

# **ANALYSIS OF FLOODS USING THE HEC-1 PROGRAM PACKAGE**

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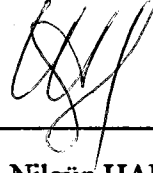
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## M.Sc THESIS EXAMINATION RESULT FORM

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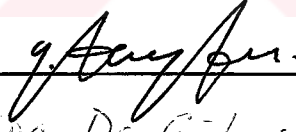
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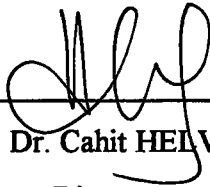
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## ABSTRACT

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The basic property of the floods; one of the most important disaster that takes place with the interference of humanbeing or nature, is constantly a problem which cannot be avoided anytime. Social life continuously lasted struggling in order to control this extreme event, as they are always under threat. Unfortunately, no solution has been obtained in spite of the knowledge deposit of the 20<sup>th</sup> century. Thus, consistent and active methods are desired in order to determine the flood characteristic in the point of view improved active prevention methods. In this conjunction, basin simulation models are being used, so developed computer models became the basic tools of the control and management work, driven for the flood process.

The most frequently used model in the concerned models is the HEC-1 model, developed by Hydrologic Engineering Center (HEC) in USA. To have the ability of concerning the hole basin with a single system with the effects of the structures (dams, dikes, levees, ...) to the floods; determining the response of the structures and properties of the basin to the floods; the ability of determining the final plan and project decision in a consistent way as it gives the possibility to product the scenarios that consist of different flood peaks and finally the ability of arranging economical optimization in the model content.

In this thesis work, HEC-1 model is studied in detail, tested on the sample basis in order to demonstrate the serviceability. In addition, to have a sample in our country, Muteferrik Marmara Basin is chosen. The conclusions are presented with the help of the tables and graphics, besides.

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## ÖZET

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Doğal ya da insan müdahalesi sonucu oluşan afetlerin en önemlilerinden biri olan taşkınların temel özelliği, insanlar için hiçbir zaman bertaraf edilemeyen, sürekli bir problem oluşturmalarıdır. Toplumlar, yüzyıllardan beri taşkın tehdidi ile karşı karşıya kaldıklarından, bu ekstrem olayın kontrolü amacıyla araştırma ve mücadele faaliyetlerini sürdürmüşlerdir. Ancak günümüzde ulaşılan bilgi birikimine rağmen sorun çözümlenmiş değildir. Bu nedenle taşkın karakteristiklerinin tanınması ve etkin kontrol önlemlerinin geliştirilmesi açısından tutarlı ve etkili yöntemlere ihtiyaç vardır. Bu çerçevede, gelişmiş ülkelerde havza benzeşim modellerinden yararlanılmaya başlanmış ve geliştirilen bilgisayar modelleri taşkınlar konusunda yürütülen kontrol ve yönetim faaliyetlerinin temel aracı haline gelmiştir.

Söz konusu modeller arasında en yaygın kullanıma ulaşmış olanı, A.B.D’de Hydrologic Engineering Center (HEC) tarafından geliştirilmiş olan HEC-1 modelidir. HEC-1 yağış-akış modelinde, havzanın bir bütün (sistem) olarak tüm özellikleriyle ele alınarak, bu sistemde yer alan yapıların da (baraj, regülatör, vs) taşkınlara olan etkilerinin dikkate alınabilmesi; havzanın yapısı ve özellikleri dikkate alındığında, bunun taşkınlara karşı davranışının belirlenebilmesi; farklı mertebelerde taşkınlardan oluşan senaryoların üretilmesine imkan tanıdığından nihai planlama ve tasarım kararlarının verilmesini tutarlı biçimde yapabilmesi ve yine model kapsamında ekonomik optimizasyonun yapılması HEC-1 modelinin kullanımını yaygın hale getiren özelliklerinin başında gelmektedir.

HEC-1 modeli, bu tez kapsamı içerisinde detaylı olarak ele alınmış, işleyebilirliğinin gösterilmesi açısından örnek havzalar üzerinde denenmiş ve yurdumuzda da bir uygulama teşkil etmesi açısından Muteferrik Marmara Havzası örnek alınarak çözüm yoluna gidilmiştir. Sonuçlar, tablolar ve bunun yanında grafik çizimlerle birlikte sunulmuştur.

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## CONTENTS

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|                      | <b>Page</b> |
|----------------------|-------------|
| CONTENTS.....        | IV          |
| LIST OF FIGURES..... | X           |
| LIST OF TABLES.....  | XI          |

### Chapter One

#### INTRODUCTION

|                      |   |
|----------------------|---|
| 1. INTRODUCTION..... | 1 |
|----------------------|---|

### Chapter Two

#### ANALYSIS OF FLOODS

|                                              |   |
|----------------------------------------------|---|
| 2. ANALYSIS OF FLOODS.....                   | 4 |
| 2.1 Coping with Floods.....                  | 4 |
| 2.2 Flood Estimation Methods.....            | 5 |
| 2.2.1 Basic Approaches.....                  | 5 |
| 2.3 Estimation of Flood Characteristics..... | 7 |
| 2.3.1 Basic Concepts on Floods.....          | 7 |

|                                                                                            |    |
|--------------------------------------------------------------------------------------------|----|
| 2.3.2 Methods of Estimation.....                                                           | 10 |
| 2.3.3 Brief Description of Methods.....                                                    | 11 |
| 2.3.4 Three Ranges of Flood Probabilities.....                                             | 14 |
| 2.3.5 Determination of Probability of Exceedence of PMF and<br>the Other Large Floods..... | 15 |
| 2.3.6 Properties of Tails and Their Extrapolation.....                                     | 16 |
| 2.4 The Use Mathematical Models in Analysis of Floods.....                                 | 18 |
| 2.4.1 Watershed Modeling.....                                                              | 18 |
| 2.4.2 Deterministic Hydrologic Models.....                                                 | 20 |
| 2.4.3 Types of Deterministic Models.....                                                   | 21 |
| 2.4.4 Flood Characteristics Based on Single Event Models.....                              | 23 |
| 2.5 Application of HEC Models in Analysis of Floods.....                                   | 24 |

### Chapter Three

#### HEC-1 MODEL

|                                                      |    |
|------------------------------------------------------|----|
| 3. HEC-1 MODEL.....                                  | 27 |
| 3.1 History of HEC-1.....                            | 27 |
| 3.2 Purpose of HEC-1.....                            | 29 |
| 3.3 The HEC Modeling Philosophy.....                 | 29 |
| 3.4 Basic Concepts (River Basin Representation)..... | 30 |
| 3.4.1 Sizes and Types of Basins.....                 | 30 |
| 3.4.2 Model Structure.....                           | 30 |
| 3.4.3 Limitations of HEC-1.....                      | 31 |
| 3.4.4 Hydrologic Simulation Components.....          | 32 |
| 3.5 Model Development.....                           | 32 |
| 3.5.1 Identification of the Stream Network.....      | 32 |
| 3.5.2 Subbasin Runoff.....                           | 34 |

|                                                                   |    |
|-------------------------------------------------------------------|----|
| 3.5.2.1 Subbasin Processes.....                                   | 34 |
| 3.5.2.2 Precipitation.....                                        | 34 |
| 3.5.2.3 Snow.....                                                 | 35 |
| 3.5.2.4 Losses.....                                               | 36 |
| 3.5.2.5 Excess-to-Runoff Transformation.....                      | 37 |
| 3.5.2.6 Base Flow.....                                            | 38 |
| 3.5.3 Channel Routing.....                                        | 38 |
| 3.5.4 Lake Routing.....                                           | 39 |
| 3.5.5 Confluences.....                                            | 39 |
| 3.5.6 Reservoir Routing.....                                      | 40 |
| 3.5.7 Diversions.....                                             | 40 |
| 3.6 Preparation of Input Data.....                                | 42 |
| 3.6.1 Types of Input Data.....                                    | 42 |
| 3.6.2 Special Features for Input Data.....                        | 43 |
| 3.6.2.1 Input Control.....                                        | 43 |
| 3.6.2.2 Time Series Input .....                                   | 44 |
| 3.6.2.3 Data Repetition Conventions.....                          | 44 |
| 3.6.3 Hydrologic/Hydraulic Simulation Options.....                | 45 |
| 3.7 Variables and Parameters of HEC-1 Model.....                  | 46 |
| 3.8 Outputs of HEC-1 Model.....                                   | 46 |
| 3.9 Computation Methodologies for Rainfall-Runoff Simulation..... | 54 |
| 3.9.1 Rainfall-Runoff Simulation.....                             | 54 |
| 3.9.2 Precipitation.....                                          | 54 |
| 3.9.2.1 Precipitation Hyetograph.....                             | 54 |
| 3.9.2.2 Historical Storms.....                                    | 55 |
| 3.9.2.3 Synthetic Storms.....                                     | 56 |
| 3.9.3 Interception/Infiltration.....                              | 58 |
| 3.9.4 Unit Hydrograph.....                                        | 60 |
| 3.9.4.1 Basic Methodology.....                                    | 60 |
| 3.9.4.2 Synthetic Unit Hydrographs.....                           | 61 |
| 3.9.5 Base Flow.....                                              | 65 |
| 3.9.6 Flood Routing.....                                          | 67 |

|                                                                          |    |
|--------------------------------------------------------------------------|----|
| 3.10 Multiplan-Multiflood Analysis.....                                  | 69 |
| 3.11 Model Calibration and Verification.....                             | 70 |
| 3.11.1 Methodology.....                                                  | 70 |
| 3.11.2 Calibration for the Unit Hydrograph and Loss Rate Parameters..... | 72 |
| 3.11.2.1 Optimization Methodology.....                                   | 72 |
| 3.11.2.2 Algorithm for Model Calibration.....                            | 75 |
| 3.11.2.3 Application of the Calibration Capability of HEC-1.....         | 77 |
| 3.11.3 Routing Parameters.....                                           | 79 |
| 3.12 Assessment of Model Reliability.....                                | 79 |
| 3.12.1 Requirements.....                                                 | 79 |
| 3.12.2 Uncertainties That Affect Model Reliability.....                  | 80 |
| 3.12.2.1 Natural Uncertainties.....                                      | 80 |
| 3.12.2.2 Data Uncertainties.....                                         | 81 |
| 3.12.2.3 Uncertainties in Model Parameters.....                          | 82 |
| 3.12.2.4 Uncertainties in Model Structure.....                           | 82 |
| 3.13 Computer Requirements for HEC-1.....                                | 83 |
| 3.13.1 Hardware Requirements.....                                        | 83 |
| 3.13.2 Software Characteristics.....                                     | 84 |
| 3.14 Application Procedure.....                                          | 85 |
| 3.15 Trial Runs of HEC-1 Program Package.....                            | 86 |
| 3.15.1 Example Problems.....                                             | 86 |
| 3.15.2 Example Problem 1.....                                            | 86 |
| 3.15.2.1 Inputs for the Problem.....                                     | 86 |
| 3.15.2.2 Solution.....                                                   | 89 |
| 3.15.3 Example Problem 2.....                                            | 94 |
| 3.15.3.1 Problem Definition.....                                         | 94 |
| 3.15.3.2 Solution.....                                                   | 96 |

**Chapter Four**  
**BASIN SELECTED FOR THE CASE STUDY**

|                                           |     |
|-------------------------------------------|-----|
| 4. BASIN SELECTED FOR THE CASE STUDY..... | 104 |
| 4.1 Basin Characteristic.....             | 104 |
| 4.2 Available Data.....                   | 104 |
| 4.3 Data Used for Model Calibration.....  | 107 |

**Chapter Five**  
**CASE STUDY**

|                                            |     |
|--------------------------------------------|-----|
| 5. CASE STUDY.....                         | 110 |
| 5.1 General.....                           | 110 |
| 5.2 Preparation of Input Data.....         | 110 |
| 5.3 Determination of Model Parameters..... | 114 |
| 5.4 Multiplan Analysis.....                | 129 |

**Chapter Six**  
**CONCLUSION**

|                    |     |
|--------------------|-----|
| 6. CONCLUSION..... | 136 |
|--------------------|-----|

**REFERENCES**

|                 |     |
|-----------------|-----|
| REFERENCES..... | 139 |
|-----------------|-----|

**APPENDIX**

|                            |     |
|----------------------------|-----|
| APPENDIX – Discharges..... | A-1 |
|----------------------------|-----|



---

## LIST OF FIGURES

---

|                                                                                                       | <b>Page</b> |
|-------------------------------------------------------------------------------------------------------|-------------|
| Figure 3.1 HEC-1 Representation of Watershed Components.....                                          | 33          |
| Figure 3.2 Treatment of Diversions in Input Data.....                                                 | 41          |
| Figure 3.3 Rainfall Hyetograph.....                                                                   | 54          |
| Figure 3.4 Loss rate, Rainfall Excess Hyetograph.....                                                 | 59          |
| Figure 3.5 SCS Dimensionless Unit Graph.....                                                          | 65          |
| Figure 3.6 Base Flow Diagram.....                                                                     | 66          |
| Figure 3.7 Multiflood and Multiplan Hydrographs.....                                                  | 71          |
| Figure 3.8 Error Calculation for Hydrologic Optimization.....                                         | 73          |
| Figure 3.9 The Castro Valley Watershed.....                                                           | 87          |
| Figure 3.10 Schematic Diagram of Castro Valley Watershed Showing<br>Components of HEC-1 Analysis..... | 88          |
| Figure 3.11 Hydrographs at RES 4.....                                                                 | 93          |
| Figure 3.12 Hydrographs at Outflow of the Watershed.....                                              | 94          |
| Figure 3.13 Watershed for Example Problem 2.....                                                      | 95          |
| Figure 4.1 Muteferrik Marmara Basin.....                                                              | 105         |
| Figure 5.1 Precipitation and Runoff Stations.....                                                     | 111         |
| Figure 5.2 Subcatchments of Muteferrik Marmara Basin.....                                             | 112         |
| Figure 5.3 A Schematic Diagram of the Basin.....                                                      | 113         |
| Figure 5.4 The Resulting Hydrographs for Rooting<br>between Stations 214 and 212.....                 | 115         |
| Figure 5.5 Comparison of Model Output Hydrograph<br>and the Observed Hydrograph for Run I.....        | 123         |
| Figure 5.6 Comparison of Model Output Hydrograph<br>and the Observed Hydrograph for Run II.....       | 129         |
| Figure 5.7 Outlet Hydrographs for Plan I and Plan II.....                                             | 135         |

# **LIST OF TABLES**

|                                                                                                 | <b>Page</b> |
|-------------------------------------------------------------------------------------------------|-------------|
| Table 2.1 Techniques Used in Flood Estimation.....                                              | 7           |
| Table 3.1 HEC-1 Loss Rate Methods.....                                                          | 36          |
| Table 3.2 HEC-1 River Channel Routing Methods.....                                              | 39          |
| Table 3.3 Subdivisions of Simulation Data.....                                                  | 42          |
| Table 3.4 Data Repetition Options.....                                                          | 45          |
| Table 3.5 Variables and Parameters Specified in HEC-1 Model.....                                | 48          |
| Table 3.6 Constraints on Unit Graph and Loss Rate Parameters.....                               | 74          |
| Table 3.7 HEC-1 Default Initial Estimates for Unit Hydrograph and<br>Loss Rate Parameters.....  | 76          |
| Table 3.8 Characteristics of Subcatchments in the Castro Valley Watershed....                   | 88          |
| Table 3.9 Characteristics of the Detention Reservoir<br>at the Outlet of Subcatchment 4.....    | 89          |
| Table 3.10 Parameters for Snyder's Unit Hydrograph.....                                         | 90          |
| Table 3.11 Characteristics of the Sample Watershed .....                                        | 95          |
| Table 3.12 Characteristics of the Detention Reservoir.....                                      | 96          |
| Table 4.1 Characteristics of Streamgaging Stations<br>in the Muteferrik Marmara Basin.....      | 106         |
| Table 4.2 Characteristics of the Precipitation Stations<br>in the Muteferrik Marmara Basin..... | 106         |
| Table 5.1 Existing Characteristics of the Watershed.....                                        | 117         |
| Table 5.2 Muteferrik Marmara Basin: Rainfall and<br>Observed Hydrograph Data.....               | 117         |
| Table 5.3 Summary of the Run I.....                                                             | 122         |

|                                                                     |     |
|---------------------------------------------------------------------|-----|
| Table 5.4 Comparison of the Computed and Observed Hydrographs.....  | 122 |
| Table 5.5 Summary of the Run II.....                                | 128 |
| Table 5.6 Comparison of the Computed and Observed Hydrographs ..... | 128 |
| Table 5.7 Summary of the Multiplan Analysis.....                    | 133 |



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## CHAPTER ONE

# INTRODUCTION

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In many countries and regions of the world, floods are the most costly hazards in terms of both loss of human lives and material damage. Experiences from the distant past, during which people interacted with floods in various ways, show clearly that flood dangers and impacts can be mitigated or alleviated but they can never be completely eliminated or avoided.

The basic requirement towards minimization of flood hazards is the identification of flood characteristics in a basin. Several methods have been used to this end, ranging from frequency analyses to regional flood analyses and recently to computer-based mathematical models. The use of the latter has become highly popular in recent years as computer-based mathematical models provide effective tools for decision making and management of flood control measures.

In the general sense, the ultimate value of models is twofold: first, to understand and reproduce how the modeled system works, and then to predict how the system will respond to future alternative conditions and actions. It is recognized now that reliance on intuition, engineering experience and/or simplified analysis techniques is not an appropriate means for making management decisions that can have extreme economic societal and technical consequences.

Following the above considerations, it is intended in this study to investigate the use of a computer-based mathematical model, HEC-1, in analysis of floods within a decision making perspective. The Hydrologic Engineering Center, HEC, developed several small computer programs for simulating various parts of the precipitation-

runoff and river-routing processes in the early 1960's. The small, individual programs were necessary because of the limited computer memory at that time. The first version of the HEC-1 Flood Hydrograph Package was published in October 1968. It has been expanded and revised and published many times since then to incorporate improvements in hydrologic algorithms and advances in computer hardware and software. To simplify input requirements and to make the program output more meaningful and readable, a major revision was made in 1973. In 1981, major revisions were made to the 1973 version of the program. The program input and output formats were completely revised, and the computational capabilities of the special versions of HEC-1dam-break (HEC-1DB), project optimization (HEC-1GS) and kinematic wave (HEC-1KW), were combined in one program. The new program included the powerful analysis features available in all the previous programs, together with some additional capabilities and merged them into a single easy to use package. In late 1984, a microcomputer version (PC version) was developed.

The current standard version, Version 4.0 (September 1990), represents improvements and expansions to the hydrologic simulation capabilities together with interfaces to the HEC Data Storage System (HEC-DSS). The DSS connection allows HEC-1 to interact with the input and output of many HEC and other models. New hydrologic capabilities in HEC-1 include Green and Ampt infiltration, Muskingum-Cunge flood routing, reservoir releases input, and improved numerical solution of kinematic wave equations. The Muskingum-Cunge routing may also be used for collectors and main channels in a kinematic wave land-surface runoff calculation. This new release also automatically performs numerical analysis stability checks for the kinematic wave and Muskingum-Cunge routings. The numerical stability check was added because many users did not check the validity of the time and steps used in the model.

The main purpose of HEC-1 in its initial development was simulation of hydrologic processes. This capability would enable the Corps to analyze historical floods and design projects for flood damage reduction. HEC-1 was limited to flood

event studies because flood protection was the Corps's major responsibility. Those flood events were typically major historical storms or hypothetical design storms. The simulation would begin with precipitation, then compute infiltration losses, and finally route the runoff throughout a river basin. HEC-1 soon expanded from just a hydrologic simulation tool; it increased its scope to include special hydrologic analyses that the hydrologic engineer typically performs using the hydrologic data derived from HEC-1. Hydrologic analysis tools include automated parameter estimation, multiple-storm simulation, precipitation depth-area relationship coordination, and flood frequency curve determination. The flood damage reduction project analysis capabilities include multiple-projects analysis, modified frequency curves, flood damage computation, and project-size optimization.

In the presented study, three accomplishments are foreseen:

- a) understanding of HEC-1 and its capabilities;
- b) getting the model package run under different inputs;
- c) running the model on an actual test case.

The first two accomplishments are tested in two example problems which are available in literature with given inputs and model outputs.

After the model is found to be working properly, it was applied to an observed storm event in the Mütfeferrik Marmara Basin. This case study includes model calibration and multiplan analysis to investigate effects of urbanization in flood characteristics of the basin.

The next chapter reviews methods of flood analyses, including modeling techniques and previous applications of HEC-1. Chapter 3 presents the model package with its basic features. The test basin and input data for the case study are described in Chapter 4, followed by the case study in Chapter 5. Chapter 6 summarizes the basic conclusions derived from the study.

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## CHAPTER TWO

# ANALYSIS OF FLOODS

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### 2.1 Coping with Floods

Floods are defined as extremely high flows or levels of rivers, whereby water inundates flood plains or terrains outside the water-confined major river channels. Floods also occur when water levels of lakes, ponds, reservoirs, aquifers and estuaries exceed some critical values and inundate the adjacent land, or when the sea surges on coastal lands much above the average sea level. In many countries and regions of the world, floods are the most costly hazards in terms of both loss of human lives and material damage. Experience demonstrates that flood dangers can be alleviated but can not be completely eliminated. Coping with floods refers to all measures which society can undertake to alleviate impacts of floods. In general, flood disasters may be classified in three major types (Yevjevich, 1994, p.3):

1. Natural floods, meaning the group of floods which will occur regardless whether humans live in a river basin or not;
2. Modified natural floods, which include the group of flood events which are affected by humans, resulting in the attenuated or the amplified floods compared with the natural floods; and
3. The completely human-generated or human-induced flood disasters, which include the group of floods caused by breaks of structures built by society (i.e., break of a levee, break of a dam or dike), or by errors in operation (such as mismanagement of flood control gates or equipment).

In coping with floods, measures that can be or have been used by societies may be classified into four groups of alternatives (Yevjevich, 1994, pp.3, 4):

1. Do nothing, either structurally or administratively. This means inducing people only to adjust in some known or new way to flood phenomenon, such as leaving flood plains mainly to agriculture, or the flood plain is inundated by “quiescent flood” depositing nutrients and leaving moisture in soil;
2. Implement only the non-structural measures for alleviating flood impacts. This is mainly done by regulating the way of using flood plains and other flood-prone land, carrying out flood defence, and, when feasible, using insurance to distribute flood risks.
3. Implement structural flood control measures only, which include intensive and extensive physical measures which change a flood generating environment (such intensive measures are: reservoirs, levees, dikes, new flood-related channel; or extensive measures usually spread over the catchment).
4. Combination of structural and non-structural measures.

There are three major groups of flood problems and solutions. The entire interaction between society and floods may be divided into three basic groups (Yevjevich, 1994, pp.5, 8):

1. Learning about floods, as the major information input into design of any policy or strategy for coping with floods, including the alternative of doing nothing except the information dissemination;
2. Estimation of flood impacts of previous floods, or synthesizing the potential future flood damages under the most recent or future life and property conditions in flooding areas; and

3. Coping with floods in order to minimize flood impacts either by value judgement, selected flood policy or economic optimization.

Five groups of methods which are discussed in the second section for determining flood characteristics are: regional flood estimation methods, frequency distribution methods, historic and flood-related paleo-data methods, Bayes Approaches, probable maximum flood, and information transferring methods (Yevjevich, 1994, pp.5, 8, 9).

## **2.2 Flood Estimation Methods**

### **2.2.1 Basic Approaches**

In order to prevent flood damages, the most important flood characteristics which should be known are peak flows and frequencies. Floods that have less probability of occurrence such as  $10^{-3}$  and  $10^{-7}$  are important for engineering design and planning related to water resources. There are few historical data available on about these floods which have high recurrence intervals. Thus, it's impossible to ascertain the estimates on peak flows, their probabilities and recurrence intervals by using observed data. However, such flood characteristics must be known for purposes of planning. Therefore, research on modeling and statistical analysis of rare floods still continue to improve our estimates on extreme events.

There are various methods for estimating rare floods. It is seen in Table 1 that these methods can be combined in to 3 groups (National Research Council, 1988). In the first method of estimating PMP, although the word "probable" is used, the approach used does not assign a probability to PMP so that flood risks can not be defined in a probabilistic basis. In the other approaches, peak flood flows of flood can be described with their probabilities.

Whichever method is used, the purpose of flood analysis is to derive maximum information about the potential and orders of flood events at a point. Apart from the methods which are based on PMP-PMF estimates, the other methods may be divided in two groups: (a) statistical analysis of flow observations; (b) statistical analysis of

meteorological data and using the results of analysis in estimation of flow. Researchers emphasize that both approaches should be used to produce the required information about floods (Ashkar, 1996, pp.63-83).

In general, according to the data used and the mathematical and conceptual approaches applied, it is possible to define 6 principal groups of methods for flood estimation. These methods are summarized in Table 2.1.

**Table 2.1 Techniques Used in Flood Estimation (Fıstıkoğlu & Özkul & Harmancıoğlu, 1997)**

| <u>INPUT</u>                                                                                                                                              | <u>ANALYSIS</u>                                                                                                               | <u>OUTPUT</u>                                                                                                        |
|-----------------------------------------------------------------------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------|
| I. Estimation of Probable Maximum Precipitation (PMP)                                                                                                     | Rainfall-Runoff Transformation                                                                                                | Probable Maximum Flood (PMF)                                                                                         |
| II. Observations of Flood Flows<br>a) Continues observations (historical and new observations)<br>b) Collecting paleo-flood data                          | Flood Frequency Analysis<br>a) Flood analysis at a point<br><br>b) Estimation of floods at a point by regional flood analysis | Probabilities and Recurrence Intervals of Peak Flood Flows                                                           |
| III.<br>A) Analysis of Intense Rainfall at a Region<br><br>B) Stochastic Modeling of Rare Rainfall in Time and Space; Obtaining Synthetic Rainfall Series | Deterministic Rainfall-Runoff Models<br><br><br>Deterministic Rainfall-Runoff Models                                          | Determination of Flood Hydrographs And Probabilities<br><br><br>Determination of Flood Hydrographs And Probabilities |

## **2.3 Estimation of Flood Characteristics**

### **2.3.1 Basic Concepts on Floods**

Both statistical and deterministic concepts on floods have been developed. The variation in the magnitude of flood peak or flood volume either as the largest event each year or as all the events above a threshold event during the period of observations, are easily shown to be random variables following frequency distributions which are then conceived as estimates of the hypothesized probability

distributions. This concept does not assume the upper bound for distribution functions. It assumes that whatever maximum was observed in the past, an incrementally larger value could still occur in the future. This implication for the boundless upper tail of fitted probability distribution functions means in practice that floods are chance-governed natural phenomenon which is not subject to a clear limitation in its extremes except by the probability related restrictions (the larger the flood, the smaller its probability of exceedence).

The concept of the boundless flood probability distribution conflicted (mainly in the United States) with two other prevailing concepts: (1) the nature can not be boundless (say a small creek can not produce precipitation and a flood equal to the largest flood of the Lower Mississippi River); and (2) the ocean-atmospheric processes must have a limitation in the supply of moisture to river basins through storms, or that an upper precipitation boundary exists. They resulted in the deterministic concept of maximum precipitation (MP) in the 1930's. Second thoughts among the originators of this concept on whether the MP could or could not be further exceeded resulted in a revised concept called the probable maximum precipitation (PMP), as it is used at present. However, the word "probable" in this name did not mean that a probability could be attached to MP. It should have been called "likely maximum precipitation". The word "likely" would then imply that it is an upper limit except for the uncertainty in its estimation. By the use of word "probable" two confusions resulted: (1) that PMP is a kind of maximum, assumed to exist in nature, except for the errors in its estimation; and (2) that the quantitative probability value can not, or should not, be attached to the PMP concept.

The PMP was usually computed by maximizing some basic factors which produce observed large storms. The concept of the storm is the first link between PMP and the resulting flood. The observed but maximized storm is transposed over a river basin in a position which produces the largest flood. This position may be conceived probabilistically as the possible event, however with a probability of zero for its occurrence.

The assumed conditions of various river basin factors, especially the antecedent moisture condition index, are the next link between PMP and the resulting flood event. Because this index is conceived as a random variable, without an upper limit, the resulting flood should also be distributed without an upper limit regardless whether PMP has or has not an upper boundary. The use of the unit hydrograph for the estimated effective precipitation (rainfall minus all abstraction), or the use of any rainfall/runoff model, is the further link between PMP and the resulting flood, named during the 1930's as the probable maximum flood (PMF). In this case "probable" was meant as the possibility of rare occurrence of flood events which exceed PMF, however no probability could be attached to PMF. The fact that a storm of rainfall can fall on the ground already covered by snow, with the rain melting it, leads to the conclusion that PMF can not be conceived as an upper bound even if PMP could be accepted as having that bound.

These two concepts, frequency-based determination of floods, and the PMF-based design of flood control measures, have been used in USA in parallel for already half a century. Several controversies still exist. Fitting frequency curves by probability distribution functions, and reliability of their extrapolations to determine probabilities of rare floods, floods with the probabilities of exceedence of  $10^{-3}$  to  $10^{-5}$  (or even  $10^{-5}$  to  $10^{-7}$ ), are still unsettled and controversial topics. This is the case even uncertainty is taken into account by using the confidence limits around the estimated critical or design flood values.

The PMF concept did not go well with some practitioners, because of the following: (1) it can not be used in economic optimizations by searching for a design flood less than PMF; (2) if a coefficient  $a < 1.00$  is used as a ratio of the design flood to PMF, this design flood falls in the range of probabilities which should be subject to economic optimization, while there is no method of assigning the probability to (a.PMF); and (3) when PMF is used for design purposes the question nearly always arises whether it is economically affordable.

The present state of problems in coping with floods within the general problems of water resources development, conservation, control and protection seems to require new solutions, specifically:

- 1) to find a bridge between the two opposite concepts of extreme large floods by resolving the basic controversy of whether the upper bound exist or not for storm precipitation for a given river basin;
- 2) to determine whether the antecedent moisture conditions, including snow cover and melt during the occurrence of PMP, do or do not have an upper bound, with the resulting conclusion whether PMF has or has not an upper bound;
- 3) to find the feasibility whether PMF can be located on the extrapolated tail of probability distribution functions fitted to flood frequency curves by any method available, in case the conclusion is made that floods do not have an upper bound, with this being equivalent of finding  $P(Q_{\max} > \text{PMF})$  as the exceedence probability of PMF;
- 4) to find out how to determine the fraction “a” for the design flood (a.PMF) by a rational method; and
- 5) to sharpen the techniques of the extrapolating the fitted flood probability distribution functions much beyond the return periods equal to observational lengths, say for the range of probabilities of  $10^{-2}$  to  $10^{-7}$  (Yevjevich & Harmancioglu, 1987).

