DESIGN OF STORM WATER DRAINAGE SYSTEM FOR THE CENTER OF DURA CITY

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The Project Entitled:

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In accordance with the recommendation of the project supervisor, and the acceptance of all examining committee members, this project has been submitted to the Department of Civil and Architectural Engineering in the college of Engineering and Technology in partial fulfillment of the requirements of the department for the degree of Bachelor of Engineering.

Project Supervisor	Department Chairman
…………………..	………………………

2005-2006

الإهداء

إلى من سهرت الليالي الطوال............. صاحبة الحنان .... إلى أمي

إلى الماس الذي لا ينكسر................نبع العطاء .... إلى والدي

إلى ملائكة الأرض ....شقائق النعمان.................إلي أشقائي

إلى قناديل الدرب..............الشموع التي لا تنطفئ .... إلى أساتذتي

إلى رفاق الدرب .... بناة المستقبل .... إلى أصدقائي

إلى صناع الكرامة .... رايات المجد .... إلى شهدائنا

إلى من رفضوا الخضوع .... من طلبوا العزة ....إلى أسرانا

إلى أستاذنا ....رمز الوفاء...............إلى الدكتور ماجد أبو شرخ

إلى من ناضل من اجل العلم والتعلم........إلى الأب الراحل د. كمال غطاشة

إليكم جميعا أحبتنا نهدي هذا الجهد المتواضع

فريق العمل

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محمد حوامده

ياسمين دودين

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ABSTRACT

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Supervisor: Dr. Majed Abu sharkh

The wide expansion and accelerated development and grown of Dura city had led to change in the hydrological and geomorphological features and the drainage system had become more complex, hence, the amount of running water has increased, and Dura city has been lied now on more one catchments. At the same time, storm water drainage is not exit and in some streets the water is conveyed to wastewater collection network which unable to meat the present water excess.

The design engineer must conduct the preliminary investigations to develop a layout plan of the sewerage system. Site visits and contacts with the city and local planning agencies and state officials should be made to determine the land use plans, zoning regulations, and probable future changes that may affect both the developed and undeveloped land. Data must be developed on topography, geology, hydrology,

climate, ecological elements, and social and economic conditions. Topographic maps with existing and proposed streets provide the most important information for preliminary flow routing

TABLE OF CONTENTS

CHAPTER 1: INTRODUCTION

CHAPTER 2: STUDY AREA

CHAPTER 3:GENERATING WATERSHEDE

CHAPTER 4: DESIGN OF STORM WATER DRAINAGE SYSTEM

CHAPTER 5: DESIGN AND PLANNING CRITEREA

CHAPTER 6 : SYSTEM DESIGN

CHAPTER SEVEN

LIST OF TABLES

LIST OF FIGURES

CHAPTER ONE

INTRODUCTION

1.1 GENERAL BACKGROUND

1.2 PROBLEM ANALYSIS

1.3 OBJECTIVES OF THE PROJECT

1.4 PROJECT METHODOLOGY

1.5 STAGES OF THE PROJECT

1.6 ORGANIZATION OF THE REPORT

1.7 SUMMARY

CHAPTER ONE

INTRODUCTION

1.1 General background

"Drainage" is the term applied to systems for dealing with excess water. It is important for the disposal of surplus irrigation water, storm water, and wastewater. Water drainage is a natural phenomenon which takes place naturally and depends on the geomorphological and hydrological features. Water drainage is often considered as minor problem, but with rapid increase in population and consequent in all round activities of man, the problem has been accentuated.

The wide expansion and accelerated development Dura city had led to change in the hydrological and geomorphological features and the drainage system had become more complex, hence, the amount of running water has increased .At the same time, storm water drainage is not exit and in some streets the water is conveyed to wastewater collection network which unable to meet the present water excess.

In view of this prevailing condition, the drainage system in Dura city would have a new characteristics and the development of new water drainage water is very necessary to drainage excess water from streets. This study is conducted to design a storm water drainage system for the center of Dura city.

1.2 Problem definition:

Drainage as a mean of disposal, till recently- has been largely a neglected aspect in the West Bank now, water drainage is very important due to water accumulation on the streets as a result of heavy precipitation (running water), population growth, and the development and extension of West Bank cities.

Dura city is located in a semi – arid region with rainfall generally limited to autumn and winter months. In the past, the open areas of much of Dura city easily observed most of this rainfall and provided the primary source for recharging the ground water aquifer.

Most of the areas in Dura city do not have a natural drainage outlet. Heavy rainfall causes storm water to collect in low areas and flood streets and walk ways. Rapid growth has decreased the open areas available for percolation of rainwater and has greatly increased the runoff to low lying areas.

In view of this condition, design of a new storm water drainage system in Dura city become very essential. A new drainage system which admits all the flood discharge from the catchments and with low initial and maintenance cost.

1.3 Objectives

The overall objective of this study is to investigate water drainage system in Dura city and propose storm water drainage system for the city. Achievement of this objective requires estimation of the accumulated areas, the quantities of water, topography of the city, the existing drainage system, etc. More specifically the main objectives of this project are:

1- Study in general, drainage system patterns in Dura city.

2- Determine the sub catchment and catchments of the study area with the help of aerial photograph metric map and Geographical Information System (GIS).

3- Design of a new storm water drainage network for the center of the city.

4- Finally, providing suggestion and recommendations regarding the reuse of collected water at the end of disposal.

1.4 Project methodology

The main tasks, which have been under taken in order to develop this project are as follows:

1. Make some visits to the municipality to discuss the problems, which Dura city faces annually from cumulated water in the streets.
2. Make visit to Meteorological Station of the Hebron city and obtained different information about rainfall, temperature and relative humidity of the study area.
3. Needed maps that show the contour lines, roads, houses and their elevations and the previous studies that contain different information about Dura city were obtained.
5. The necessary hydraulic calculations and other design requirements were carried out .
6. Prepare the necessary report, maps and drawings for the project.

1.5 Stages of the project

The project consists of three stages and is to be completed in accordance with time schedule shown in Table 1.1. The description of each of the three stages of the project and the tasks involved are listed below:

Table 1.1: Stages of the Project with Their Expected Duration

Stages No.	Title	Duration								
		2005			2006					
		10	11	12	01	02	03	04	05	06
One	Data Collection And survey									
Two	Design Of Storm water Drainage									
Three	writing the report									

1.5.1 First Stage: Data collection and survey

Available data and information were collected from different sources. Moreover, many visits to municipality and meteorological station were done. This phase includes the following tasks:

1. Collection of aerial and topographical maps for the project area.
2. Collection of meteorological and hydrological data (rainfall, temperature, wind speed, evaporation …etc.) from different sources.

1.5.2 Second Stage: Design of storm water collection system

In this stage, the necessary hydraulic calculations needed for the design of the main trunk will be carried out. This stage includes the following tasks:

1. Determine the catchments and sub-catchments areas and routes of the storm water pipes.
2. Establish a system layout, which includes the areas that are going to be served, existing streets and roads, topography ….etc.
3. Establish the design criteria and conducting the needed hydraulic calculations.
4. Preparing needed drawings for the designed storm water pipes.

1.5.3 Third Stage: Writing the report

After finishing the design calculations of the main trunks, and preparing the different drawings .

1.6 Organization of the project

The study report has been prepared in accordance with the objectives of work. The report consists of seven chapters.

The first chapter entitled "Introduction" describes the background of the project, problem statement, project objectives, project methodology, and the summary of the chapter.

The second chapter entitled "The Study Area" describes geography of the project area, topography, climate, land use, road network, and the summary of the chapter.

The third chapter entitled "Generating Watershed", this chapter describes how to prepare watershed of study area using arcview GIS.

The fourth chapter entitled "Design of Storm Drainage System" describes the storm water runoff, hydraulic consideration; design of storm water sewers, and the summary of the chapter.

The fifth chapter entitled "Design and Planning Criteria" describes introduction, catchments areas, rainfall characteristics, runoff flow, design parameters, and the summary of the chapte.

The sixth chapter entitled "System Design" describes introduction, layout of the system, design computations, the proposed storm water drainage system, profiles of drainage channels, and the summary of the chapter.

The seventh chapter entitled "Conclusions" discusses the conclusions of the study.

1.7 Summary

In this chapter, the problem of storm water drainage in Dura city area has been identified and the necessity of establishing and design storm water drainage system to the area has been pointed. The objectives of the present study have been brought out and the procedures to achieve these objectives have been discussed. At the end, the structure of the final report of this study has been presented.

CHAPTER TWO

THE STUDY AREA

2.1 INTRODUCTION

2.2 GEOGRAPHY

2.3 CLIMATE

2.4 LAND USE AND ROAD NETWORK

2.5 SUMMARY

CHAPTER TWO

THE STUDY AREA

2.1 Introduction

In this chapter, basic data of Dura city will be briefly narrated. The topography and climate will be described.

2.2 Geography

Dura is situated some 8 Km southwest of Hebron, the location map of Dura is shown in Fig.2.1. The project area comprises the future storm water drainage system for Dura city.

The population within the municipal boundary has been determined by a census carried out by the Palestinian Bureau of Statistics on the night of 9 December (1997). The population is (46000) habitant. This population is expected to grow substantially up to the year (2028) planning horizon of this project .

Topography: The study area consist of mountains with steep slopes and few plain area. The city is about 900 meters above mean sea level (AMSL) .The area of city center is located between 820m and 870m AMSL . The built-up area of the city spreads between the elevation of 760m to 890m AMSL .

