1 - P) is substituted for P equation 3.2.1.4 and the value of Z computed by equation 3.2.1.3 is given a negative sign.

5. Calculate the rainfalls for each probability level (or return period) by taking the anti-logarithms of the logRF<sub>j</sub> values.

### 3.2.2 ALGORITHM FOR GUMBEL'S EXTREME VALUE TYPE I ANALYSIS

- 1 Assemble the rainfall series.
- 2 Calculate the mean X and standard deviation S<sub>x</sub> of the series.
3. Select several return periods T<sub>j</sub> and associated exceedence probabilities P<sub>j</sub>.
4. Calculate the rainfalls corresponding to the return periods T<sub>j</sub> by using the equation (Lattermaier and Burges,(after ponce) ) as modification to the basic Gumbel method:

$$Y_j = X - \left(\frac{\sqrt{6}}{\pi} (0.5772 + \ln(\ln(\frac{T_j}{T_j - 1})))\right) * S_x \quad (\text{Eq. 3.2.3})$$

The peak rainfall intensity so obtained, in conjunction with a careful selection of runoff coefficients, can constitute a watershed size-and-shape-independent rainfall-runoff model that is as close a simulation of actual conditions as needed for minor highway drainage design purposes.

### **3.3. HYDROLOGIC ANALYSIS: SCS CURVE NUMBER METHOD**

The SCS Curve Number Method has been recommended in the *ERA Drainage Manual (2002)* as one of the major rainfall-runoff models suitable for Ethiopian Conditions. The approach adopted by ERA, however, is an oversimplified one in that it limits itself to a 24-hour storm event with a type II time distribution of rainfall intensity. It employs 24-hour depth-frequency curves to directly determine the peak rainfall-depth and does not deal with the time distribution of the direct-runoff rate. The software to be developed will not only automate SCS unit hydrograph method for Ethiopian conditions but will also address this major shortfall in the ERA manual as it outputs the time distribution of direct runoff rate for a duration (3-hr, 6-hr, 12-hr durations) to be specified by the designer.

#### **3.3.1 ALGORITHM FOR THE SCS CURVE NUMBER METHOD**

1. User inputs – Catchment Area
  - Average slope of the terrain (H/L), read from topographic map
  - Max length of travel (distance from the furthest point on the catchment to the outlet), read from topographic map
  - Design Frequency (Return period) (5 to 50 yrs for culverts)
  - Land use
  - Treatment or Practice
  - Hydrological Condition
  - Hydrological Soil Group
  - Antecedent Moisture Condition Class
  - Hydrologic Region ( to select applicable IDF equation)

2. Using input from user, select appropriate IDF curve for the region. (Note here that all IDF curves have been regressed, there are 4 IDF regions and at least 5 return periods: a total of at least 20 curves have been regressed into power equations for use in the software. It must be noted here that the figures computed from the regressed IDF equations for more than 3 hours tend to overestimate the design depth. This is because the IDF curves have been developed for less than 3 hours and projected data for more than 3 hours will be an overstretching of the equations and hence only a gross estimate.

3. From the IDF curve, read total depth for the selected duration. Convert intensity data to depth data by multiplying by the duration (number of minutes). The resulting 3-, 6- or 12-hour depth data is taken as the design rainfall.

4. Calculate  $T_p = 0.7T_c$

where

$$T_c = 0.02L^{0.77}S^{-0.385}$$

where

$T_c$  - time of concentration (min)

L - max. length of travel (m)

S - Slope equal to  $H/L$  where H is the difference in elevation between the most remote point in the basin and the outlet

5. Split up the design rainfall as selected in step 3 into a number of consecutive unit storm periods. Theoretically, this unit storm period should be equal to or less than  $\frac{1}{4}$  of the time to peak as calculated in step 4; however, in practice, a  $\frac{1}{2}$ -hour unit storm period is usually selected. The design rainfall is distributed throughout the selected duration with an SCS Type II rainfall distribution.

6. Determine the Curve Number value for the drainage basin under consideration. Use Tables provided as built-in by the program. Adjust the CN value, if necessary, according to AMC I or AMC III with the following equations:

$$CN I = \frac{4.2CN II}{10 - 0.058CN II}$$

$$CN III = \frac{23CN II}{10 + 0.13CN II}$$

7. Calculate the depths of excess rainfall (=direct runoff) using the CN method for design rainfall depths of accumulated unit storm periods as determined in step 5 and the CN value as determined in step 6.

$$S = \frac{25400}{CN} - 254$$

For each of the successive unit storm periods calculate the contribution of excess rainfall depth from the equation:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \quad \text{For } P > 0.2S$$

8. Calculate the peak runoff rate of the unit hydrograph for the drainage basin under consideration, using the empirical relationship

$$q_p = 0.208 \frac{AQ}{T_p}$$

A - area of drainage (km<sup>2</sup>)

Q – Excess rainfall (mm), 1mm for the Unit hydrograph

q<sub>p</sub> – Peak runoff rate unit hydrograph (m<sup>3</sup>/s)

T<sub>p</sub> – time to peak runoff unit hydrograph (hrs)

9. Calculate the ordinates of the dimensional unit hydrograph, using the dimensionless ratios given in the table (non-dimensional UH to be built into the program and made visible to the user but not modifiable). T<sub>p</sub> and q<sub>p</sub> are to be calculated here for the dimensional unit hydrograph using the dimensionless ratios as given in Table 2.2.