### 2.3.2 Methods of Estimation

The large number of methods for estimating flood characteristics, with their many variations, can be classified in several ways. Here, six groups of methods are selected to cover the techniques that exist for determining flood characteristics. Basically, and according to the source of data and to mathematical or conceptual approaches, these groups are:

- 1) frequency-curve methods, which use the directly observed flood data, by fitting the probability distribution functions to frequency curves, and by extrapolating these functions to ranges of large floods;

- 2) transfer of information to floods from other variables, by such methods as the rainfall/runoff models, with frequency analysis of the resulting floods by the established mathematical or physical relationships;
- 3) regional data methods by estimating parameters of frequency curves or the curves themselves at given points by using the regional data on floods or flood-related variables;
- 4) paleohydrology methods by using either the old chronicles on floods (historical markers or records), and/or the proxy chronological data either resulting from floods or related to past flood events;
- 5) Bayesian methods by using the Bayes theorem, in combining the information on floods from several sources; and
- 6) Methods which lead to PMF through the estimation of PMP.

A large book manual (or manuals, such as the five volumes of United Kingdom Flood Study Report, 1975) are needed for a full description of all the methods which have been either used or proposed for the estimation of flood characteristics. Some general remarks on the above groups of methods are in order if one has to project into the future the need for research that will give the largest payoffs (Yevjevich et al., 1987).

### **2.3.3 Brief Description of Methods**

Frequency curve methods depend on the accuracy of flood data and the length of record. Several problems are often encountered with these methods: (a) the reluctance to extrapolate the fitted probability distribution functions much beyond the flood of the probability  $1/N$ , with  $N$  = the number of annual flood peaks; (b) largest floods with probabilities smaller than  $1/N$  often occur in samples of  $N$  years, thus creating the problem of upper outliers, and largest floods with probabilities greater than  $1/N$  also occur in sample of size  $N$ , thus creating the problem of inliers (lower outliers), with both cases pulling away the fitted probability distribution functions either much above or much below the expected population curves (overestimation or underestimation of floods for given probabilities; (c) by using

only the largest annual flood values (annual flood series) instead of all the floods above a threshold which are greater in number than  $N$  (partial flood series), some additional valuable information often is discarded; and (d) the fitted probability function is selected only by the goodness-of fit criterion or by the convention for the uniform use of the same function (such as the Log-Pearson function), instead of searching what must be or should be the true distribution function starting with the time series characteristics of instantaneous or daily runoff. A broad scope for research is further needed for these frequency methods, and also beyond the above four types of problems. In general, these methods are considered among the most reliable for floods of probability greater than  $10^{-2}$ , or for return period of less than 100 years, if sufficient data are available (Hydrology Subcommittee, 1982; Boughton & Shirley, 1983; Kovacs, 1984; Lane, 1985).

Transfer methods for flood estimation are basically related to techniques of computing floods from precipitation (such as by the rainfall/runoff models) for given river basin moisture conditions, thus representing a joint distribution of rainfall and soil moisture measuring index. By searching for information in variables outside the runoff data, new sources of uncertainty are added, such as the sparse information on regional storm distributions, inaccuracy in computing various abstractions, deviations due to separation of hydrographs into surface and subsurface components, errors in models, errors in estimation of parameters, etc. The problem arises when one tries to measure how much information is contained in the resulting flood series, or how large is uncertainty in individual flood values. These methods are basically beneficial in areas with significant information on precipitation and limited information on direct runoff. They can also be used to fill in the missing records on floods or to extend the flood series based on runoff data to periods for which rainfall but no runoff is observed. Definitely, significant further contributions to estimation of flood characteristics can be made by improving the transfer techniques of hydrologic information (Eagleson, 1972; Linsley & Kraeger, 1985).

Regional flood estimation methods combine all or most of the information available in a region in order to obtain flood characteristics either at a point or at all

the points of interests in that region. This approach has led to methods which estimate the parameters of regional flood frequency distributions so that they could be plotted or their equations established. Often, only the characteristic floods (say the 20-year, 50-year, 100-year, and similar floods) are estimated by regional equations. The basic objection to regional methods may be the fact that even if storms are similar over a large region, the response hydrographs may be result of different unit hydrographs or they change from one basin to another. Some flood parameters may be quite different among these basins from those generalized by the regional information. A sound concept is that a region should contain much more information on floods than a point within it. However, the technology of generalization and transfer of that information are far from being reliable or satisfactory at present. The condensation of regional information and its transfer need some basic improvements. Future research is likely to produce many advancements. In general, these methods may be ranked by their accuracy immediately following the good frequency analysis methods (Linsley et al., 1985; Rowe & Long & Royce, 1975; Hydrology Committee, 1981; Stedinger, 1983; Buishand, 1984).

Paleohydrology methods, whether they use the chronicle floods or paleoflood proxy data, must be considered as containing much more uncertainty than the three previous groups because of large errors in individual values, in the first case, and various inaccuracies in relationships of past floods and proxy data, in the second case. Regardless of limited precision, these methods provide an insight into the past events, though they might lead to premature conclusions of “climatic changes”, or that “floods have changed “significantly, due to significant inconsistencies (systematic errors) in basic data (Helley & LaMarche, 1973; Baker, 1982; Kochel & Baker & Patton, 1982).

Bayesian methods are a recent approach to flood hydrology, basically intended to combine information on floods from various sources. If the apriori probabilities could be reliably estimated by data-based frequencies, the aposteriori probabilities computed by Bayesian theorem would also be reliable. However, if subjective probabilities are used (therefore justifying the name of the “theorem of hypotheses”),

the aposteriori probabilities can only tell how sensitive they are to variations in the apriori probabilities. The problems of objectivity or subjectivity in the estimation of the aposteriori probabilities have a very long history of controversies in probability theory and mathematical statistics. This history should not be neglected when applying the Bayesian methods to flood hydrology. May approaches are used in the application of these methods (Gentillon, 1981; Kuczera, 1982; Russel, 1982; Stedinger, 1983).

Methods of computation of PMP as well as synthesizing the storm hydrographs for the estimation of PMF need a review from time to time, as the new data on storms become available in each region. The objectivity or subjectivity in maximization of meteorological and oceanic variables leading to PMP need a constant vigil, since the consequences of selecting these values have a significant impact on PMF and its affordability in design (Myers, 1969; Hansen & Miller & Schreiner, 1982; Newton, 1983).

### **2.3.4 Three Ranges of Flood Probabilities**

Three ranges of flood probabilities can be singled out from the point of view of accuracy in their determination:

(1) floods of probabilities  $2 \cdot 10^{-1}$ - $10^{-2}$  (5-year to 100-year return period floods), or those used for design of many drainage or flood evacuation structures, which can be relatively reliably estimated by the first three groups of methods (frequency, transfer and regional methods);

(2) floods with probabilities  $10^{-2}$  to  $10^{-4}$  (100-year to 10,000-year return period flood), those that are usually estimated with the least accuracy by the first five groups of methods, and with the large uncertainty-based confidence intervals; this range of probabilities of floods require most attention in future research; and

(3) floods with probabilities equal or greater than the 10,000-year floods (including PMF and the very rare floods, with probabilities of exceedence of  $10^{-4}$ - $10^{-7}$ ), which are crucial for survival of important and critical structures and

systems; they represent the challenge to profession of nearly unprecedented scientific difficulties.

This division is made according to approaches needed to solve the problems of future increases in accuracies of flood estimates. For the lower range of floods, the multitude of techniques are already available, so that the problem may be improvement and selection of the best existing method rather than the search for new methods. The main problem is the estimation of confidence intervals to computed floods once the method is selected and/or applied. For the middle range, with the design floods of these magnitudes both critical and affordable, especially for its lower range of  $10^{-2}$ - $10^{-3}$  and important flood mitigation cases, both the improvement of existing methods and the design of new methods of estimation are needed. The probability of PMF would likely fall at the upper part of this range, say at or above  $10^{-4}$ ; however, this is still a hypothesis to be proven. Therefore, a location of PMF on the assumed unbounded tail of flood probability distribution functions might help the estimation of floods for both the middle and the upper ranges of probabilities (Yevjevich et al., 1987).

### **2.3.5 Determination of Probability of Exceedence of PMF and the Other Large Floods**

To estimate the probability of exceedence of the probable maximum flood (PMF), as computed by using the probable maximum precipitation (PMP), it seems that future research should concentrate, if feasible also by parallel efforts, on the three basic lines of investigation: (1) by searching for the shape (mathematical form) and location of the upper tail of probability distribution functions most likely to represent the population flood distributions, and by extrapolating this tail to low probability values to encompass the estimated PMF value; (2) by using both the envelopes and the average curves of the largest observed floods in regions of relatively similar geophysical conditions, for a step-wise increase of periods of observation, by extrapolating these curves to curves of still smaller probabilities of exceedence, with the location of PMF value among these envelope or average curves; and (3) random

variables that lead in their maximization to PMP and then to PMF, and by a composite aggregation of these variables in order to determine frequencies (as estimates of probabilities) of both PMP and PMF.

By finding the exceedence probabilities of PMF values, the other crucial problem, the extrapolation flood frequency curves to probabilities in the range of  $10^{-2}$  to  $10^{-7}$  may be partially or fully resolved. Therefore, the problem is reduced to finding the most reliable upper tail of extreme floods, both in shape and in location (Yevjevich et al., 1987).

### **2.3.6 Properties of Tails and Their Extrapolation**

The results of studies in tails of flood frequency curves are controversial, namely whether the tails are light (exponential), normal (Gaussian) or very heavy (those of the stable probability distribution functions). Some studies (Tao & Yevjevich & Kottegoda, 1976) of daily flow series show tails of independent stochastic components to be nearly indistinguishable from the simple exponential tails. Then, sampling of extremes from such tails should also produce the exponential tails. Some other studies (Shen & Bryson & Ochoa, 1980; Ochoa & Bryson & Shen, 1980) have found the other flood frequency tails to be heavy. Therefore, the problem must be resolved by finding what are the types of tails of most flood probability distributions by investigating their characteristics.

A hypothesis is advanced here, namely that tails of probability distributions of basic runoff processes (say of instantaneous discharge, often well approximated by daily runoff series for middle-sized and large rivers) can tell us as much if not more about the tails of distributions of flood peaks than the limited amount of data on annual flood peaks can tell about the tails of their distributions. Simply, much more data are available to assess accurately the character of tails of daily or instantaneous flows from which flood peaks are sampled than for flood distribution tails themselves. Distribution parameters of daily instantaneous flow change periodically over the year, even if one assumes that the probability function does not change with

seasons. This latter hypothesis may or may not be valid . However, it may likely be valid for humid seasons in which floods usually occur than for the entire year composed of both the high flow and the low flow periods.

Equations of tails should have at least one parameter less (if not two parameters less) than the flood probability distribution function. Tails should be determined by its equation shape with parameters and its location (say by the initial tail coordinate values), such as  $Q_1$  and  $F(Q_1)$ , but with no more than two parameters. An example may be

$$P(Q_{max} > Q) = \int_Q^{\infty} e^{-\alpha Q} dQ$$

For  $Q_{max}=Q_1$ ,  $Q_1$  and  $F(Q_1)$  determine the tail location. If  $\alpha$ =shape and  $\beta$ =scale parameters are those of the gamma distributions,  $\beta=1$  may be often assumed for simple tails. In that case  $Q_i$  and  $F(Q_i)$  uniquely determine the location of the tail. The larger  $Q_i$ , the closer the tail should be to an exponential tail. For small  $Q_i$ ,  $\beta \neq 1$  may be needed to take care of a more complex tail, with  $\alpha$  and  $\beta$  estimates related (as both change with a change of  $Q_i$ ). The location point  $Q_i$  should be selected so as to be to the right of the inflection point, and  $\alpha$  (and eventually  $\beta$ ) to be determined by the full use of any information available on tails, both of the underlying process and of flood frequency curve.

A concept is often advanced among flood specialist, namely that the superpopulation of floods is composed of different types of populations: those of frontal rainfall, convective rainfall, hurricane-related rainfall, rainfall/snowmelt combination, and similarly generated floods, often with various degrees of combination of these distinct general causes of extreme floods (Canfield & Olsen & Hawkins & Chen, 1980). The question arises whether all these flood populations have the same type of tails, so that their combination in one "heterogeneous sample of floods" still preserves the same type of tail, but only with different parameters and location than is the case for a homogeneous population. Therefore, it will be

necessary to study the tails in detail in order to investigate the problem posed by the concept of superpopulation of floods. This can be done both by assuming it to be valid and by contesting it on the basis of a myriad of flood producing causes, when each type of flood producing inputs is only one of those causal factors. In the cause-effect relationships with a very large number of causes, some causes may be absent or may have zero effects in some floods but not in others, and still the concept of a homogeneous population remains valid (Yevjevich et al., 1987, pp.1-22).

## **2.4 The Use Mathematical Models in Analysis of Floods**

### **2.4.1 Watershed Modeling**

In recent years, a risk-based management approach to water resources problems is gaining increased popularity. This approach relies upon the identification and targeting of worst case problems as those needing initial emphasis. One of the currently available engineering tools that is believed to satisfied the greatest needs and provide the best foundation for future problem solving is the development and application of watershed models to understand and predict the relationship of watershed conditions and management activities (Donigian & Imhoff & Ambrose, 1995).

The ultimate value of models is twofold: first, to understand and reproduce how the modeled system works, and then to predict how the system will respond to future alternative conditions and actions. This is also the basis for modeling of flood events. For decades, the need has existed to address certain water quantity problems such as flood hazard and control and the water supply on a watershed scale. Hydrologic and hydraulic models developed for watershed-scale analysis have been popular in many parts of the world since the early 1970's. One of the first products of these efforts are the "HEC" series of models created and maintained by the U.S. Army Corps of Engineers Hydrologic Engineering Center in Davis, CA, USA (Donigian et al., 1995). These series include HEC-1, HEC-2, HEC-3, HEC-4, HEC-5, HEC-5Q, HEC-6.

HEC-1 is developed for the calculation of a runoff hydrograph from a watershed with specified precipitation pattern and precipitation rates for a single storm event. In addition, hydrographs at any number of intermediate points in the watershed may be produced.

The most popular program of this series is the HEC-2, which calculates the surface profile for a steady flow discharge. Pertinent hydraulic parameters are determined in the process: depth of flow, water surface width elevation of total energy line, friction slope, flow velocity, critical depth and volume of water beneath the computed profile. A secondary purpose of HEC-2 is to calculate the permissible encroachment width for a specified rise in the water surface profiles.

The purpose of HEC-3 is to perform a routing of flows through a system of multipurpose reservoirs. Inflows, water demands and reservoir operating rules are prescribed. System performance is measured in terms of storage requirements and in terms of dollars, with shortages declared during any period that a demand is not met.

HEC-4 is used to generate a sequence of monthly flow volumes of any desired length. Statistical characteristics used in the generation are calculated from observed monthly stream flows or from regionalized statistics. Missing records are filled in based on concurrent flows at other stations. Maximum and minimum monthly flows are flagged in the observed, reconstituted and generated flow records.

The purpose of the HEC-5 program is to determine the release from each reservoir project in a system of reservoirs; both conservation and flood control purposes may be included. The system is operated automatically according to prescribed criteria. Inflow may be deterministic, as in planning studies, or forecasted, with an assigned contingency factor for error.

HEC-5Q is developed to simulate the operation of river-reservoir systems in basin wide studies. The stream component is a one-dimensional unsteady model driven by a selection of available hydrologic routing models. The reservoir component is a

one-dimensional vertical model. The model also simulates DO, BOD, a selection of conservative and non-conservative constituents, and contains a phytoplankton option (Donigian et al., 1995).

HEC-6 calculates water surface profiles and stream bed profiles. The water velocity, water depth, energy slope, sediment load, gradation of the bed surface are also computed. Because a large number of water surface profiles are calculated, a completely different algorithm to HEC-2 was developed.

HEC series of models are deterministic hydrologic models with particular features. In the following section, a brief discussion on deterministic models is presented to better describe the characteristics of the HEC model used in the study.

#### **2.4.2 Deterministic Hydrologic Models**

In the mathematical sense, the word *model* describes a system of assumptions, equations, and procedures intended to describe the performance of a prototype system. Most of mathematical models are relatively simple; but the definition of the hydrologic system shows that it is complex and not easily described by a simple model. Since the advent of the digital computer, the term *hydrologic model* has come to mean a relatively complex mathematical description of the hydrologic cycle designed for solution on a computer. Some models attempt to describe the actual physical process of the hydrologic cycle so as to simulate actual hydrologic events such as the transformation of a series of rainfall inputs to the resulting streamflow hydrograph. These models have been described as *deterministic*, *physically based*, and *conceptual*. Another class of hydrologic models seeks to reproduce the statistical behavior of a hydrologic time series without regard to an actual event.

Models, both deterministic and stochastic, have contributed much to hydrology. They have given us a better understanding of the hydrologic processes, are useful research tools, and can be used to develop solutions to engineering problems with a

detail and accuracy which could not be achieved by more conventional paper and pencil analysis (Linsley & Kohler & Paulhus, 1988, p.325).

### 2.4.3 Types of Deterministic Models

If a model precisely imitates the processes of the hydrologic system, it might be supposed that only one such model would be needed. For many reasons, such precise imitation of the processes of hydrology has not been attained, and many models have been developed for different purposes and with different philosophical concepts.

‘Models intended primarily for research are usually designed to simulate part of the hydrologic cycle at a point’ (Freeze, 1978). These *research models* utilize formal mathematical expressions for the algorithms, which are usually differential equations. Such models are useful for exploring the sensitivity of the hydrologic process to specific site characteristics or to watch the variation of the process over short time periods (Linsley et al., 1988, p.326).

*Conceptual models* involve a series of functions which are considered to describe the catchment process involved. The algorithms are usually simplified by use of empirical relations in order to speed the solution and to adapt the model to cope with the point-to-point variations in the hydrologic process within the catchment (Crawford & Linsley, 1966).

*Lumped models* treat a whole catchment, or a significant portion of it, as if it were homogenous in character and subject to uniform rainfall. *Distributed models* divide the catchment into a large number of small subareas, simulate each separately, and combine them to obtain catchment response. In principle, the distributed model deals with the variations over a catchment more logically than the lumped model. However, unless the input rainfall and the catchment characteristics are known with comparable detail, the solution may be no better than that of a lumped model. Computations in detail of the distributed model are usually too time-consuming for engineering applications. However, the distributed model is useful for research on

the consequences of heterogeneity in catchment (Linsley et al., 1988). Essentially, the HEC-1 model used in this study is a deterministic, conceptual/empirical and lumped model for rainfall-runoff simulation.

*Continuous models* are capable of generating outflow hydrographs over long periods of time (several years). Of course, such models use finite time periods in the computation and the computed flows are discrete points in time. *Event models* are designed to simulate a single event such as the hydrograph of a single storm. A continuous model must include algorithms which maintain a continuous water balance for the catchment so that the conditions antecedent to each storm event are known. In the event model, initial conditions must be specified as part of the input. Most event models were developed specifically to design urban drainage systems and other small projects (Chen & Shubinski, 1971, pp. 289-301). The probability of a computed peak flow is sometimes assumed equal to that of the input rainfall. Continuous models are useful for simulation of long flow records for use in design, evaluating the impact of changes in catchment on streamflow, and forecasting of streamflow (Linsley et al, 1988, p.326). Examples of event simulation models include: the U. S. Army Corps of Engineers (1981) HEC-1 flood hydrograph model; the Soil Conservation Service (1965) TR-20 computer program for project hydrology; the U. S. Environmental Protection Agency (1977) SWMM storm water management model; and the Illinois State Water Survey ILLUDAS model, by Terstriep and Stall (1974). Examples of continuous simulation models include: the U. S. National Weather Service runoff forecast system (Day, 1985); the U. S. Army Corps of Engineers (1976) STORM model; and the U. S. Army Corps of Engineers (1972) SSARR streamflow synthesis and reservoir regulation model. This is by no means a complete list of available models, but it covers most of the models commonly used in hydrologic practice (Chow & Maidment & Mays, 1988, p.507).

*General models* are designed to be used on any catchment, with catchment characteristics represented by input parameters. *Catchment specific models* are prepared for a single catchment and have appropriate catchment characteristics built in. Specific models are often constrained to the range of conditions actually

represented in the database, on which they are built, while general models are adapted to a much wider range of conditions.

Models are rarely static. They undergo frequent modification by their developer or by a subsequent user. Consequently, it is difficult to fit a specific model into a class (Linsley et al, 1988, p. 327).

#### **2.4.4 Flood Characteristics Based on Single Event Models**

In single event models the estimation of single flood events are considered, i.e. the transformation of a single rainfall event into a single flood hydrograph. Event-based models generally involve a simple approximation of the rainfall-runoff transformation process in a watershed. Some of the underlying hydrologic transport processes are neglected or simplified. However, these models are quite useful in engineering practice since they are simple and provide good estimates of flood hydrographs from given storm events.

In modeling single flood events, the effects of evaporation in the watershed are ignored, as well as the interaction between the aquifer and the streams. Evapotranspiration may be ignored because its magnitude during the time period in which the flood event develops is negligible when compared to other abstractions like infiltration. Likewise, the effect of the stream-aquifer interaction is generally ignored because the response time of the subsurface soil system is much longer than that of the surface or direct runoff process. Event-based modeling generally involves the following aspects:

(a) evaluation of the rainfall flux over the watershed  $I(u, t)$  as a function of space and time;

(b) evaluation of surface runoff production, i.e. rainfall excess,  $I_e(u, t)$  which is the rainfall available for runoff after infiltration and other abstractions have been accounted for;

(c) routing of the rainfall excess to the watershed outlet in order to determine the corresponding flood hydrograph,  $Q(t)$  (Ramirez & Salas & Rosso, 1994, p.81).

## **2.5 Application of HEC Models in Analysis of Floods**

Several examples can be cited on the application of HEC-1 in flood studies. Some of them are summarized given below ( these cases, except the last three, are found on internet so that exact references could not be obtained ):

◆ One case study focused on using the HEC-1 hydrologic model to simulate the flood through the Highwood River, one of the major rivers in Southern Alberta. The use of computer assisted hydrological modeling in simulating storm events has developed into a very useful tool since its inception in 1959. In using empirical and mathematical relationships to simplify hydrologic principles, the hydrologist is able to simulate natural processes that occur with results that are useable in the prediction of future events. These relationships are represented by hydrologic components of the precipitation-runoff process and are then combined quantitatively to present the estimated outflow of the basin. In accomplishing the computed outflows for the Highwood River, the HEC-1 modeling program was used. The Horton model was used to define infiltration rates and capacities, whereas the Clark method was used for unit hydrograph representation.

◆ Another study was realized in North Carolina. The HEC-1 and HEC-2 model parameter development, model setup, model calibration and verification, pollutant loading calculations, lake water quality spreadsheet model set-up and calibration, and storm water master plan was made for the Big Branch Watershed in Raleigh, North Carolina. The project includes definition of sub-basins, runoff curve numbers, time of concentration, channel cross sections, and floodplain boundary coverages for the GIS database. In addition, the project develops conceptual alternative solutions and cost estimates for flooding and water quality problems identified in the modeling process. Public involvement is an integral part of this project.

◆ HEC-1 and HEC-2 were also used in Lake Orinda Dam, and spillway investigations in California. The adequacy of the dam spillway was evaluated in view of the regulations of the California Division for Safety of Dams (DSOD). A probable Maximum Flood (PMF), a 72-hour, 20,000-year event, was used. Numerical modeling of spillway routings under existing conditions and with various alternative spillway modifications were constituted by using HEC-1 and HEC-2.

◆ HEC-1 was used in the meteorologic/hydraulic investigation of the Sacramento River Delta island/reservoir in San Joaquin County, California. The purpose was the conceptual design of the spillway and appurtenances to convert Bouldin Island into a fresh water supply reservoir. Stormwater runoff to the reservoir was estimated using computer program HEC-1. The HEC-1 program was set up to simulate a 4,500-year, 96-hour storm using the Station Angle Method of determining precipitation depths from the records of a surrounding rain gage network.

◆ In another project, HEC-1 was used for water supply feasibility study in the Geysers, California. It was employed for the feasibility and preliminary screening studies for the dam and the reservoir and instream extraction facility water projects on Big Sulphur Creek in Sonoma County, California. Analysis involved hydrologic simulation of the watershed and simulation of operations of alternative projects was realized by using HEC-1. Project construction is now complete.

◆ Flood control measures were evaluated along Sulphur Springs Creek, Benica, California. Flood hydrographs with various frequencies up to 100-year flood for the study area were synthesized by using the rainfall-runoff simulation model HEC-1. Various operating plans for upstream reservoir (Lake Herman) were simulated.

◆ In Vallejo, California, a complete review and update of watershed meteorology, hydrology and stream and storm drain system hydraulics was conducted in order to remap the 100-and 500-year floodplains on three urban watersheds. Derivation and routing of the base floods were done with computer model HEC-1, and plotting and mapping of floodplain backwaters was done with program HEC-2.

◆ In Sunshine Landfill, Los Angeles, California, the conceptual design of the sediment basin, its inlet and outlet pipes and energy dissipation structures, and the sizing of all v-ditches and piping to convey runoff from the fill slopes to the sediment basin were developed. All drainage improvements were designed to handle the 100-year design discharge, utilizing HEC-1, to estimate storm water runoff hydrographs, and HEC-2, to determine culvert sizing and inlet configurations.

◆ Hydraulic analysis of flood control alternatives for the Northgate Project, Vallejo, California, was made by using HEC-1 and HEC-2 programs. Alternative plans included the design of various reaches of channel along Rindler and Blue Rock Springs Creek and the diversion of flood flows to underground conduit systems, through the Solano County Fairgrounds, to the confluence with Lake Chabot. The system was modeled using computer programs HEC-1 and HEC-2.

◆ Cline (1989) have examined the potential of a using commercially available computer-aided drafting package AutoCAD as a micro-computer tool for a three phase Water Information System (W.I.S.). The system is developed to obtain, organize and display watershed data; calculate some model parameter values; and create model input files. The watershed model HEC-1 has been selected for the rainfall-runoff simulation and example application to Macks Creek Watershed in Idaho, USA, is presented.

◆ Bhaskar (1988) investigated trends in the Clark Instantaneous Unit Hydrograph parameters to predict flood conditions in a future year when the Beargrass Creeek watershed was expected to undergo complete urbanization. For this purpose, the methodology for trend analysis was used in conjunction with the HEC-1 model to simulate rainfall-runoff processes.

◆ Bravo (1994) determined the Muskingum-Cunge Flood Routing Method parameters for streams where measured hydrographs were nor available. The study was realized with the Muskingum Routing option of the HEC-1 program.

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## CHAPTER THREE

### HEC-1 MODEL

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#### 3.1 History of HEC-1

The Hydrologic Engineering Center, HEC, developed several small computer programs for simulating various parts of the precipitation-runoff and river-routing processes in the early 1960's. The small, individual programs were necessary because of the limited computer memory at that time. When computer memory became large enough in the mid to late 1960's, the several small programs were combined into a single package for river basin hydrology. The first version of the HEC-1 Flood Hydrograph Package was published in October 1968. It has been expanded and revised and published many times since then to incorporate improvements in hydrologic algorithms and advances in computer hardware and software. To simplify input requirements and to make the program output more meaningful and readable, a major revision was made in 1973 (Feldman, 1995, p.119). In 1981, major revisions were made to the 1973 version of the program. The program input and output formats were completely revised, and the computational capabilities of the special versions of HEC-1dam-break (HEC-1DB), project optimization (HEC-1GS) and kinematic wave (HEC-1KW), were combined in one program. The new program included the powerful analysis features available in all the previous programs, together with some additional capabilities and merged them into a single easy to use package (Hydrologic Engineering Center, 1990). In late 1984, a microcomputer version (PC version) was developed. A menu capability was added to facilitate user interaction with the model; an interactive input developer, a data editor, and output display features were also added.

The current standard version, Version 4.0 (September 1990), represents improvements and expansions to the hydrologic simulation capabilities together with interfaces to the HEC Data Storage System (HEC-DSS). The DSS connection allows HEC-1 to interact with the input and output of many HEC and other models. New hydrologic capabilities in HEC-1 include Green and Ampt infiltration, Muskingum-Cunge flood routing, reservoir releases input, and improved numerical solution of kinematic wave equations. The Muskingum-Cunge routing may also be used for collectors and main channels in a kinematic wave land-surface runoff calculation. This new release also automatically performs numerical analysis stability checks for the kinematic wave and Muskingum-Cunge routings. The numerical stability check was added because many users did not check the validity of the time and steps used in the model.

In September 1991, version 4.0.1E of HEC-1 was released for use on extended memory PC's. A hydrograph-array size of 2,000 ordinates is available in this version. The increased array size reduces limitations encountered when simulating long storms using short time intervals. For example, simulation of the fixed-length (96-hour) standard project storm at 15 minute intervals requires 384 ordinates just for the storm; more time intervals would be required to simulate the full runoff hydrograph and route it through a river basin. The large-array version also allows greater flexibility in checking for numerical stability of simulation process (e.g., kinematic runoff and routing computations). The large-array version uses an extended-memory operating system (386 or newer machines) with three megabytes of total memory.

There are many different hydrologic simulation options in HEC-1. This evolves from the autonomy of the Corps field offices and different methods being preferred. Different geographic regions and/or the preferences of the hydrologic engineers have brought about the many methods. No one method for hydrologic analysis is prescribed by Corps headquarters; the only requirement is that the validity of the selected method must be demonstrated.

A next generation of HEC hydrologic modeling system is now under development. The new system will have broader capabilities than HEC-1 but will still primarily be a flood hydrology analysis model (Feldman, 1995, pp.119-120).

### **3.2 Purpose of HEC-1**

The main purpose of HEC-1 in its initial development was simulation of hydrologic processes. This capability would enable the Corps to analyze historical floods and design projects for flood damage reduction. HEC-1 was limited to flood event studies because flood protection was the Corps's major responsibility. Those flood events were typically major historical storms or hypothetical design storms. The simulation would begin with precipitation, then compute infiltration losses, and finally route the runoff throughout a river basin. HEC-1 soon expanded from just a hydrologic simulation tool; it increased its scope to include special hydrologic analyses that the hydrologic engineer typically performs using the hydrologic data derived from HEC-1. Hydrologic analysis tools include automated parameter estimation, multiple-storm simulation, precipitation depth-area relationship coordination, and flood frequency curve determination. The flood damage reduction project analysis capabilities include multiple-projects analysis, modified frequency curves, flood damage computation, and project-size optimization (Feldman, 1995, pp.119-120).

### **3.3 The HEC Modeling Philosophy**

HEC has developed and supported a full range of simulation models for understanding how water resources systems function. It is assumed that experienced professionals can deduce the appropriate solution to a problem given the insight provided by selective execution of the simulation models. This deduction process has historically been the dominant methodology for planning and operational decisions in the water resources community, and it still continues. HEC has used hydrologic simulation methods in a variety of situations with notable success. Some optimization models are now being used to evaluate reservoir system operation.

### **3.4 Basic Concepts (River Basin Representation)**

#### **3.4.1 Sizes and Types of Basins**

HEC-1 is not limited to any watershed or river basin size except that it can not simulate at time intervals less than one minute. It has been used for small urban areas ( $1 \text{ km}^2$  or less), and large river basins ( $100,000 \text{ km}^2$ ). The river basins may be of any complexity of subbasins, river reaches, reservoirs, etc., but they must be in a converging dendritic form. HEC-1 is not a network model which will allow water to be reintroduced upstream.