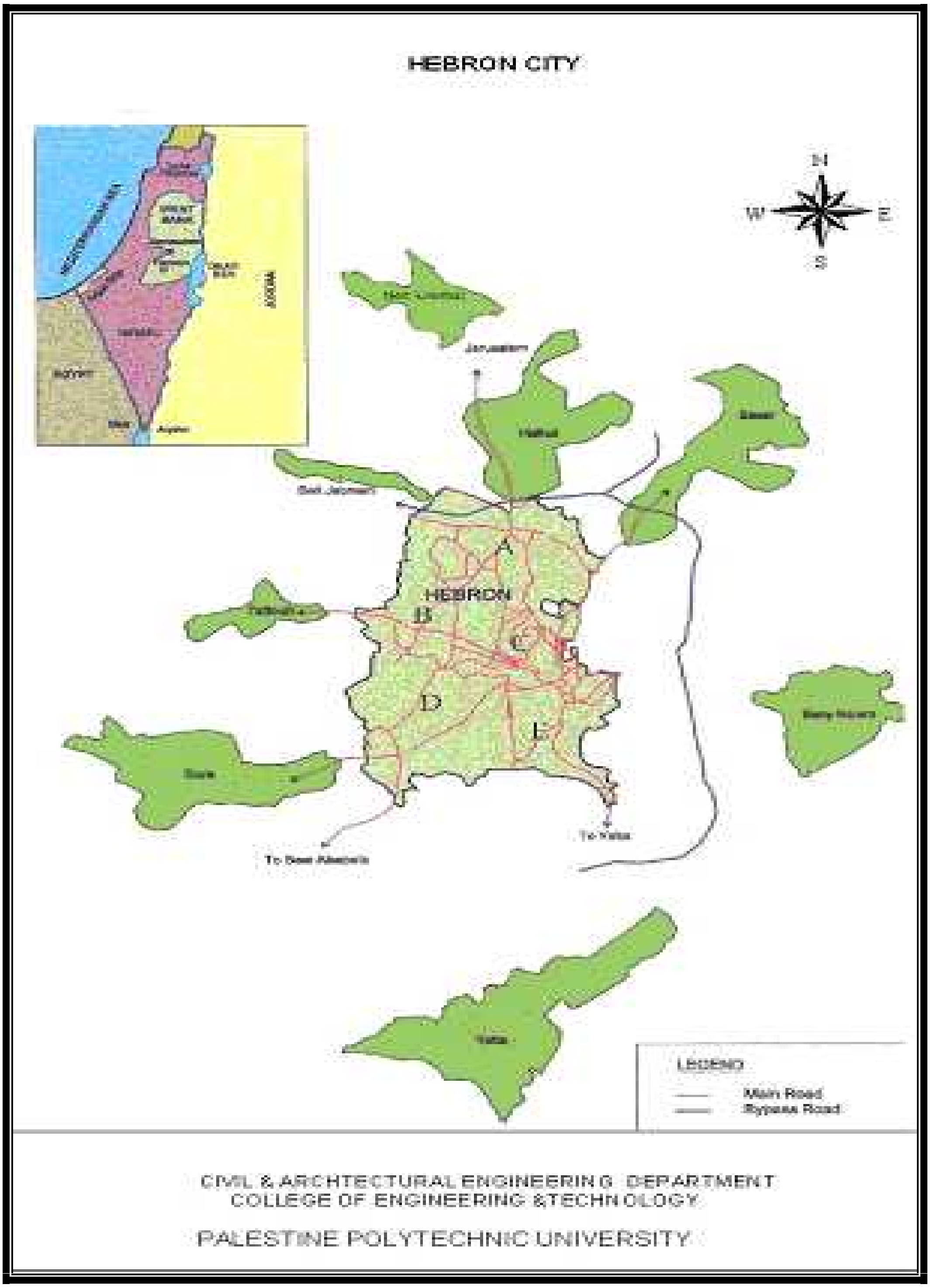


Fig.2.1-Location Map of Dura city wrt Hebron district

2.3 Climate

Dura city has atypical Mediterranean climate two main seasons: the dray season from May to October, and the rainy season from November to April, spring and autumn are short and have special characteristics. The climatologically data presented in the following paragraphs were obtained from the survey carried out by Meteorological Station which is located at the elevation of (+900) m AMS.

2.3.1 Rainfall

The average annual rainfall in Dura city for five years is approximately (460) mm, of which about 98 percent falls between October and April

The maximum monthly rainfall for the same years was recorded in December (1999) and amounted (220.6) mm. The maximum annual rainfall in the period from (1995) to (2000) is (569.50) mm in the year (1999 / 2000), the minimum annual rainfall is (261.8) mm in the year (1998 / 1999). Table (2.1) shows the monthly rainfall and number of raining days during the period from (1995 / 1996)to (1999 / 2000) at the Hebron Meteorological Station.

Table(2.1)- Monthly Rainfall and Number of Raining Days During the period from (2000 – 2005)

As Given From Hebron Metrological Station

Year / Month	2000		2001		2003		2004		2005		5 – years Mean
	total Rainfall (mm)	No. of Raining days	Monthly Rainfall (mm)	No. of Raining days	Monthly Rainfall (mm)	No. of Raining days	Monthly Rainfall (mm)	No. of Raining days	Monthly Rainfall (mm)	No. of Raining days	Average Rainfall (mm)
Septembe	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
October	22.9	1	8.6	4	12.5	6	4.2	1	10.4	5	11.72
November	10.3	6	53.3	5	2.7	3	211.2	1	48.3	3	65.16
December	211.6	7	136.5	10	144.5	11	49.6	5	84.2	11	125.28
January	297.1	15	145.7	9	75.6	11	182.6	10	153.0	10	170.8
February	65.2	9	92.2	90	184.9	9	97.7	5	118.3	9	111.66
March	74.7	11	24.8	15	115.9	2	24.1	7	49.0	3	57.70
April	0.0	6	5.1	3	2.6	1	1.8	3	12.7	1	4.44
May	0.0	0.0	53.9	2	0.0	2	0.0	0.0	0.0	2	10.78
Total	681.8	55	519.9	57	538.7	45	261.8	32	569.5	44	575.54

2.3.2 Temperature

The temperature range is characterized by considerable variations between summer and winter. The mean temperature values at the Hebron Meteorological Station for the period 1996 – 2000 are given in Table 2.2. The following characteristics values where shown:

- Mean maximum temperature: 28 ˚C
- Mean minimum temperature: 5 ˚C
- Maximum temperature recorded: 30 ˚C
- Minimum temperature recorded: 2 ˚C

2.3.3 Relative Humidity

Situated at a considerable distance from the sea in a mountains region on the outskirts of the desert, Dura has low values of relative humidity as compared to those in the plain. As shown in Table 2.3 the relative humidity in Dura city ranges from 30% - 80%, it reaches the maximum value in January.

2.3.4 Winds

The directions and velocities of wind vary depending on the season of the year. In winter, the wind blows in the morning from the southwest, around noon from southwest and west, and at night from west and northwest. In summer, a northeasterly wind blows all day long. According to the data obtained from the Hebron Meteorological Station, average wind velocity in winter is about 12 m/s and in summer 8 m/s.

Table(2.2)- Mean Monthly Minimum and Maximum Temperature (˚C) for the last Five Years (2000–2005)

As Given From Hebron Meteorological Station

Year / Month	2000		2001		2003		2004		2005		5–years Mean Temperature	
	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
January	3.8	8.8	6.2	11.9	7.4	12.9	5	10.5	3.6	11.6	5.2	11.14
February	4	10.4	5.7	11.2	3.4	9.1	5.1	12.1	5.2	11.2	4.68	10.8
March	5.8	12.8	11.5	19.5	5.3	11.6	9.5	17.9	8.1	16.1	8.04	15.58
April	12	21.0	11.8	21.2	11.3	20.1	10.9	20.5	11	20.5	11.4	20.66
May	13.4	23.5	14.3	23.4	18	27.7	14	23	13.2	23.3	14.58	24.18
June	16	30.4	17.5	27.7	18.5	29.3	16	26.8	15.7	26.1	16.74	27.28
July	20.7	27.6	19.6	29.1	18.6	28.5	19.1	29.6	18.9	29.5	19.38	29.42
August	17.4	27.6	18.3	28.7	19	29.6	17.6	28.1	18.6	29.2	18.18	28.64
September	16.8	26	16.6	26.5	16.4	25.5	17	28	17	27.6	16.76	26.72
October	13.6	20.8	14.2	23	15	24.6	16.7	25.5	17.1	23.1	15	23.4
November	11.4	17.7	10.3	17.1	11.9	18.4	11.3	17.4	10.1	17.3	11	17.58
December	7.4	11.7	7	12.5	6.5	12.5	6	12.4	9.8	16.8	7.34	13.18

Table (2.3)- Mean Monthly Relative Humidity (%) During the period (2000 – 2005)

As Given From Hebron Meteorological Station

Year / Month	2000	2001	2003	2004	2005	5–years Mean Relative Humidity
January	72	65	81	71	70	70
February	67	72	75	64	67	69
March	69	70	71	58	69	64
April	55	50	55	51	55	52
May	39	38	48	43	45	41
June	34	51	51	60	55	40
July	52	49	43	57	57	53
August	54	60	49	56	53	51
September	55	52	59	63	61	56
October	60	50	55	67	60	58
November	67	58	61	53	58	56
December	64	70	63	50	57	63

Note: No Data Taken in 2003 Because of the Israeli Invasion

2.4 Land use and road network

2.4.1 Land Use

The land area of the Dura city is approximately 14000 dounms. There is no clear city plan defining land use in the various zones of Dura. However, the present residential zone will maintain the same character in the future.

With the development of the city, the agricultural areas within municipal boundaries will gradually be built up.

2.4.2 Roads

The plan of the existing and planned roads in the Dura city has been obtained from Dura municipality .The main roads traverse as shown in Fig (2.2) and Fig (2.3) the city from north to south in the same direction as the main drainage channels.

2.2main roads

Main roads and house distribution

2.3

2.5 Summary

In this chapter, the basic data of Dura city including location, topography and meteorological data have been discussed.

CAPTER THREE

GENERATING WATERSHED

3.1 INTRODUCTION

3.2 HOW TO GENERATE WATERSHED

3.3 SUMMERY

CAPTER THREE

GENERATING WATERSHED

3.1 Introduction

It is very important in each related projects, to find the water catchments that tell us the direction of the water flow, this direction can be known with respect to topographic surface of the study area.

In this chapter we will explain how to find water catchments using GIS and the Contour map of the study area, and we will show you the steps to get that.

What is ArcView GIS?

- A desktop geographic information system (GIS).

- Software with an easy-to-use,point and click graphical user interface.

- A product of ESRI .`

What you get with GIS ?

- Thematic maps.

- Data creation and editing.

- Map registration.