10. Calculate the ordinates of the individual hydrographs of direct runoff for each of the unit storm periods, using the ordinates of the unit hydrograph as calculated in step 9 and the corresponding excess rainfall depths as calculated in step 7.
11. Calculate the ordinates of the total composite hydrograph of direct runoff by adding the ordinates of the individual hydrographs of direct runoff as calculated in step 10. The ordinates of these individual hydrographs are lagged in time one unit storm period with respect to each other.
12. The highest value on the composite hydrograph is the peak runoff rate for the design rainfall with duration as selected in step 3.

The purpose of computing a design discharge for a structure with the above given data for a desired return period has been achieved.

### **3.4. METHODS OF HYDRAULIC DESIGN**

#### **3.4.1. HYDRAULIC DESIGN OF SLAB AND BOX CULVERTS.**

Once the design discharge quantity from the catchment has been determined, the next task would be determination of nominal hydraulic dimensions of the structure to accommodate that quantity of flow. Although flow in a culvert is typically non-uniform unsteady flow, due to the scope of this thesis work and associated time constraints, the hydraulics has been simplified to satisfy Manning's equations for steady, uniform open-channel flow.

It is intended that the computer program perform this task, namely hydraulic design, employing Manning's equation (Eq. 3.4.1) for Steady Open Channel Uniform flow, which is considered to be most practical for road drainage purposes.

The principal advantage of hydraulic design that the program should offer over conventional manual design would be an expedient optimization interface between reduction of upstream flooding and highway damage due to under-design on the one hand, and reduction of construction costs by avoiding gross over-design on the other

hand. The designer using the software must be able to make an on-the-spot fine-tuning of any design decision, including hydrologic parameters, to evaluate and compare the tradeoffs among several scenarios before deciding on the one that best suits his budget and risk requirements.

The following data are to be used in the software development for hydraulic design of minor drainage structures, as per Manning's Equation:

$$Q = \frac{1}{n} AR^{2/3} S^{1/2} \quad (\text{Eq. 3.4.1})$$

Q – discharge obtained from hydrologic analysis (m<sup>3</sup>/s)

$$R = \frac{A}{P}$$

R – hydraulic radius of the structure's section

where A – area of the flow section

P – wetted perimeter of the flow section

n - Manning's coefficient for the desired channel section type

S – design bed slope of the drainage structure

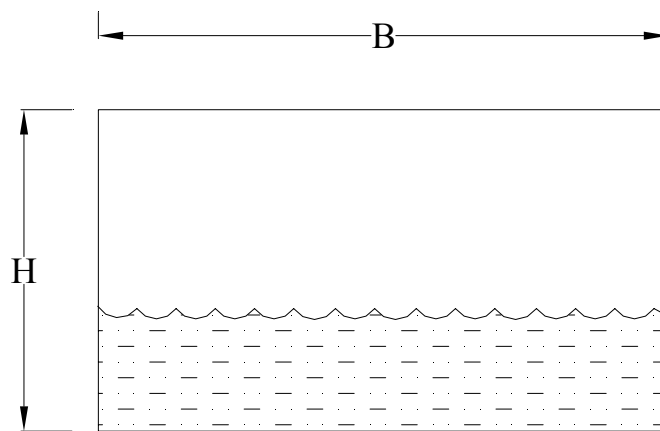


Fig. 3.4.1. A rectangular (slab or box culvert) flow section

For a rectangular Section (Slab or Box Culvert)

$$\text{Wetted Perimeter } P = 2 \cdot H + B$$

$$\text{Flow Area } A = B \cdot H$$

$$\text{Hydraulic radius } R = \frac{A}{P} = \frac{B \cdot H}{2 \cdot H + B}$$

where B – Clear Span of the structure (entered by user from actual measurement of gully width minus abutment thickness)

H – Flow Depth

Substituting the above in Manning's equation results in:

$$Q = \frac{1}{n} (B \cdot H) \left( \frac{B \cdot H}{2H + B} \right)^{2/3} \cdot S^{1/2}$$

$$Q = \frac{1}{n} \cdot \frac{(B \cdot H)^{5/3}}{(2H + B)^{2/3}} \cdot S^{1/2} \quad (\text{Eq. 3.4.2})$$

The software shall perform analytical solution of the fractional power relationship shown in Eq. 3.4.2 to determine the flow depth, H.

So, clear height of the structure (including free board) equals:

$$\text{Clr.Ht.} = \frac{H}{PFF} \cdot 100 \quad (\text{Eq. 3.4.3})$$

where PFF – percentage of full flow (proportion allowing free

board in consideration of intermittent turbulent flow,

67% is usual in the Ethiopian Practice) is selected by user as a percentage value between 0 and 99 %.

Using a realistic percentage of full flow between 60 and 80 % as warranted by roadway geometric design considerations in favor of:

- Sight-distance requirements
- The riding quality of the road (avoiding short bumps or sags along the vertical alignment)
- Aesthetics of the vertical alignment
- Earthwork economy

would ensure more practical hydraulic dimensions. One would do well here to remember that Manning's equation is developed for *Steady Open Channel Uniform Flow* and this is a simplified, albeit commonly used, assumption of culvert hydraulics.