Within the smallest unit of watershed area, termed a subbasin, it is assumed that hydrologic processes occur uniformly. This is the so-called lumped watershed model. That is, precipitation and infiltration are assumed to be uniform over the entire subbasin. There is a minor modification of this concept when using kinematic wave runoff procedures.

The lumped-parameter assumption in HEC-1 is the same as that found in almost all hydrologic models. The assumption is not particularly limiting because the subbasins and reaches can be made small enough to accommodate any practical spatial variations in the watershed characteristics.

#### **3.4.2 Model Structure**

The river network is defined by HEC-1 model building blocks which include subbasins, river reaches, confluences, reservoirs, diversions, and pumps. Each building block describes the hydrologic process necessary to compute the hydrograph at the downstream outlet of the subbasin, river reach, reservoir, or confluence. Diversions and pumps allow water to be withdrawn from a building block, and this information saved on disk. Those diverted flows are labeled and may be retrieved at any subsequent building block; on retrieval, they are treated like a subbasin runoff and may be routed and combined with other flows.

### 3.4.3 Limitations of HEC-1

HEC-1 hydrologic simulation must be made at a constant computational time interval. This is a limitation only when some components of the model, i.e. building blocks, require more detail than others. For example, in a large river basin, it may be desired to simulate runoff in small urban areas in some detail. That detail would require the computational time interval to be much smaller than that used in other parts of the basin. Two solutions are possible: choose an average time interval that suffices for both the large and the small components; or simulate the small areas separately. If the small areas are simulated separately at a short time interval, they may be interfaced with the larger river basin model through the HEC Data Storage System, DSS. The time-series input data to HEC-1 do not have to be the same as the computation time interval; HEC-1 interpolates the input time series to the prescribed computational time interval (Feldman, 1995, pp.120-125).

There is no recovery of precipitation loss rates (infiltration, interception, detention storage, etc.) during periods of no precipitation. That is, the soils, etc., do not dry out and recover their loss potential. Thus, HEC-1 is limited to flood event studies where the loss rates do not undergo recovery. The storm events may be very complicated in terms of precipitation patterns. If there are significant dry periods, then the storm should be separated into one or more flood events and the loss rates determined separately for each event. This is usually not a significant limitation in flood studies where the losses are usually small with respect to the flood runoff volumes (Feldman, 1995, pp.125-126).

The river basin must be dendritic, i.e. a converging tree network. If flow reversals occur, then HEC-1 would need to be interfaced with an unsteady flow model; this may be accomplished through the data storage system.

The hydrologic processes are considered to be average (lumped) over each time increment and spatial component of the model. This is a limitation common to almost

all models. It is solved by making the simulation building blocks small enough to capture the desired variability.

The reservoirs in HEC-1 cannot be operated for downstream flow conditions. HEC-1 computes all hydrograph points in time for the current point in space, e.g. the reservoir. It does not know what is happening downstream. Thus, reservoir outflows are only a function of reservoir storage or stage. This limitation can also be overcome by use of the data storage system. The HEC-1 results are saved in DSS and then retrieved by HEC-5, which operates reservoirs to meet a variety of downstream requirements.

#### **3.4.4 Hydrologic Simulation Components**

HEC-1 views a river basin as a series of hydrologic building blocks representing various parts of the precipitation to runoff process. Figure 3.1 shows a simple river basin and several of those building blocks. The hydrologic process building blocks are: watershed runoff, river routing, reservoir routing, confluences, and diversions (Feldman, 1995, pp.121-126).

### **3.5 Model Development**

#### **3.5.1 Identification of the Stream Network**

A river basin is subdivided into an interconnected system of stream network components (e.g., Figure 3.1) using topographic maps and other geographic information. A basin schematic diagram (e.g., Figure 3.1) of these components is developed by the following steps:

1. The watershed boundary of the study area is delineated first. In a natural or open area, this can be done by using a topographic map. However, supplementary information, such as municipal drainage maps, may be necessary to obtain an accurate depiction of an urban basin's extent.

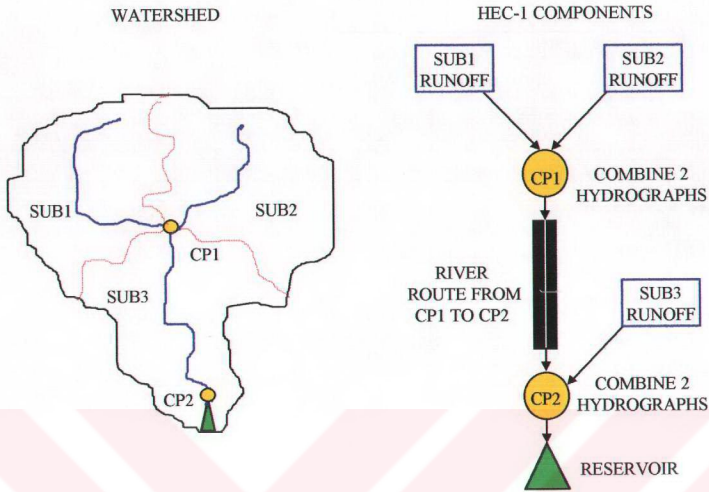


**Figure 3.1 HEC-1 Representation of Watershed Components**

2. Segmentation of the basin into a number of subbasins determines the number and types of stream network components to be used in the model. Two factors impact on the basin segmentation: the purpose of the study and the hydrometeorological variability throughout the basin. First, the purpose of the study defines the areas of interest in the basin, and hence, the points where subbasin boundaries should occur.

Second, the variability of the hydrometeorological processes and basin characteristics impacts on the number and locations of subbasins. Each subbasin is intended to represent an area of the watershed which, on the average, has the same hydraulic/hydrologic properties. Further, the assumption of uniform precipitation and infiltration over a subbasin becomes less accurate as the subbasin becomes larger. Consequently, if the subbasins are chosen appropriately, the average parameters used in the components will more accurately model the subbasins.

3. Each subbasin is to be represented by a combination of model components. Subbasin runoff, river routing, reservoir, diversion and pump components are available.

4. The subbasins and their components are linked together to represent the connectivity of the basin. HEC-1 has available a number of methods for combining or linking together outflow from different components. This step finalizes the basin schematic (HEC, 1990).

### **3.5.2 Subbasin Runoff**

#### **3.5.2.1 Subbasin Processes**

The subbasin (also known as a watershed or catchment) runoff block involves four processes: precipitation, losses (interception, infiltration, and detention storage), excess-to-runoff transformation, and baseflow. HEC-1 contains several options for simulating each of these processes. The subbasin is the smallest areal element in the watershed conceptualization for which the precipitation-to-runoff process can be simulated. As noted before, it can be of any size limited only by the minimum of a minute computation over the subbasin.

#### **3.5.2.2 Precipitation**

For each subbasin, the average precipitation in each time interval must be determined. The precipitation will fall as rain or snow depending upon the air temperature (only if snowmelt computation is indicated). HEC-1 has the provision to use gaged precipitation or hypothetical storms. If subbasin average precipitation is already known, it may be input directly. If gages are used, the subbasin average precipitation must be calculated for each simulation interval. The gages are weighted by Thiessen or other methods. The resulting time series of subbasin-average precipitation is then available for the loss calculations.

Hypothetical storms may also be used. These storms may be derived separately from HEC-1 and input, or they may be computed by HEC-1 by inputting storm statistics. Design storms such as the Standard Project or Probable Maximum can be calculated by HEC-1 but only for limited purposes. It is recommended that the design storm be computed by current criteria and then input to HEC-1 for runoff computations. An example of such a computation is the criteria of Hydrometeorological Report No.52 (National Weather Service, 1982). HEC has a separate computer program, HMR52, to perform those calculations and provide the resulting storm input to HEC-1 via the data storage system.

For frequency-based hypothetical storms, the storm statistics required are: storm area, storm frequency, and intensity-duration data. The storm area is required because area adjustments must be made for point rainfall statistics. The storm frequency is required if the storm is frequency-based so that partial-duration adjustments can be made to the frequent storms. HEC-1 then creates a balanced storm in which each intensity of a shorter duration is imbedded within the intensity of the next longer duration (Feldman, 1995, pp.126-128).

### **3.5.2.3 Snow**

Snowfall and snowmelt can be simulated within each subbasin. Either a simple degree-day or the more comprehensive energy-budget method may be used. The degree-day approach requires air temperatures at the outlet of the subbasin for each time interval. An adiabatic lapse rate is input to compute the average air temperature in up to 10 elevation zones. Within each elevation zone, it is determined whether the precipitation falls as rain or snow. If snow, it is accumulated in a snow pack. If rain, it is subject to losses and runoff during each time interval. If the air temperature is warm enough, then melt occur. The energy-budget method works in a similar manner except that the melt computation is also a function of wind speed, incoming solar radiation, and dew point. These data must also be provided for each time interval.

### 3.5.2.4 Losses

Losses refer to all rainfall or snowmelt that is not available for direct runoff due to interception, infiltration, and detention storage. These losses are not available for further use in the runoff simulation. Thus, the outflowing hydrograph from a subbasin is the result of only direct runoff and base flow. The loss rate methods available in HEC-1 are shown in Table 3.1.

**Table 3.1 HEC-1 Loss Rate Methods**

| <b>Method</b>      | <b>Approach</b>                                          |
|--------------------|----------------------------------------------------------|
| Initial & Constant | Satisfy an initial volume, then a constant rate          |
| HEC Exponential    | A logarithmic decay function based on accumulated losses |
| SCS Curve Number   | Initial abstraction and infiltration per SCS             |
| Holtan             | Infiltration based on accumulated losses                 |
| Green & Ampt       | Infiltration based on soil characteristics               |

The Initial & Constant method is commonly used for hypothetical storms or observed floods with poor definition of rainfall. The HEC Exponential method has logical parameters, but they are not readily related to watershed characteristics, e.g. land use and soil type. The SCS method is easy to use because the Curve Number can be easily obtained from land use and soil maps; unfortunately, that “book” value may not be appropriate for historical storms of differing antecedent moisture conditions. The SCS now recommends calibration of the curve number to observed rainfall and runoff. The Holtan method is based on watershed characteristics, but its development stopped when the Green & Ampt method became popular. The Green & Ampt parameters may be obtained from soil characteristics, and much work is currently being done to enhance its usage. As in any hydrologic model, it must be remembered that these loss characteristics are considered homogenous over the subbasin, while the real makeup of the subbasin is quite heterogeneous. Thus, whatever method is used, it

must be considered to be “hydrologically representative” for the whole subbasin (Feldman, 1995, pp.126-129).

The choice and validity of rainfall loss methods depends on the type of the problem, the data available, and the runoff process which are likely to be dominant (Pilgrim & Cordery, 1993, p.9.7).

#### **3.5.2.5 Excess-to-Runoff Transformation**

The unit hydrograph and kinematic wave methods are available in HEC-1. For the unit hydrograph, it may be input directly if the ordinates are known, or several analytical functions may be used. If the ordinates are input, they must be for a duration equal to the simulation time interval. The Clark, Snyder, or SCS analytical methods will compute a unit graph whose duration is equal to the simulation time interval. The SCS method is formulated as the single parameter LAG relationship using the SCS dimensionless unit shape. The Clark method uses the standard two parameters and a time-area curve. If the time-area curve is not known, HEC-1 supplies one with a typical elliptic shape. The Snyder method is simulated using the Clark method because Snyder, by itself, does not provide all ordinates of the unit graph.

Kinematic wave is a nonlinear runoff transformation function (HEC, 1993). HEC-1's formulation of the kinematic wave method takes two forms: the standard runoff plane approach and a HEC storm-drain-subsystem approach for urban areas. The subsystem approach is described in the following hydrologic analysis section. Instability of the kinematic wave computation with respect to time and distance steps has been a frequent problem in kinematic wave solutions. The HEC representation of the method provides for automatic stability checks of time and distance intervals until the computation remains stable. Continuity and other diagnostic information also is provided.

### 3.5.2.6 Base Flow

HEC-1 has a rather simple representation of base flow because its primary interest is in simulating flood hydrographs where base flow is a relatively small contributor. Also, there is little agreement in the profession as to how base flow can be determined. Thus, a simple method is provided, or the user can determine the base flow by some other technique and directly input it to the program. In the later case, HEC-1 base flow can be set to zero.

The base flow capability consists of a starting flow, a recession threshold, and a logarithmic decay recession rate. The base flow begins with the starting flow which is recessed at the decay rate. Then, the direct runoff is added to that recessed flow as the storm occurs. When the flood hydrograph falls to a recession threshold level, then the subsequent calculations are simply the recessed threshold value. If rainfall starts again, then rainfall excesses are again used. Usually the recession threshold value is small compared to the flood peak. The recession threshold value is chosen as the point on the falling limb of the hydrograph where the flows plot as a straight line on semi-log paper.

### 3.5.3 Channel Routing

Several methods for channel routing are available (as was the case with subbasin runoff methods). Channel infiltration may be computed in connection with any of the routing methods. The available channel routing methods are shown in Table 3.2.

The Muskingum method is widely used in relative steep channels with little floodplain storage. If floodplain storage has major attenuating effects on the flow, then the modified puls with the storage-outflow relationship determined with a program like HEC-2 is preferred. Modified Puls works well but it must be calibrated to the flood events in question; this is difficult in ungaged areas. The Muskingum-Cunge method has performed well for all but the flattest rivers and/or sharply rising hydrographs (Garbrecht & Brunner, 1991).

**Table 3.2 HEC-1 River Channel Routing Methods**

| <b>Method</b>   | <b>Approach</b>                                                                                   |
|-----------------|---------------------------------------------------------------------------------------------------|
| Average-Lag     | Averages flows over a period and lags the result                                                  |
| Muskingum       | Classic method for upland, nonfloodplain rivers                                                   |
| Muskingum-Cunge | Close approximation to full unsteady flow in most cases                                           |
| Modified Puls   | Uses floodplain storage-outflow characteristics developed from steady-flow backwater calculations |
| Kinematic wave  | Flood wave translation without attenuation                                                        |

It is a close approximation to the full unsteady flow computations and doesn't require calibration in the sense that Modified Puls does. It must be remembered that these hydrologic routings do not know what is happening downstream so it is not a full hydraulic solution considering flow resistances transmitted upstream.

### **3.5.4 Lake Routing**

Flow through lakes is simulated with the same Modified Puls storage-outflow relationship as that used for channels. The lake application is actually the more theoretically correct use of Modified Puls. It assumes a level pool (lake surface) whose outflow is dependent only on the storage (or elevation) of the lake. The channel usage is an approximation of channel flow as a series of reservoirs.

### **3.5.5 Confluences**

Confluences are simply locations where one or more separate hydrographs are combined into a single hydrograph. They are treated as a separate building block because they compute a new hydrograph.

### 3.5.6 Reservoir Routing

Two methods are available for reservoir routing : Modified Puls and Level Pool. The Modified Puls is the same as described above for lakes. The Level-Pool routing allows the user to prescribe the following reservoir outlet characteristics: low level outlet; spillway; and flow over the top of dam. A volume-elevation relationship (or surface area versus elevation) is also required. Low-level outflow is computed as orifice flow, and spillway and top-of-dam flows are computed as weir flow. Continuity is maintained between the inflow, change in storage and outflow. A level pool water surface is assumed throughout the reservoir. If the reservoir outflow rating curve is known or computed from other sources, it may be input directly instead of the information.

The Level-Pool reservoir routing can also be used to represent other types of flow constrictions such as roadway embankments with culverts. This simulation is appropriate if the culvert is sufficiently submerged so that the orifice flow equations apply. If the open channel flow through the culverts is significant, then a separate rating curve must be developed and input to HEC-1.

Reservoir releases may also be specified directly, and HEC-1 will calculate the associated reservoir storage. This option is useful for quickly evaluating various reservoir release alternatives. If the reservoir storage is depleted or exceeded, then the release pattern is revised and simulated again.

### 3.5.7 Diversions

The diversion building block bifurcates the main-channel flow according to an input relationship of total inflow versus the flow to be diverted. There are several possible reasons for the diversion: natural separations in braided channels; flow constrictions which cause some of the flow to take a separate path; and inadequate channel capacities such as those in urban storm drains. In some cases, the restricted

flow will simply pond at that location and re-enter the stream when channel capacity permits. In other cases, the restricted flow will take a separate flow path, e.g. over the streets instead of the storm drains in an urban area. The flow which is not diverted continues down the main channel.

Because there is no general solution to these diversion possibilities, HEC-1 makes the user prescribe the path to be taken. The diverted flow is stored on disk for future retrieval in any subsequent subbasin building block (see Figure 3.2). The diverted flow is retrieved in a subbasin building block and may be routed through reservoir storage or must be pumped over the obstacle. The pump option provides for several pumps, each of which may be turned on and off at different water surface elevations. Pumping is accomplished in a storage (reservoir) routing building block. The pumped flows are saved like the diverted flows for retrieval in any subsequent building block (Feldman, 1995, pp.129-132).

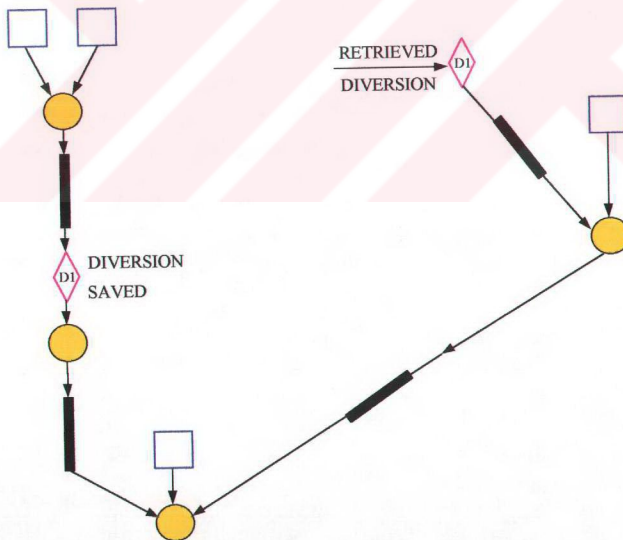


**Figure 3.2 Treatment of Diversions in Input Data**

### 3.6 Preparation of Input Data

#### 3.6.1 Types of Input Data

There are two general types of data records for HEC-1: input control and river basin simulation data. The input control records give the program format of the river basin data and also control certain diagnostic output. All input control records begin with an asterisk (\*) in the first column followed by a command.

The river basin simulation data are all identified by a unique two-character alphabetic code in columns one and two of each record. These codes serve two functions: they identify the data to be read from the record; and they activate various simulation options. The first character of the code identifies the general category, and the second character identifies a specific type of data within a category.

The river basin simulation data records are structured by the user to reflect the topology of the basin. The sequence of the input data prescribes how the river basin is simulated. There are three general subdivisions of these data as shown in Table 3.3, i.e., job control; hydrology and hydraulics; and economics (HEC, 1990).

**Table 3.3 Subdivisions of Simulation Data**

| <b>Job Control</b>          | <b>Hydrology &amp; Hydraulics</b> | <b>Economics &amp; End of Job</b> |
|-----------------------------|-----------------------------------|-----------------------------------|
| I _ Job Initialization      | K _ Job step control              | E _ etc., Economics data          |
| V _ Variable Output Summary | H _ Hydrograph transformation     | ZZ, End of Job                    |
| O _ Optimization            | Q _ Hydrograph data               |                                   |
| J _ Job Type                | B _ Basin data                    |                                   |
|                             | P _ Precipitation data            |                                   |
|                             | L _ Loss (infiltration) data      |                                   |
|                             | U _ Unit Graph data               |                                   |
|                             | M _ Melt data                     |                                   |
|                             | R _ Routing data                  |                                   |
|                             | S _ Storage data                  |                                   |
|                             | D _ Diversion data                |                                   |
|                             | W _ Pump withdrawal data          |                                   |

### 3.6.2 Special Features for Input Data

#### 3.6.2.1 Input Control

There are six input control commands: \*FREE, \*FIX, \*LIST, \*NOLIST, \*MESSAGE, and \*DIAGRAM. Data can be input to the HEC-1 model in a fixed and/or free format. The traditional HEC fixed-format input structure (ten 8-column fields) is the default option of the program. The program also provides the capability to enter data in a free format. All records following a \*FREE record in the data are considered as being in free format. Free format data fields are separated by commas or one or more spaces, and successive commas represent blank fields. The fixed format can be returned to at any point in the data set by providing a \*FIX record. The \*FIX will be in control until another \*FREE record is encountered, etc.

A preprocessor in the program converts free-format data to the standard 8-character field structure and prints the reformatted data. This “echo print” may be turned off and on with \*NOLIST and \*LIST records.

Messages, notes, explanations of data, etc., can be inserted anywhere in the data set by using the \*MESSAGE record.

The stream network structure can be portrayed diagrammatically by using the \*DIAGRAM record at the beginning of the data set. This option causes the program to search the input data set for KK (station computation identifier) records and determine the computation job step associated with each KK record group. A flow chart of the stream network simulation as recognized from the KK record sequences is printed. The user should verify that this flow chart conforms to the intended network of subbasins and routing reaches.

### 3.6.2.2 Time Series Input

The IN (time interval for input data) record allows the user to enter time-series data, either hyetographs or hydrographs, at time steps other than the computation interval specified on the IT (time specification) record. This option is convenient when entering data generated by another program or in a separate HEC-1 simulation. Note that if direct input unit hydrograph ordinates are used (UI record), they must be at the same time step as the simulation computation interval and cannot be input with the IN record.

### 3.6.2.3 Data Repetition Conventions

In many instances, certain physical characteristics are the same for a number of subbasins in the stream network model (for example, infiltration characteristics). Further, in a multiplan analysis, much of the PLAN 1 subbasin data remain unchanged in subsequent plans. The HEC-1 program input conventions make it unnecessary to repeat much of this information in the data set.

Data groups for subbasin runoff simulation which need not be repeated (if they are the same as input for the previous subbasin) are shown in Table 3.4. HEC-1 automatically uses the previous subbasin's input data for these data types unless new data are provided for the current subbasin. The source of the data used as identified by the input record number is printed in the left hand margin. If a zero is printed as the input record number, this means no data records have been provided up to that point, which contain the required information. Great care should be taken to verify that the input data used was so intended. No data are repeatable for routing reaches (HEC, 1990).

**Table 3.4 Data Repetition Options**

| <b>Data Types Which Are Automatically Repeated</b> | <b>Record Identification</b> |
|----------------------------------------------------|------------------------------|
| Rainfall                                           | P                            |
| Infiltration                                       | L                            |
| Base Flow                                          | BF                           |
| Snowmelt                                           | M                            |
| *Unit Hydrograph                                   | US, UC, UD                   |
| *Kinematic Wave                                    | **UK, RK                     |
| * Not recommended                                  |                              |
| * Only if all records remain unchanged             |                              |

In the multiplan analysis, data may be supplied for a number of plans for the same subbasin. Data need not be repeated for each plan by the following two conventions:

1. Plans not specified in the data set by a KP (plan label) record are assumed to be the same as the first plan in the KK record group (data for a particular plan follows a KP record data set).
2. Data specified subsequent to a KP record are considered to update the previous plan data. If no data follows a KP record, then the indicated plan will be considered to be equivalent to the immediately preceding plan in the data set.

### 3.6.3 Hydrologic/Hydraulic Simulation Options

The HEC-1 program has a number of alternative methods available for simulating some aspects of the hydrologic/hydraulic processes. The different methods were also noted in the several data types available for one data category. For example loss rates may be calculated by any of the 5 different methods: exponential, initial/constant, SCS, Green & Ampt or Holtan.

There are a number of methods available for specifying rainfall hyetographs in the stream network computation. The gage data consists of PG records for nonrecording gages and PG and PI or PC records for recording gages. These data are usually

grouped toward the beginning of the data set before the first KK record runoff computation. Within each KK-record group, the PR (recording stations to be weighted), PW (weighting for precipitation stations) and PT (storm total stations to be weighted), PW records (weighting for precipitation stations) are used to specify which gages and corresponding weightings are to be used for computation of that subbasin's average precipitation. Note that a recording gage can be used as both a storm total and a recording gage station. This is indicated by using gage WEST of PT and PW records. If the storm total value is not specified on the PG record for the recording station, the program sums the incremental values on the PI records to compute that value (HEC, 1990).

### **3.7 Variables and Parameters of HEC-1 Model**

Table 3.5 shows the variables and parameters of HEC-1, which were used in this thesis.

### **3.8 Outputs of HEC-1 Model**

The outputs of the HEC-1 model are:

- Runoff Hydrographs,
- Losses,
- Peak Flow,
- Time of Peak,
- Average Flow for Maximum Period,
- Maximum Stage,
- Time of Maximum Stage,
- Stage/Storage plots,

- Routing Outputs (inflow-excess-outflow, basin storage, percent error, volume, maximum celerity,...),
- Optimization Results,
- Statistics Based On Optimization Region (sum of flows, equivalent depth, mean flow, time to center of mass, lag, peak flow, time of peak, standard error, objective function, average absolute error, average percent absolute error,...),
- Reservoir Routing (storage-elevation, outflow-elevation, storage-outflow-elevation values, peak stages, maximum reservoir elevation, maximum depth over dam, maximum stage, maximum outflow, duration over top, time of maximum outflow, time of failure,...)
- Dam Breach Hydrograph,
- System Cost and Performance Value (total system capital cost, total system amortized capital cost, total system annual O, M, power and replacement cost, total system annual cost, average annual damages, average annual damage reduction (benefits), average annual system net benefits,...)

**Table 3.5 Variables and Parameters Specified in HEC-1 Model**

|                            | <i>Variables</i>                                                                                                                                                                                                                                | <i>Parameters</i>                                                                                                                                                                                                |
|----------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <b>BASIN RUNOFF DATA</b>   |                                                                                                                                                                                                                                                 |                                                                                                                                                                                                                  |
| Subbasin Area              | TAREA, drainage area in square miles (sq km),<br>SNAP, normal annual precipitation for the drainage area,                                                                                                                                       | RATIO, multiply each hydrograph ordinate by this value,                                                                                                                                                          |
| Base Flow Characteristics  | STRTQ, flow at start of storm in cfs (cu m/s),<br>QRCSN, flow in cfs (cu m/s) where base flow recession occurs in accordance with the recession constant RTIOR,<br>RTIOR, ratio of recession flow, QRCSN to that flow occurring one hour later, |                                                                                                                                                                                                                  |
| <b>LOSS RATE DATA</b>      |                                                                                                                                                                                                                                                 |                                                                                                                                                                                                                  |
| SCS Curve Number Loss Rate |                                                                                                                                                                                                                                                 | STRTL, initial rainfall abstraction in inches (or mm) for snow free ground,<br>CRVNB, SCS curve number for rainfall/snowmelt losses on snow-free ground,<br>RTIMP, percent of drainage basin that is impervious, |

**Table 3.5 Variables and Parameters Specified in HEC-1 Model (Continued)**

|                                                       | <i>Variables</i>                                                                | <i>Parameters</i> |
|-------------------------------------------------------|---------------------------------------------------------------------------------|-------------------|
| <b>PRECIPITATION DATA</b>                             |                                                                                 |                   |
| <b>Basin Average Precipitation</b>                    | <b>STORM</b> , total storm,                                                     |                   |
| <b>Incremental Precipitation Time Series</b>          | <b>PRCPR(1)</b> , precipitation in inches (mm) during the first time interval,  |                   |
|                                                       | <b>PRCPR(2)</b> , precipitation in inches (mm) during the second time interval, |                   |
| <b>Cumulative Precipitation Time Series</b>           | <b>PRCPR(1)</b> , cumulative precipitation at the beginning of storm,           |                   |
|                                                       | <b>PRCPR(2)</b> , cumulative precipitation at the end of the first period,      |                   |
|                                                       | <b>PRCPR(3)</b> , cumulative precipitation at the end of the second period,     |                   |
|                                                       |                                                                                 |                   |
| <b>Storm Total Precipitation for a Station (Gage)</b> | <b>STATION</b> , station identification,                                        |                   |
|                                                       | <b>PRCPN</b> , total storm precipitation in inches (mm),                        |                   |
|                                                       | <b>ANAPN</b> , normal annual precipitation for a station,                       |                   |
| <b>Hypothetical Storms</b>                            | <b>PFREQ</b> , storm frequency in percent,                                      |                   |

Table 3.5 Variables and Parameters Specified in HEC-1 Model (Continued)

|                                    | <i>Variables</i>                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                               | <i>Parameters</i> |
|------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-------------------|
| Hypothetical Storms<br>(continued) | <p><b>TRSDA</b>, storm area to be used in computing reduction of point rainfall depths Per TP-40</p> <p><b>PNHR(1)</b>, 5-minute duration depth for PFREQ storm,</p> <p><b>PNHR(2)</b>, 15-minute duration depth for PFREQ storm,</p> <p><b>PNHR(3)</b>, 60-minute duration depth for PFREQ storm,</p> <p><b>PNHR(4)</b>, 2-hour duration depth for PFREQ storm,</p> <p><b>PNHR(5)</b>, 3-hour duration depth for PFREQ storm,</p> <p><b>PNHR(6)</b>, 6-hour duration depth for PFREQ storm,</p> <p><b>PNHR(7)</b>, 12-hour duration depth for PFREQ storm,</p> <p><b>PNHR(8)</b>, 24-hour duration depth for PFREQ storm,</p> |                   |

Table 3.5 Variables and Parameters Specified in HEC-1 Model (Continued)

|                                              | <i>Variables</i>                                                                                                                                                                                                                                         | <i>Parameters</i> |
|----------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-------------------|
| <b>Hypothetical Storms<br/>(continued)</b>   | <p><b>PNHR(9)</b>, 2-day duration depth for PFREQ storm,</p> <p><b>PNHR(10)</b>, 4-day duration depth for PFREQ storm</p> <p><b>PNHR(11)</b>, 7-hour duration depth for PFREQ storm,</p> <p><b>PNHR(12)</b>, 10-hour duration depth for PFREQ storm,</p> |                   |
| <b>Precipitation Gage Weighting</b>          | <b>ISTR(I)</b> , alphanumeric station identification of recording gage to be used and corresponding to weighting in field 1 on the following PW record,                                                                                                  |                   |
| <b>Storm Total Stations to be Weighted</b>   | <b>ISTN(I)</b> , alphanumeric station identification for the total storm station,                                                                                                                                                                        |                   |
| <b>Weightings for Precipitation Stations</b> | <b>WTR(I)</b> , relative weight in any units for the station,                                                                                                                                                                                            |                   |

Table 3.5 Variables and Parameters Specified in HEC-1 Model (Continued)

|                                    | <i>Variables</i>                                                                                                                                                                                                     | <i>Parameters</i>                                                                                                                                                                      |
|------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <b>HYDROGRAPH TIME-SERIES DATA</b> |                                                                                                                                                                                                                      |                                                                                                                                                                                        |
| Direct Input Hydrograph            | <p><b>QI(1)</b>, hydrograph ordinate in cfs (cu m/s) at the beginning of the first period,</p> <p><b>QI(2)</b>, etc.,</p>                                                                                            |                                                                                                                                                                                        |
| Pattern Hydrograph                 | <p><b>QP(1)</b>, pattern hydrograph for local inflow which will be adjusted for volume in routing coefficient derivation</p> <p><b>QP(2)</b>, etc.,</p>                                                              |                                                                                                                                                                                        |
| <b>ROUTING DATA</b>                |                                                                                                                                                                                                                      |                                                                                                                                                                                        |
| Channel Loss                       | <p><b>CLOSS</b>, ratio of remaining flow (after <b>QLOSS</b>) which is lost for entire routing,</p> <p><b>ELVINV</b>, average invert elevation of channel L used to compute flow surface area for <b>PERCRT</b>,</p> | <p><b>QLOSS</b>, constant channel loss in entire routing in cfs</p> <p>(cu m/s),</p> <p><b>PERCRT</b>, percolation rate cfs/acre (cu m/s-acre) for wetted surface area of channel,</p> |

Table 3.5 Variables and Parameters Specified in HEC-1 Model (Continued)

|                                     | <i>Variables</i> | <i>Parameters</i>                                                                                                                                                                                                 |
|-------------------------------------|------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <b>Muskingum Routing</b>            |                  | <p>NSTPS, integer steps (equal to number of subreaches),</p> <p>AMSKK, Muskingum K coefficient in hours for entire reach,</p> <p>X, Muskingum X coefficient for Muskingum routing or working R&amp;D routing,</p> |
| <b>UNIT GRAPH/KINEMATIC DATA</b>    |                  |                                                                                                                                                                                                                   |
| <b>Clark Unit Graph</b>             |                  | <p>TC, time of concentration in hours,</p> <p>R, Clark storage coefficient in hours,</p>                                                                                                                          |
| <b>Snyder Unit Graph</b>            |                  | <p>TP, Snyder's standard lag in hours,</p> <p>CP, Snyder's peaking coefficient,</p>                                                                                                                               |
| <b>SCS Dimensionless Unit Graph</b> |                  | <p>TLAG, SCS lag in hours</p>                                                                                                                                                                                     |

### 3.9 Computation Methodologies for Rainfall-Runoff Simulation

#### 3.9.1 Rainfall-Runoff Simulation

The HEC-1 model components are used to simulate the rainfall-runoff process as it occurs in an actual river basin. The model components function on the basis of simple mathematical relationships which are intended to represent individual meteorologic, hydrologic and hydraulic processes which comprise the precipitation-runoff process. These processes are separated into precipitation, interception/infiltration, transformation of precipitation excess to subbasin outflow, addition of baseflow and flood routing. The computation equations described are equally applicable to English or metric units except where noted.