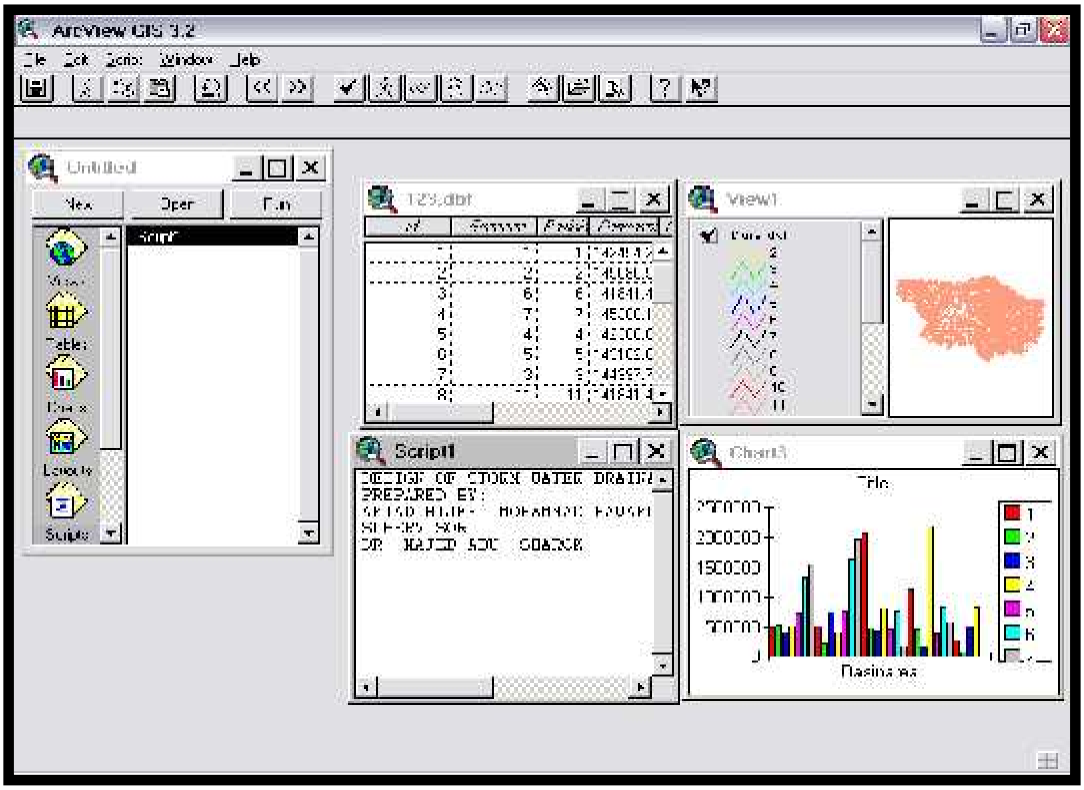
- Extensions that provide additional GIS functionally.

Exploring the ArcView interface and documents:

- User interface.

- Project window.

- View ,layout ,table ,charts and scripts.



Introducing themes

- A Theme is a collection of similar geographic features and their attributes.
- Themes are displayed in views.
- Each theme has a title and a legend in the views table of contents.

Introducing tables

- A document for displaying tabular information.
- Formed into records (rows) and fields(columns).
- Contains descriptive information about theme features.

Introducing layouts

- Provide away to create presentation-quality maps.
- Can be sent to a plotter or printer.

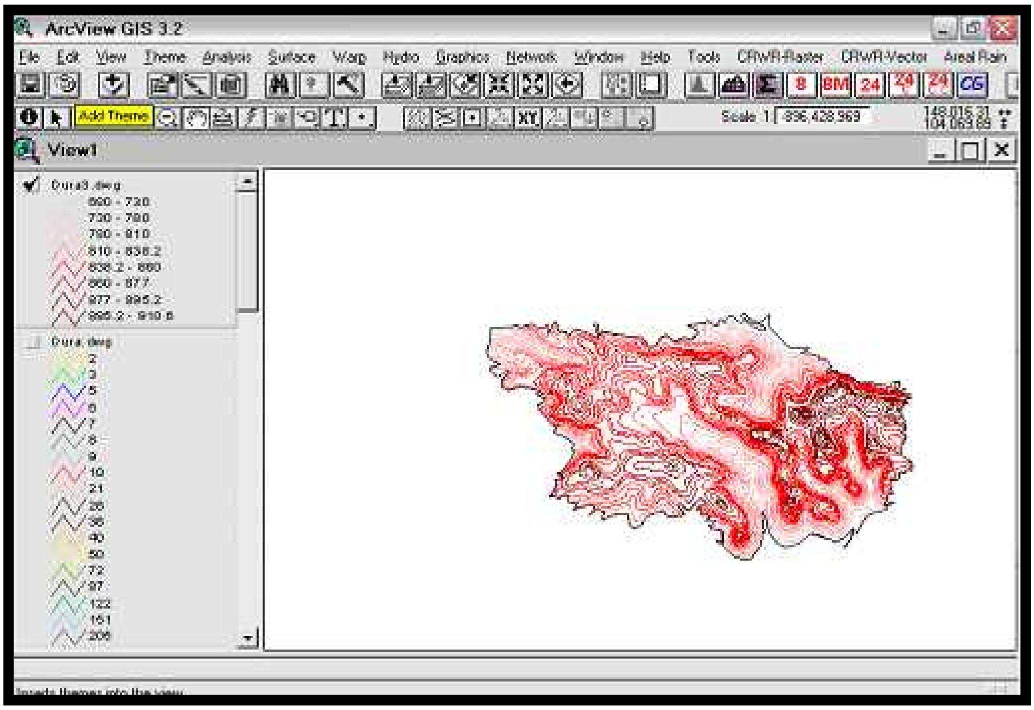
Introducing scripts

- A document where you write programs.
- Create complete applications.

3.2 Main Steps to Generate Watershed:

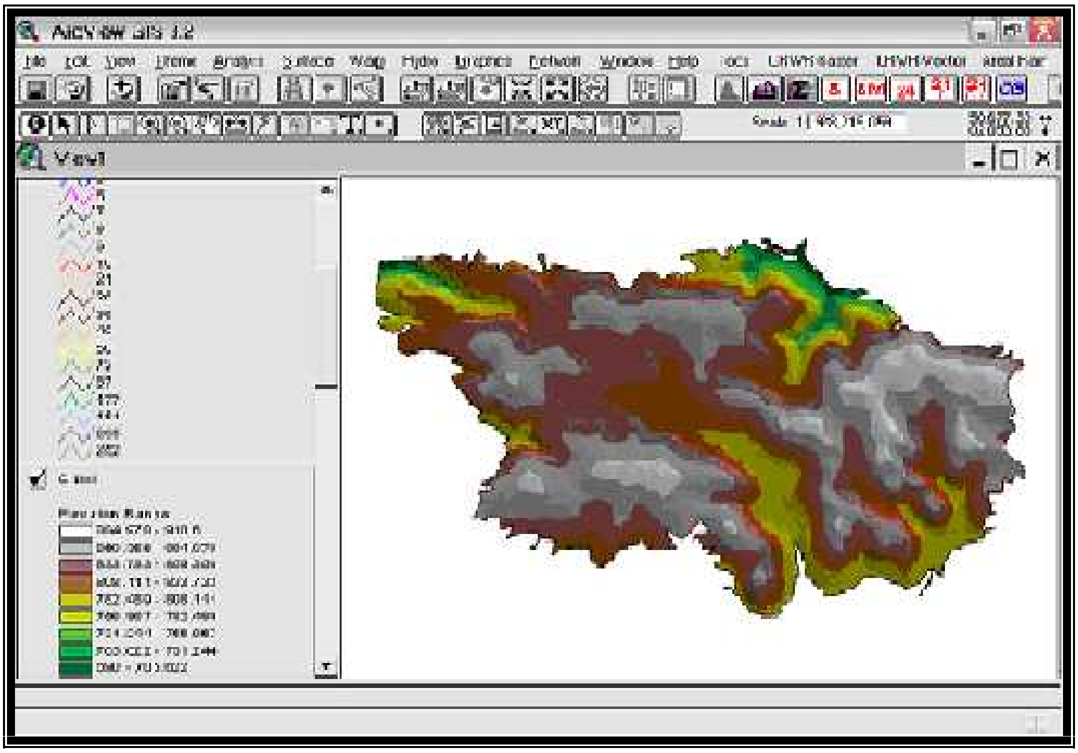
1. Prepare the contour map of the needed area …..
2. Open the main page of the arc view GIS, then open new view.
3. Make sure that the following extensions be active (spatial analysis, 3D analyst, hydro, cad reader, export USGS DEM).

4. Add the contour map as *.dxf theme.



5. From view menu select new theme, then choose polygon to draw around the contour map borders.
6. From surface menu select "create TIN from features",now you get Triangle Irregular Net "TIN".

TIN: It is a net work of irregular triangle adapts to terrain.



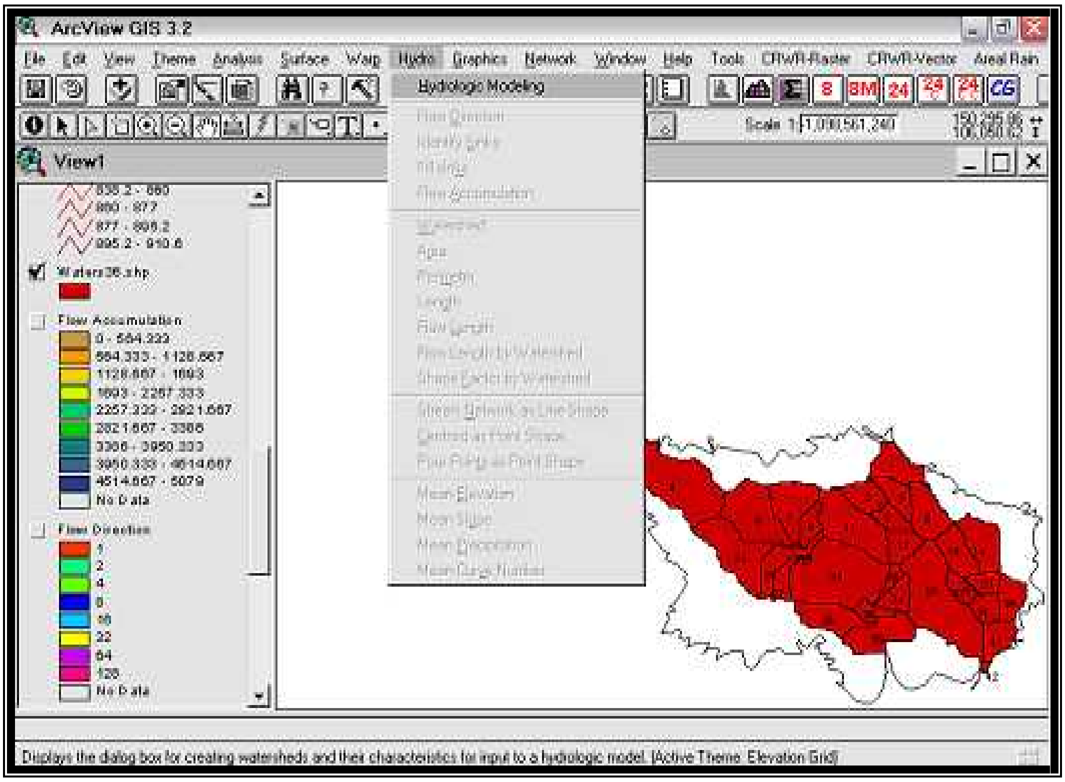
7.To prepare the grid ,make sure that TIN theme is active, then , select "convert to grid "from theme menu. Now you get Digital Elevation Model (DEM).