### 3.4.2. HYDRAULIC DESIGN OF PIPE CULVERTS

Consider a partially full flow in a pipe of diameter  $D$  with flow depth  $d$ , assuming all angles in radians, the following can be deduced:

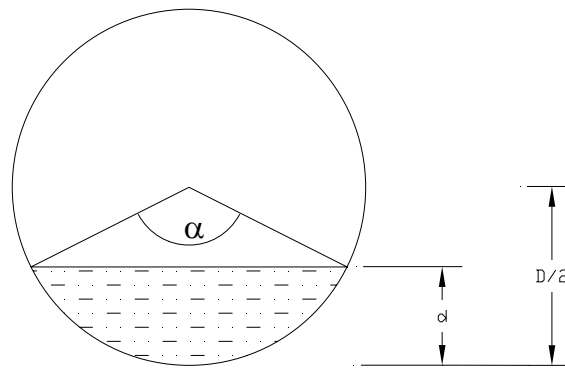


Fig. 3.4.2 Flow in a partially full pipe

Taking a triangular section of the pipe area above the flow surface,

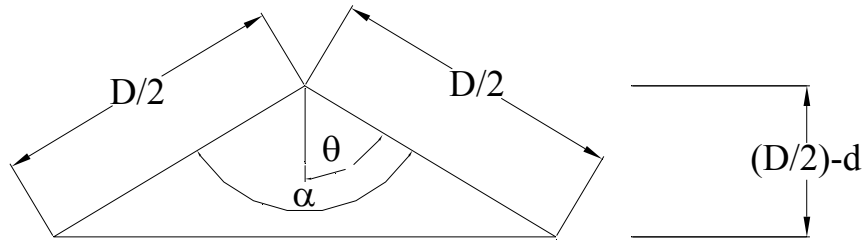


Fig. 3.4.3 Triangular section of partially full pipe above flow surface

$$\cos \theta = \frac{\left(\frac{D}{2} - d\right)}{\left(\frac{D}{2}\right)} \Rightarrow \theta = \cos^{-1}\left(1 - 2 * \left(\frac{d}{D}\right)\right)$$

$$\Rightarrow \theta = \cos^{-1}\left(1 - 2 * \left(\frac{d}{D}\right)\right) \quad (\text{Eq. 3.4.4})$$

$$\Rightarrow \alpha = 2 * \cos^{-1}\left(1 - 2 * \left(\frac{d}{D}\right)\right) \quad (\text{Eq. 3.4.5})$$

Again considering the bottom section of the pipe with the triangular section,

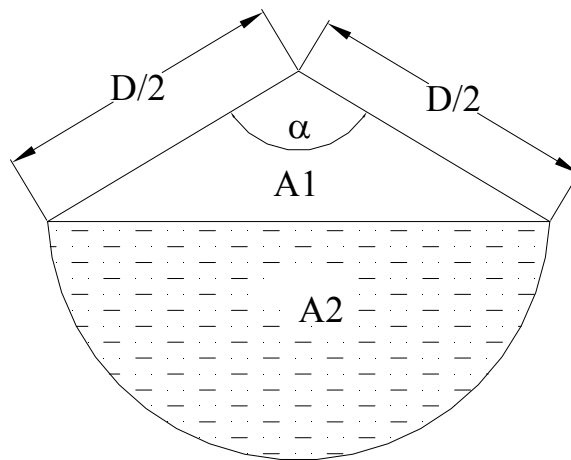


Fig. 3.4.4 Bottom section of partially full pipe

$$\text{Area of sector} = \frac{\alpha * D^2}{8}$$

$$\text{Area of Triangle: } A_1 = \frac{1}{2} * \left(\frac{D}{2}\right) * \left(\frac{D}{2}\right) * \sin \alpha = \frac{D^2}{8} * \sin \alpha \quad (\text{from the Law of Sines})$$

Area of flow  $A_2 = \text{Area of Sector} - \text{Area of triangle}$

$$\Rightarrow \frac{D^2}{8} * (\alpha - \sin \alpha) \quad (\text{Eq. 3.4.6})$$

$$\text{Wetted Perimeter } P = \frac{\alpha * D}{2}$$

Hydraulic Radius:

$$R = \frac{A}{P} = \frac{D^2}{8} * (\alpha - \sin \alpha) * \frac{2}{\alpha * D} \Rightarrow \frac{D}{4 * \alpha} (\alpha - \sin \alpha) \quad (\text{Eq. 3.4.7})$$

From the simple trigonometric manipulation above, and substituting the expressions for flow area Eq. (3.4.6) and hydraulic radius Eq. (3.4.7), Manning's equation applied for a pipe in terms of the flow depth and pipe diameter results in the rather lengthy equation:

$$Q = \frac{1}{n} * \frac{D^2}{8} * (2 * \cos^{-1}(1 - 2 * (d/D)) - \sin(2 * \cos^{-1}(1 - 2 * (d/D)))) * \left( \frac{D}{4 * 2 * \cos^{-1}(1 - 2 * (d/D))} * (2 * \cos^{-1}(1 - 2 * (d/D)) - \sin(2 * \cos^{-1}(1 - 2 * (d/D)))) \right)^{2/3} * S^{1/2}$$

which, when simplified, shrinks down to the form:

$$Q = \frac{1}{n} * \frac{D^{8/3}}{2^{16/3} * \cos^{-1}(1 - 2 * (d/D))} * (2 * \cos^{-1}(1 - 2 * (d/D)) - \sin(2 * \cos^{-1}(1 - 2 * (d/D))))^{5/3} * S^{1/2} \quad (\text{Eq. 3.4.8}) \dots\dots$$

where Q - design discharge obtained from hydrologic analysis

$(\frac{d}{D})$  – the ratio of full flow desired by designer and entered as a percentage of full flow .

d - Flow depth

D - Diameter of pipe

n - Manning's n

S - design bed slope of the pipe

The guidelines mentioned in slab and box culvert design for the choice of the percentage of full flow apply here also.