#### 3.9.2 Precipitation

##### 3.9.2.1 Precipitation Hyetograph

A precipitation hyetograph is used as the input for all runoff calculations. The specified precipitation is assumed to be basin average (i.e., uniformly distributed over the subbasin). Any of the options used to specify precipitation produce a hyetograph such as that shown in Figure 3.3. The hyetograph represents average precipitation (either rainfall or snowfall) depths over a computation interval.

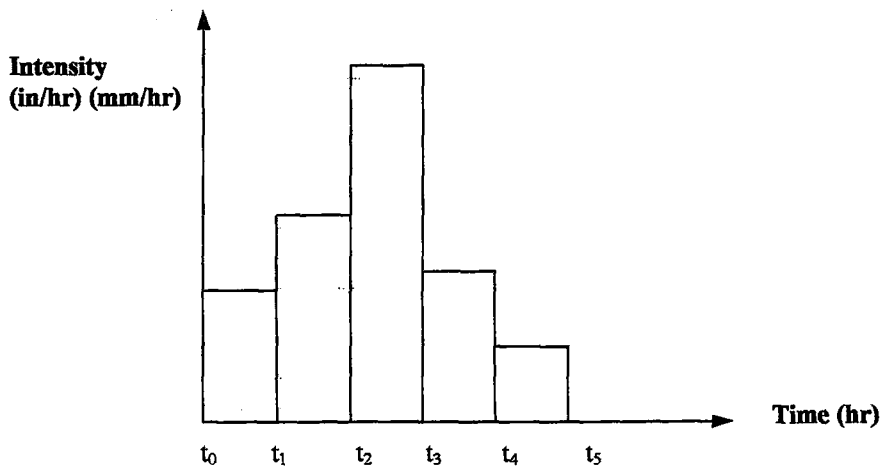


**Figure 3.3 Rainfall Hyetograph**

### 3.9.2.2. Historical Storms

Either of two methods will be used for an observed storm event:

- (1) **Basin-Average Precipitation:** Any storm may be specified for a subbasin as a total amount of precipitation for the storm and a temporal pattern for distributing the total precipitation.
- (2) **Weighted Precipitation Gages:** The total storm precipitation for a subbasin may be computed as the weighted average of measurements from several gages according to the following equations:

$$PRCPA = \frac{\sum_{J=1}^n PRCPN(J) * WTN(J)}{\sum_{J=1}^n WTN(J)} \quad (1)$$

where  $PRCPA$  is the subbasin-average total precipitation,  $PRCPN(J)$  is the total precipitation for gage  $J$ ,  $WTN(J)$  is the relative weight for gage  $J$ , and  $n$  is the number of gages.

If normal annual precipitation for the subbasin is given, equation (1) is modified to include weighting by station normal annual precipitation by:

$$PRCPA = SNAP * \frac{\sum_{J=1}^n PRCPN(J) * WTN(J)}{\sum_{J=1}^n ANAPN(J) * WTN(J)} \quad (2)$$

where  $ANAPN(J)$  is the station normal annual precipitation, and  $SNAP$  is the subbasin-average normal annual precipitation. Use of this option may be desirable in cases where precipitation measurements are known to be biased. For example, data

obtained from a gage located on the floor of a valley may consistently underestimate subbasin average precipitation for higher elevations. *ANAPN* may be used to adjust for this bias.

The temporal pattern for distribution of the storm-total precipitation is computed as a weighted average of temporal distributions from recording stations:

$$PRCP(I) = \frac{\sum_{J=1}^n PRCPR(I, J) * WTR(J)}{\sum_{J=1}^n WTR(J)} \quad (3)$$

where  $PRCP(I)$  is the basin-average precipitation for the  $I$ th time interval,  $PRCPR(I, J)$  is the recording station precipitation for the  $I$ th time interval, and  $WTR(J)$  is the relative weight for gage  $J$ .

The subbasin-average hyetograph is computed using the temporal pattern,  $PRCP$ , to distribute the total,  $PRCPA$ .

### 3.9.2.3 Synthetic Storms

Synthetic storms are frequently used for planning and design studies. Criteria for synthetic storms are generally based on a detailed analysis of long term precipitation data for a region. There are three methods in HEC-1 for generating synthetic storm distributions:

**(1) Standard Project Storm:** The procedure for computing Standard Project Storms, *SPS*, programmed in HEC-1 is applicable to basins of area 10 to 1,000 square miles located east of 105° longitude. The *SPS* is determined by specifying an index precipitation, *SPFE*, a storm reduction coefficient, *TRSPC*, and the area over which the storm occurs, *TRSDA*. *SPFE* and *TRSPC* are determined by referring to manual EM-1110-2-1411 (Corps of Engineers, 1952). A total storm depth is determined and

distributed over a 96-hour duration based on the following formulas which were derived from design charts in the referenced manual.

$$R24HR(3) = 182.15 - 14.3537 * LOG_6(TRSDA + 80) \quad (4)$$

$$R24HR(1) = 3.5$$

$$R24HR(2) = 15.5$$

$$R24HR(4) = 6.0$$

where  $R24HR(I)$  is the percent of the index precipitation occurring during the  $I$ th 24-hour period.

Each 24-hour period is divided into four 6-hour periods. The ratio of the 24-hour precipitation occurring during each 6-hour period is calculated as:

$$R6HR(3) = \frac{13.42}{(SPFE + 11.0)^{0.93}} \quad (5)$$

$$R6HR(2) = 0.055 * (SPFE - 6.0)^{0.51} \quad (6)$$

$$R6HR(4) = 0.5 * (1 - R6HR(3) - R6HR(2)) + 0.0165 \quad (7)$$

$$R6HR(1) = R6HR(4) - 0.033 \quad (8)$$

where  $R6HR(I)$  is the ratio of 24-hour precipitation occurring during the  $I$ th 6-hour period and  $SPFE$  is the index precipitation in inches.

The precipitation for each time interval, except during the peak 6-hour period, is computed as:

$$PRCP = 0.01 * R24HR * R6HR * SPFE * \frac{TRHR}{6} \quad (9)$$

where  $TRHR$  is the computation time interval in hours.

### 3.9.3 Interception/Infiltration

Land surface interception, depression storage and infiltration are referred to in the HEC-1 model as precipitation losses. Interception and depression storage are intended to represent the surface storage of water by trees or grass, local depressions in the ground surface, in cracks and crevices in parking lots or roofs, or in an surface area where water is not free to move as overland flow. Infiltration represents the movement of water to areas beneath the land surface.

Two important factors should be noted about the computation of precipitation loss in the model. First, precipitation which does not contribute to the runoff process is considered to be lost from the system. Second, the equations used to compute the losses do not provide for soil moisture or surface storage recovery. This fact dictates that the HEC-1 program is a single-event-oriented model. The precipitation loss computations can be used with either the unit hydrograph or kinematic wave model components. In the case of the unit hydrograph component, the precipitation loss is considered to be a subbasin average (uniformly distributed over an entire subbasin). On the other hand, separate precipitation losses can be specified for each overland flow plane (if two are used) in the kinematic wave component. The losses are assumed to be uniformly distributed over each overland flow plane (HEC, 1990).

In some instances, there are negligible losses for portion of a subbasin. This would be true for an area containing a lake, reservoir or impervious area. In this case, precipitation losses will not be computed for a specified percentage of the area labeled as impervious.

There are five methods that can be used to calculate the precipitation loss. Using any one of the methods, an average precipitation loss is determined for a computation interval and subtracted from the rainfall/snowmelt hyetograph as shown in Figure 3.4. The resulting precipitation excess is used to compute an outflow hydrograph for a

subbasin. A percent imperviousness factor can be used with any of the loss rate methods to guarantee 100% runoff from that portion of the basin.

A percent impervious factor can be used with any of the loss rate methods; it guarantees 100% runoff from that percent of the subbasin.

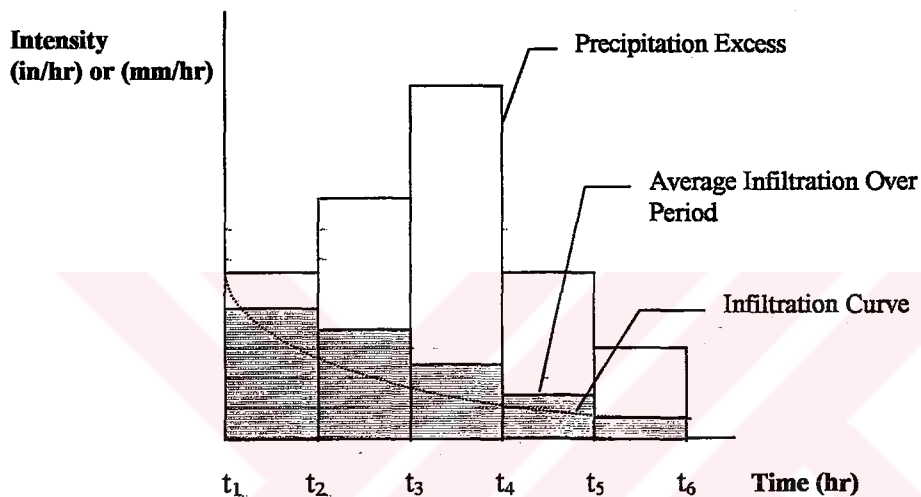


**Figure 3.4 Loss Rate, Rainfall Excess Hyetograph**

The SCS Curve Number Method may be used to identify the precipitation loss. The Soil Conservation Service (SCS), U.S. Department of Agriculture, has instituted a soil classification system for use in soil survey maps across the country. Based on experimentation and experience, the agency has been able to relate the drainage characteristics of soil groups to a curve number, CN (Soil Conservation Service, 1972 and 1975). The SCS provides information on relating soil group type to the curve number as a function of soil cover, land use type and antecedent moisture conditions.

Precipitation loss is calculated based on supplied values of CN and IA (where IA is an initial surface moisture storage capacity in units of depth). CN and IA are related to a total runoff depth for a storm by the following relationships:

$$ACEXS = (ACRAN - IA)^2 / (ACRAN - IA + S) \quad (10)$$

$$S = (1000 - 10 * CN) / CN \text{ or}$$

$$S = (25400 - 254 * CN) / CN \text{ (Metric Units)} \quad (11)$$

where *ACEXS* is the accumulated excess in inches (mm), *ACRAN* is the accumulated rainfall depth in inches (mm), and *S* is the currently available soil moisture storage deficit in inches (mm).

In the case that the user does not wish to specify IA, a default value is computed as

$$IA = 0.2 * S \quad (12)$$

This relation is based on empirical evidence established by the Soil Conservation Service.

Since the SCS method gives total excess for a storm, the incremental excess (the difference between rainfall and precipitation loss) for a time period is computed as the difference between the accumulated excess at the end of the current period and the accumulated excess at the end of the previous period.

### 3.9.4 Unit Hydrograph

#### 3.9.4.1 Basic Methodology

The unit hydrograph technique has been discussed extensively in the literature (Corps of Engineers, 1959; Linsley & Kohler & Paulhus, 1975, 1988; Viessman &

Knapp & Lewis & Harbaugh, 1977). This technique is used in the subbasin runoff component to transform rainfall/snowmelt excess to subbasin outflow. A unit hydrograph can be directly input to the program or a synthetic unit hydrograph can be computed from user supplied parameters.

A 1-hour unit hydrograph is defined as the subbasin surface outflow due to a unit (1 inch or mm) rainfall excess applied uniformly over a subbasin in a period of one hour. Unit hydrograph durations other than an hour are common. HEC-1 automatically sets the duration of unit excess equal to the computation interval selected for watershed simulation.

The rainfall excess hyetograph is transformed to a subbasin outflow by utilizing the general equation:

$$Q(i) = \sum_{j=1} U(j) * X(i - j + 1) \quad (13)$$

where  $Q(i)$  is the subbasin outflow at the end of computation interval  $i$ ,  $U(j)$  is the  $j$ th ordinate of the unit hydrograph,  $X(i)$  is the average rainfall excess for computation interval  $i$  (HEC, 1990).

The equation is based on two important assumptions. First, the unit hydrograph is characteristic for a subbasin and is not storm dependent. Second, the runoff due to excess from different periods of rainfall excess can be linearly superposed.

#### 3.9.4.2 Synthetic Unit Hydrographs

The parameters for the synthetic unit hydrograph can be determined from gage data by employing the parameter optimization option. Otherwise, these parameters can be determined from regional studies or from guidelines given in references for each synthetic technique. There are three synthetic unit hydrograph methods available

in the model: Clark Unit Hydrograph, Snyder Unit Hydrograph and SCS Dimensionless Unit Hydrograph. The methods which are used in this application are explained below.

### ***Clark Unit Hydrograph***

The Clark method (1945) requires three parameters to calculate unit hydrograph:  $TC$ , the time of concentration for the basin,  $R$ , a storage coefficient, and a time-area curve. A time-area curve defines the cumulative area of the watershed contributing runoff to the subbasin outlet as a function of the time (expressed as a proportion of  $TC$ ).

In the case that a time curve is not supplied, the program utilizes a dimensionless time area curve:

$$AI = 1.414T^{1.5} \quad 0 \leq T < 0.5 \quad (14)$$

$$1 - AI = 1.414(1 - T)^{1.5} \quad 0.5 < T < 1 \quad (15)$$

where  $AI$  is the cumulative area as a fraction of total subbasin area and  $T$  is the fraction of time of concentration. The ordinates of the time-area curve are converted to volume of runoff per second for unit excess and interpolated to the given time interval. The resulting translation hydrograph is then routed through a linear reservoir to simulate the storage effects of the basin; and the resulting unit hydrograph for instantaneous excess is averaged to produce the hydrograph for unit excess occurring in the given time interval.

The linear reservoir routing is accomplished using the general equation:

$$Q(2) = CA * I + CB * Q(1) \quad (16)$$

The routing coefficient are calculated from:

$$CA = \Delta t / (R + 0.5 * \Delta t) \quad (17)$$

$$CB = 1 - CA \quad (18)$$

$$QUNGR = 0.5 (Q(1) + Q(2)) \quad (19)$$

where  $Q(2)$  is the instantaneous flow at the end of period,  $Q(1)$  is the instantaneous flow at the beginning of period,  $I$  is the ordinate of the translation hydrograph,  $\Delta t$  is the computation time interval in hours (also duration of unit excess),  $R$  is the basin storage factor in hours, and  $QUNGR$  is the unit hydrograph ordinate at the end of the computation interval. The computation of unit hydrograph ordinates is terminated when its volume exceeds 0.995 inch (mm) or 150 ordinates, whichever occurs first.

### ***Snyder Unit Hydrograph***

The Snyder method (1938) determines the unit graph peak discharge, time to peak, and widths of the unit graph at 50 and 75% of the peak discharge. The method does not produce the complete unit graph required by HEC-1. Thus, HEC-1 uses the Clark method to affect a Snyder unit graph. The initial Clark parameters are estimated from the given Snyder's parameters,  $T_p$  and  $C_p$ . A unit hydrograph is computed using Clark's method, and Snyder parameters are computed from the resulting unit hydrograph by the following equations:

$$CPTMP = QMAX * (TPEAK - 0.5 * \Delta t) / (C * A) \quad (20)$$

$$ALAG = 1.048 * (TPEAK - 0.75 * \Delta t) \quad (21)$$

$$TPEAK = C * C_t (L * L_{CA})^{0.3} \quad (22)$$

where  $CPTMP$  is Snyder's  $C_p$  for the computed unit hydrograph,  $L$  is the length of the main stream in kilometers (or miles) from the outlet to the upstream divide,  $L_{CA}$  is the distance in kilometers (miles) from the outlet to a point on the stream nearest the centroid of the watershed area,  $QMAX$  is the maximum ordinate of unit hydrograph,

$T_{PEAK}$  is the time when  $Q_{MAX}$  occurs (in hours),  $\Delta t$  is the duration of excess (in hours),  $A$  is the subbasin area in square miles (sq km),  $C$  is a conversion factor, and  $ALAG$  is Snyder's standard lag,  $T_p$ , for the computed unit hydrograph. Snyder's standard lag is for a unit hydrograph which has a duration of excess equal to  $T_p/5.5$ . The constant, 1.048, in the above equation results from converting the duration of excess to the given time interval.

Clark's  $TC$  (time of concentration in hours) and  $R$  (storage coefficient) are adjusted to compensate for differences between the values of  $T_p$  and  $C_p$  calculated by equations (20) and (21) and the given values. A new unit hydrograph is computed using these adjusted values. This procedure continues through 20 iterations or until the differences between the computed and the given values of  $T_p$  and  $C_p$  are less than one percent of the given values.

### ***SCS Dimensionless Unit Hydrograph***

Input data for the Soil Conservation Service, SCS, dimensionless unit hydrograph method (Soil Conservation Service, 1972) consists of a single parameter,  $TLAG$ , which is equal to the lag (hrs) between the center of mass of rainfall excess and the peak of the unit hydrograph. Peak flow and time to peak are computed as:

$$T_{PEAK} = 0.5 * \Delta t + TLAG \quad (23)$$

$$Q_{PK} = 484 * AREA / T_{PEAK} \quad (24)$$

where  $T_{PEAK}$  is the time to peak of the unit hydrograph in hours,  $\Delta t$  is the duration of excess in hours or the computation interval,  $Q_{PK}$  is the peak flow of unit hydrograph in cfs/inch, and  $AREA$  is the subbasin area in square miles. The unit hydrograph is interpolated for the specified computation interval and computed peak flow from the dimensionless unit hydrograph shown in Figure 3.5.

The selection of the computation interval in the program, which is also the duration of the unit hydrograph, is based on the relationship  $\Delta t = 0.2 * T_{PEAK}$  (SCS, 1972, chapters 15, 16). There is some latitude allowed in this relationship; however, the duration of the unit graph should not exceed  $\Delta t \leq 0.25 * T_{peak}$ . These relations are based on an empirical relationship,  $TLAG = 0.6T_c$ , and  $1.7 * T_{PEAK} = \Delta t + T_c$  where  $T_c$  is the time of concentration of the watershed. The duration should not be greater than  $\Delta t \leq 0.29 * TLAG$  (HEC, 1990).

**Dimensionless Flow  $Q/Q_{peak}$**

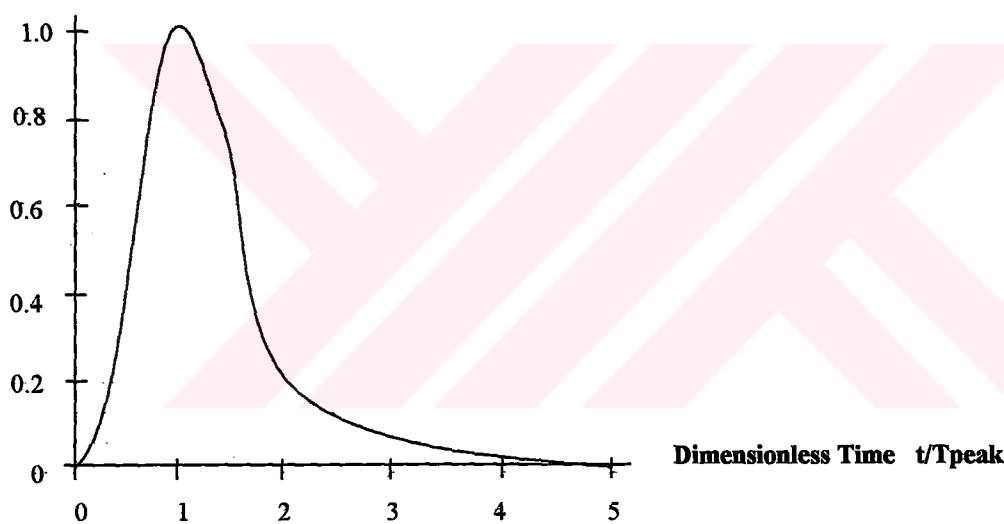


**Figure 3.5 SCS Dimensionless Unit Graph**

### 3.9.5 Base Flow

Two distinguishable contributions to a streamflow hydrograph are direct runoff and base flow which results from releases of water from subsurface storage. The

HEC-1 model provides means to include the effects of base flow on the streamflow hydrograph as a function of three parameters, *STRTQ*, *QRCSN* and *RTIOR*. Figure 3.6 defines the relation between the streamflow hydrograph and these variables.

The variable *STRTQ* represents the initial flow in the river. It is affected by the long term contribution of groundwater releases in the absence of precipitation and is a function of antecedent conditions (e.g., the time between the storm being modeled and the last occurrence of precipitation). The variable *QRCSN* indicates the flow at which an exponential recession begins on the receding limb of the computed hydrograph.

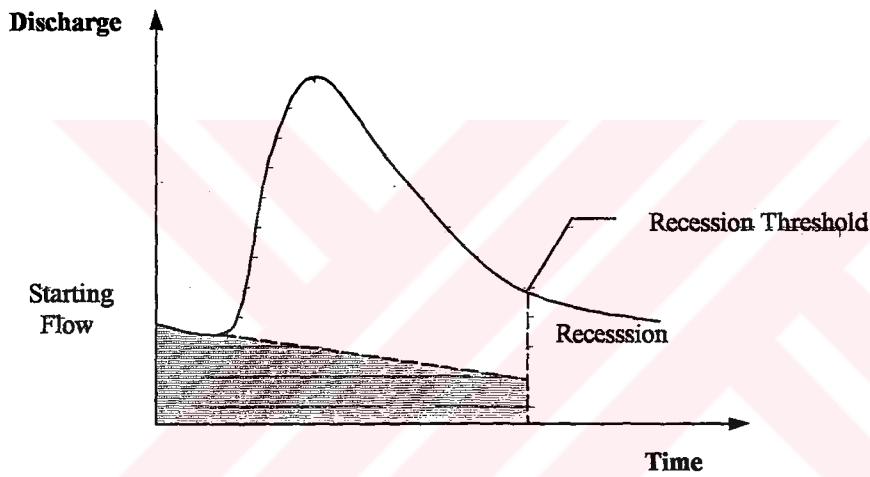


**Figure 3.6 Base Flow Diagram**

Recession of the starting flow and “falling limb” follow a user specified exponential decay rate, *RTIOR*, which is assumed to be characteristic of the basin. *RTIOR* is equal to the ratio of a recession limb flow to the recession limb flow occurring one hour later. The program computes the recession flow  $Q$  as:

$$Q = Q_0 (RTIOR)^{-n\Delta t} \quad (25)$$

where  $Q_0$  is *STRTO* or *QRCSN*, and  $n\Delta t$  is the time in hours since the initiation of recession. *QRCSN* and *RTIOR* can be obtained by plotting the log of observed flows versus time. The point at which the recession limb fits a straight line defines *QRCSN* and the slope of the straight line is used to define *RTIOR*. Alternatively, *QRCSN* can be specified as a ratio of the peak flow. For example, the user can specify that the exponential recession is to begin when the “falling limb” discharge drops to 0.1 of the calculated peak discharge.

The rising limb of the streamflow hydrograph is adjusted for base by adding the recessed starting flow to the computed direct runoff. The falling limb is determined in the same manner until the computed flow is determined to be less than *QRCSN*. At this point, the time at which the value of *QRCSN* is reached is estimated from the computed hydrograph. From this time on, the streamflow hydrograph is computed using the recession equation unless the computed flow rises above the base flow recession. This is the case of a double peaked streamflow hydrograph where a rising limb of the second peak is computed by combining the starting flow recessed from the beginning of the simulation and the direct runoff (HEC,1990).

### 3.9.6 Flood Routing

Flood routing is used to simulate flood wave movement through river reaches and reservoirs. Most of the flood-routing methods available in HEC-1 are based on the continuity equation and some relationship between flow and storage or stage. These methods are Muskingum, Muskingum-Cunge, Kinematic wave, Modified Puls, Working R and D, and Level-pool reservoir routing. In all of these methods, routing proceeds on an independent-reach basis from upstream to downstream; neither backwater effects nor discontinuities in the water surface such as jumps or bores are considered.

Storage routing methods in HEC-1 are those methods which require data that define the storage characteristics of a routing reach or reservoir. These methods are: Modified Puls, Working R and D, and Level-Pool routing.

There are also two routing methods in HEC-1 which are based on lagging averaged hydrograph ordinates. These methods are not based on reservoir storage characteristics, but have been used on several rivers with good results.

### ***Muskingum Method***

The Muskingum method (Corps of Engineers, 1960) computes outflow from a reach using the following equation:

$$QOUT(2) = (CA - CB) * QIN(1) + (1 - CA) * QOUT(1) + CB * QIN(2) \quad (26)$$

$$CA = \frac{2 * \Delta t}{2 * AMSKK * (1 - X) + \Delta t} \quad (27)$$

$$CB = \frac{\Delta t - 2 * AMSKK \Delta t}{2 * AMSKK * (1 - X) + \Delta t} \quad (28)$$

where  $QIN$  is the inflow to the routing reach in cfs ( $m^3/sec$ ),  $QOUT$  is the outflow from the routing reach in cfs ( $m^3/sec$ ),  $AMSKK$  is the travel time through the reach in hours, and  $X$  is the Muskingum weighting factor ( $0 \leq X \leq 0.5$ ). The routing procedure may be repeated for several subreaches (designated as  $NSTPS$ ) so the total travel time through the reach is  $AMSKK$ . To insure the methods' s computational stability and the accuracy of computed hydrograph, the routing reach should be chosen so that:

$$\frac{1}{2(1 - X)} \leq \frac{AMSKK}{NSTPS * Dt} \leq \frac{1}{2X} \quad (29)$$

### ***Muskingum-Cunge Method***

This routing technique can also be used independently of the subbasin runoff computation; it can be used for any routing reach. The Muskingum-Cunge method is not limited to the standard prismatic channel shapes. Muskingum-Cunge allows more detailed main channel and overbank flow areas to be specified with an eight-point cross section. The Muskingum-Cunge routing is applicable to a wide range of channel and hydrograph conditions. It has the same limitations as all other HEC-1 routing methods in that the downstream backwater effects cannot be simulated.

### **3.10 MultiPlan-MultiFlood Analysis**

The multiplan-multiflood simulation option allows the user to investigate a series of floods for a number of different characterizations (plans) of the watershed in a single computer run. The advantage in this option is that multiple storms and flood control projects can be simulated efficiently, and the results can be compared with a minimum of effort by the user.

The multiflood simulation allows the user to analyze several different floods in the same computer run. The multifloods are computed as ratios of a base event (e.g., 0.5, 1.0, 1.5, etc.) which may be either precipitation or runoff. The ratio hydrographs are computed for every component of the river basin. In the case of rainfall, each ordinate of the input base-event hyetograph is multiplied by a ratio and a stream network rainfall-runoff simulation carried out for each ratio. This is done for every ratio of the base event. In the case of runoff ratios, the ratios are applied to the computed or direct-input hydrograph, and no rainfall-runoff calculations are made for individual ratios.

The multiplan option allows a user to conveniently modify a basin model to reflect desired flood control projects and changes in the basin's runoff response characteristics. This is useful when, for example a comparison of flood control options

or the effects of urbanization are being analyzed. The user designates PLAN 1 as the existing river basin model, and then modifies the existing plan data to reflect basin changes (such as reservoirs, channel improvements, or changes in land use) in PLANS 2, 3, etc.

If the basin's rainfall-runoff response characteristics are modified in one of the plans, then precipitation ratios but not runoff ratios must be used. Otherwise, ratios of hydrographs should be used. The program performs a stream network analysis, or multiflood analysis, for each plan as in Figure 3.7. The results of the analysis provide flood hydrograph data for each plan and each ratio of the base event. The summary of the results at the end of the program output provides the user with a convenient method for comparing the differences between plans and the differences between different flood ratios for the same plan (HEC, 1990).

### **3.11 Model Calibration and Verification**

#### **3.11.1 Methodology**

Calibration and verification are essential parts of the modeling process. Rough estimates for the parameters in the HEC-1 model can be obtained from the methods which are used in HEC-1. However, the model should be calibrated to observed flood data whenever possible. HEC-1 provides a powerful optimization technique for the estimation of some of the parameters when gaged precipitation and runoff data are available. Parameters can be calibrated automatically by using a gradient search procedure. By using this technique and regionalizing the results, rainfall-runoff parameters for ungaged areas can also be estimated (HEC, 1990).