DEM in simple words is: regularly spaced grid of elevation points or it is the simplest form of digital representation of topography , DEM is frequently used to refer to any digital representation of a topographic surface. And it is used to refer specifically to regular grid of spot heights. The resolution, or the distance between adjacent grid points, is a critical parameter.

Grid DEM	TIN
. simple storage . slower to compute . possibility of redundant data points . uniform pixel size	. fewer points needed for the same accuracy resolution adapts to terrain . initial construction is time consuming . some operations do not have efficient algorithms

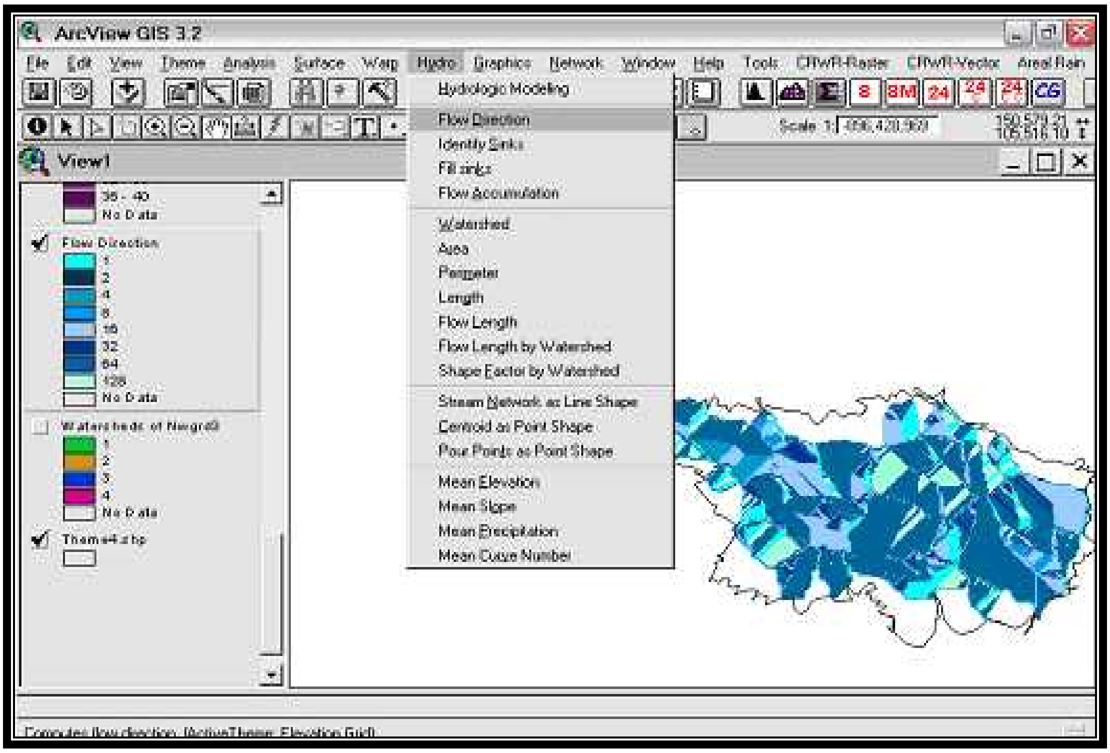
9. After getting main catchment, you must repeat step "8" to determine the sub-catchment area by changing the minimum number of cells that define watershed.

NOTE: "The minimum cells of main catchments (chosen 2000) are greater than the minimum cells of sub-catchments (chosen 240)".

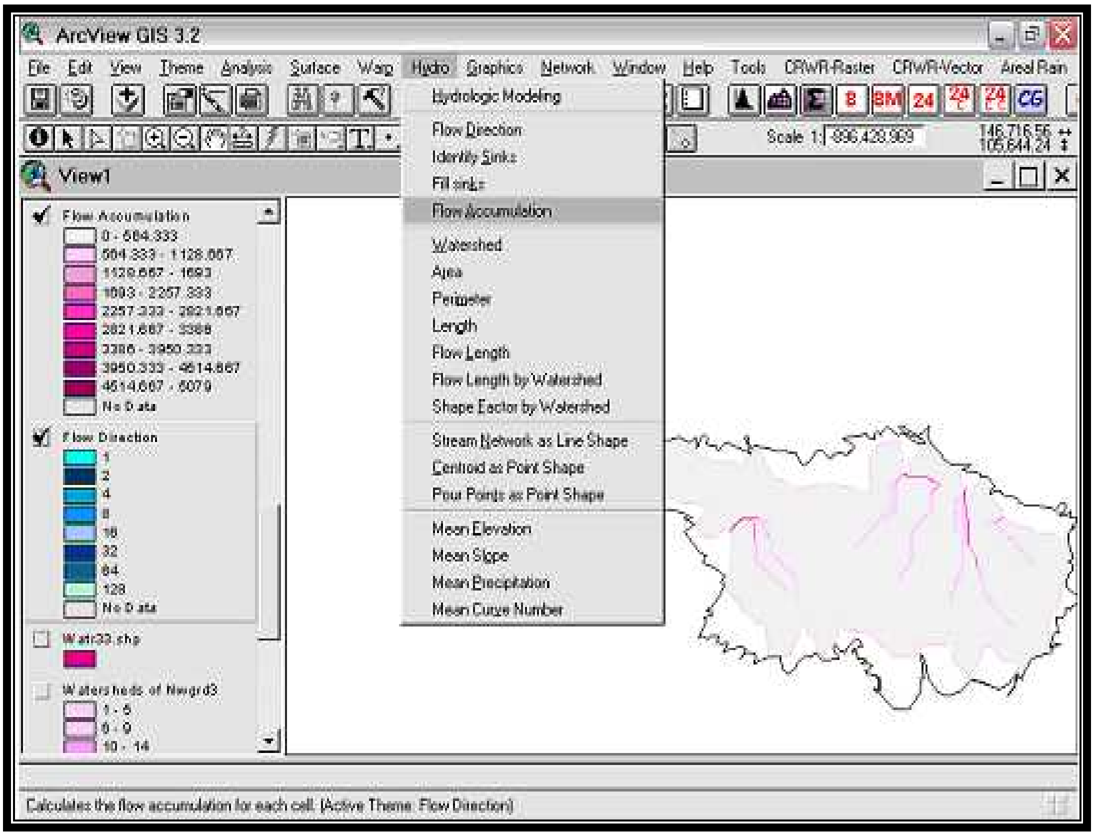


10. Determination of flow direction require sub-catchments theme active, then, go to hydro menu, select flow direction.

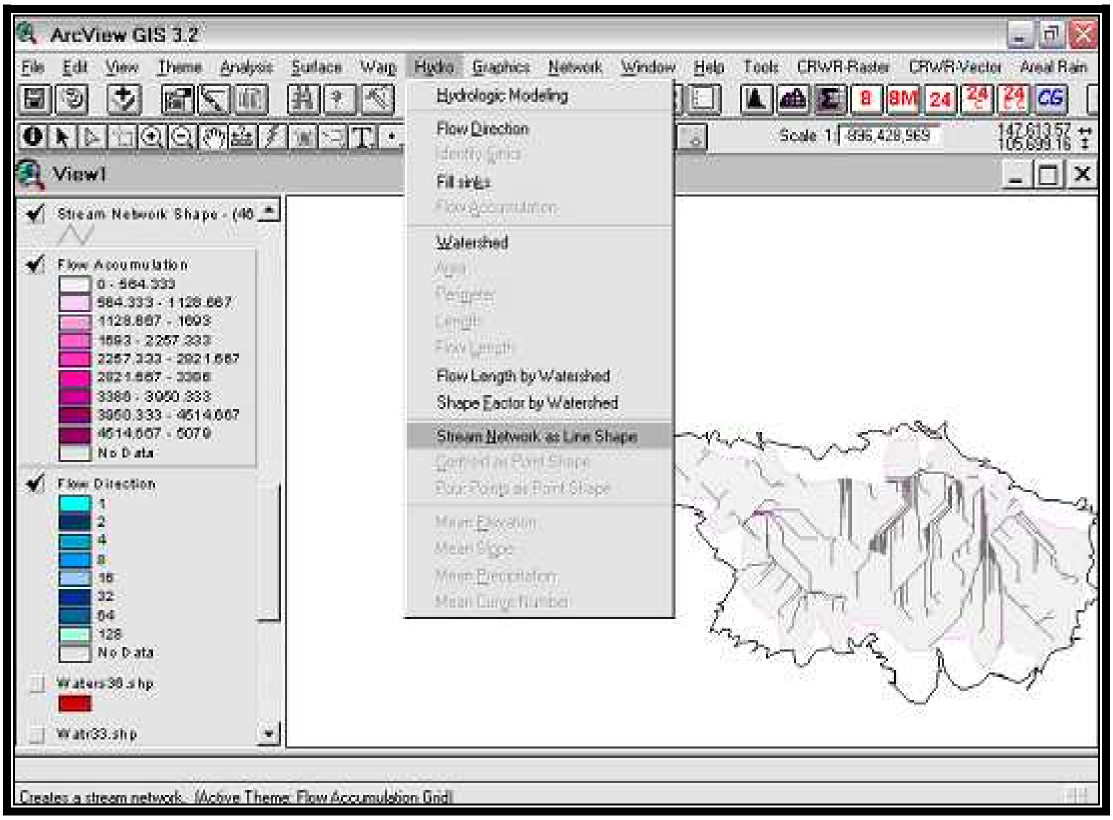
Flow direction: it is one of the keys to deriving characteristics about a surface is the ability to determine the direction of flow from every cell in the grid.



11. Flow direction is theme without determination the actual stream line, and to do that you want to select flow accumulation from hydro menu.



12. After flow accumulation determinenation you want to get stream line network by it from hydro menu, and notice that flow accumulation theme is active, and from stream line list, choose flow direction.



3.3 Summary

In the previous chapter, the basic data about using Arc view GIS, in order to generate watershed using the same program and the contour map of the study area.