The algorithm to be developed shall employ direct numerical methods for the solution of the transcendental relationship of Eq. (3.4.8) to determine the diameter of the pipe needed to accommodate the design discharge with the percentage of full flow specified by the designer.

### 3.5. COMPONENTS AND FEATURES OF THE SOFTWARE

The software has been designed in Visual Basic programming language as a MDI (multiple document interface) computer program with one parent module for data storage and another child module for data entry.

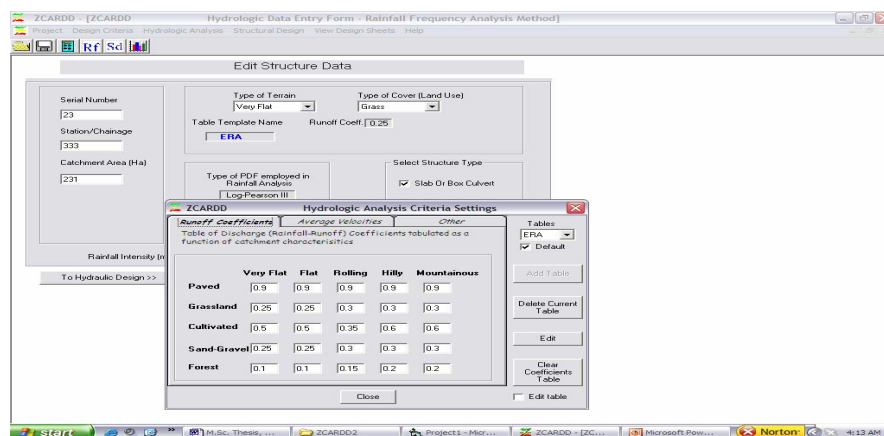


Fig. 3.5.1 Parent hydrologic interface with a child module active

As can be seen from Fig 3.5.1, the data entry form with one of the sub-modules will help the user to either opt to edit and send the data back to the main data-entry module or reject it altogether. This is accomplished using the Object-Oriented Programming feature of Visual Basic, which is powerful when dealing with multiple separate objects (modules) that work independent of each other but are able to exchange data when necessary. This is virtually impossible to achieve in standard spreadsheet programming software like Microsoft Excel.

A Windows graphical user interface that is very familiar to most users was selected to make the software user-friendly owing to Windows' long-standing usage and popularity. The main form contains three different phases of drainage design, which are hydrologic analysis, hydraulic and structural design. Fig. 3.5.2 shows the main data-entry form with the hydraulic module active.

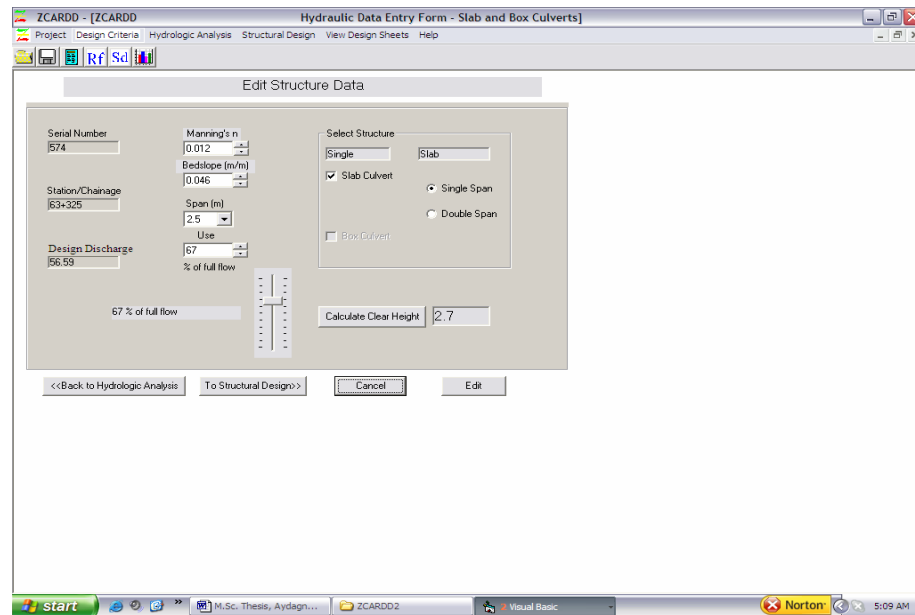


Fig. 3.5.2 Parent interface with hydraulic module active

The parent form also contains 4 other child forms (modules that work in coordination and data exchange capabilities with the main module). The four other child modules that work in coordination with the parent modules for data entry and storage are:

- Hydrologic analysis criteria settings
- Rainfall frequency analysis

- Structural design criteria settings
- SCS unit hydrograph discharge calculation

These forms or modules can be launched from the buttons or the pull-down menus on the parent form and are able to pass data back to it when required to do so. Each of the modules will be dealt with separately.