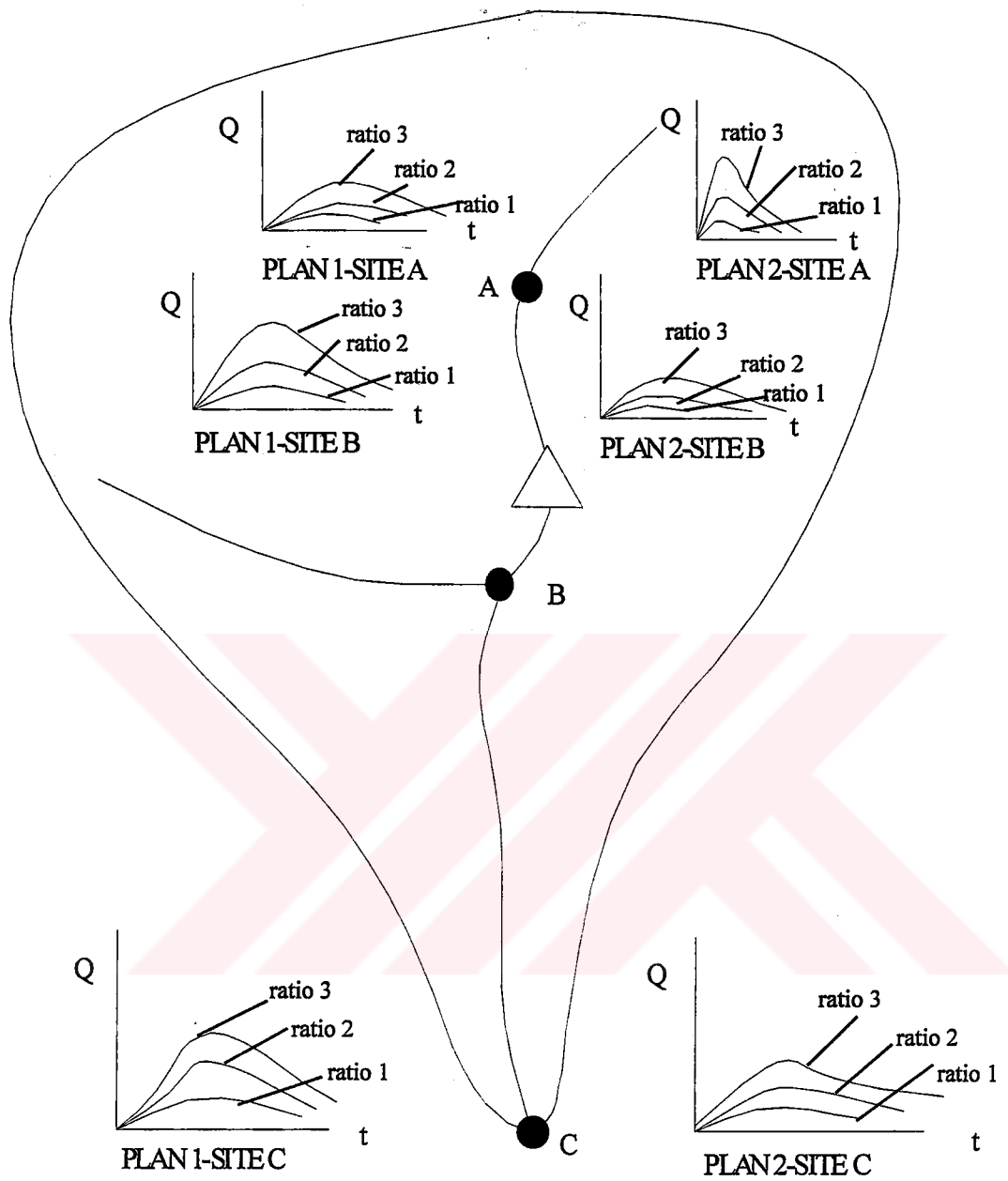


**Figure 3.7 Multiflood and Multiplan Hydrographs**

### 3.11.2 Calibration for the Unit Hydrograph and Loss Rate Parameters

#### 3.11.2.1 Optimization Methodology

The parameter calibration option has the capability to automatically determine a set of unit hydrograph and loss rate parameters that “best” reconstitute an observed runoff hydrograph for a subbasin. The data which must be provided to the model are: basin average precipitation; basin area; starting flow and baseflow parameters *STRTQ*, *QRCSN* and *RTIOR*; and the outflow hydrograph. Unit hydrograph and loss rate parameters can be determined individually or in combination. Parameters that are not to be determined from the optimization process must be estimated and provided to the model. Initial estimates of the parameters to be determined can be input by the user or chosen by the program’s optimization procedure.

The runoff parameters that can be determined in the calibration are the unit hydrograph parameters of the Snyder, Clark and SCS methods. The melt rate and threshold melt temperature can also be optimized for snow hydrology studies. If the Snyder method is employed, the Clark coefficients will be determined and converted to the Snyder parameters.

The “best” reconstitution is considered to be that which minimizes an objective function, *STDER* which is the square root of the weighted square difference between the observed hydrograph and the computed hydrograph. Presumably, this difference will be a minimum for the optimal parameter estimates. *STDER* is depicted in Figure 3.9 and is computed as follows.

$$STDER = \left[ \sum_{i=1}^n (QOBS_i - QCOMP_i)^2 * \frac{WT_i}{n} \right] \quad (30)$$

where  $QCOMP_i$  is the runoff hydrograph ordinate for time period  $i$  computed by HEC-1,  $QOBS_i$  is the observed runoff hydrograph ordinate  $i$ ,  $n$  is the total number of hydrograph ordinates, and  $WT_i$  is the weighting factor for the hydrograph ordinate  $i$  computed from the following equation:

$$WT_i = (QOBS_i + QAVE) / (2 * QAVE) \quad (31)$$

where  $QAVE$  is the average observed discharge.

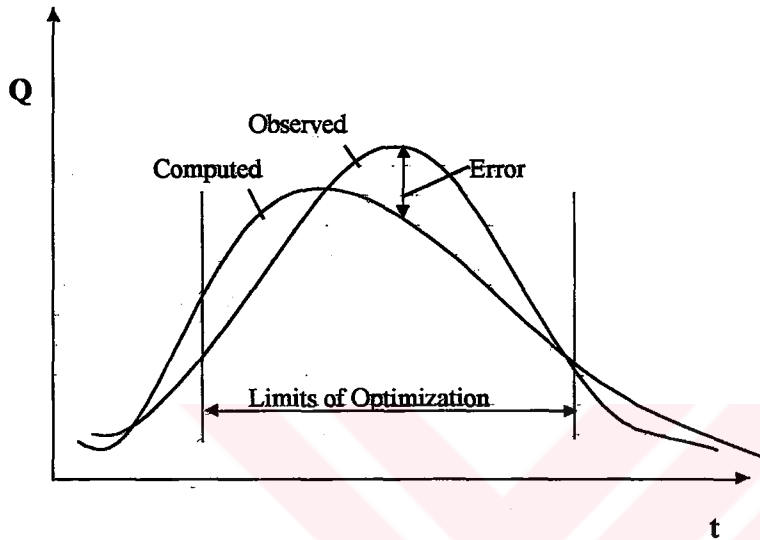


**Figure 3.8 Error Calculation for Hydrologic Optimization**

This weighting function emphasizes accurate reproduction of peak flows rather than low flows by biasing the objective function. Any errors for computed discharges that exceed the average discharge will be weighted more heavily, and hence the optimization scheme should focus on reduction of these errors.

The minimum of the objective function is found by employing the univariate search technique (Ford & Morris & Feldman, 1980). The univariate search method computes values of the objective function for various values of the optimization parameters. The values of the parameters are systematically altered until  $STDER$  is minimized.

The range of feasible values of the parameters is bounded because of physical limitations on the values that the various unit hydrograph, loss rate, and snowmelt parameters may have, and also because of numerical limitations imposed by the

mathematical functions. In addition to bounds on the maximum and minimum values of certain parameters, the interaction of some parameters is also restricted because of physical or numerical limitations. These constraints are summarized in Table 3.6. The constraints shown here are limited to those imposed explicitly by the program. Additional constraints may be appropriate in certain circumstances; however, these must be imposed externally to the program when the user must decide whether to accept, modify, or reject a given parameter set, based on engineering judgement (HEC, 1990).

**Table 3.6 Constraints on Unit Graph and Loss Rate Parameters**

| Clark Unit Graph Parameters                                                               |                                                                  |                               |
|-------------------------------------------------------------------------------------------|------------------------------------------------------------------|-------------------------------|
| $TC \geq 1.03\Delta t$                                                                    |                                                                  |                               |
| $R \geq .52$                                                                              |                                                                  |                               |
| $\Delta t = \text{Computation Interval}$                                                  |                                                                  |                               |
| Loss Rate Parameters                                                                      |                                                                  |                               |
| <b>Exponential</b>                                                                        |                                                                  | <b>SCS</b>                    |
| $ERAIN \leq 1.0$                                                                          |                                                                  | $0 \leq CN \leq 100$          |
| $RTIOL \geq 1.0$                                                                          |                                                                  |                               |
| <b>Snowmelt</b>                                                                           |                                                                  | <b>Green and Ampt</b>         |
| $RTIOK \geq 1.0$                                                                          |                                                                  | $IA \geq 0$                   |
| $-1.11^\circ\text{C} \leq \text{FRZTP} \leq 3.33^\circ\text{C}$                           |                                                                  | $DTHETA \geq 0$               |
|                                                                                           |                                                                  | $PSIF \geq 0$                 |
|                                                                                           |                                                                  | $XKSAT \geq 0$                |
| <b>Uniform</b>                                                                            |                                                                  | <b>Holtan</b>                 |
| $STRIL \geq 0$                                                                            |                                                                  | $FC \geq 0$                   |
| $CNSTL \geq 0$                                                                            |                                                                  | $GIA \geq 1.0$                |
|                                                                                           |                                                                  | $BEXP \geq 0$                 |
| <b>ERAIN:</b> Exponent of precipitation                                                   | <b>RTIOL:</b> Rate of change of the rain loss-rate parameter     | <b>CN:</b> SCS                |
| curve number on snow-free ground                                                          | <b>RTIOK:</b> Rate of change of the snowmelt loss rate parameter |                               |
| <b>FRZTP:</b> Index temperature at which snow will melt in degrees F ( $^\circ\text{C}$ ) | <b>IA:</b> Initial loss (inches or mm)                           |                               |
| <b>DTHETA:</b> Volumetric moisture deficit                                                | <b>PSIF:</b> Wetting front suction (inches or mm)                | <b>XKSAT:</b>                 |
| Hydraulic conductivity at natural saturation (inches/hour or mm/hour)                     | <b>STRIL:</b> Initial rainfall /snowmelt                         |                               |
| loss (inches or mm)                                                                       | <b>CNSTL:</b> Uniform rainfall/snowmelt loss (inches or mm)      | <b>FC:</b> Holtan's long term |
| equalibrium loss rate on snow-free ground (inches/hour or mm/hour)                        | <b>BEXP:</b> Exponent of available soil                          |                               |
| moisture storage                                                                          |                                                                  |                               |

The optimization procedure does not guarantee that a “global” optimum (or a global minimum of the objective function) will be found for the runoff parameter; a local minimum of the objective function might be found by the procedure. To help assess the results of the optimization, HEC-1 provides graphical and statistical comparisons of the observed and computed hydrographs. From this, the user can then judge the accuracy of the optimization result. It is possible that the computed hydrograph will not meet the criteria established by the user. An improvement in the reconstitution might be affected by specifying different starting values for the parameters to be optimized. This can be accomplished by varying the starting values in a number of optimization runs in order to better sample the objective function and to find a global optimum.

### 3.11.2.2 Algorithm for Model Calibration

The algorithm for calibration of the model proceeds as follows:

1. Initial values are assigned for all parameters. These values may be assigned by the user or they may be program-assigned default values as in Table 3.7.
2. The response of the river basin as simulated with initial parameter estimates, and the initial value of the objective function is calculated. The volume of the simulated hydrograph is adjusted to within one percent of the observed hydrograph if the option to adjust infiltration parameters has been selected. This is demonstrated by the asterisked (\*) values. The asterisk (\*) denotes which variable was changed and its “optimum” value.
3. Each parameter to be estimated is decreased by one percent and then by two percent; the system response is evaluated, and the objective function is calculated for each change, respectively. This gives three separate system evaluations at equally-spaced values of the parameter with all other parameters held constant. The “best” value of the parameter is then estimated using Newton’s method. A parameter which does not improve the objective function under this procedure is maintained at its original value. This is indicated by a plus (+) in place of an asterisk (\*) in the computed output.

**Table 3.7 HEC-1 Default Initial Estimates for Unit Hydrograph and Loss Rate Parameters (HEC, 1990)**

|                   | Unit Graph |                 |
|-------------------|------------|-----------------|
|                   | Parameter  | Initial Value   |
| Clark             | TC+R       | $(TAREA)^{1/2}$ |
|                   | R/(TC+R)   |                 |
|                   | 0.50       |                 |
|                   | Loss Rates |                 |
|                   | Parameter  | Initial Value   |
| Exponential       | COEF       | 0.07            |
|                   | STRKR      | 0.20            |
|                   | STRKS      | 0.20            |
|                   | RTIOK      | 2.00            |
|                   | ERAIN      | 0.50            |
|                   | FRZTP      | 0.00            |
|                   | DLTKR      | 0.50            |
|                   | RTIOL      | 2.00            |
| Initial & Uniform | STRTL      | 1.00            |
|                   | CNSTL      | 0.10            |
| Holtan            | FC         | 0.01            |
|                   | GIA        | 0.50            |
|                   | SA         | 1.00            |
|                   | BEXP       | 1.40            |
| Curve Number      | STRTL      | 1.08            |
|                   | CRVNBR     | 65.00           |
| Green & Ampt      | IA         | 0.10            |
|                   | DTHETA     | 0.50            |
|                   | PSIF       | 10.00           |
|                   | XKSAT      | 0.10            |

TAREA = Drainage area, in square miles or square kilometer

STRKR: Initial value of STRKR in in/h or (mm/h) for HEC exponential rain loss rate function.

STRKS: Initial value of STRKS in in/h or (mm/h) for HEC exponential snowmelt loss rate function.

RTIOK: Rate of change of the snowmelt loss rate parameter.

DLTKR: Amount of initial accumulated rain loss during which the loss coefficient is increased (inches or mm).

SA: Available soil moisture capacity (inches or mm).

STRTL: Initial rainfall abstraction for snow-free ground (inches or mm).

CRVNBR: SCS curve number for rainfall/snowmelt losses on snow-free ground.

4. Step 3 is repeated. This results in adjustments to all of the optimization parameters.

5. Step 3 is then repeated for the parameter that most improved the value of the objective function in its last change. This is continued until no single change in any parameter yields a reduction of the objective function of more than one percent.

6. One more complete search of all parameters is made.

7. A final adjustment to the infiltration parameters is made to adjust the computed hydrograph volume to within one percent of the observed hydrograph volume. Note that this leads to a small change in the objective function from optimal.

Additional information is displayed comparing computed and observed hydrograph statistics, which are defined as follows:

|                                        |                                                                                                        |
|----------------------------------------|--------------------------------------------------------------------------------------------------------|
| <b>Standard Error:</b>                 | The root mean squared sum of the difference between observed and computed hydrographs                  |
| <b>Objective Function:</b>             | The weighted root mean squared sum of the difference between observed and computed hydrographs         |
| <b>Average Absolute Error:</b>         | The average of the absolute value of the differences between observed and computed hydrographs         |
| <b>Average Percent Absolute Error:</b> | The average of absolute value of percent difference between observed and computed hydrograph ordinates |

### 3.11.2.3 Application of the Calibration Capability of HEC-1

Due to the varying quantity of data available for precipitation-runoff analysis, the exact sequence of steps in application of the automatic calibration capability of HEC-1 varies from study to study. An often-used strategy employs the following steps when using the complete exponential loss rate equation (HEC, 1990):

1. For each storm selected, determine the baseflow and recession parameters that are event dependent. These are not included in the set of parameters that can be estimated automatically. These parameters are the recession flow for antecedent runoff ( $STRTO$ ), the discharge at which recession flow begins ( $QRCSN$ ), and the recession coefficient that is the ratio of flow at some time to the flow one hour later ( $RTIOR$ ).

2. For each storm at each gage, determine the optimum estimates of all unknown unit hydrograph and loss rate parameters using automatic calibration

3. If  $ERAIN$  is to be estimated, select a regional value of  $ERAIN$ , based on analysis of the results of Step 2 for all storms for the representative gages.

4. Using the optimization scheme, estimate the unknown parameters with  $ERAIN$  now fixed at the selected value. Select an appropriate regional value of  $RTIOL$  if  $RTIOL$  is unknown. If the temporal and spatial distribution of precipitation is not well defined, an initial loss followed by a uniform loss rate may be appropriate. If these values are used, as they often are in studies accomplished by HEC, steps 2, 3, and 4 are omitted.

5. With  $ERAIN$  and  $RTIOL$  fixed, estimate the remaining unknown parameters using the optimization scheme. Select a value of  $STRKR$  for each storm being used for calibration. If parameter values for adjacent basins have been determined, check the selected value for regional consistency.

6. With  $ERAIN$ ,  $RTIOL$ , and  $STRKR$  fixed, use the parameter estimation algorithm to compute all remaining unknown parameters.  $DLTKR$  can be generalized and fixed if desired at this point, although this parameter is considered to be relatively event-dependent.

7. Using the calibration capability of HEC-1, determine values of  $TC+R$  and  $R/(TC+R)$ . Select appropriate values of  $TC+R$  for each gage. In order to determine  $TC$  and  $R$ , an average value of  $R/(TC+R)$  is typically selected for the region.

8. Once all parameters have been selected, the values should be verified by simulating the response of the gaged basins to other events not included in the calibration process.

### 3.11.3 Routing Parameters

HEC-1 may also be used to automatically derive routing criteria for certain hydrologic routing techniques. Criteria can be derived for the Tatum, Straddle-Stagger and Muskingum routing methods only.

Inputs to this method are observed inflow and outflow hydrographs and a pattern local inflow hydrograph for the river reach. The pattern hydrograph is used to compensate for the difference between observed inflow and outflow hydrographs. The assumed pattern hydrograph can have a significant effect on the optimized routing criteria.

Observed hydrographs are reconstituted to minimize the squared sum of the deviations between the observed hydrograph and the reconstituted hydrograph. The procedure used is essentially the same as that used for in the unit hydrograph and loss rate parameters (HEC, 1990).

## 3.12 Assessment of Model Reliability

### 3.12.1 Requirements

When using computer models of watershed for extrapolation, hydrologists, engineers, and planners are very interested in the reliability of model output. Model-output reliability can also be used to assess model verification and the selection of appropriate models (Melching & Yen & Wenzel, 1991). Operational forecasts are greatly enhanced by estimates of the probability that discharges or volumes will exceed various levels (Georgakakos, 1986). Risk-cost design procedures require estimates of the probability that various target discharges, volumes, or stages will be exceeded. Beven (1989) has reviewed the limitations of the current generation of distributed, physical- simulation models of watershed hydrology and has suggested that a possible way forward requires a realistic estimation of prediction uncertainty

(Beven, 1989). Therefore, estimation of the reliability of model output is important to hydrologic engineering and water-resources planning.

Beck (1987) noted that uncertainties affect primarily four problem areas that engineers and scientists must address:

1. uncertainty about model structure, i.e., what basic processes are involved, how they interact, and how these processes and interactions can be mathematically characterized in an efficient and parsimonious manner;
2. uncertainty in the model parameters, i.e., parameter identification and calibration problems;
3. uncertainty associated with estimates of the future behavior of the system, i.e., aggregation of uncertainties in model structure, model parameters, and in the definition of the design or decision scenario into overall estimation uncertainty; and
4. reduction of critical modeling uncertainties via carefully designed experiments and/or monitoring programs (Beck, 1987, pp.1393-1441).

### **3.12.2 Uncertainties That Affect Model Reliability**

#### **3.12.2.1 Natural Uncertainties**

The analysis of natural uncertainties is typically limited to the magnitude of uncertainties prevailing in the data which are used to develop the watershed model. Hydrologic frequency analysis also assumes that hydrologic processes are stationary and independent in allowing natural uncertainties to be considered as a purely random probabilistic processes. Time series analysis techniques assume that there are underlying processes causing variations, e.g., trends, periodicities, and correlations, in flow data series around which natural randomness causes additional variation. The underlying processes are represented by linear or nonlinear relations (trends),

harmonic analysis or seasonal standardization (periodicities), and autocorrelation and moving averages; and the natural randomness is represented by an error term which is independent, has a mean of zero, follows a designated PDF, and is additive.

The reliability analysis techniques take an approach similar to that of time-series analysis in that the underlying process is described by the deterministic model, and the additional, natural uncertainties are handled separately. That is, natural uncertainties affect the input data, model-parameter, and the model structure uncertainties. The uncertainties in the input data, model parameters, and model structure propagate through the model such that a probability distribution for the output from a deterministic model can be obtained.

### **3.12.2.2 Data Uncertainties**

The effects of data uncertainties must be considered in different ways depending on:

1. whether the model parameters are determined from calibration or from physical measurements and principles; and,
2. whether the model is used to estimate real events (flood forecasting and recreation of historic flows).

For calibrated models, it was reported that the bias (systematic errors and uncertainty) in recorded rainfall at each gage location is compensated by the curve-fitting of the model to adjust parameters. Thus, the bias in the amount of recorded rainfall affects the resulting fitted-parameter values, rather than the accuracy of the fit of simulated to observed runoff (Dawdy & Lichty & Bergman, 1972).

For synthetic rainfall events, the uncertainties of the basic rainfall data have been incorporated into the stochastic or probabilistic structure of the procedures used to establish the design storms or synthetic rainfall series. Thus, the synthetic event

uncertainties may be modeled by the confidence intervals on rainfall frequency analysis or the random term in temporal/spatial stochastic models of storm rainfall.

### 3.12.2.3 Uncertainties in Model Parameters

The best parameter values obtained provide useful information regarding the parameter and propagated data uncertainties although there are identification uncertainties. In fact, Beck (1987) points out that:

*In simple terms the result of the whole process of identification and the consequences of a lack of identifiability are crystallized in the a posteriori parameter estimation error variance-covariance matrix....Qualitatively, well (or poorly) estimated parameters will have relatively low (or high) error variances; parameters associated with redundant hypotheses should be highly uncertain; and parameters associated with multiple hypotheses, the collective effects of which cannot be disentangled, should yield significant off-diagonal covariances in the matrix.*

The reliability analysis either explicitly (weighting coefficients) or implicitly (multiple simulations) considers output sensitivity to the parameters, which partially compensates for some parameter-identification problems.

### 3.12.2.4 Uncertainties in Model Structure

Model structure uncertainty stems from the inability to truly represent the watershed's physical runoff process in model simulation. The uncertainties introduced by the approximations used in the model affect the prediction of different runoff features; peak discharge, time to peak, runoff volume, etc.; in different ways and to different degrees. For example, the time to peak of a runoff event is much less sensitive to uncertainties in the abstraction model than those in the runoff volume. Thus, it is difficult to consider and account for the total effect of model-structure uncertainty on the complete estimated runoff hydrograph. Therefore, it is necessary to

focus on the most important outputs from the model for the problem being studied and to develop a simple, multiplicative correction factor,  $\lambda_m$  between optimal model performance and the true value of the output of interest (Melching, 1995, pp.69-85).

Garen and Burges (1981) pointed out that if a historical flow record representative of a wide range of watershed responses and of sufficient length to constitute a statistically significant sample is available, comparison of simulated and recorded flows might be extrapolated to estimate uncertainty in subsequent model estimates. For a physical simulation model, the variance between the measured discharge and the estimated discharge using the true input and model parameters mainly results from model structure inadequacies. For calibrated models, the variance between the measured discharge and estimated discharge stems from model-structure inadequacies, input-data and model-parameter uncertainties. Fortunately, the error compensation properties of the calibration process minimize the bias in the estimate resulting from error laden data and parameters allowing a reasonable estimate of the effects of model-structure inadequacies. Therefore, the expected value and the standard deviation of the difference between measured and calibrated values for the runoff characteristics of interest taken over a wide range of storms provides a reasonable estimate of  $\lambda_m$  and the uncertainty. This approach was used by Melching (1987) and Melching et al. (1990, 1991) in their reliability analysis of peak discharge estimates made by HEC-1 and RORB (Melching, 1995, pp.69-85).

### **3.13 Computer Requirements for HEC-1**

#### **3.13.1 Hardware Requirements**

The HEC-1 program is written in ANSI standard FORTRAN77 and requires 633 thousand bytes of core storage. Disk storage is needed for the 16 output and scratch files used by the program.

HEC-1 is also available for microcomputers (PC's). The PC version has all the same capabilities as the mainframe version (HEC, 1990). The MS-DOS version of the

program is distributed for the PC in two forms: (1) the standard program, which permits up to 300 ordinates for each computed hydrograph, and (2) a large array version which permits up to 2000 ordinates to be used. The standard version occupies 475 Kbytes of random-access memory (RAM) on the PC. It can run with or without a math coprocessor chip (8087, 80287, or 80387). The large array version requires extended memory for the PC (2.5Mbytes of RAM). This version requires a 80386 system with a math coprocessor or an 80486 central processing unit (CPU). The program source code is also available for installation on PC, mainframe, or minicomputer systems. HEC-1 is also available for Macintosh systems.

The personal computer version of HEC-1 is generally supplied as part of a package that includes a menu program with which execution of HEC-1 and associated tasks can be performed, an editor for creating and editing input files, and a utility program for viewing program output (DeVries & Hromadka, 1993, p. 21.8).

### **3.13.2 Software Characteristics**

The programs distributed with HEC-1 package provide the necessary tools to accomplish all the basic steps for a rainfall-runoff simulation on the personal computer (PC). A menu program, MENU 1, has been developed to provide program users convenient access to the HEC-1 package of programs and related files, when operating a PC with a hard disk.

Text editors can be used to create or modify an input data file for the HEC-1 program. The Corps editor, COED, has been developed with some features specifically designed around the HEC-1 format for computer program input. COED will place the input data in the format expected by HEC-1 as the data file is created. It also has help information for HEC-1 input data.

An interactive data input program (HEC1IN) has been developed for new to intermediate users of the HEC-1 program. This program leads the user through a

sequence of formatted screens and tables that describe the watershed and the type of hydrologic processes to be used in the computations. After all the screens and tables are filled out, the program creates a skeleton HEC-1 input file. This file will contain all of the data records required to simulate the rainfall-runoff process, but not all of the data. It is the users' responsibility to edit the file (using COED or any other text editor) and fill in the necessary data associated with each record.

HEC-1 is available from program vendors who supply the compiled program or source code and also provide various degrees of program support. HEC-1 is available directly from the Hydrologic Engineering Center only to U.S government agencies. A list of program vendors is available from the Hydrologic Engineering Center, 609 Second St., Davis, CA 95616 (HEC, 1990).

### **3.14 Application Procedure**

The basic steps in rainfall-runoff simulation by HEC-1 include:

1. Gathering data (topographic maps, precipitation and stream flow data, aerial photos, soils and land use information, etc.),
2. Estimate model parameters (unit hydrograph, loss rate and routing parameters, etc.),
3. Develop HEC-1 input data file representing the watershed and rainfall-runoff,
4. Simulate the flood event by executing HEC-1,
5. Review and evaluate model results as compared to observed information,
6. Calibrate model parameters to obtain the best fit for several observed events,
7. Use calibrated model for design or analysis purposes.

### **3.15 Trial Runs of HEC-1 Program Package**

#### **3.15.1 Example Problems**

Before attempting to apply the HEC-1 model to a real case to be presented in this study, two example problems are solved to test the run of the model. Both problems are discussed by Chow & Maidment & Mays (1988, pp. 507-547). For the first problem, both the inputs and the outputs are given together with selected values of model parameters and output hydrographs. The model is run with these given inputs and parameter values, and the resulting hydrographs are found to coincide with those given by Chow et al. (1988). The second example is defined by the same authors as a problem to be solved, where only the inputs and parameter values are given. The model is also run for this case and the results are found to be quite reasonable. In the following, these two trial runs of HEC-1 are explained in the case of two example problems.

#### **3.15.2 Example Problem1**

##### **3.15.2.1 Inputs for the Problem**

In this problem, a rainfall runoff model using HEC-1 computer program is developed for the Castro Valley Creek catchment, shown in Figure 3.9, in order to analyze the effects of urbanization on floods. The catchment is divided into four subcatchments: a schematic diagram of the watershed is shown in Figure 3.10 to describe the basic components to be considered in HEC-1 analysis.

Subcatchment 4 is described to undergo urbanization through development of a new residential area, and a detention reservoir in subcatchment 4 and downstream channel modifications are investigated, the purpose of which is to reduce the effects of the additional flow resulting from the development. The objective of the problem is to calculate the runoff hydrograph at the catchment outlet for three different conditions: (1) the existing conditions throughout the catchment, (2) the existing condition in

subcatchments 1 to 3 with subcatchment 4 urbanized, and (3) the same as (2) but with a modified channel and a reservoir in subcatchment 4.

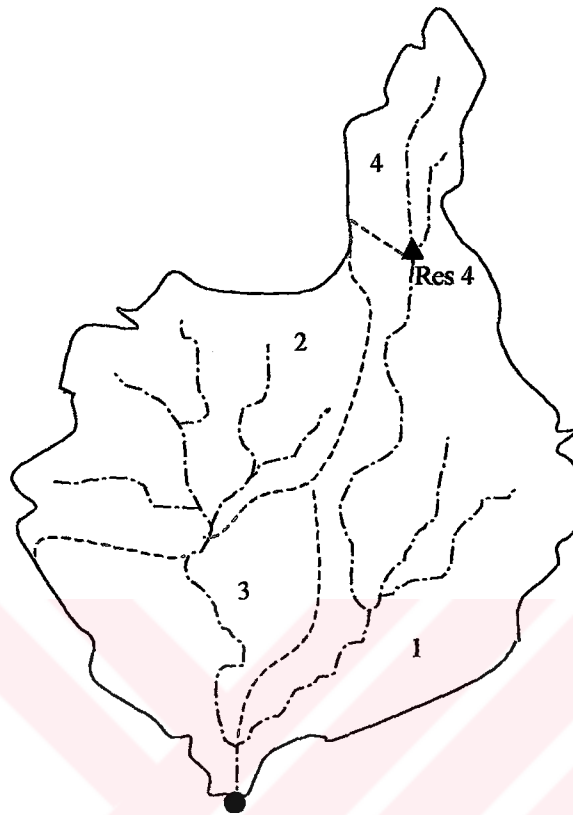


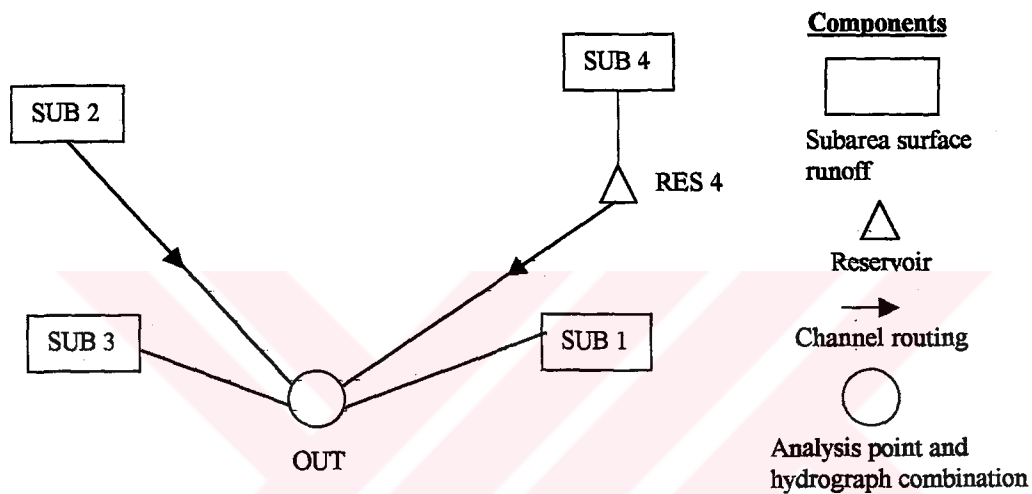
**Figure 3.9 The Castro Valley Watershed**

Subarea runoff computations are performed using Snyder's synthetic unit hydrograph with rainfall loss rates determined by the SCS curve number method. Channel routing is carried out by the Muskingum method and routing through the reservoir by the level-pool method.