CHAPTER FOUR

DESIGN OF STORM WATER DRAINAGE SYSTEM

4.1 INTRODUCTION

4.2 STORM WATER RUNOFF

4.3 HYDRAULIC CONSIDERATION

4.4 STORM WATER SEWERS DESIGN

4.5 SUMMARY

CHAPTER FOUR

DESIGN OF STORM WATER DRAINAGE SYSTEM

4.1 Introduction

Rapid effective removal of storm runoff was a luxury not found in many cities in the early nineteenth century. Today, the modern city dweller has come to think of this as an essential service. Urban drainage facilities have progressed from crude ditches and stepping stones to the present intricate coordinates systems of curbs, gutters, inlets, and under ground conveyance.

The design must consider meteorological factors, geomorphological factors, and the economic value of the land, as well as human value considerations such as aesthetic and public safety aspects of the design. The design of storm water detention basins should also consider the possible effects of inadequate maintenance of the facility.

4.2 Storm water runoff

Storm water runoff is that portion of precipitation which flows over the ground surface during and a short time after a storm. The dependence parameters that controlled the quantity of the storm water which carried by a storm or combined sewer are the surface of the drainage area (A, ha), the intensity of the rainfall (i, l/s.ha), and runoff coefficient C dimensionless (the condition of the surface). There are many methods and formulas to determine the storm flow .

4.2.1 Rational Method

The rational method has probably been the most popular method for designing storm systems. It has been applied all over the world and runoff is related to rainfall intensity by the formula,

$$\boxed{Q = C.i.A} \tag{4.1}$$

Where Q= peak runoff rate (l/sec)

C= runoff coefficient, which is actually the ratio of the peak runoff rate to the average rainfall for a period known as the time of concentration.

i = average rainfall intensity, mm/min, for period equal to the time of concentration.

A= drainage area, ha.

For small catchments areas, it continues to be a reasonable method, provided that it is used properly and that results and design concepts are assessed for reasonableness. This procedure is suitable for small systems where the establishment of a computer model is not warranted.

The steps in the rational method calculation procedure are summarised below:

1- The drainage area is first subdivided into sub-areas with homogeneous land use according to the existing or planned development.

2- For each sub-area, estimate the runoff coefficient *C* and the corresponding area *A*.

3- The layout of the drainage system is then drawn according to the topography, the existing or planned streets and roads and local design practices.

4- Inlet points are then defined according to the detail of design considerations. For main drains, for example, the outlets of the earlier mentioned homogeneous sub-areas should serve as the inlet nodes. On the other hand in very detailed calculations, all the inlet points should be defined according to local design practices.

5- After the inlet points have been chosen, the designer must specify the drainage sub-area for each inlet point A and the corresponding mean runoff coefficient *C*. If the sub-area for a given inlet has non-homogeneous land use, a weighted coefficient may be estimated.

6- The runoff calculations are then done by means of the general rational method equations for each inlet point, proceeding from the upper parts of the watershed to the final outlet. The peak runoff, which is calculated at each point, is then used to determine the size of the downstream trunk drain using a hydraulic formula for pipes flowing full.

7- After the preliminary minor system is designed and checked for its interaction with the major system, reviews are made of alternatives, hydrological assumptions are verified, new computations are made, and final data obtained on street grades and elevations. The engineer then should proceed with final hydraulic design of the system.

4.2.2 Runoff Coefficient, C

Runoff coefficient is a function of infiltration capacity, interception by vegetation, depression storage, and evapotranspiration. It is requires greatest exercise of judgment by engineer and assumed constant, actually variable with time. It is desirable to develop composite runoff coefficient (weighted average) for each drainage area as:

$$\boxed{C = \frac{\sum Ci.Ai}{\sum Ai}} \tag{4.2}$$

Where Ai = i th area.

Ci = i th runoff coefficient.

The range of coefficients with respect to general character of the area is given in the following tables (Table 4.1 and Table4.2).

Table4.1-The Range of Coefficient With Respect to General Character of the Area (Sarikaya, 1984)

Description of Area	Runoff Coefficients
Business	
Down town	0.70 to 0.95
Neighborhood	0.50 to 0.70
Residential	
Single-Family	0.30 to 0.50
Multi-unit, detached	0.40 to 0.60
Multi-unit, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, Cemeteries	0.10 to 0.25
Playground	0.20 to 0.35
Railroad yard	0.20 to 0.35
Unimproved	0.10 to 0.30

Table4.2- The Range of Coefficient With Respect to Surface Type of the Area (Sarikaya, 1984)

Character of Surface	Runoff Coefficients
Pavement	
Asphalt and concrete	0.70 to 0.95
Brick	0.70 to 0.85
Lawns, Sandy soil	
Flat, 2 percent	0.05 to 0.10
Average,2to7percent	0.10 to 0.15
Steep, 7 percent	0.15 to 0.20
Roofs	0.75 to 0.95
Lawns, heavy soil	
Flat, 2 percent	0.13 to 0.17
Average,2 to 7percent	0.18 to 0.22
Steep, 7 percent	0.25 to 0.35

4.2.3 Infiltration

Infiltration is a movement of water from the surface of ground into the soil. In a rainstorm, Infiltration normally begins at a high rate and decreases to a minimum as rain continues. At any instant, the Infiltration capacity (Fc) of soil is the maximum rate at which water will enters the soil in a given condition. The rate at which water actually enters the soil during a storm is known as Infiltration rate (f) and is equal to the Infiltration capacity or the rainfall rate whichever is less which means when the rainfall intensity is less than the Infiltration capacity, the prevailing Infiltration rates are approximately equal to the rainfall rates, and if the rainfall intensity is always above the infiltration capacity, the infiltration rate follow the capacity curve.

Infiltration capacity depends on many factors such as; soil type, moisture content, compactor, due to rain, man, and animals, vegetative cover, and temperature. The limiting value of infiltration capacity is controlled by the soil permeability and can be determined experimentally by subjecting an experimental plot to rainfall rates in excess of infiltration capacity and by measuring surface runoff. The other method is by use of infiltrometer of which there are many different types.

For the rainfall rates in excess of the infiltration capacity found that infiltration capacity varies as:

$$\boxed{f = f_c + (f_i - f_c)e^{-kx}} \tag{4.3}$$

Where:

f: is the infiltration capacity at any time t from start of rainfall.

fi: is the initial infiltration capacity, generally mm/hr, at t=0

fc: is the infiltration capacity after it attains a constant value.

K: is a positive constant value depending on soil and vegetation cover.

T: is the duration of rainfall, reckoned from the beginning of rainfall.

4.2.4 Rainfall Intensity, i

In determining rainfall intensity for use in rational formula it must be recognized that the shorter the duration, the greater the expected average intensity will be. The critical duration of rainfall will be that which produces maximum runoff and this will be that which is sufficient to produce flow from the entire drainage area. Shorter periods will provide lower flows since the total area is not involved and longer periods will produce lower average intensities. The storm sewer designer thus requires some relationship between duration and expected intensity. Intensities vary from place to another and curves or equations are specified for the areas for which they were developed.

The rainfall intensity depends on many factors through which we can do our calculations; we can list these factors as follow:

1- Average frequency of occurrence of storm ($\frac{1}{n}$) or (f).

   Average frequency of occurrence is the frequency, with which a given event is equaled or exceeded on the average, once in a period of years. Probability of occurrence, which is the reciprocal of frequency, (n) is preferred by sum engineers. Thus, if the frequency of a rain once a 5-year ($\frac{1}{n}$=5), then probability of occurrence n=0.20. Selection of storm design rain frequency based on cost-benefit analysis or experience. There is range of frequency of often used:

   a. Residential area: f = 2 to 10 years (5 year most common).
   b. Commercial and high value districts: f = 10 to 50 (15 year common).
   c. Flood protection: f = 50 year.

2- Intensity, duration and frequency characteristics of rainfall.

Basic data derived from gage measurement of rainfall (Point rainfall) over along period can be used to obtain a rainfall height diagram that show the relation between the height of rain (mm) and time (min). The slope of the curve or rain height per unit time is defined as rain intensity:

i= (Δ height of rain / Δ time) $\left[\frac{mm}{\min}\right]$

The rain intensity in letter per second . hectare is equal:

$$i(\frac{l}{s.ha}) = 166.7i\left[\frac{mm}{\min}\right]$$

in order to drive intensity-duration-frequency curves long-term observation of rainfall is needed. Analysis of such observation is given in any text in sanitary engineering.

3- Time of Concentration

The time of concentration is the time required for the runoff to become established and flow from the most remote part (in time) of the drainage area to the point under design.

$$t_c = t_i + t_f \tag{4.4}$$

Where t_c: time of concentration.

t_i: inlet time.

t_f: flow time.

Time of flow in storm, $t_f = \frac{\text{Length of pipe line (L)}}{\text{Velocity of flow (v)}}$

Inlet time (t_i): is the time required for water to flow over ground surface and along gutters to drainage inlet. Inlet time is function of rainfall intensity, surface slope, surface roughness, flow distance, and infiltration capacity and depression storage.

4.2.4 Catchments Area, A

Most of the catchments are partly developed with residential facilities. The catchments are moderately flat with rural, residential and commercial land uses. The rural areas are located at the downstream end of the catchments.As shown in fig(4.1,).fig(4.2)

maincatchments

subcatchment

4.3 Hydraulic consideration

4.3.1 Introduction

Storm water systems are usually designed as close channels, except where lift stations are required to overcome topographic barriers. The hydraulic problems associated with these flows are complicated in some cases by the quality of the fluid, the highly variable nature of the flows, and the fact that an unconfined or free surface exists. The driving force for sewer flow is gravity. For the hydraulic calculations of sewers, it is usually assumed uniform flow in which the velocity of flow is constant, and steady flow condition in which the rate discharge at any point of a sewer remains constant .

4.3.2 Hydraulic design equations

In principle all closed channel flow formulas can be used in hydraulic design of sewer pipes. The following are the most important formulas:

1. Chezy's formula:

$$V = C\sqrt{RS} \qquad (4.5)$$

Where V: the velocity of flow (m/s).

C: the Chezy coefficient; $C = \frac{100\sqrt{R}}{m + \sqrt{R}}$, where m = 0.35 for concrete pipe

or 0.25 for vitrified clay pipe

R: the hydraulic radius (m)

S: the slope of the sewer pipe (m/m).

2. Darcy-Weisbach formula: It is not widely used in wastewater collection design and evaluation because a trial and error solution is required to determine pipe size for a given flow and head loss, since the friction factor is based on the relative roughness which involves the pipe diameter, making it complicated. Darcy-Weishbach formula states that

$$H = a \frac{L \times V^2}{D \times 2g} \tag{4.6}$$

Where H: the pressure head loss

L: the length of pipe (m).