### 3.5.1 HYDROLOGIC ANALYSIS CRITERIA SETTINGS

This module is a data-entry/exchange form where the user can set up a site-specific table for runoff coefficients and average velocities and save it as a template for use in multiple catchments or projects. By default it contains the standard table from ERA/TCDE manuals (1968) that is still in use when the occasion is warranted.

	Very Flat	Flat	Rolling	Hilly	Mountainous
<b>Paved</b>	0.9	0.9	0.9	0.9	0.9
<b>Grassland</b>	0.25	0.25	0.3	0.3	0.3
<b>Cultivated</b>	0.5	0.5	0.35	0.6	0.6
<b>Sand-Gravel</b>	0.25	0.25	0.3	0.3	0.3
<b>Forest</b>	0.1	0.1	0.15	0.2	0.2

Fig. 3.5.3. Module for user-defined hydrologic analysis criteria table

But the shortcoming of the standard in using a single table has been dealt with by allowing the user to save and apply different table templates for the catchments that are being analyzed. The main module, while dealing with the runoff coefficient of the catchment automatically takes the table template saved and made active at the time the hydrologic analysis module is launched. Fig. 3.5.3 shows the hydrologic analysis

criteria settings module with the *Runoff Coefficients* tab and the standard ERA table active.

### 3.5.2 RAINFALL FREQUENCY ANALYSIS MODULE

This module allows the designer to enter a rainfall intensity series, compute the peak discharge rates using either of the two of the statistical distributions selected previously for a return period to be specified. This module is also capable of letting the designer save a rainfall series under a template name of choice, performing the analysis and sending data back to the main module. The designer will then be able to see on the main module which rainfall intensity series and which frequency distribution has been used for each individual catchment.

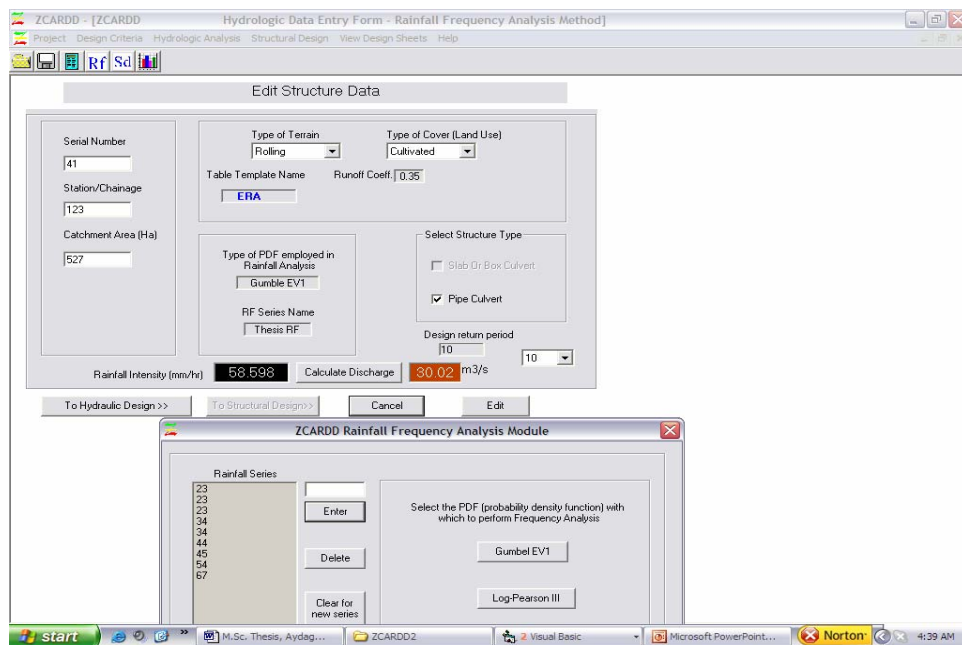


Fig. 3.5.4 Module for rainfall frequency analysis

### 3.5.3 STRUCTURAL DESIGN CRITERIA SETTINGS

This module has been designed to take care of the structural portion of the drainage structure, where the designer can set up his options for the type of road, cross slope, width of lane, etc are specified to enable the software to determine the finished road level at the crossing point and even calculate some quantities.