The following table presents the existing characteristics of the subcatchments. The total watershed area is 5.51mi<sup>2</sup>.

**Table 3.8 Characteristics of Subcatchments in the Castro Valley Watershed**

| Subcatchment | Area<br>(mi <sup>2</sup> ) | Watershed<br>Length<br>L (mi) | Length to<br>Centroid<br>L <sub>CA</sub> (mi) | SCS Curve<br>Number<br>CN |
|--------------|----------------------------|-------------------------------|-----------------------------------------------|---------------------------|
| 1            | 1.52                       | 2.65                          | 1.40                                          | 70                        |
| 2            | 2.17                       | 1.85                          | 0.68                                          | 84                        |
| 3            | 0.96                       | 1.13                          | 0.60                                          | 80                        |
| 4            | 0.86                       | 1.49                          | 0.79                                          | 70                        |

**Figure 3.10 Schematic Diagram of Castro Valley Watershed Showing Components of HEC-1 Analysis**

The parameters for Snyder's synthetic unit hydrograph for the existing conditions are given as  $C_p = 0.25$  and  $C_t = 0.38$ . The flood wave travel time (Muskingum coefficient  $K$ ) for the stream reach passing through subarea 3 is estimated as 0.3 h, and the travel time for subarea 1 is estimated as 0.6 h. The Muskingum  $X$  has been approximated as 0.2 for each of the two stream reaches.

The design rainfall is a hypothetical 100-year-return period storm defined by the following depth-duration data.

| Duration      | 5 min | 15 min | 1 h  | 2 h  | 3 h  | 6 h  | 12 h | 24 h |
|---------------|-------|--------|------|------|------|------|------|------|
| Rainfall (in) | 0.38  | 0.74   | 1.30 | 1.70 | 2.10 | 3.00 | 5.00 | 7.00 |

A residential development in subcatchment 4 is expected to increase the impervious area so that the developed SCS curve number becomes 85. The unit hydrograph parameters are anticipated to change to  $C_p = 0.5$  and  $C_t = 0.19$ . Modification of the channel through subcatchment 1 will change its Muskingum routing parameters to  $K = 0.4$  h and  $X = 0.3$ . Then the detention reservoir to be constructed at the outlet of subcatchment 4 is described to have the characteristics described in Table 3.9.

**Table 3.9 Characteristics of the Detention Reservoir at the Outlet of Subcatchment 4**

| Low-level outlet                    |                       | Reservoir capacity<br>(acre.ft) | Elevation (ft above MSL) |
|-------------------------------------|-----------------------|---------------------------------|--------------------------|
| Diameter<br>ft                      | 5                     | 0                               | 388.5                    |
| Cross-sectional area                | 19.63 ft <sup>2</sup> | 6                               | 394.2                    |
| Orifice coefficient                 | 0.71                  | 12                              | 398.2                    |
| Centerline elevation<br>(above MSL) | 391 ft                | 18                              | 400.8                    |
| Overflow spillway (ogee type)       |                       | 23                              | 401.8                    |
| Length<br>ft                        | 30                    | 30                              | 405.8                    |
| Weir coefficient                    | 2.86                  |                                 |                          |
| Crest elevation<br>ft (above MSL)   | 401.8                 |                                 |                          |

### 3.15.2.2 Solution

The parameters used for Snyder's unit hydrograph in HEC-1 are  $t_p$  and  $C_p$ ;  $t_p$  is calculated by using equation (22) for the existing condition with  $C_t = 1.0$ , and  $C_t$ ,  $L$ , and  $L_{CA}$  as given above. For example, for subcatchment 1,

$$t_p = C_t(L * L_{CA})^{0.3} = 0.38(2.65 * 1.40)^{0.3} = 0.56 \text{ h}$$

The results of these calculations for the four subcatchments are given in Table 3.10.

**Table 3.10 Parameters for Snyder's Unit Hydrograph**

|       | Subcatchment |      |      |      |             |
|-------|--------------|------|------|------|-------------|
|       | 1            | 2    | 3    | 4    | 4 urbanized |
| $C_t$ | 0.38         | 0.38 | 0.38 | 0.38 | 0.19        |
| $t_p$ | 0.56         | 0.41 | 0.34 | 0.40 | 0.20        |

The HEC-1 input for the Castro Valley Creek catchment is shown below. Here, the use of HEC-1's multiplan option enables the runoff hydrographs for all three conditions to be calculated in one computer run. Plan 1 represents existing conditions, Plan 2 has subcatchment 4 urbanized, and Plan 3 introduces the reservoir and channel modifications.

#### *HEC-1 INPUT*

ID CASRO VALLEY CREEK CATHMENT

ID CONSIDER EXISTING CONDITIONS, DEVELOPED CONDITIONS AND

ID DEVELOPED CONDITIONS WITH IMPROVEMENTS

\*DIAGRAM

IT 5 0 0 289

IO 2 2

JP 3

\* PLAN1 =EXISTING CONDITIONS

\* PLAN2 =URBANIZED CONDITIONS

\* PLAN3 =URBANIZED CONDITIONS WITH IMPROVEMENTS

KK SUB4

KM RUNOFF COMPUTATIONS FOR SUBCATHMENT 4

PH 1 5.51 0.38 0.74 1.30 1.70 2.10 3.00 5.00 7.00

BA 0.86

LS 0 70

US 0.40 0.25

KP 2

LS 0 85

US 0.20 0.5

KP 3

LS 0 85

US 0.20 0.5

KK RES4

KM ROUTE SUB4 THROUGH RESERVOIR

K 1

RN

KP 2

RN

KP 3

RS 1 STOR 0

SV 0 6 12 18 23 30

SE 388.5 394.2 398.2 400.8 401.8 405.8

SL 391 19.63 0.71 0.5

SS 401.8 30 2.86 1.5

KK OUT

KM ROUTE SUBCATHMENT 4 RUNOFF TO OUTLET

KP 1

RM 1 0.6 0.2

KP 3

RM 1 0.4 0.3

KK SUB1

KM RUNOFF COMPUTATIONS FOR SUBCATHMENT 1

BA 1.52

LS 0 70

US 0.56 0.25

```

KK OUT
KM COMBINE SUBCATHMENT 1 RUNOFF WITH SUBCATHMENT
RUNOFF ROUTED TO OUTLET
HC 2
KK SUB2
KM RUNOFF COMPUTATION FOR SUBCATHMENT 2
BA 2.17
LS 0 84
US 0.41 0.25
KK OUT
KM ROUTE SUBCATHMENT 2 RUNOFF TO OUTLET
RM 1 0.3 0.2
KK OUT
KM COMBINE HYDROGRAPHS AT OUTLET
HC 2
KK SUB3
KM RUNOFF COMPUTATIONS FOR SUBCATHMENT 3
BA 0.96
LS 0 80
US 0.34 0.25
KK OUT
KM COMBINE HYDROGRAPHS AT OUTLET
HC 2
ZZ

```

Each component operation begins with a KK card. The input is set up so that the runoff from subcatchment 4 is determined first, then routing through the proposed detention reservoir is performed, followed by the Muskingum routing through subcatchment 1. Next, the rainfall-runoff computation is performed for subcatchment 1 and the resulting runoff hydrograph added to the runoff hydrograph from subcatchment 4. Next, rainfall-runoff computations are performed for subcatchment 2,

and this runoff is routed through subcatchment 3 and added to the outlet hydrograph. The final step is to perform the rainfall-runoff computations for subcatchment 3 and to add this result to the outlet hydrograph.

The resulting runoff hydrographs at the outlet of subcatchment 4 and at the outlet of the entire catchment for each of the three plans shown in Figures 3.11 and 3.12, respectively. The peak discharge from subcatchment 4 under existing conditions is 271 cfs, and under urbanized conditions, 909 cfs. The detention reservoir reduces the peak discharge to 482 cfs. The peak water surface elevation in the reservoir is 402.88 ft above mean sea level (MSL) at time 12.67 h. The peak discharges at the outlet are 1906 cfs for existing conditions, 2258 cfs for urbanized conditions, and 2105 cfs for urbanized conditions with the reservoir and channel modifications.

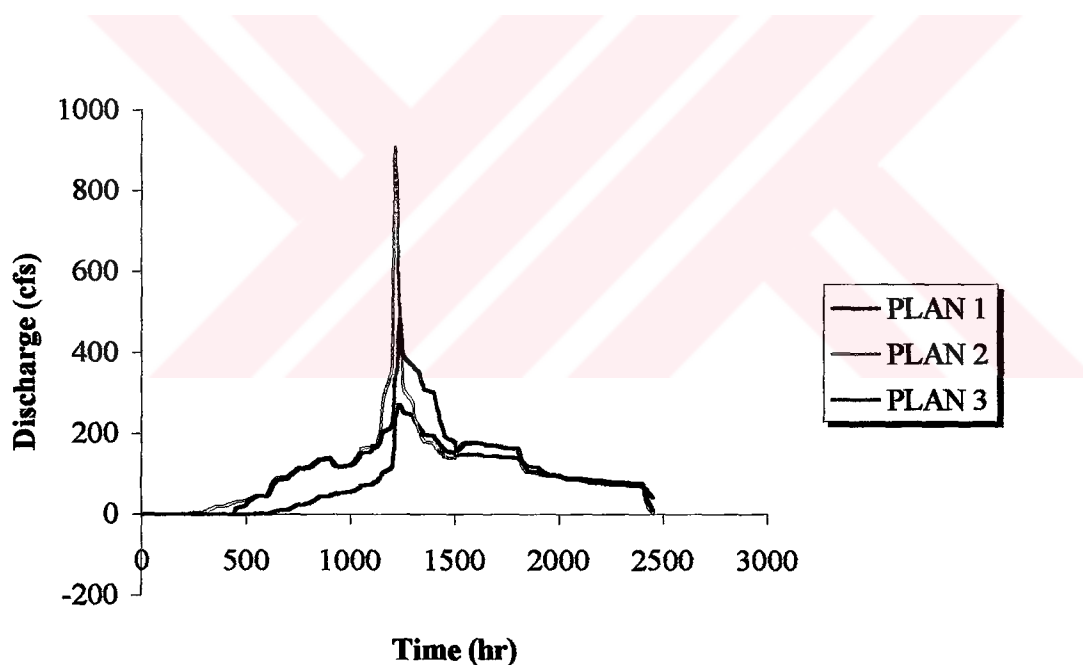


**Figure 3.11 Hydrographs at RES 4**

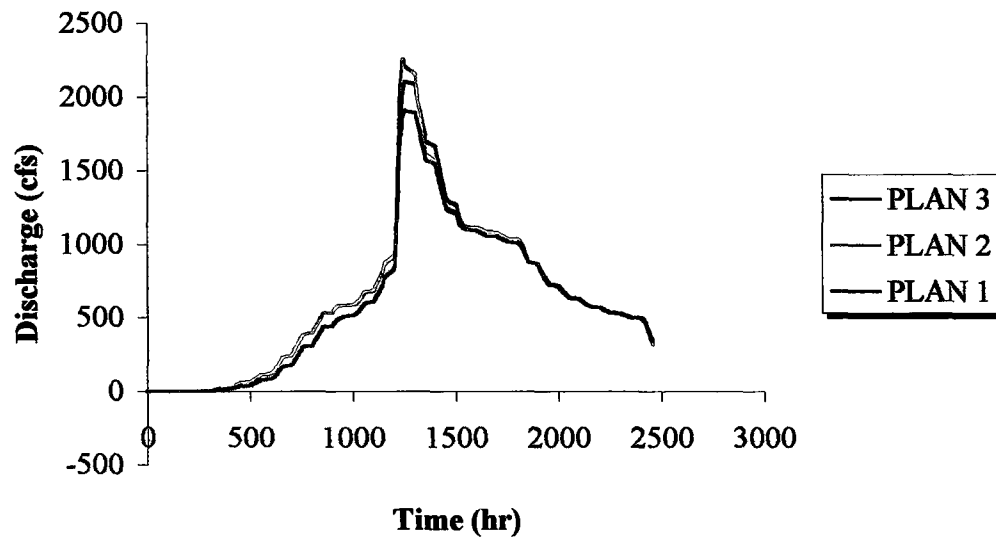


**Figure 3.12 Hydrographs at Outlet of the Watershed**

Figures 3.11 and 3.12 and the numerical results obtained as a result of this run have agreed pretty well with those given by Chow et al. (1988) so that, upon completion of this problem, it is considered that the HEC-1 model available in this study can be used in a real case study.

### **3.15.3 Example Problem 2**

#### **3.15.3.1 Problem Definition**

The problem in the second example is to determine the runoff hydrograph for the watershed shown in Figure 3.13. The watershed is divided into two subcatchments, A and B, with the following characteristics given in Table 3.11.

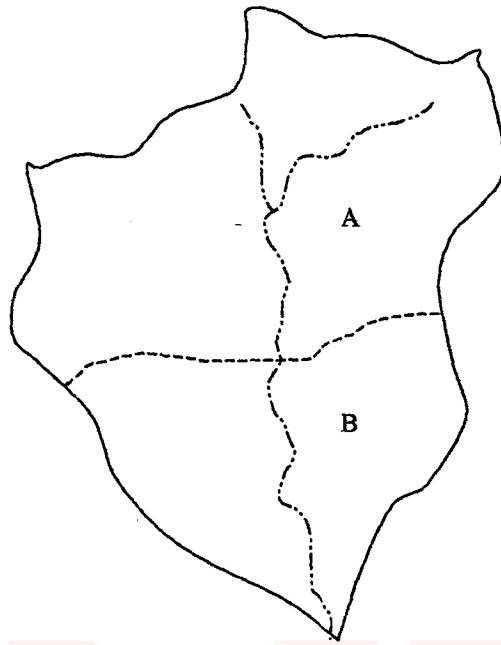


**Figure 3.13 Watershed for Example Problem 2**

**Table 3.11 Characteristics of the Sample Watershed**

| Subcatchment | Area<br>(mi <sup>2</sup> ) | Watershed<br>Length<br>L (mi) | Length to<br>centroid<br>L <sub>CA</sub> (mi) | SCS curve<br>Number<br>CN |
|--------------|----------------------------|-------------------------------|-----------------------------------------------|---------------------------|
| A            | 2.17                       | 1.85                          | 0.68                                          | 70                        |
| B            | 0.96                       | 1.13                          | 0.60                                          | 75                        |

In this problem, the modeler is asked to:

a) use Snyder's synthetic unit hydrograph with  $C_p = 0.25$  and  $C_t = 0.38$  for both subcatchments. Muskingum routing is to be used for flow through subcatchment B with  $K = 0.3h$  and  $X = 0.25$ . 15-minute unit hydrograph is to be considered and use the 12-hour storm defined by the following depth-duration data is to be used.

| Duration      | 15 min | 1 h  | 2 h  | 3 h  | 6 h  | 12 h |
|---------------|--------|------|------|------|------|------|
| Rainfall (in) | 0.74   | 1.30 | 1.70 | 2.10 | 3.00 | 5.00 |

b) solve the same problem for urbanized conditions, with the area developed so that the curve numbers are 85 for subcatchment A and 90 for subcatchment B. The Muskingum K for the channel through subcatchment B is now 0.2h. The Snyder parameters  $C_p$  and  $C_t$  change to 0.35 and 0.30, respectively.

c) using the urbanized conditions in (b), determine the runoff hydrograph in the same input file with (a) and (b) if a following detention reservoir is placed at the outlet of subcatchment B, with characteristics given in Table 3.12.

**Table 3.12 Characteristics of The Detention Reservoir**

| Low-level outlet                     |           | Reservoir capacity | Elevation      |
|--------------------------------------|-----------|--------------------|----------------|
| Diameter                             | 5ft       | (acre.ft)          | (ft above MSL) |
| Orifice coefficient                  | 0.71      | 0                  | 388.5          |
| Centerline elevation                 | 391 ft    | 6                  | 394.2          |
|                                      | above MSL | 12                 | 398.2          |
| <b>Overflow spillway (ogee type)</b> |           | 18                 | 400.8          |
| Length                               | 30ft      | 23                 | 401.8          |
| Weir coefficient                     | 2.86      | 30                 | 405.8          |
| Crest elevation                      | 400ft     |                    |                |
|                                      | Above MSL |                    |                |

### 3.15.3.2 Solution

In the following, HEC-1 model runs and their results are given for parts a, b, and c of the problem defined.

a)

**HEC-1 INPUT FILE FOR EXISTING CONDITIONS**

ID EXAMPLE

IT 15 0 0 49

IO 2 2

KK SUB1

KM RUNOFF COMPUTATIONS FOR SUBCATCHMENT 1

PH 1 3.13 0 0.38 0.74 1.30 1.70 2.10 3.00 5.00

BA 2.17

LS 0 70

US 0.41 0.25

KK OUT

KM ROUTE SUBCATCHMENT 1 RUNOFF TO-OUTLET

RM 1 0.30 0.25

KK SUB2

KM RUNOFF COMPUTATION FOR SUBCATCHMENT 2

BA 0.96

LS 0 75

US 0.34 0.25

KK OUT

KM ROUTE SUBCATCHMENT 2 RUNOFF TO-OUTLET

RM 1 0.30 0.25

KK OUT

KM COMBINE HYDROGRAPHS AT OUTLET

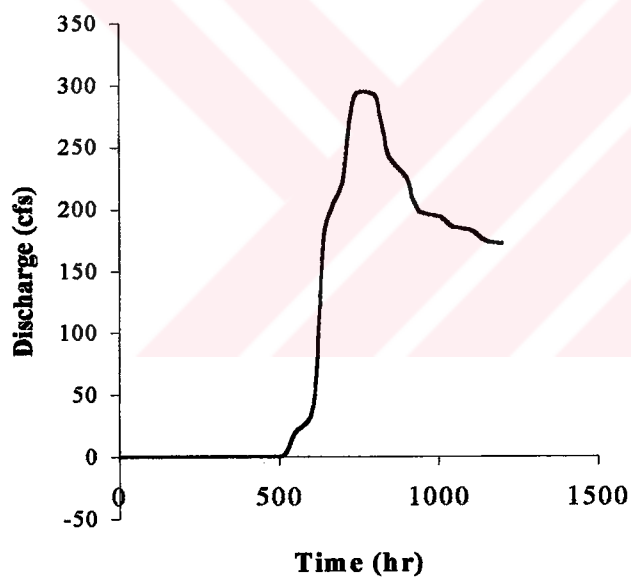
HC 2

ZZ

## **RESULTS**

### **RUNOFF SUMMARY** **FLOW IN CUBIC FEET PER SECOND** **TIME IN HOURS, AREA IN SQUARE MILES**

| OPERATION<br>STATION | PEAK<br>FLOW | TIME OF<br>PEAK | AVERAGE FLOW FOR MAXIMUM PERIOD |         |         | BASIN<br>AREA |
|----------------------|--------------|-----------------|---------------------------------|---------|---------|---------------|
|                      |              |                 | 6-HOUR                          | 24-HOUR | 72-HOUR |               |
| HYDROGRAPH AT SUB1   | 186.         | 7.50            | 132.                            | 67.     | 67.     | 2.17          |
| ROUTED TO OUT        | 181.         | 7.75            | 127.                            | 64.     | 64.     | 2.17          |
| HYDROGRAPH AT SUB2   | 115.         | 7.50            | 79.                             | 41.     | 41.     | 0.96          |
| ROUTED TO OUT        | 112.         | 7.75            | 77.                             | 39.     | 39.     | 0.96          |
| 2 COMBINED AT OUT    | 294.         | 7.75            | 205.                            | 103.    | 103.    | 3.13          |



b)

### **HEC-1 INPUT FILE FOR URBANIZED CONDITIONS**

ID EXAMPLE

ID CONSIDER URBANIZED CONDITIONS

IT 15 0 0 49

KK SUB1

KM RUNOFF COMPUTATIONS FOR SUBCATCHMENT 1

PH 1 3.13 0 0.38 0.74 1.30 1.70 2.10 3.00 5.00

BA 2.17

LS 0 85

US 0.32 0.25

KK OUT

KM ROUTE SUBCATCHMENT 1 RUNOFF TO OUTLET

RM 1 0.30 0.25

KK SUB2

KM RUNOFF COMPUTATION FOR SUBCATCHMENT 2

BA 0.96

LS 0 90

US 0.27 0.25

KK OUT

KM ROUTE SUBCATCHMENT 2 RUNOFF TO OUTLET

RM 1 0.20 0.25

KM COMBINE HYDROGRAPHS AT OUTLET

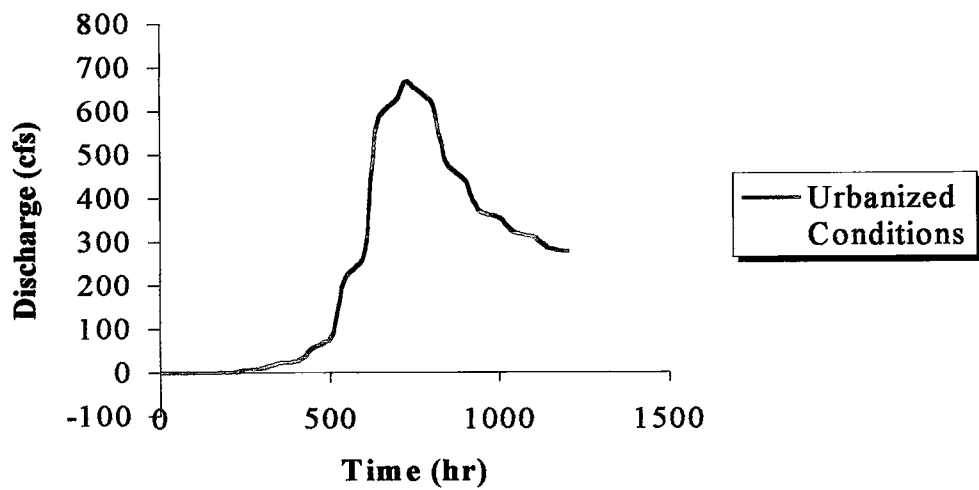
HC 2

ZZ

## RESULTS

### RUNOFF SUMMARY FLOW IN CUBIC FEET PER SECOND TIME IN HOURS, AREA IN SQUARE MILES

| OPERATION<br>STATION | PEAK<br>FLOW | TIME OF<br>PEAK | AVERAGE FLOW FOR MAXIMUM PERIOD |         |         | BASIN<br>AREA |
|----------------------|--------------|-----------------|---------------------------------|---------|---------|---------------|
|                      |              |                 | 6-HOUR                          | 24-HOUR | 72-HOUR |               |
| HYDROGRAPH AT SUB1   | 437.         | 7.25            | 291.                            | 158.    | 158.    | 2.17          |
| ROUTED TO OUT        | 432.         | 7.75            | 289.                            | 154.    | 154.    | 2.17          |
| HYDROGRAPH AT SUB2   | 241.         | 7.25            | 157.                            | 89.     | 89.     | 0.96          |
| ROUTED TO * OUT      | 240.         | 7.50            | 157.                            | 88.     | 88.     | 0.96          |
| 2 COMBINED AT OUT    | 671.         | 7.50            | 446.                            | 241.    | 241.    | 3.13          |



c)

### *HEC-1 INPUT FILE FOR 3 PLAN*

ID EXAMPLE PROBLEM 3

\*DIAGRAM

IT 15 0 0 49

JP 3

IO 2 2

\* PLAN1 =EXISTING CONDITIONS

\* PLAN2 =URBANIZED CONDITIONS

\* PLAN3 =URBANIZED CONDITIONS WITH IMPROVEMENTS

KK SUB1

KM RUNOFF COMPUTATIONS FOR SUBCATCHMENT 1

PH 1 3.13 0 0.38 0.74 1.30 1.70 2.10 3.00 5.00

BA 2.17

LS 0 70

US 0.41 0.25

KP 2  
 LS 0 85  
 US 0.32 0.25  
 KK OUT  
 KM ROUTE SUBCATCHMENT 1 RUNOFF TO OUTLET  
 KP 1  
 RM 1 0.30 0.25  
 KK SUB2  
 KM RUNOFF COMPUTATION FOR SUBCATCHMENT 2  
 BA 0.96  
 LS 0 75  
 US 0.34 0.25  
 KP 2  
 LS 0 90  
 US 0.27 0.25  
 KK OUT  
 KM ROUTE SUBCATCHMENT 2 RUNOFF TO OUTLET  
 KP 1  
 RM 1 0.30 0.25  
 KP 2  
 RM 1 0.20 0.25  
 KK OUT  
 KM COMBINE HYDROGRAPHS AT OUTLET  
 HC 2  
 KK RES2  
 KM ROUTE HYDROGRAPH THROUGH RESERVOIR  
 KP 1  
 RN  
 KP 2  
 RN  
 KP 3

RS 1 STOR 0  
 SV 0 6 12 18 23 30  
 SE 388.5 394.2 398.2 400.8 401.8 405.8  
 SL 391 19.63 0.71 0.5  
 SS 400 30 2.86 1.5  
 ZZ

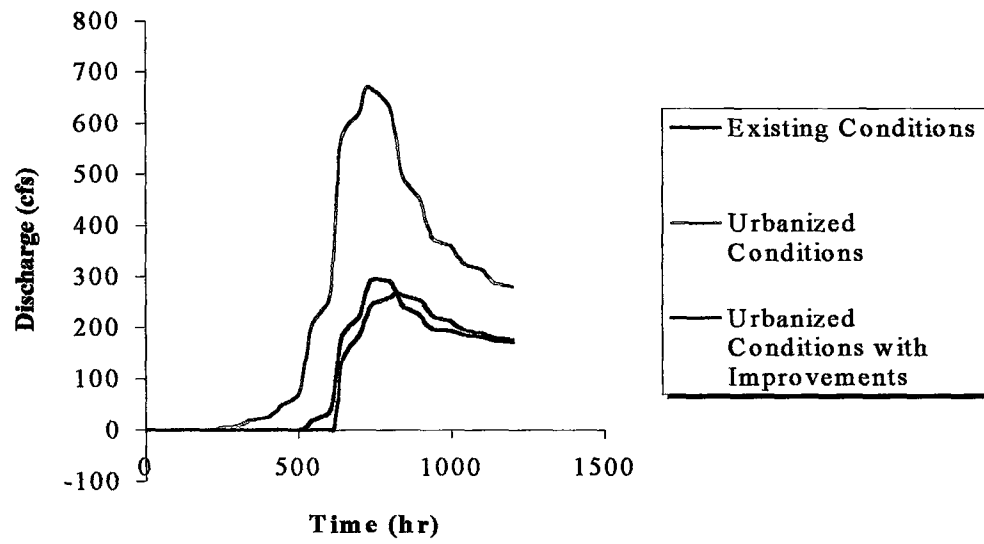
## RESULTS

### RUNOFF SUMMARY FLOW IN CUBIC FEET PER SECOND TIME IN HOURS, AREA IN SQUARE MILES

| OPERATION STATION | BASIN<br>AREA | PLAN<br>NO | PEAK<br>FLOW | TIME OF PEAK |
|-------------------|---------------|------------|--------------|--------------|
| HYDROGRAPH AT+    | SUB1 2.17     | 1 FLOW     | 186.         | TIME 7.50    |
|                   |               | 2 FLOW     | 437.         | TIME 7.25    |
|                   |               | 3 FLOW     | 186.         | TIME 7.50    |
| ROUTED TO +       | OUT 2.17      | 1 FLOW     | 181.         | TIME 7.75    |
|                   |               | 2 FLOW     | 432.         | TIME 7.75    |
|                   |               | 3 FLOW     | 181.         | TIME 7.75    |
| HYDROGRAPH AT+    | SUB2 0.96     | 1 FLOW     | 115.         | TIME 7.50    |
|                   |               | 2 FLOW     | 241.         | TIME 7.25    |
|                   |               | 3 FLOW     | 115.         | TIME 7.50    |
| ROUTED TO +       | OUT 0.96      | 1 FLOW     | 112.         | TIME 7.75    |
|                   |               | 2 FLOW     | 240.         | TIME 7.50    |
|                   |               | 3 FLOW     | 112.         | TIME 7.75    |
| 2 COMBINED AT +   | OUT 3.13      | 1 FLOW     | 294.         | TIME 7.75    |
|                   |               | 2 FLOW     | 671.         | TIME 7.50    |
|                   |               | 3 FLOW     | 294.         | TIME 7.75    |
| ROUTED TO +       | RES2 3.13     | 1 FLOW     | 294.         | TIME 7.75    |
|                   |               | 2 FLOW     | 671.         | TIME 7.50    |
|                   |               | 3 FLOW     | 267.         | TIME 8.50    |

#### \*\* PEAK STAGES IN FEET \*\*

|         |        |
|---------|--------|
| 1 STAGE | .00    |
| TIME    | .00    |
| 2 STAGE | .00    |
| TIME    | .00    |
| 3 STAGE | 396.72 |
| TIME    | 8.50   |



---

## CHAPTER FOUR

### BASIN SELECTED FOR THE CASE STUDY

---

#### 4.1 Basin Characteristics

The application of the HEC-1 model to a real case requires simultaneous measurements of rainfall and runoff for a particular storm event so that the model can be calibrated for the watershed analyzed. Furthermore, pluviograph records are needed as precipitation inputs.

Several basins have been investigated to select one that fulfills the above requirements. Unfortunately, among the data available in almost all basins considered, simultaneous measurements of rainfall and runoff could not be found. The only basin that satisfied the above conditions was the Muteferrik Marmara Basin although precipitation records still had some shortcomings. As a result, this basin was selected as the test case for purpose of the study.

Muteferrik Marmara Basin is located in northwest Turkey and is bordered by the Marmara Sea (Figure 4.1). It extends between 24-48 west meridians and 40-42 north parallels. The total area of the basin is 2094.2 km<sup>2</sup>.