D: the diameter of pipe (m)

λ : the dimensionless friction factor generally varying between 0.02 to 0.075.

3. The Manning formula: Manning's formula, though generally used for gravity conduits like closed channel, it is also applicable to turbulent flow in pressure conduits and yields good results, provided the roughness coefficient n is accurately estimated. Velocity, according to Manning's equation is given by:

$$\boxed{V = \frac{1}{n} R^{2/3} S^{1/2}} \tag{4.7}$$

Where n: the Manning's roughness coefficient.

R: the hydraulic radius.

- For circular pipe flowing full, R=D/4.
- For open channel flowing full, R=A/P.

The Manning's roughness coefficient depends on the material and age of the conduit. Commonly used values of n for different materials are given in Table (4.3).

Table4.3: Common Values of Roughness Coefficient Used in the Manning Equation (Sarikaya, 1984)

Material	Commonly Used Values of n
Concrete	0.013 and 0.015
Vitrified clay	0.013 and 0.015
Cast iron	0.013 and 0.015
Brick	0.015 and 0.017
Corrugated metal pipe	0.022 and 0.025
Asbestos cement	0.013 and 0.015
Earthen channels	0.025 and 0.003

4.3.3 Hydraulics of Partially Field Section

The filling rate of a sewer is an important consideration, as sewers are seldom running full, so storm water sewers designed for 40% or 50% running full, that is means only 40 % to 50 % of the pipe capacity should be utilized to carry the peak flow.

Partially filled sewers, are calculated by using partial flow diagram and tables indicating the relation between water depth, velocity of flow and rate flow .

4.4 Storm water sewers design

Designing a community storm system is not a simple task. It requires considerable experience and a great deal of information to make proper decisions concerning the layout, sizing, and construction of a storm network that is efficient and cost-effective. The design engineer needs to generally undertake the following tasks (Qasim,1985, Peavy,1985):

1. Define the service area.
    3. Conduct preliminary investigations.
    4. Develop preliminary layout plan and profile.
    5. Selection of design parameters.
    6. Review construction considerations.
    7. Conduct field investigation and complete design and final profiles.
    8. Prepare contract drawing and specifications.

4.4.1 Service Area

Service area is defined as the total area that will eventually be served by the drainage system.. It is important that the design engineers and project team become familiar with the surface area of the proposed project.

4.4.2 Preliminary Investigation

The design engineer must conduct the preliminary investigations to develop a layout plan of the drainage system. Site visits and contacts with the city and local planning agencies and state officials should be made to determine the land use plans, zoning regulations, and probable future changes that may affect both the developed and undeveloped land. Data must be developed on topography, geology, hydrology, climate, ecological elements, and social and economic conditions. Topographic maps with existing and proposed streets and other utility lines provide the most important information for preliminary flow routing (Qasim, 1985).

If reliable topographic maps are not available, field investigations must be conducted to prepare the contours, place bench marks, locate building, utility lines, drainage ditches, low and high areas, stream, and the like. All these factors influence the sewer layout.

4.4.3 Layout Plan

Proper storm sewer layout plan as shown in fig (4.3), and profiles must be completed before design flows can be established. The following is a list of basic rules that must be followed in developing a sewer plan and profile (Qasim, 1985).

1. Select the site for disposal of the storm water at the end of the network, generally the lowest elevation of the entire drainage area.
2. The preliminary layout of storm sewers is made from the topographic maps. In general, sewers are located on streets, or on available right-of-way; and sloped in the same direction as the slope of the natural ground surface.

3. The trunk storm sewers are commonly located in valleys. Each line is started from the intercepting sewer and extended uphill until the edge of the drainage area is reached, and further extension is not possible without working downhill.
4. Main storm sewers are started from the trunk line and extended uphill intercepting the laterals.
5. Preliminary layout and routing of storm sewage flow is done by considering several feasible alternatives. In each alternative, factors such as total length of storm sewers, and cost of construction of laying deeper lines versus cost of construction, operation, and maintenance of lift station, should be evaluated to arrive at a cost- effective drainage system.
6. After the preliminary storm sewer layout plan is prepared, the street profiles are drawn. These profiles should show the street elevations, existing storm sewer lines, and manholes and inlets. These profiles are used to design the proposed lines.

Finally, these layout plans and profiles are revised after the field investigations and storm sewer designs are complete (Viessman, 1985).

4.4.4 Flow accumulations network

One of the most important things you must do before designing a storm drainage system, is to select the direction of the main line of flow accumulations and stream lines. Which depend on the gravity and elevations deference. See Fig.(4.4) & Fig.(4.5).

Layout plan

4.3

flow

4.4

tin

4.5

4.4.5 Selection of Design Parameters

Many design factors must be investigated before storm sewer design can be completed. Factors such as design period; peak, average, and minimum flow; storm sewer slopes and minimum velocities; design equations …etc. are all important in developing storm sewer design. Many of the factors are briefly discussed below.

1. **Design Flow Rate:** Storm water sewers should be designed to carry the largest storm that occurred in the period of design; commonly it is 5 years because of consideration of the cost and the frequently factors.

2. **Minimum Size:** As mentioned earlier, the minimum storm sewer size recommended is 250 to 300 mm for closed system.

3. **Minimum and Maximum Velocities:** In storm water sewers, solids tend to settle under low-velocity conditions. Self-cleaning velocities must be developed regularly to flush out the solids. Most countries specify minimum velocity in the

sewers under low flow conditions. The minimum allowable velocity is 0.75 m/s, and 0.9 m/s is desirable. This way the lines will be flushed out at least once or twice a day. The maximum velocities for storm water system are between 4 to 5 m/s. The maximum velocity is limited to prevent the erosion of sewer inverts.

4. **Slope:** For closed system minimum slopes determined from minimum velocities, for minimum velocity 0.9 m/s, the slopes are shown in Table 4.4.

Maximum slopes determined from maximum velocities, 1/D (cm) can be used as a guide.

Table4.4: Minimum Recommended Slopes of Storm Sewer (n = 0.015) (Sarikaya, 1984)

Pipe Diameter (D)		Slope (min)	Slope (max) =1/D
mm	**Inch**	**mm**	**Cm**
250	10	0.00735	0.04
300	12	0.00576	0.033
450	18	0.00336	0.0222
600	24	0.00229	0.0167

5. **Depth:** The depth of storm sewers when using closed system is generally just enough to receive flow but not less than 1 m below the ground surface. Depth depends on the water table, lowest point to be served, topography, and the freeze depth.

6. Appurtenances: Storm Sewer appurtenances include manholes, inlets, outlets and outfall, and others. Appropriate storm sewer appurtenances must be selected in design of storm water sewers.

7. Design Equations and Procedures: Storm water sewers are mostly designed to flow partially full. Once the peak, average, and minimum flow estimates and made general layout and topographic features for each line are established, the design engineer begins to size the sewers. Design equations proposed by Manning, Chezy, Gangullet, Kutter, and Scobey have been used for designing sewers and drains. The Manning equation, however, has received most widespread application. This equation is expressed below:

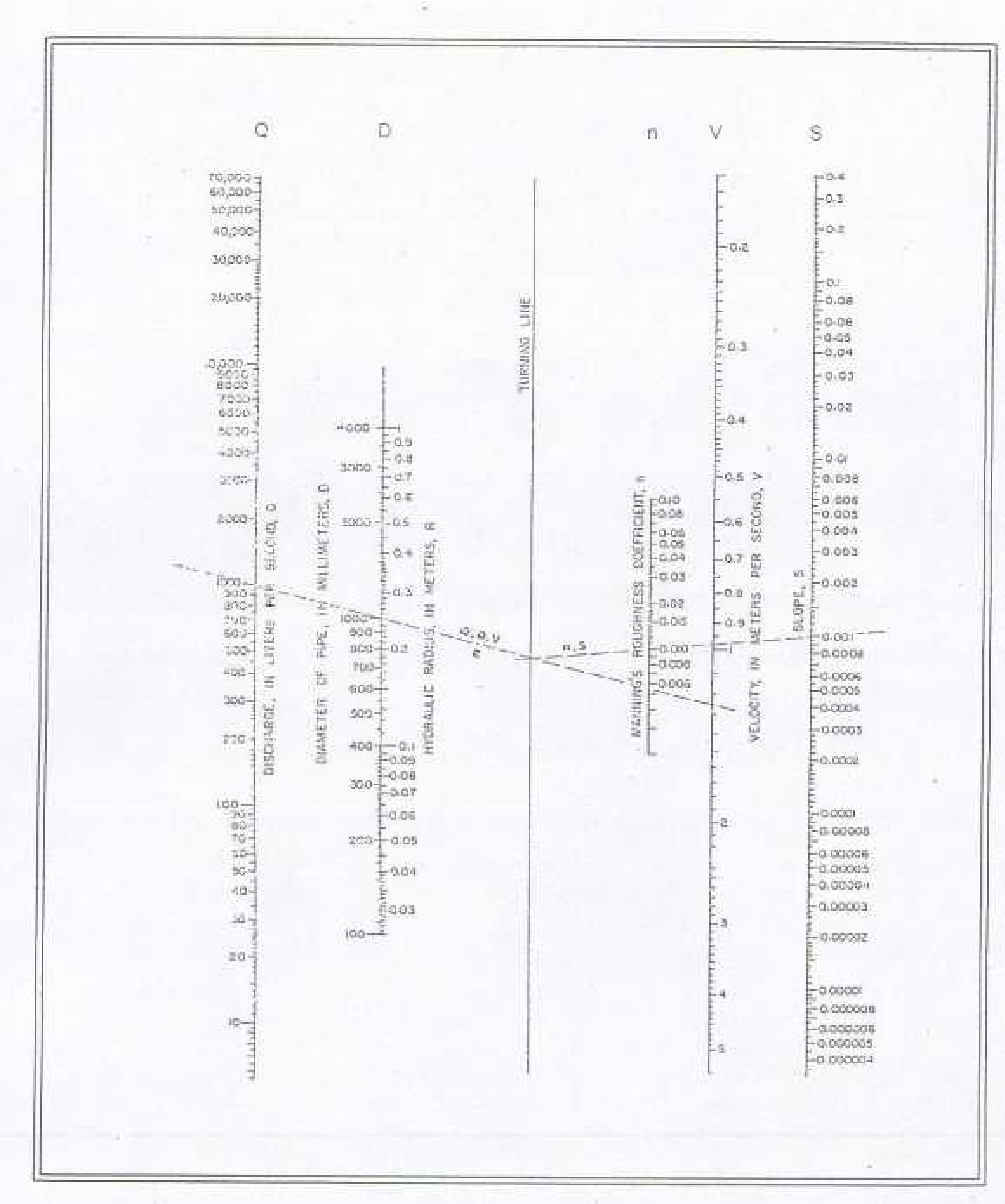
$$V = \frac{1}{n} R^{2/3} S^{1/2} \tag{4.7}$$

And as mentioned earlier, the runoff flow is calculated using the following formula:

$$Q = C.i.A \tag{4.1}$$

Various types of nomographs have been developed for solution of problems involving sewers flowing full. Nomographs based on Manning's equation for circular pipe flowing full and variable n values are provided in Figure (4.6). Hydraulic elements of circular pipes under partially-full flow conditions are provided in Figure (4.7). It may be noted that the value of n decreases with the depth of flows Figure 4.2. However, in most designs n is assumed constant for all flow depths. Also, it is a common practice to use d, v, and q notations for depth of flow, velocity, and discharge under partial flow condition while D, V, Q notations for diameter, velocity, and discharge for sewer flowing full.