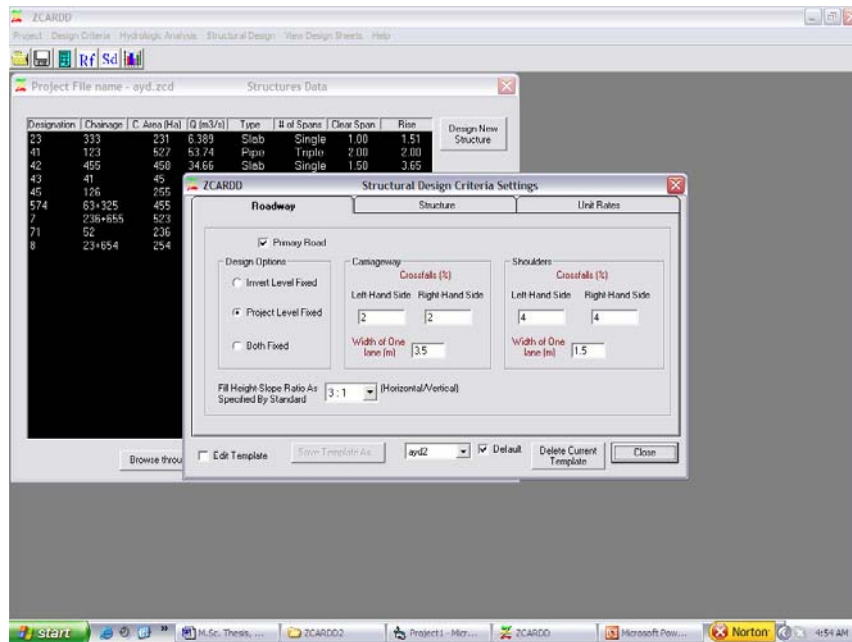


Fig. 3.5.5. Structural design criteria settings module

### 3.5.4 SCS UNIT HYDROGRAPH DISCHARGE CALCULATION MODULE

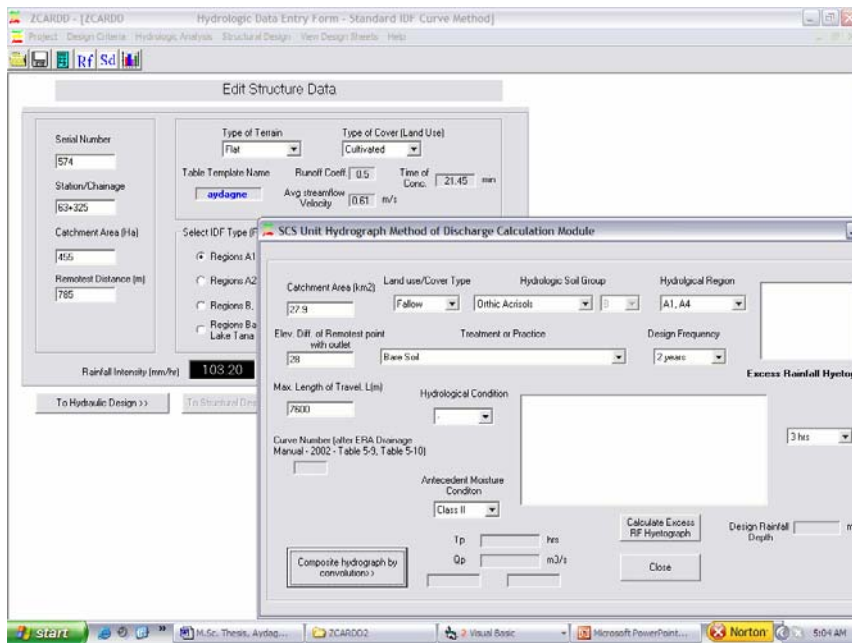


Fig. 3.5.6 SCS discharge calculation module

This module is a calculation platform for SCS unit hydrograph. Its features include automatic selection of the Curve Number value versus catchment soil and various

geologic characteristics; conversion of antecedent moisture conditions is also automatically accomplished.

The module calculates the design rainfall depth from the IDF curves provided by ERA, and the excess rainfall hyetograph for durations specified by the designer, and finally arrives at the peak discharge by using hydrograph convolution. This module has also the capability to send its data (peak discharge) to the main module.

### 3.5.5 ERROR-HANDLING

The software has quite a number of data-validation and error-handling features that help the designer to carry out analysis, design within acceptable norms, and avoid what is known as GIGO (Garbage In, Garbage Out) in software design.