#### 4.2 Available Data

The topographic map of the basin, with the scale of 1:100000 was obtained from EIE (Electrical Works Authority). No data were available on soil types and land cover of the basin. Accordingly, in selection of curve numbers in the SCS method for



Figure 4.1 Mutferrik Marmara Basin

flood routing, the topographic map of the basin and information available in literature were used.

There are 8 streamgaging stations in the basin, the characteristics of which are given in Table 4.1. The existing 6 precipitation stations are described in Table 4.2 (DSI, 1990, pp. XXV-1, XXV-16). The locations of these stations are indicated in Figure 4.1.

**Table 4.1 Characteristics of Streamgaging Stations in the Muteferrik Marmara Basin**

| DSI Basin Number | Station Number | Observing Agency | Stream Name & Station Name | Drainage Area (km <sup>2</sup> ) | Elevation (m) | Observation Period |
|------------------|----------------|------------------|----------------------------|----------------------------------|---------------|--------------------|
| 25               | 209            | EIE              | Koca Ç., Gürçeşme          | 267.6                            | 48            | 1969-              |
| 25               | 2-069          | DSI              | Kaynarca D., Cihadiye      | 73.70                            | 40            | 1979-              |
| 25               | 2-085          | DSI              | Kocabaş Ç., Uzunalan       | 165.0                            | 141           | 1984-              |
| 25               | 2-064          | DSI              | Çan Ç.-Çan                 | 756.0                            | 65            | 1977-1980          |
| 25               | 2-010          | DSI              | Kocabaş Ç., Akkayrak       | 975.0                            | 29            | 1964-              |
| 25               | 212            | EIE              | Biga Ç., Çınarköprü        | 2094.2                           | 8             | 1953-              |
| 25               | 2-044          | DSI              | Hosap Ç., Taşoluk          | 270.6                            | 75            | 1968-              |
| 25               | 214            | EIE              | Hoşab D., Gümüşçay         | 380.0                            | 15            | 1979-              |

**Table 4.2 Characteristics of the Precipitation Stations in the Muteferrik Marmara Basin**

| DSI Basin Number | Observing Agency | Stream Name & Station Name | Elevation (m) | Observation Period |
|------------------|------------------|----------------------------|---------------|--------------------|
| 25               | DMI              | Biga                       | 25            | 1931-1987          |
| 25               | DMI              | Çan                        | 100           | 1958-1987          |
| 25               | DMI              | Etili                      | 100           | 1969-1987          |
| 25               | DMI              | Gümüşçay                   | 25            | 1967-1980          |
| 25               | DMI              | Gündoğdu                   | 50            | 1966-1986          |
| 25               | DMI              | Yenice                     | 275           | 1956-1987          |

### 4.3 Data Used for Model Calibration

The most important step of a model application is calibration where the model is made to “fit” the basin via estimation of parameters that represent the basin. Calibration is based on the comparison between observed outputs (flows) and model outputs; thus, it requires in the case of HEC-1, an observed flow hydrograph resulting from a given storm. Model verification is realized after calibration, on another set of data observed in a different period.

In the Muteferrik Marmara Basin, two important flood events were observed on January 15, 1981 and November 25, 1988. Very little information is available on the second event; furthermore, other storms after 1988 could not be estimated since the available flow data have lost their homogeneity after the construction of hydraulic structures (dams) in the basin. Accordingly, the storm event of January 15, 1981 is used here for calibration purposes.

For estimation of Muskingum parameters through the automatic calibration facility of HEC-1, flow data were available for routing between the streamgaging stations:

2-44 → 214 , 214 → 212 , 209 → 212 , 2-10 → 212

The data used for this calibration purpose are given in Appendix.

On the other hand, stations 2-69, 2-64 and 2-85 were closed at the time of the selected storm event so that routing parameters between 2-69 → 212 were considered the same those between the neighbouring stations. Further more, the subcatchments represented by these stations were inevitably combined into one subbasin in application of the model.

In the calibration of Snyder’s unit hydrograph parameters, pluviograph records are needed for rainfall gaging stations that represent each subbasin. However, since there is only one rainfall gaging station in the basin (Biga), that has pluviograph records,

the  $T_p$  parameter of Snyder's method is computed by equation (22) of Chapter 3 in the form of:

$$T_p = C_1 * C_t (L * L_{CA})^{0.3}$$

In the above,  $C_1$  is given as 1 in the English system and 0.75 in the metric system; thus it is taken as 0.75 in this study.  $C_t$  and  $C_p$  represent basin characteristics and vary between 0.4-2.2 and 0.56-0.94 respectively. It is given by Bayazit (1995) that  $C_t$  assumes small values while  $C_p$  obtains large values in mountainous areas. Thus, two different values are tested for each of  $C_t$  and  $C_p$  parameters as the results will be given in Chapter 5.

After determination of the above Muskingum and Snyder parameters, the rainfall-runoff relationship has to be calibrated. The major difficulty encountered at this point was that the pluviograph records at Biga station were not readily available. However, the total amounts of rainfall on January 15, 1981 at other stations were given as:

| Station  | Total Precipitation (mm) |
|----------|--------------------------|
| Çan      | 25.4                     |
| Etili    | 29.0                     |
| Gündoğdu | 20.2                     |
| Yenice   | 42.7                     |

These values were obtained from unpublished documents of EİE.

Furthermore, observed maximum precipitation values for the month considered were available as (DSİ, 1990, pp. XXV-1, XXV-16):

| <b>Station</b> | <b>Observed Maximum Precipitation (mm)</b> |
|----------------|--------------------------------------------|
| Biga           | 66.5                                       |
| Çan            | 75.0                                       |
| Etili          | 78.5                                       |
| Gündoğdu       | 59.7                                       |
| Y. İnova       | 30.0                                       |
| Yenice         | 88.9                                       |

Considering the above figures, precipitation values were estimated computational intervals of 2 hrs as the values of the observed hydrograph were available at intervals of 2 hrs.

The above values and parameters estimated were then used to prepare input files for HEC and to run the model as described in the next chapter.

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## CHAPTER FIVE

### CASE STUDY

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#### 5.1 General

The analysis of floods in the Muteferrik Marmara Basin by HEC-1 model comprises the following steps:

- a) preparation of input data and description of the basin;
- b) calibration of the model to select model parameter values;
- c) multiplan analysis, where the calibrated model was run first for existing conditions in the basin (PLAN 1) and then for assumed urban development (PLAN 2) to assess the effects of urbanization.

In the following sections, these steps are explained in detail.

#### 5.2 Preparation of Input Data

The study was initiated by description of the basin on a topographical map (scale: 1:/100000) and definition of the boundary and locations of precipitation and runoff station on the map as in Figure 5.1.

The locations of the precipitation and streamflow gaging stations were specified on the basis of information obtained from EIE and DSI. These initial investigations included also the delineation of the primary and secondary tributaries of the major creeks in the basin.

In the next step, the basin was divided into subcatchments on the basis of the drainage areas pertaining to each streamflow gaging station. Figure 5.2 shows the subcatchments specified.

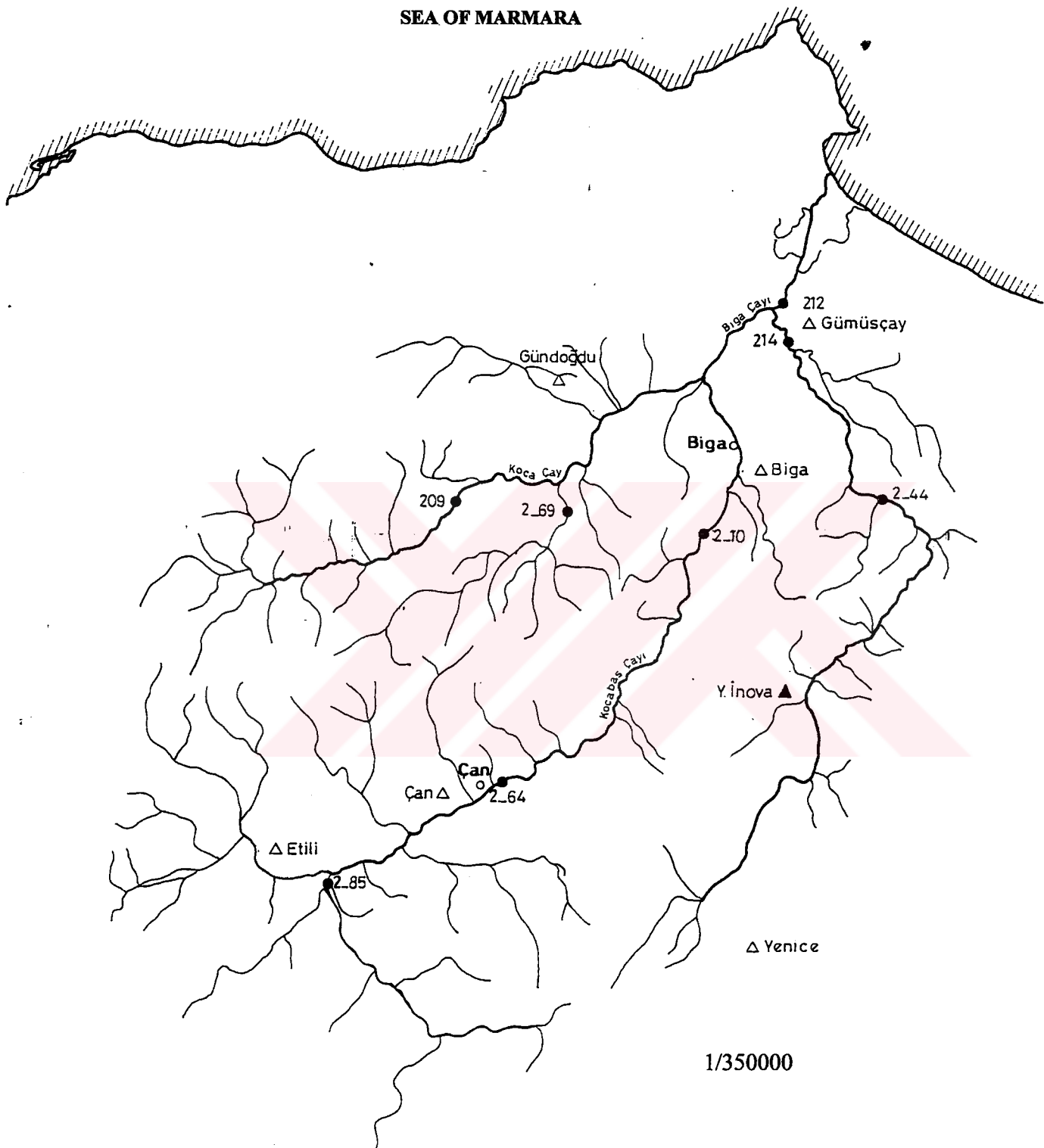


**Figure 5.1 Precipitation and Runoff Stations**

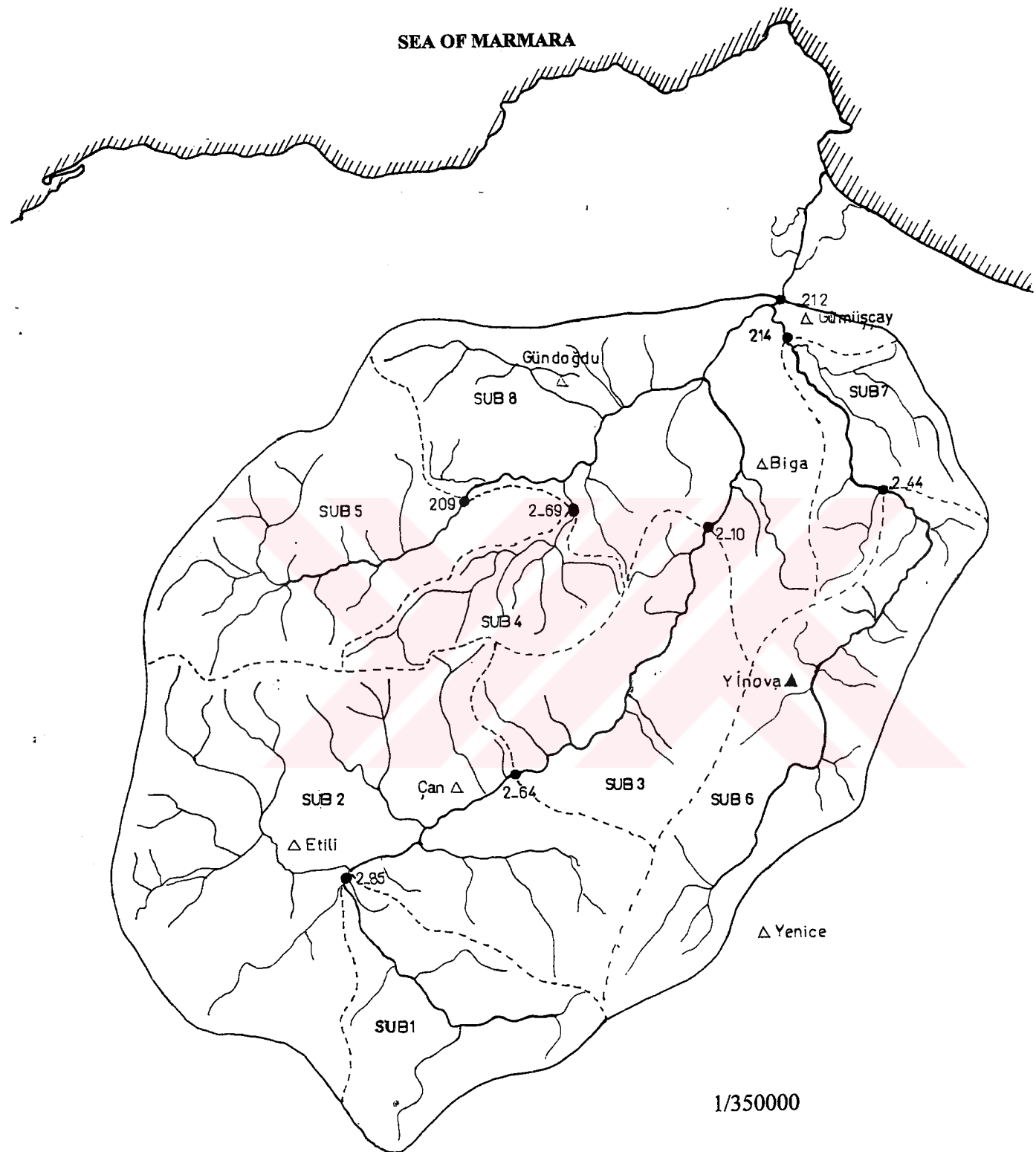


**Figure 5.2 Subcatchments of Mutferrik Marmara Basin**

Further, the basin was schematized as in Figure 5.3 to introduce the basin structure and the subbasins to the model.

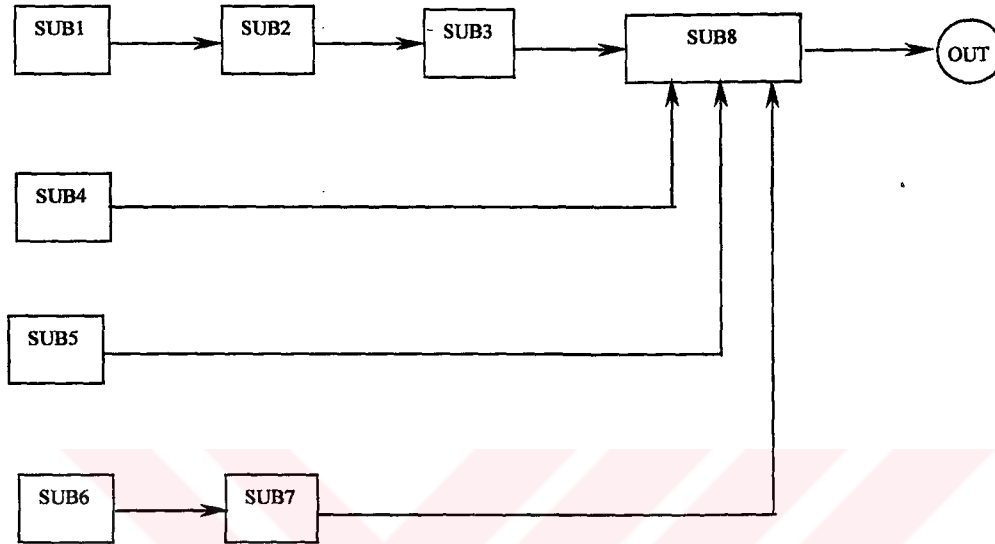


**Figure 5.3 A Schematic Diagram of the Basin**

Next, runoff data were compiled to describe the flood hydrographs resulting from the storm event of January 15, 1981. For stations 2-10 and 2-44, runoff data at 2-hr intervals were obtained from DSI XXV. Regional Directorate. At the time of the storm event, station 2-85 was closed so that no records were available for this station. For the EIE stations, rating curves and hydrographs were provided by the EIE General Directorate, from which runoff data at 2-hr intervals could be derived. At the time of the storm event, EIE stations 264 and 269 were also closed.

As described in Chapter 4, several difficulties were encountered in finding the appropriate precipitation data. The only information available was the total precipitation (mm) at stations Çan, Etili, Gündoğdu and Yenice on the day of the storm. In addition, the monthly total precipitation at 6 gages was available (DSI,

1990). These two pieces of information were used to estimate precipitation values for the simulation interval between January 12, 1981 and January 17, 1981.

### 5.3. Determination of Model Parameters

The next step of study comprised the determination of unit hydrograph parameters and routing parameters, which can be specified via the automatic calibration facility of HEC-1. The Muskingum parameters ( $T_p$  and  $C_p$ ) were determined for runoff stations where streamflow data were available. Using this information, routing parameters were specified between these stations as the following:

| Stations | NSTPS | AMSKK | X    |
|----------|-------|-------|------|
| 209-212  | 9     | 7.29  | 0.14 |
| 2-44-214 | 7     | 0     | 0.01 |
| 214-212  | 9     | 3.33  | 0.30 |
| 2-10-212 | 9     | 0     | 0.03 |

At stations where no runoff data were available, parameter values at neighboring stations were used as for the case of routing between 2-69 and 212. For stations 2-85 and 2-64, subbasins 1 and 2 were considered as a single basin to route the flow to 2-10.

To describe the specification of routing parameters, HEC-1 model run is given below where routing is realized between stations 214 and 212.

#### *HEC-1 INPUT*

```
ID STREAMFLOW ROUTING OPTIMIZATION
ID MUSKINGUM METHOD 214/212
IT 120 13OCT88 2400 15OCT88 2400
IO 1 2
IM
OR 2
```

KK SUB8

|    |     |       |       |      |      |      |      |      |      |     |
|----|-----|-------|-------|------|------|------|------|------|------|-----|
| QP | 102 | 153   | 174   | 214  | 246  | 281  | 312  | 362  | 420  | 474 |
| QP | 529 | 656   | 950   | 1460 | 2040 | 1920 | 1680 | 1360 | 1100 | 924 |
| QP | 798 | 656   | 550   | 498  | 456  |      |      |      |      |     |
| QI | 109 | 109   | 109   | 109  | 114  | 138  | 171  | 219  | 244  | 334 |
| QI | 457 | 503.1 | 469.8 | 414  | 365  | 322  | 262  | 222  | 187  | 155 |
| QI | 125 | 109   | 80    | 80   | 80   |      |      |      |      |     |
| QO | 102 | 153   | 174   | 214  | 246  | 281  | 312  | 362  | 420  | 474 |
| QO | 529 | 656   | 950   | 1460 | 2040 | 1920 | 1680 | 1360 | 1100 | 924 |
| QO | 798 | 656   | 550   | 498  | 456  |      |      |      |      |     |
| RM | -1  | -1    | -1    |      |      |      |      |      |      |     |
| ZZ |     |       |       |      |      |      |      |      |      |     |

The results are shown below and Figure 5.4.

#### DERIVED COEFFICIENTS

| NSTPS | NSTDL | LAG | AMSKK | X  | TSK |
|-------|-------|-----|-------|----|-----|
| 9     | 0     | 0   | 3.33  | .3 | .00 |

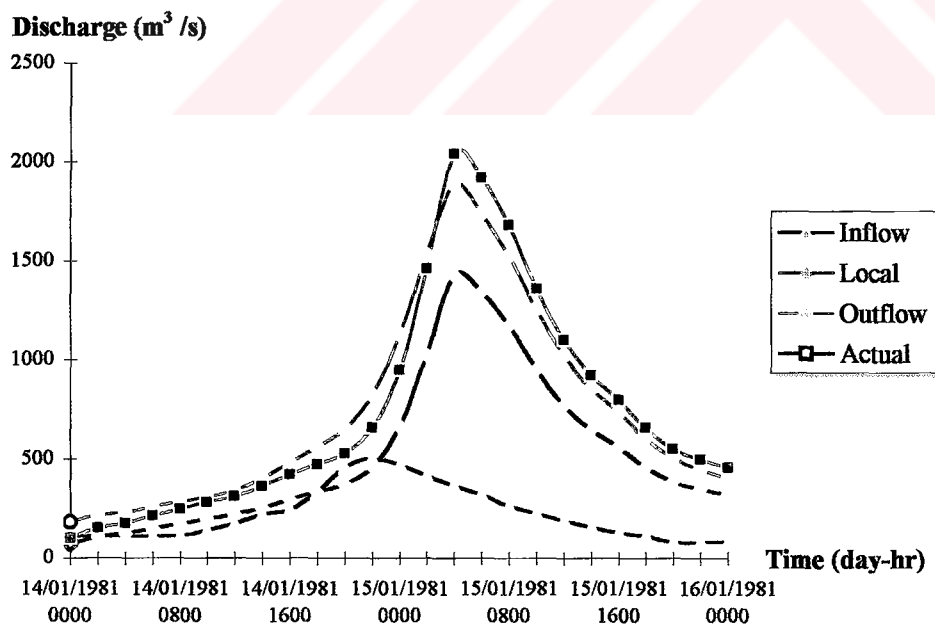


**Figure 5.4 The Resulting Hydrographs for Routing between Stations 214 and 212**

In specification of unit hydrograph parameters, the automatic calibration facility of HEC-1 could not be used since appropriate precipitation data, i.e., pluviograph records, could not be obtained. Instead, equation (22) of Chapter 3 was used to estimate the parameters. In this equation, the  $L$  and  $L_{CA}$  parameters were determined, using the topographical map of the basin. Since  $C_1$  is given as 0.75 for the metric system, it is taken as 0.75 in the computations. As noted in Chapter 4,  $C_t$  varies between 0.4 and 2.2 and  $C_p$  between 0.56 and 0.94. Two model runs are executed to determine the values of these parameters for the Muteferrik Marmara Basin. In the first run,  $C_t$  is taken as 1.5 and  $C_p$  as 0.6. In the second run,  $C_t$  is kept the same while  $C_p$  is changed to 0.8.

On the other hand, to determine the infiltration rates, curve number for a single parameter SCS method was selected on the basis of information obtained from EIE and other sources (e.g., literature values and surveys of basin topographical map). Next, each precipitation gage was given a weight on the basis of the percent area represented by that gage in comparison with the total basin area.

Tables 5.1 and 5.2 display the watershed characteristics and rainfall/runoff data which are input to the model. The data record identifiers used to input each type of data are also indicated in Table 5.2. An important point to note here is that both recording and non-recording gage stations can be used as total-storm stations for a subbasin as specified on the PT, PW card. (The total depth associated with incremental or cumulative rainfall data is automatically calculated for each recording gage). In this example, 25P (Biga) gage was used only for the temporal pattern. The subbasin storm pattern was calculated as a weighted average of the recording gage storm patterns indicated on the PR, PW cards.

**Table 5.1 Existing Characteristics of the Watershed**

| Subbasin | Area<br>(km <sup>2</sup> ) | Watershed Length<br>(km) | Length to<br>Centroid | SCS Curve<br>Number |
|----------|----------------------------|--------------------------|-----------------------|---------------------|
| 1        | 165                        | 18.1                     | 11.25                 | 70                  |
| 2        | 591                        | 25.8                     | 12.5                  | 70                  |
| 3        | 219                        | 23.5                     | 12.75                 | 80                  |
| 4        | 73.7                       | 15.8                     | 7                     | 70                  |
| 5        | 267.6                      | 18                       | 16.25                 | 80                  |
| 6        | 270.6                      | 42.3                     | 16.75                 | 70                  |
| 7        | 109.4                      | 12.9                     | 9.5                   | 80                  |
| 8        | 397.9                      | 19.5                     | 11.3                  | 70                  |

**Table 5.2 Muteferrik Marmara Basin: Rainfall and Observed Hydrograph Data**

| RAINFALL DATA                                                                                                                                                                                                         |                 |  | Record Identifier |
|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------|--|-------------------|
| Total Storm Data                                                                                                                                                                                                      |                 |  |                   |
| Gage #                                                                                                                                                                                                                | Storm Depth(mm) |  | PG                |
| Gündoğdu (1)                                                                                                                                                                                                          | 80.70           |  |                   |
| Çan (3)                                                                                                                                                                                                               | 95.00           |  |                   |
| Etili (4)                                                                                                                                                                                                             | 108.5           |  |                   |
| Yenice (5)                                                                                                                                                                                                            | 120.1           |  |                   |
| Y.İnova (6)                                                                                                                                                                                                           | 110.0           |  |                   |
| Hourly Precipitation Data                                                                                                                                                                                             |                 |  |                   |
| Starting Time : 24:00                                                                                                                                                                                                 |                 |  | IN                |
| Date : January,12,1981                                                                                                                                                                                                |                 |  |                   |
| Station # : Biga (2)                                                                                                                                                                                                  |                 |  |                   |
| Hourly Incremental rainfall:                                                                                                                                                                                          |                 |  | PG, PI            |
| 2, 3, 6, 7, 3, 8, 9, 9, 15, 10, 11, 11, 11, 12, 10, 10, 9, 9,                                                                                                                                                         |                 |  |                   |
| 9, 21.5, 61.25, 70.25, 79.80, 15, 4, 1, 1.8, 2.1, 2, 2, 2.5,                                                                                                                                                          |                 |  |                   |
| 2.5, 3.2, 3.5, 3.5, 3.2, 3.1, 3.1, 3, 3.2, 3.3, 3.5, 3, 4.5,                                                                                                                                                          |                 |  |                   |
| 4.5, 4.5, 4, 4, 5, 5                                                                                                                                                                                                  |                 |  |                   |
| OBSERVED HYDROGRAPH                                                                                                                                                                                                   |                 |  | Record Identifier |
| Station 212                                                                                                                                                                                                           |                 |  |                   |
| Observed flow beginning at same time simulation starts, see input data listing. IN card is required preceding the flow data because the data tabulation interval is different than the previous IN card for rainfall. |                 |  | IN                |
|                                                                                                                                                                                                                       |                 |  | QO                |

Using the input data of Tables 5.1 and 5.2, two runs 7 HEC-1 were realized for different values of  $C_p$  with  $C_t$  being selected as 1.5. The result of these two runs are summarized in the following.