Fig, 4.6: Diagram for solution by Manning Formula



4.4.6 Design Computation

After the preliminary sewer layout plan and profile are prepared, the design computations are accomplished. Design computations for sewers are repetitious and therefore, are best performed in a tabular format. Table (4.5) is typical of the way in which data can be organized to facilitate computations for closed system.

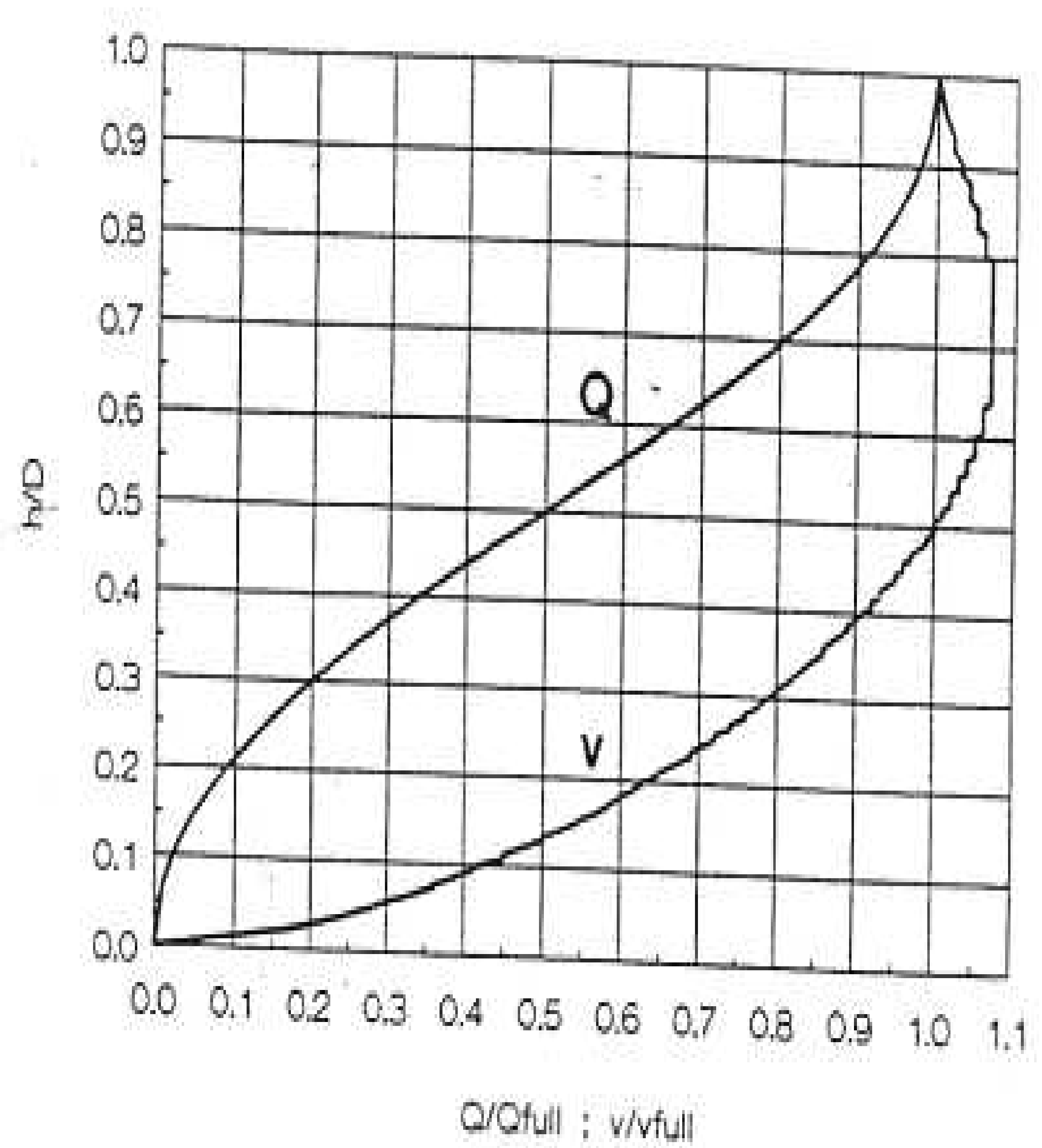
4.4.7 Preparation of Maps and Profile

It is important that the detailed drawings be prepared and specifications completed before the bide can be requested. The contract drawings should show (1) surface features, (2) depth and character of material to be excavated, (3) the existing structures that are likely to be encountered, and (4) the details of sewer and appurtenances to be constructed.

The specifications should be prepared by writing clearly and completely all work requirements and conditions affecting the contracts. As an example, technical specifications should cover items such as site preparation, excavation and backfill, concrete work, sewer materials and pipe lying, and acceptance tests (Qasim, 1985).

Table 4.5 computation table

Fig, 4.7: Hydraulic Properties Of Circular Sewer


1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
h/D
Q
V
0.0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0 1.1
Q/Qfull ; v/vfull

4.5 Summery

In this chapter, municipal storm water drainage systems in general have been described. The method of calculating the storm water runoff has been presented. The flow equations of sewer pipes have been brought out. Finally the design and construction of community storm water drainage system has been briefly discussed.

CHAPTER FIVE

DESIGN AND PLANNING CRITEREA

5.1 Introduction

In the previous chapters, the problem of the study has been defined and the objectives of the project have been listed. The characteristics of the project area of Dura city have been described. Storm water drainage system and design of storm water sewers were explained. In this chapter, basis for planning and design will be discussed including catchments areas, rainfall characteristics, runoff flow, and the design parameters.

5.2 Catchment areas

Dura city is divided into five main drainage catchments areas; the city is divided into regional catchments areas based on the topography of the area. Figure(5.1) shows the main five catchments areas of the city. The five main catchments are divided into many sub-catchments areas as illustrated in Figure (5.2). The sub-catchments and there areas have been assorted in Table (5.1).

Figure 5.1

Table 5.1- The Sub- Catchments Areas of Dura city

5.3 Rainfall characteristics

5.3.1 General Condition

There is no significant variation in annual rainfall along Dura city from the north to the south, the average annual rainfall in Dura city for the last five years is 460 mm, of which about 98 percent falls between October and April. There are two well defined seasons, the wet season start in October and extending into April, and the dry season extending from May to September. The monthly average rainfall varies widely, the highest in the last five years (1995 to 2000) being 220.60 mm occurring in December, 1999, and the second highest of 200 mm on February, 1996.

5.3.2 Intensity-Duration Curve

Standard runoff calculations are based on rainfall intensity for a given time period (rainfall intensity-duration curve). Hebron station measurements have been used as the basis for rainfall throughout Dura city, the mean annual rainfall rate at Dura station area is approximately equal for the Hebron city. The data obtained from Hebron station on rainfall intensity are used to draw the intensity- duration curve for Hebron city. The calculation is presented in Table(5.2).

Table 5.2: Intensity-Duration Relationship for Five Years Period in Hebron i(mm/hr) = b*m^T

RETURN PERIOD(5YEARS), B = 66.53 AND M = 0.993					
Duration(min)	15	75	105	165	180
Rainfall(mm)	15.5	10.2	12.3	9.8	21.3

The intensity-duration frequency relations could be expressed by the formula:

$$i = b*m^{T} \tag{5.1}$$

Where i: intensity (mm/hr).

T: duration time (min).

b,m: constants.

The values of **m** and **b** are determined using the data in the Table (5.2) and by the help of Excel software. This allowing tracing the curve of Figure (5.3) for Hebron city.

Figure (5.3) presents the rainfall intensity-duration curve for Hebron city. The typical curve along with Gaza city curve is also presented in the figure. As shown in the figure, the typical curve is higher; hence, the rainfall intensity is more.