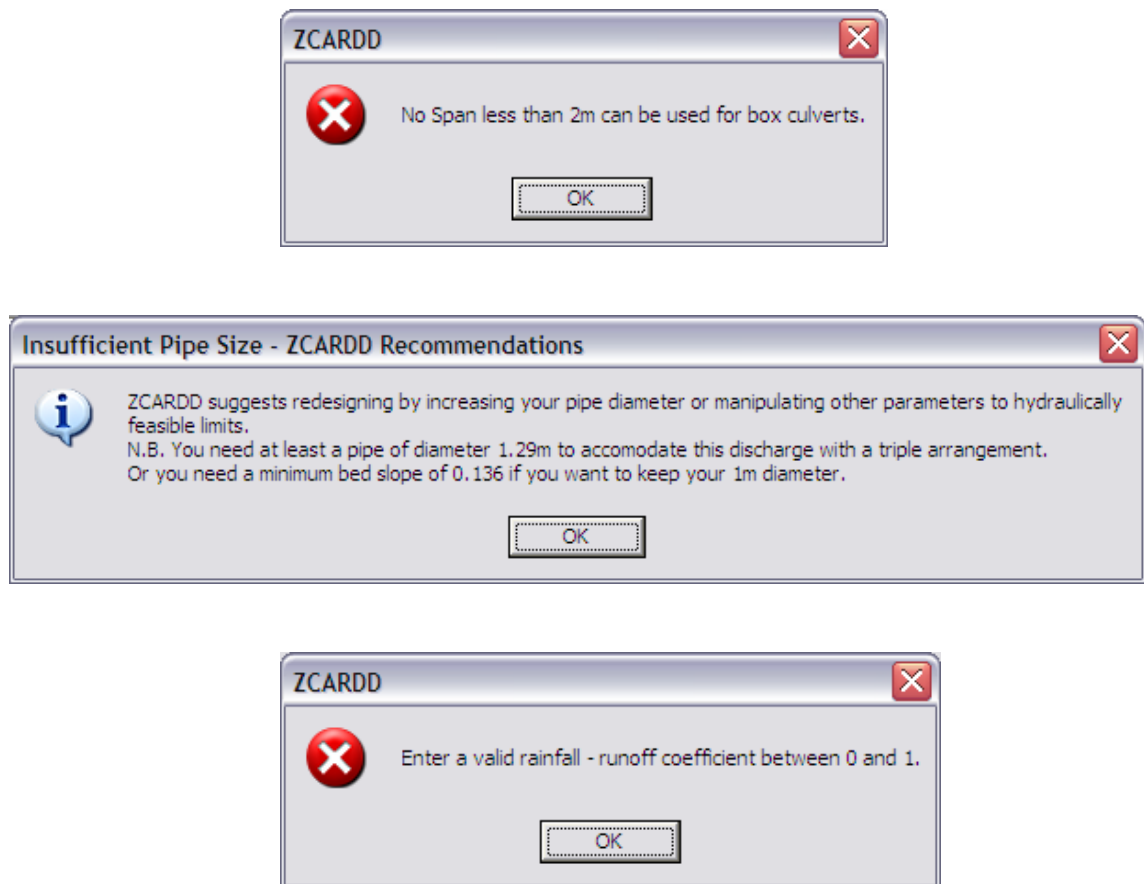


Fig. 3.5.7 Examples of alerting and error-handling message boxes

Several routines are written that help the designer to carry out tasks in compliance with the standards and norms of the practice and validate data-entry, whereby the

software immediately flashes with a message box by highlighting the offending parameter and informing/alerting the user to work within specified norms of practice. Three sample message box are shown in Fig. 3.5.7: two message boxes alerting the designer to keep within the norms of practice and the third one validating the discharge coefficient data and informing the user that the data entered is not valid.

### 3.6 RUNNING THE SOFTWARE

The software has been tested on several real projects and has been found to be helpful in coming up with quick solutions for the design of minor drainage structures.

Below is a sample data taken from a real catchment on a real project where SCS manual method has been used with its output of hydrograph. In this section, real data will be fed to the software and the results thereof will be compared with the hydrograph output from the manual calculation. The SCS unit hydrograph method shall be used to test the software.

It is desired to generate the flood hydrograph and arrive at the peak discharge for a 3-hour rainfall duration using SCS type II rainfall distribution for the drainage structure at Abuko river on Gedo-Nekemt Road Upgrading Design Review Project. (*Sava Engineering, 2006*)

Given data:

Name of project: Gedo-Nekemt Road Upgrading Project

Location: Abuko River at Ch 051+130

Catchment area: 87.98 km<sup>2</sup>

Average slope of the terrain, S: 0.0484

Maximum length of travel (remotest distance): 23551m

Design frequency: 50 years

Treatment or practice: Small grain straight row

Hydrological condition: Good

Hydrological soil group: Chromic Cambisols, Group B

Antecedent moisture condition class: Class II

Hydrologic region (to select applicable IDF equation): Gedo-Nekemt (region B2)

The data listed above will be entered in the SCS unit hydrograph module of the software and the output hydrograph will be compared with that carried out manually with the help of a Microsoft Excel spreadsheet model.