**Run 1 :**

The input file of the first one is given below. The model is run for  $C_t = 1.5$  and  $C_p = 0.6$ . The results of this run are given in Tables 5.3 and 5.4, and the observed hydrograph

***HEC-1 INPUT***

ID MARMARA BASIN

IM

\*DIAGRAM

IO 1 2

IT 120 12JAN81 2400 17JAN81 2400

IN 120 12JAN81 2400

PG 1 80.7

PG 3 95

PG 4 108.5

PG 5 120.1

PG 7 110

PG 2 0

PI 2 3 6 7 3 8 9 9 15 10

PI 11 11 11 12 10 10 9 9 9 21.5

PI 61.25 70.25 79.80 15 4 1 1.8 2.1 2.0 2.0

PI 2.5 2.5 3.2 3.5 3.5 3.2 3.1 3.1 3.0 3.2

PI 3.3 3.5 3.0 4.5 4.5 4.5 4 4 5 5

KK SUB2

KM SCS RUNOFF COMPUTATION

BA 756

BF

PR 2

PW 1  
 PT 3 4 5  
 PW 0.34 0.56 0.10  
 LS 70  
 US 6.63 0.60  
 KK 210

KM ROUTE SUB2 TO 210

RM 7 9.8 0.50

KK SUB3

KM SCS RUNOFF COMPUTATION

BA 219

BF

PR 2

PW 1

PT 3 2 5 7

PW 0.41 0.14 0.04 0.42

LS 80

US 6.23 0.60

KK 210

KM COMBINE HYDROGRAPHS AT 210

HC 2

KK OUTLET

KM ROUTE SUB3 TO OUTLET

RM 9 0 0.03

KK SUB6

KM SCS RUNOFF COMPUTATION

BA 270.6

BF

PR 2

PW 1

PT 2 5 7

PW 0.12 0.33 0.55

LS            70

US    8.07   0.60

KK    214

KM ROUTE SUB6 TO 214

RM     7    0.00   0.01

KK SUB7

KM SCS RUNOFF COMPUTATION

BA 109.4

BF

PR     2

PW     1

PT     2       7

PW 0.965   0.035

LS            80

US    4.76   0.60

KK    214

KM COMBINE HYDROGRAPHS

HC     2

KK OUTLET

KM ROUTE SUB7 TO OUTLET

RM     9    3.33   0.30

KK SUB4

KM SCS RUNOFF COMPUTATION

BA 73.7

BF

PR     2

PW     1

PT     3       1       2

PW    0.62   0.32   0.06

LS            70

US    4.62   0.60

KK OUTLET

## KM ROUTE SUB4 TO OUTLET

RM 9 5.55 0.2

## KK SUB5

## KM SCS RUNOFF COMPUTATION

BA 267.6

BF

PR 2

PW 1

PT 1 3 4

PW 0.48 0.19 0.33

LS 80

US 6.20 0.60

## KK OUTLET

## KM ROUTE SUB5 TO OUTLET

RM 9 7.29 0.14

## KK SUB8

## KM SCS RUNOFF COMPUTATION

BA 397.9

BF 34

PR 2

PW 1

PT 1 2 7

PW 0.57 0.33 0.10

LS 70

US 5.68 0.60

## KK OUTLET

## KM COMBINE HYDROGRAPHS

HC 5

## KK OUTLET

QO 34.2 34.2 34.2 34.2 34.2 34.2 35.9 39.7 44.9 53.1

QO 65.4 85 102 153 174 214 246 281 312 362

QO 420 474 529 656 950 1460 2040 1920 1680 1360

QO 1100 924 798 656 550 498 456 408 376 346  
 QO 324 308 293 276 267 264 264 264 264 264  
 QO 266 267 266 260 251 237 228 216 206 194  
 QO 185 176  
 ZZ

**Table 5.3 Summary of the Run I**

| OPERATION<br>STATION    | PEAK<br>FLOW | TIME OF<br>PEAK | AVERAGE FLOW FOR MAXIMUM<br>PERIOD |         |         | BASIN<br>AREA |
|-------------------------|--------------|-----------------|------------------------------------|---------|---------|---------------|
|                         |              |                 | 6-HOUR                             | 24-HOUR | 72-HOUR |               |
| HYDROGRAPH AT SUB2      | 377.18       | 50.00           | 353.210                            | 205.930 | 101.910 | 756.00        |
| ROUTED TO 210           | 380.37       | 62.00           | 355.340                            | 205.410 | 101.800 | 756.00        |
| HYDROGRAPH AT SUB3      | 315.09       | 50.00           | 287.120                            | 167.310 | 078.860 | 219.00        |
| 2 COMBINED AT 210       | 472.72       | 60.00           | 441.390                            | 339.540 | 174.240 | 975.00        |
| ROUTED TO OUTLE         | 472.72       | 60.00           | 441.390                            | 339.540 | 174.240 | 975.00        |
| HYDROGRAPH AT SUB6      | 260.09       | 52.00           | 241.110                            | 154.060 | 075.820 | 270.60        |
| ROUTED TO 214           | 260.09       | 52.00           | 241.110                            | 154.060 | 075.820 | 270.60        |
| HYDROGRAPH AT SUB7      | 678.58       | 48.00           | 612.500                            | 331.040 | 157.860 | 109.40        |
| 2 COMBINED AT 214       | 882.54       | 50.00           | 817.000                            | 473.630 | 227.790 | 380.00        |
| ROUTED TO OUTLET        | 873.15       | 52.00           | 814.140                            | 473.230 | 227.760 | 380.00        |
| HYDROGRAPH AT SUB4      | 54.36        | 50.00           | 049.600                            | 024.910 | 011.810 | 73.70         |
| ROUTED TO OUTLET        | 50.84        | 56.00           | 047.110                            | 024.780 | 011.790 | 73.70         |
| HYDROGRAPH AT SUB5      | 175.37       | 50.00           | 160.250                            | 091.790 | 043.610 | 267.60        |
| ROUTED TO OUTLET        | 159.26       | 58.00           | 147.370                            | 090.790 | 043.500 | 267.60        |
| HYDROGRAPH AT SUB8      | 824.72       | 50.00           | 741.620                            | 429.480 | 218.660 | 397.90        |
| 5 COMBINED AT<br>OUTLET | 2040.46      | 52.00           | 1911.22                            | 1335.78 | 669.410 | 2094.20       |

**Table 5.4 Comparison of the Computed and Observed Hydrographs**

|                           | SUM OF<br>FLOWS                | EQUIV<br>DEPTH | MEAN<br>FLOW | TIME TO<br>CENTER<br>OF<br>MASS | LAG C.M.<br>TO C.M. | PEAK<br>FLOW | TIME<br>OF<br>PEAK |
|---------------------------|--------------------------------|----------------|--------------|---------------------------------|---------------------|--------------|--------------------|
| COMPUTED<br>HYDROGRAPH    | 27114                          | 93.219         | 444          | 63.72                           | 63.72               | 2040         | 52.00              |
| OBSERVED<br>HYDROGRAPH    | 25338                          | 87.114         | 415          | 64.97                           | 64.97               | 2040         | 52.00              |
| DIFFERENCE                | 1776                           | 6.105          | 29           | -1.25                           | -1.25               | .00          | .00                |
| PERCENT<br>DIFFERENCE     | 7.01                           |                |              |                                 | -1.92               | .02          |                    |
|                           |                                |                |              |                                 |                     |              |                    |
| STANDARD<br>ERROR 126     | AVERAGE ABSOLUTE ERROR         |                |              |                                 | 59.                 |              |                    |
| OBJECTIVE<br>FUNCTION 162 | AVERAGE PERCENT ABSOLUTE ERROR |                |              |                                 | 13.74               |              |                    |

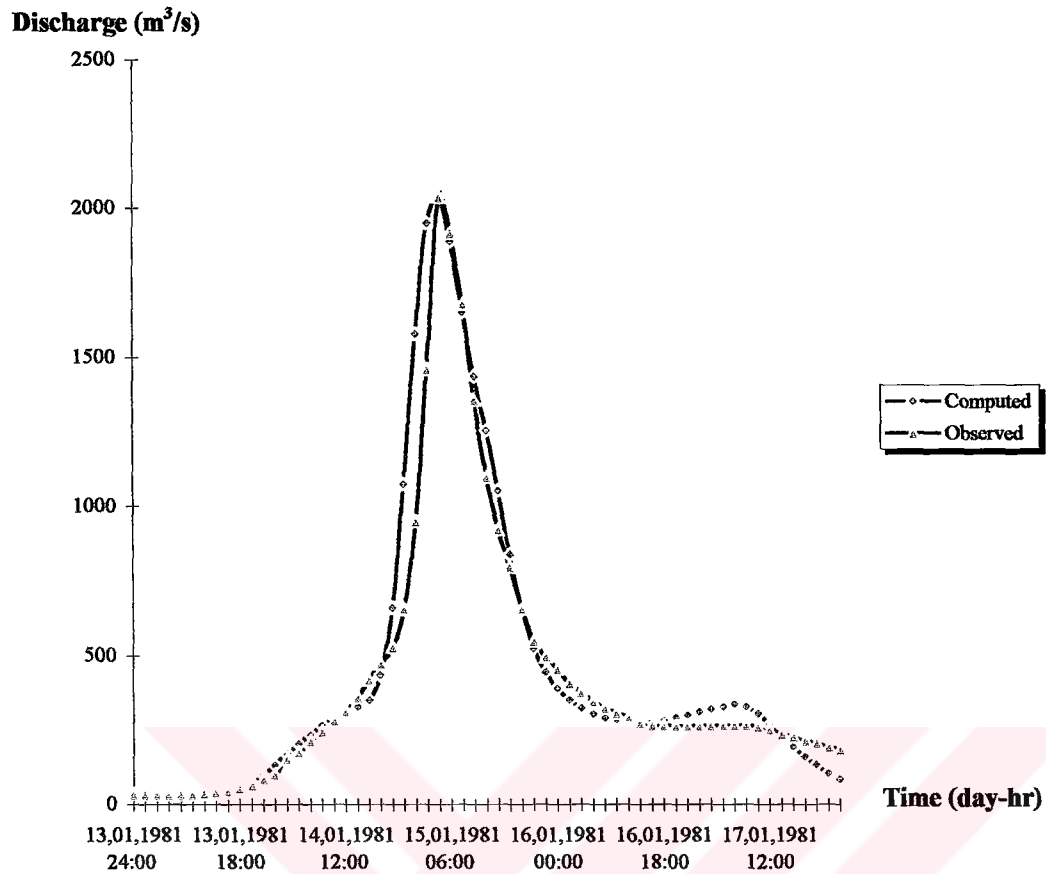


**Figure 5.5 Comparison of Model Output Hydrograph  
and the Observed Hydrograph for Run I**

### ***Run II***

The input file of this run is given below for  $C_t = 1.5$  and  $C_p = 0.8$ . Tables 5.5 and 5.6 summarize the results of this run and Figure 5.6 compares the model output hydrograph with the observed hydrograph.

### ***HEC-1 INPUT***

ID MARMARA BASIN

IM

\*DIAGRAM

IO 1 2

IT 120 12JAN81 2400 17JAN81 2400

IN 120 12JAN81 2400

PG 1 80.7

PG 3 95

PG 4 108.5

PG 5 120.1

PG 7 110

PG 2 0

PI 2 3 6 7 3 8 9 9 15 10

PI 11 11 11 12 10 10 9 9 9 21.5

PI 61.25 70.25 79.80 15 4 1 1.8 2.1 2.0 2.0

PI 2.5 2.5 3.2 3.5 3.5 3.2 3.1 3.1 3.0 3.2

PI 3.3 3.5 3.0 4.5 4.5 4.5 4 4 5 5

KK SUB2

KM SCS RUNOFF COMPUTATION

BA 756

BF

PR 2

PW 1

PT 3 4 5

PW 0.34 0.56 0.10

LS 70

US 6.63 0.80

KK 210

KM ROUTE SUB2 TO 210

RM 7 9.8 0.50

KK SUB3

KM SCS RUNOFF COMPUTATION

BA 219

BF

PR 2

PW 1

PT 3 2 5 7

PW 0.41 0.14 0.04 0.42

LS 80

US 6.23 0.80

KK 210

KM COMBINE HYDROGRAPHS AT 210

HC 2

KK OUTLET

KM ROUTE SUB3 TO OUTLET

RM 9 0 0.03

KK SUB6

KM SCS RUNOFF COMPUTATION

BA 270.6

BF

PR 2

PW 1

PT 2 5 7

PW 0.12 0.33 0.55

LS 70

US 8.07 0.80

KK 214

KM ROUTE SUB6 TO 214

RM 7 0.00 0.01

KK SUB7

KM SCS RUNOFF COMPUTATION

BA 109.4

BF

PR 2

PW 1

PT 2 7

PW 0.965 0.035

LS 80

US 4.76 0.60

KK 214

KM COMBINE HYDROGRAPHS

HC 2

KK OUTLET

KM ROUTE SUB7 TO OUTLET

RM 9 3.33 0.30

KK SUB4

KM SCS RUNOFF COMPUTATION

BA 73.7

BF

PR 2

PW 1

PT 3 1 2

PW 0.62 0.32 0.06

LS 70

US 4.62 0.80

KK OUTLET

KM ROUTE SUB4 TO OUTLET

RM 9 5.55 0.2

KK SUB5

KM SCS RUNOFF COMPUTATION

BA 267.6

BF

PR 2

PW 1

PT 1 3 4

PW 0.48 0.19 0.33

LS 80

US 6.20 0.80

KK OUTLET

KM ROUTE SUB5 TO OUTLET

RM 9 7.29 0.14

KK SUB8

KM SCS RUNOFF COMPUTATION

BA 397.9

BF 34

PR 2

PW 1

PT 1 2 7

PW 0.57 0.33 0.10

LS 70

US 5.68 0.80

KK OUTLET

KM COMBINE HYDROGRAPHS

HC 5

KK OUTLET

QO 34.2 34.2 34.2 34.2 34.2 34.2 35.9 39.7 44.9 53.1

QO 65.4 85 102 153 174 214 246 281 312 362

QO 420 474 529 656 950 1460 2040 1920 1680 1360

QO 1100 924 798 656 550 498 456 408 376 346

QO 324 308 293 276 267 264 264 264 264 264

QO 266 267 266 260 251 237 228 216 206 194

QO 185 176

ZZ

**Table 5.5 Summary of the Run II**

| OPERATION<br>STATION | PEAK<br>FLOW | TIME OF<br>PEAK | AVERAGE FLOW FOR MAXIMUM<br>PERIOD |         |         | BASIN<br>AREA |
|----------------------|--------------|-----------------|------------------------------------|---------|---------|---------------|
|                      |              |                 | 6-HOUR                             | 4-HOUR  | 72-HOUR |               |
| HYDROGRAPH AT SUB2   | 485.80       | 50.00           | 443.140                            | 218.520 | 103.390 | 756.00        |
| ROUTED TO 210        | 485.15       | 60.00           | 443.820                            | 218.380 | 103.380 | 756.00        |
| HYDROGRAPH AT SUB3   | 393.75       | 50.00           | 347.230                            | 177.650 | 079.370 | 219.00        |
| 2 COMBINED AT 210    | 519.71       | 60.00           | 476.740                            | 369.950 | 175.510 | 975.00        |
| ROUTED TO OUTLE      | 519.71       | 60.00           | 476.740                            | 369.950 | 175.510 | 975.00        |
| HYDROGRAPH AT SUB6   | 330.87       | 52.00           | 304.240                            | 167.290 | 076.870 | 270.60        |
| ROUTED TO 214        | 330.87       | 52.00           | 304.240                            | 167.290 | 076.870 | 270.60        |
| HYDROGRAPH AT SUB7   | 829.19       | 48.00           | 716.460                            | 350.180 | 158.490 | 109.40        |
| 2 COMBINED AT 214    | 1074.60      | 48.00           | 984.000                            | 500.670 | 229.060 | 380.00        |
| ROUTED TO OUTLET     | 1074.67      | 52.00           | 965.650                            | 501.370 | 229.020 | 380.00        |
| HYDROGRAPH AT SUB4   | 68.13        | 48.00           | 059.890                            | 025.440 | 011.920 | 73.70         |
| ROUTED TO OUTLET     | 62.16        | 54.00           | 055.600                            | 025.430 | 011.900 | 73.70         |
| HYDROGRAPH AT SUB5   | 222.33       | 50.00           | 195.110                            | 096.610 | 044.070 | 267.60        |
| ROUTED TO OUTLE      | 195.06       | 58.00           | 178.880                            | 096.300 | 043.940 | 267.60        |
| HYDROGRAPH AT SUB8   | 1013.08      | 50.00           | 915.55                             | 450.490 | 219.950 | 397.90        |
| 5 COMBINED AT OUTL   | 2419.85      | 52.00           | 2286.50                            | 1407.20 | 673.270 | 2094.20       |

**Table 5.6 Comparison of Computed and Observed Hydrographs**

|                        | SUM OF<br>FLOWS | EQUIV<br>DEPTH | MEAN<br>FLOW | TIME TO<br>CENTER<br>OF<br>MASS | LAG C.M.<br>TO C.M. | PEAK<br>FLOW | TIME OF<br>PEAK |
|------------------------|-----------------|----------------|--------------|---------------------------------|---------------------|--------------|-----------------|
| COMPUTED<br>HYDROGRAPH | 27232           | 93.626         | 446          | 61.73                           | 61.73               | 2420         | 52.00           |
| OBSERVED<br>HYDROGRAPH | 25338           | 87.114         | 415          | 64.97                           | 64.97               | 2040         | 52.00           |
| DIFFERENCE             | 1894            | 6.511          | 31           | -3.23                           | -3.23               | 380          | .00             |
| PERCENT<br>DIFFERENCE  | 7.47            |                |              |                                 | -4.98               | 18.62        |                 |
|                        |                 |                |              |                                 |                     |              |                 |
| STANDARD ERROR         | 226             |                |              |                                 |                     |              | 113             |
| OBJECTIVE FUNCTION     | 300             |                |              |                                 |                     |              | 24.39           |

b) urbanized conditions in all subbasins (PLAN2)

To investigate PLAN 2, the SCS curve number for each subbasin was changed to 85.

The design rainfall is a hypothetical 25-year-return-period storm defined by the following depth-duration data.

| Duration      | 5 min | 15 min | 1 h   | 2 h   | 3 h   | 6 h    | 12 h   | 24 h   |
|---------------|-------|--------|-------|-------|-------|--------|--------|--------|
| Rainfall (mm) | 15.12 | 29.11  | 56.43 | 79.15 | 88.95 | 108.32 | 132.46 | 147.50 |

Both conditions are evaluated in the same run of the model as shown in the HEC-1 input file given below. The results of this multiplan analysis are shown in Table 5.7 and Figure 5.7.

#### HEC-1 INPUT FILE

ID MARMARA BASIN CONSIDER EXISTING CONDITIONS, DEVELOPED  
CONDITIONS

ID AND DEVELOPED CONDITIONS WITH IMPROVEMENTS

\*DIAGRAM

IM

IO 1 2

IT 30 0 0 241

JP 2

\* PLAN1=EXISTING CONDITIONS

\* PLAN2=UNDER URBANIZED CONDITIONS

KK SUB2

KM SCS RUNOFF COMPUTATION

PH 2094.2 15.12 29.11 56.43 79.15 88.95 108.32 132.46 147.50

BA 756

LS 70

US 6.63 0.6

KP 2

LS 0 85

US 6.63 0.6

KK 210

KM ROUTE SUB2 TO 210

RM 7 9.8 0.50

KK SUB3

KM SCS RUNOFF COMPUTATION

BA 219

LS 80

US 6.23 0.6

KP 2

LS 0 85

US 6.23 0.6

KK 210

KM COMBINE HYDROGRAPHS AT 210

HC 2

KK OUT

KM ROUTE SUB3 RUNOFF TO OUT

RM 9 0 0.03

KK SUB6

KM SCS RUNOFF COMPUTATION

BA 270.6

LS 70

US 8.07 0.6

KP 2

LS 0 85

US 8.07 0.6

KK 214

KM ROUTE SUB6 TO 214

RM 7 0.00 0.01

KK SUB7

KM SCS RUNOFF COMPUTATION

BA 109.4

LS 80

US 4.76 0.6

KP 2

LS 0 85

US 4.76 0.6

KK 214

#### KM COMBINE HYDROGRAPHS

HC 2

KK OUT

#### KM ROUTE SUB7 TO OUT

RM 9 3.33 0.30

KK SUB4

#### KM SCS RUNOFF COMPUTATION

BA 73.7

LS 70

US 4.62 0.6

KP 2

LS 0 85

US 4.62 0.6

KK OUTLET

#### KM ROUTE SUB4 TO OUTLET

RM 9 5.55 0.2

KK SUB5

#### KM SCS RUNOFF COMPUTATION

BA 267.6

LS 80

US 6.20 0.6

KP 2

LS 0 85

US 6.20 0.6

KK OUTLET

KM ROUTE SUB5 TO OUTLET

RM 9 7.29 0.14

KK SUB8

KM SCS RUNOFF COMPUTATION

BA 397.9

BF 34

LS 70

US 5.68 0.6

KP 2

LS 0 85

US 5.68 0.6

KK OUTLET

KM COMBINE HYDROGRAPHS

HC 5

ZZ

**Table 5.7 Summary of the Multiplan Analysis**

| OPERATION STATION  | AREA   | PLAN | RATIO 1<br>1.00         |
|--------------------|--------|------|-------------------------|
| HYDROGRAPH AT SUB2 | 756.00 | 1    | FLOW 854<br>TIME 19.50  |
|                    |        | 2    | FLOW 1402<br>TIME 19.00 |
| ROUTED TO 210      | 756.00 | 1    | FLOW 828<br>TIME 29.50  |
|                    |        | 2    | FLOW 1357<br>TIME 28.50 |
| HYDROGRAPH AT SUB3 | 219.00 | 1    | FLOW 368<br>TIME 19.00  |
|                    |        | 2    | FLOW 425<br>TIME 18.50  |
| 2 COMBINED AT 210  | 975.00 | 1    | FLOW 962<br>TIME 29.00  |
|                    |        | 2    | FLOW 1520<br>TIME 28.50 |
| ROUTED TO OUTLET   | 975.00 | 1    | FLOW 962<br>TIME 29.00  |
|                    |        | 2    | FLOW 1520<br>TIME 28.50 |

**Table 5.7 Summary of the Multiplan Analysis (Continued)**

|                      |         |   |                         |
|----------------------|---------|---|-------------------------|
| HYDROGRAPH AT SUB6   | 270.60  | 1 | FLOW 263.<br>TIME 21.00 |
|                      |         | 2 | FLOW 430<br>TIME 20.50  |
|                      |         |   |                         |
| ROUTED TO 214        | 270.60  | 1 | FLOW 263<br>TIME 21.00  |
|                      |         | 2 | FLOW 430<br>TIME 20.50  |
|                      |         |   |                         |
| HYDROGRAPH AT SUB7   | 109.40  | 1 | FLOW 220<br>TIME 17.50  |
|                      |         | 2 | FLOW 254<br>TIME 17.50  |
|                      |         |   |                         |
| 2 COMBINED AT 214    | 380.00  | 1 | FLOW 437<br>TIME 20.00  |
|                      |         | 2 | FLOW 634<br>TIME 19.50  |
|                      |         |   |                         |
| ROUTED TO OUTLET     | 380.00  | 1 | FLOW 434<br>TIME 23.00  |
|                      |         | 2 | FLOW 629<br>TIME 23.00  |
|                      |         |   |                         |
| HYDROGRAPH AT SUB4   | 73.70   | 1 | FLOW 107<br>TIME 17.50  |
|                      |         | 2 | FLOW 177<br>TIME 17.00  |
|                      |         |   |                         |
| ROUTED TO OUTLET     | 73.70   | 1 | FLOW 98<br>TIME 23.50   |
|                      |         | 2 | FLOW 161<br>TIME 23.00  |
|                      |         |   |                         |
| HYDROGRAPH AT SUB5   | 267.60  | 1 | FLOW 450<br>TIME 19.00  |
|                      |         | 2 | FLOW 521<br>TIME 18.50  |
|                      |         |   |                         |
| ROUTED TO OUTLET     | 267.60  | 1 | FLOW 400<br>TIME 26.50  |
|                      |         | 2 | FLOW 464<br>TIME 26.50  |
|                      |         |   |                         |
| HYDROGRAPH AT SUB8   | 397.90  | 1 | FLOW 535<br>TIME 18.50  |
|                      |         | 2 | FLOW 859<br>TIME 18.00  |
|                      |         |   |                         |
| 5 COMBINED AT OUTLET | 2094.20 | 1 | FLOW 1987<br>TIME 27.00 |
|                      |         | 2 | FLOW 2895<br>TIME 27.00 |

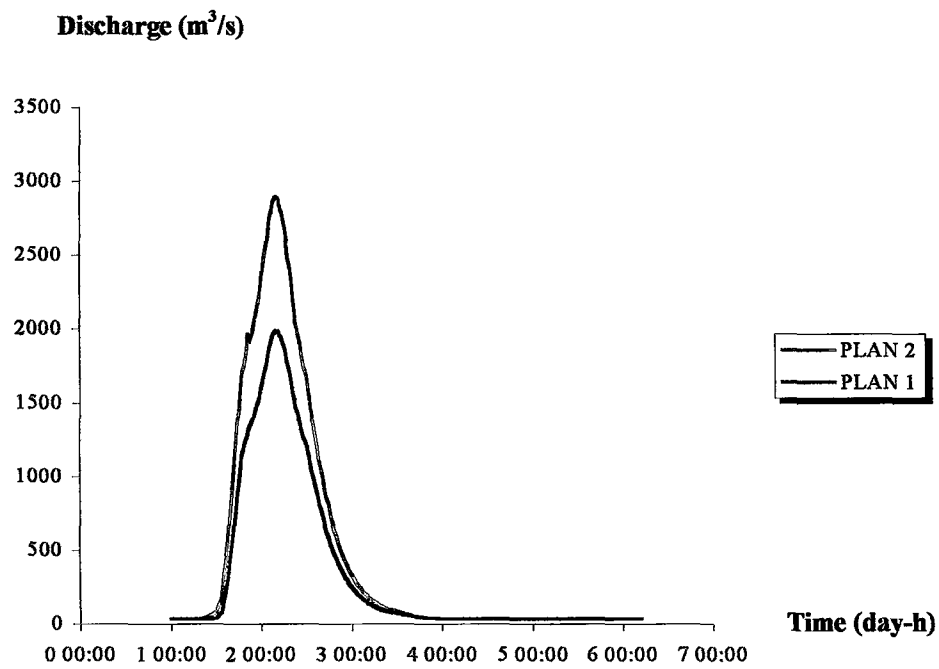


**Figure 5.6 Outlet Hydrographs for Plan I and Plan II**

The results of the multiplan analysis, where urbanized conditions are assumed in all subbasins, have increased the peak flow at the outlet of the basin about %32. The multiplan analysis can be run under different scenarios provided that appropriate data are available to describe each scenario realistically.

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## CHAPTER SIX

# CONCLUSION

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The purpose of this study has been to investigate the use of the HEC-1 model package in the analysis of flood events. Three accomplishments were foreseen through the realization of the study:

- a) understanding of HEC-1 and its capabilities;
- b) getting the model package run under different inputs;
- c) running the model on an actual test case.

The first two accomplishments are tested in two example problems which are available in literature. After the model is found to be working properly, it was applied to an observed storm event in the Muteferrik Marmara Basin.

The study on model features and capabilities has shown that all ordinary flood hydrograph computations associated with a single recorded or hypothetical storm can be accomplished with this package. The result of the modeling process is the computation of streamflow hydrographs at desired locations in a river basin. Furthermore, the following capabilities of the model were noted:

- HEC-1 provides a powerful optimization technique for the estimation of some of the parameters when gaged precipitation and runoff data are available.
- Multiplan-multiflood analysis allows the simulation of several ratios of a design flood for several different plans (or characterization) of a stream network in a single computer run.

- Dam-break simulation provides the capability to analyze the consequences of dam overtopping and structural failures.
- The depth-area option computes flood hydrographs preserving a user-supplied precipitation depth versus area relation throughout a stream network.
- The economic assessment of flood damage can be determined for reaches defined in a multiplan-multiflood analysis. The expected annual damage occurring in a damage reach and the benefits accrued due to a flood control plan are calculated flows for the reach.
- The optimal size of a flood control system can be estimated using an optimization procedure provided by HEC-1. The option utilizes data provided for the economic assessment option together with data on flood control project costs to determine a system which maximizes net benefits with or without a specified degree of protection level for the components.

In application of the model to the actual test case of the Muteferrik Marmara Basin, the basic difficulty encountered was the availability of appropriate rainfall data as input to the flood hydrographs observed for a particular storm event. This difficulty is essentially reflected in calibration of model parameters to “fit” the model to the basin. For that matter, the major conclusion derived as a result of the case study is that simultaneous rainfall and runoff records should be available to simulate a storm event realistically. If such data are not available from past records, a short-term monitoring program appears to be necessary prior to HEC-1 applications if the model is to be used effectively.

Another important conclusion to be drawn is that HEC-1 can be used as an effective decision tool in evaluating and developing flood control measures. This is realized through the multiflood-multiplan analysis facility of the model. In the study, a hypothetical scenario of urbanized conditions in the basin were investigated, the results of which indicated the need for flood control measures. There are particular hydraulic structures in the basin investigated and their effects on flood characteristics of the watershed could be investigated. However, the data required on these structures were not made available for purposes of the study so that the multiplan analysis could not be detailed further. Similarly economic features of different flood

hydraulic structures in the basin investigated and their effects on flood characteristics of the watershed could be investigated. However, the data required on these structures were not made available for purposes of the study so that the multiplan analysis could not be detailed further. Similarly economic features of different flood control measures could not be investigated due to lack of data although this facility of HEC-1 is identified.

Despite all the data-based difficulties encountered in the study, the use of the model is strongly suggested for analysis of floods as it permits multiplan-multiflood decision-making assessments as well as benefit/cost analyses.



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## APPENDIX

**Discharge (m<sup>3</sup>/s)**

| DATE | TIME |   |   |   |   |    |    |    |    |    |    |    |    |
|------|------|---|---|---|---|----|----|----|----|----|----|----|----|
|      | 0    | 2 | 4 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 |

**(2-010) Kocabaş Çayı-Akkayrak**

|            |       |       |       |       |       |       |       |       |       |       |       |       |       |
|------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| 14/01/1981 |       |       |       |       | 21,0  |       |       |       | 46,0  |       |       |       |       |
| 15/01/1981 | 135,0 | 140,0 | 145,0 | 170,0 | 230,0 | 250,0 | 300,0 | 350,0 | 390,0 | 330,0 | 300,0 | 270,0 | 240,0 |
| 16/01/1981 | 240,0 | 230,0 | 195,0 | 170,0 | 140,0 | 130,0 | 115,0 | 105,0 | 96,0  | 93,0  | 89,0  | 86,0  | 86,0  |
| 17/01/1981 | 86,0  | 87,0  | 91,0  | 95,0  | 98,0  | 97,0  | 95,0  | 92,0  | 89,0  | 85,0  | 80,0  | 77,0  | 74,0  |
| 18/01/1981 | 74,0  | 72,0  | 70,0  | 67,0  | 65,0  | 63,0  | 62,0  | 60,0  | 56,0  | 54,0  | 52,0  | 49,0  | 48,0  |

**(2-044) Hoşap Çayı-Taşoluk**

|            |      |      |      |       |       |       |       |       |       |      |      |      |      |
|------------|------|------|------|-------|-------|-------|-------|-------|-------|------|------|------|------|
| 14/01/1981 | 4,0  | 4,0  | 5,3  | 5,7   | 6,6   | 8,2   | 11,0  | 16,0  | 22,0  | 27,0 | 32,0 | 46,0 | 57,0 |
| 15/01/1981 | 57,0 | 58,0 | 82,0 | 127,0 | 225,0 | 337,0 | 340,0 | 180,0 | 141,0 | 93,0 | 70,0 | 54,0 | 43,0 |
| 16/01/1981 | 43,0 | 36,0 | 36,0 | 28,0  | 27,0  | 27,0  | 26,0  | 26,0  | 23,0  | 23,0 | 22,0 | 22,0 | 22,0 |
| 17/01/1981 | 22,0 |      |      |       | 22,0  |       |       |       | 18,5  |      |      |      |      |

**(209) Koca Çay-Gürçeşme**

|            |       |       |       |      |      |      |       |       |       |       |       |       |       |
|------------|-------|-------|-------|------|------|------|-------|-------|-------|-------|-------|-------|-------|
| 14/01/1981 | 19,4  | 22,1  | 28,2  | 39,6 | 57,3 | 82,0 | 111,0 | 138,0 | 179,0 | 206,0 | 220,0 | 190,0 | 157,0 |
| 15/01/1981 | 157,0 | 132,0 | 113,0 | 98,8 | 66,0 | 73,0 | 64,0  | 55,5  | 46,4  | 38,8  | 33,0  | 28,6  | 25,5  |
| 16/01/1981 | 25,5  | 24,3  | 24,7  | 25,1 | 25,1 | 25,1 | 24,7  | 24,3  | 24,3  | 24,3  | 24,3  | 24,3  | 24,7  |

**(212) Biga Çayı-Çınarköprü**

|            |       |       |       |       |       |       |       |       |       |       |       |       |       |
|------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| 13/01/1981 | 34,2  | 34,2  | 34,2  | 34,2  | 34,2  | 34,2  | 35,9  | 39,7  | 44,9  | 53,1  | 65,4  | 85,0  | 102,0 |
| 14/01/1981 | 102,0 | 153,0 | 174,0 | 214,0 | 246,0 | 281,0 | 312,0 | 362,0 | 420,0 | 474,0 | 529,0 | 656,0 | 950,0 |
| 15/01/1981 | 950,0 | 1460  | 2040  | 1920  | 1680  | 1360  | 1100  | 924,0 | 798,0 | 656,0 | 550,0 | 498,0 | 456,0 |
| 16/01/1981 | 456,0 | 408,0 | 376,0 | 346,0 | 324,0 | 308,0 | 293,0 | 276,0 | 267,0 | 264,0 | 264,0 | 264,0 | 264,0 |
| 17/01/1981 | 264,0 | 266,0 | 267,0 | 266,0 | 260,0 | 251,0 | 237,0 | 228,0 | 216,0 | 206,0 | 194,0 | 185,0 | 176,0 |

| DATE | TIME |   |   |   |   |    |    |    |    |    |    |    |    |
|------|------|---|---|---|---|----|----|----|----|----|----|----|----|
|      | 0    | 2 | 4 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 |

## (214) Hoşap Deresi- Gümüşçay

|            |       |       |       |       |       |       |       |       |       |       |       |       |       |
|------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| 14/01/1981 | 109,0 | 109,0 | 109,0 | 109,0 | 114,0 | 138,0 | 171,0 | 219,0 | 244,0 | 334,0 | 457,0 | 503,1 | 469,8 |
| 15/01/1981 | 469,8 | 414,0 | 365,0 | 322,0 | 262,0 | 222,0 | 187,0 | 155,0 | 125,0 | 109,0 | 80,0  | 80,0  | 80,0  |



TC YÜKSEKÖĞRETİM KURULU  
DOKÜMANTASYON MERKEZİ