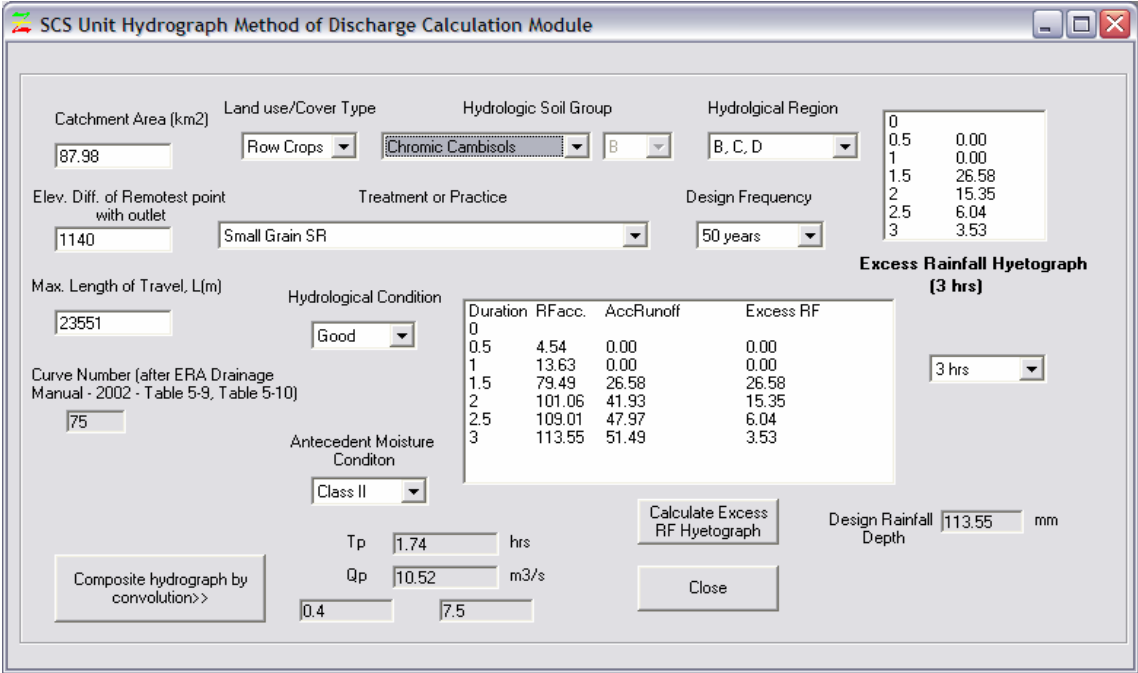


Fig. 3.5.8 Data entry and hyetograph output of SCS test-run project

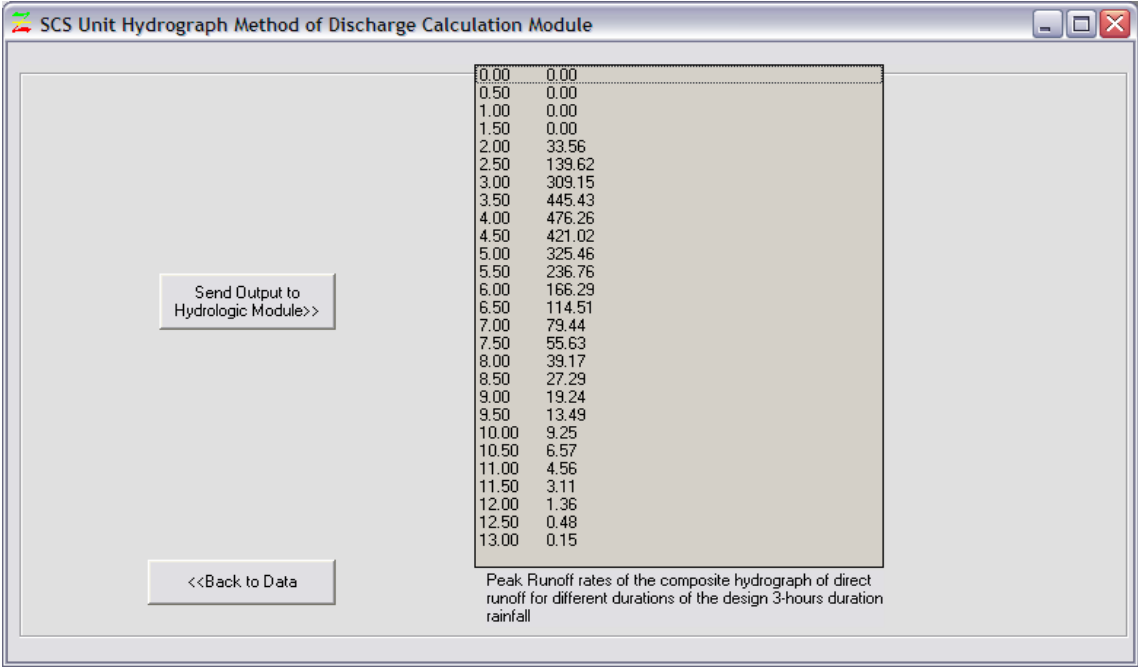


Fig. 3.5.9. The resulting composite hydrograph by convolution of the test-run project.

## **CHAPTER 4: CONCLUSIONS AND RECOMMENDATIONS**

The thesis work has attempted to address and simplify a major hurdle in the practice of Ethiopian road drainage design in an approach that is concise, expeditious and technology- conscious.

The test project comparison with a design manually carried out demonstrates that the software's expeditiousness is coupled with extreme precision, as the hydrograph outputs from the manual and the software designs manifest almost no visual distinction.

More work needs to be done in the area, as additional hydrological data, knowledge and information technology tools become available to the practice. Finer regionalization of the IDF curves using appropriate probability distributions would undoubtedly avail better and more effective tools for design engineers to better resolve the classic conflict between safety and economy in drainage design.

Hence, the works accomplished in the ERA Drainage Manual and in this thesis have to be improved upon by carrying out research projects on the applicability of probability distributions for different parts of the country. Moreover, additional meteorological and hydrological gauging stations have to be installed throughout catchments in the nation to help in the development of more reliable watershed models that better predict hydrologic behavior of Ethiopian catchments after a storm event.

Nevertheless, the computer program developed as a result of this thesis is practical and applicable in its current form as a drainage design tool. While the software cannot be considered a fundamentally groundbreaking work, it nonetheless pioneers a major paradigm shift in the Ethiopian practice of minor drainage design. It is not just a computerized version of the established practice during the last four decades, as it integrates methodologies not included therein; however, it capitalizes on the manual system already in place to transform it into a more convenient and accurate design tool.

The Windows user-interface has been designed with painstaking care, and as can be deduced already, a substantial amount of effort has gone into the writing of the code in an effort to juxtapose reliability in the system with user-friendliness and ergonomics in the interface.

The thesis has approached minor drainage design from a premise that assumed prevalent lack of hydrologic and hydraulic data on most Ethiopian small watersheds and streams. Since hydrologic data are expected to be more readily available in the future, it is anticipated that upcoming works will take advantage of proliferation of information to improve on this thesis and further simplify this critical activity in the schedule of road design projects.

The hydraulic design aspect has been formulated with a view to come up with nominal dimensions for each structure in order to provide the geometric designer with preliminary data on which the preliminary vertical profiles can be based.

Severe time constraints in the academic year and lack of material resources for the study have obliged this student to forfeit a detailed automation of the hydraulic design in favor of a prospective continuation of research in the area. A future thesis to be presented by me or another postgraduate researcher is highly recommended to focus on the finer details of both hydrologic and hydraulic design for drainage structures in the Ethiopian context.

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## **APPENDICES**