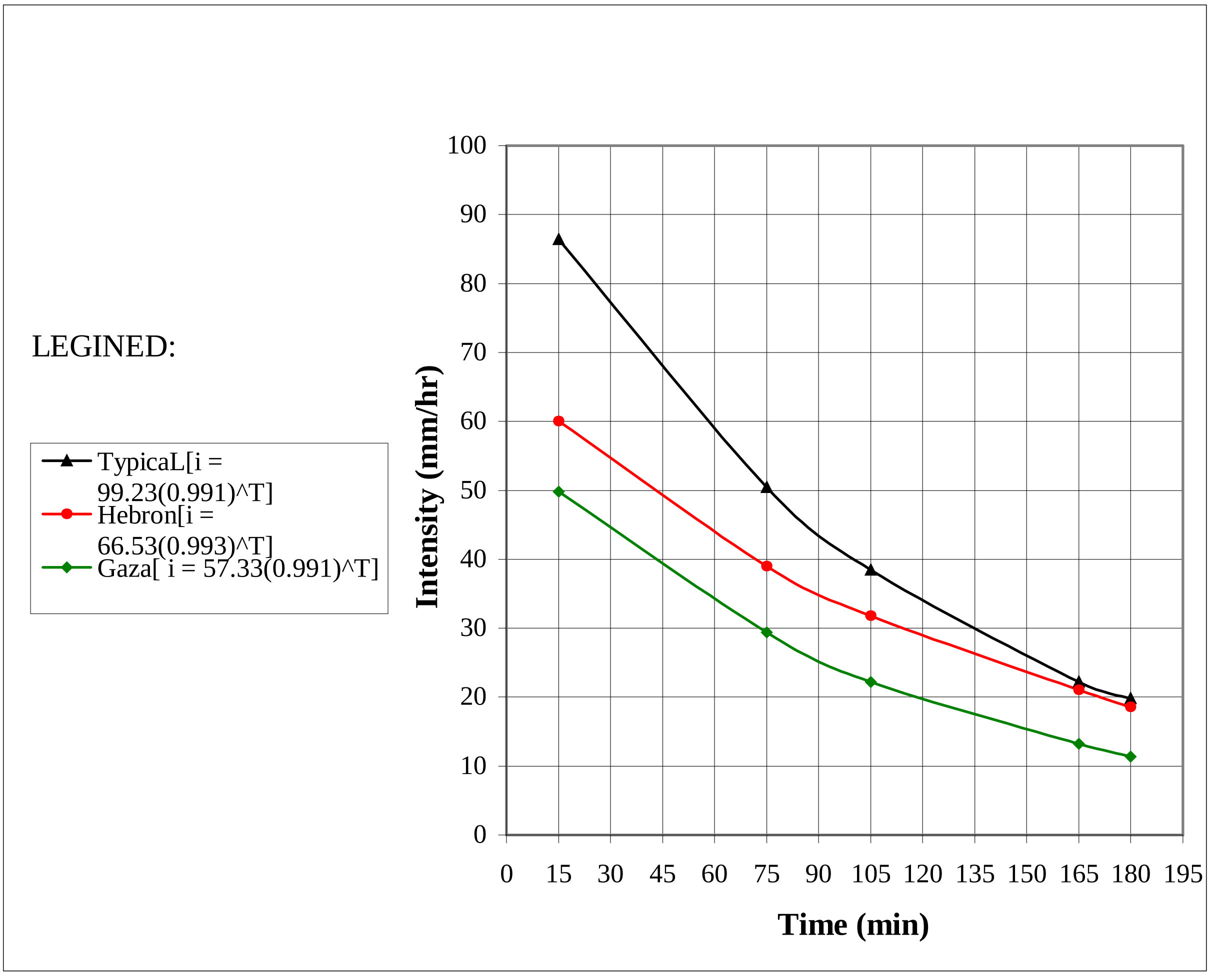


Figure 5.3 - The rainfall Intensity- Duration Curve

5.4 Runoff flow

5.4.1 Runoff coefficient

The runoff coefficient is a function of the permeability of the surfaces and interception/ retention/ infiltration of storm water in the drainage area. In an developed area, C is a function of the surface and natural soil type. In a developed area, C is a function of the amount of paving and/or development. Because of this, runoff coefficient values for developed areas are closely linked to various type of land use. Typical C values are as shown below in Table (5.3).

Table 5.3: Typical Runoff Coefficients for Developed Area

Development	Coefficient
Pavement, Road/ Parking	0.9
Commercial/ Public	0.7
Residential Communities	0.6
Parks/ Unimproved areas	0.3
Irrigation areas	0.2
Natural zones	0.05

In our project used runoff coefficients(C) = 0.3.

5.4.3 Method of Calculation

a. Concentration time (t_c): the concentration time was taken as t_c = ti+tf min, as it commonly used for consideration of safety.

b. Flow rate (Q): The discharge is calculated using Rational formula as

$$Q = C.i.A \tag{4.1}$$

c. Rainfall intensity (i): It is calculated by using the formula:

$$i = b*m^T \tag{5.1}$$

Parameters **b** and **m** correspond to the frequency used for the design rainfall. For the design rainfall used in the project, with a return period of f = 5 years, b = 66.53 and m= 0.993.

d. Manning formula: As mentioned earlier and according to usual practice, the sewers are designed for gravity flow using Manning's formula:

$$\boxed{V = \frac{1}{n} R^{2/3} S^{1/2}} \tag{4.6}$$

Depending on pipe materials, the typical values of n are:

- Reinforced Concrete (RC) n = 0.013
- Polyvinyl Chloride (PVC) n = 0.011
- Ductile Iron: n = 0.013
- Asbestos Cement: n = 0.012

5.5 Design Parameters

5.5.1 Minimum and Maximum Velocities

Sewers should be "self cleansing" to limit the settling of grit, which in case of "closed channel" determined by the gradient of the sewer invert. The minimum velocity is 0.75 m/s and 0.9 m/s is desirable. The maximum velocity is limited to prevent the erosion of sewer inverts, it is between 4 to 5 m/s.

5.5.2 Storm Water Sewers Slope

The natural ground slope(G) is used because it is the technical and economic solution. But 1f Smin<G<Smax channel slope(S)=G, G>Smax S=Smax, G<Smin S=Smin.

5.5.3 Design Period

In designing of storm water system the appropriate period that used is 25 years, which is selected in the this project.

5.6 Summary

in this chapter "Basis for Planning and Design criteria", catchment areas have been found, intensity-duration curve has been estimated, runoff flow chosen, and the design parameters have been described.

CHAPTER SIX

SYSTEM DESIGN

6.1 INTRODUCTION

6.2 LAYOUT OF THE SYSTEM

6.3 DESIGN COMPUTATION

CHAPTER SIX

SYSTEM DESIGN

6.1 Introduction

In this project, an attempt is made to evaluate and design storm water drainage system for the Dura city, in order to solve the problem causes by the cumulative flooded storm water in the streets. In this chapter, the layout of the system established will be presented followed by discussion of detailed design computations .

6.2 Layout of the system

The first step in designing a storm water drainage system is to establish an overall system layout that includes a plan of the area, showing roads, streets, buildings, other utilities, topography, and soil type.

In suggesting the layout of storm water drainage system for Dura city area, the following basic steps were followed:

1. Obtain a topographic map of the area to be served.
2. Locate the catchments and sub catchments of the city and determine the area of these catchments, the catchments are determined by using Arcview GIS 3.2 program.
3. Sketch in preliminary closed channel system to serve all the areas.
4. Sewers layout is followed natural drainage ways so as to minimize excavation and pumping requirements.
5. Establish preliminary Sewers diameters that can drain the required water runoff.

6. Revise the layout so as to optimize flow-carrying capacity at minimum cost. Sewerlengths and sizes are kept as small as possible, Sewer slopes are minimized, and followed the ground surface slope to minimize the depth of excavation, and the numbers of appurtenances are kept as small as possible.

Finally, the layout of storm water drainage system for Dura city is illustrated in the Figures (6.1) and (6.2).

6.3 Design computations

The detailed design of storm water sewers involves the selection of appropriate sewers diameter, and slopes to transport the quantity of storm water, from the surrounding and upstream areas, to the next sewer in series, subject to the appropriate design constrains. The design computations and procedure for design storm water drainage system for Dura city is illustrated in the design example given below.

Design Example: Design a gravity flow storm water drainage system:

Design a gravity flow storm water drainage channel for the area line A1 .Assume that the following design criteria have been developed and adopted based on an analysis of local conditions and codes.

1. Runoff coefficient (C) uses 0.3.
2. Concentration time (T_c) use 12.432 minuets.
3. Rainfall intensity for Dura city equal 169.385 l/s.ha.
4. Runoff rate depending on the formula:

$$Q = C.i.A \tag{4.1}$$

5. For the hydraulic design equation use the Manning equation with roughness factor . To simplify the computations, use the tables in Appendix-B.

Figures 6.1

Figures 6.2

General Steps for the Solution:

1. Lay out the storm water sewer. Draw a line to represent the proposed sewer
2. Locate and number the upper and lower manhole of the line.
3. The necessary computations for the storm water system shown in table(6.1). The data in the table (6.2) are calculated as follow:

a. The entries in columns 2 through 4 are used to identify the point locations, their numbers and the length between them.

b. The entries in column 5 are used to identify the sewered area, column5 shows the partial sewered area in hectare

c. The entries in columns 6 through 13are used to identify rainfall intensity and to calculate the design flow. Runoff coefficient (C) is entered in column 6. The partial sewered area in hectare is multiplied by runoff coefficient (C) and the result is given in column 7.The cumulative multiplication of the sewered area in hectare is multiplied by runoff coefficient (C) are given in column 8. The concentration time is shown in column 10 and rainfall intensity is shown in column 11 and 12. Column 13 shows the cumulative runoff rate (Q) which obtained by multiply column (8) by column (12).

d. The necessary layout data for the sewer (columns 14 through 16) are obtained as follows: The ground surface elevations at the upper and lower point entered in columns 14 and 15, are obtained by interpolation with the elevation data. The street slope given in column 16 is obtained by subtracting downstream elevation from upstream elevation (column 14- column 15) and dividing the result by sewer length (column 4).

e. Sewer design information is summarized in columns 17 through 27. The required sewer diameter is chosen by trial and error as follow: beginning with any appropriate diameter ,and calculate new Q_{ull} and V_{ull} of selected sewer at a given slope. Q_p and V_p are obtained from the tables (Appendix-B), calculate the ratio Q_p/Q_{ull}, where Q_p is the runoff rate design flow in l/s (column 13), from Appendix-B, the ratio of h/D and V_p/V_{full} are obtained, where h is the depth of flow, V_p is the new partial velocity at a given slop.. Column 19 and 13 show the full capacity (F_{ull}) and partial capacity (Q_p) respectively. Column 18 shows the sewer slope. Column 20 and 25 show the full velocity (V_{ull}) and partial velocity (V_p) respectively. The distance between manholes chosen 70m max. The choice manhole will be based on knowledge of the street slopes and their relations.

Table. 6.1: Information Tables of Designed System

Main line	Branched line	Length(m)	Area(ha)	From (MSL)		To (MSL)	
A	A1	433	٦.١٢٨	١	٨٩٢.١٠	٨	٨٧٠.٠٠
	A2	290	٣.٣٩٨	٨	٨٧٠.٠٠	١٣	٨٦٨.٠٠
	A3	٣٢٧	٣.٩٧٨	١٣	٨٦٨.٠٠	١٩	٨٦٤.٧٠
	A4	٢٥٤	٣.٣٣٠	١٩	٨٦٤.٧٠	٢٤	٨٥٩.٠٠
	A5	٣٠٨	٢.٧٥٠	٢٤	٨٥٩.٠٠	٢٩	٨٤٧.٩٠
	A6	٧٠	١.٩٤٠	٢٩	٨٤٧.٩٠	٣٠	٨٤٥.٣٠
	A7	٤٦	٠.٩٧٠	٣٠	٨٤٥.٣٠	31	٨٣٩.٦٠
	A8	٢٠٩	١٠.٤٦	٣١	٨٣٩.٦٠	٣٦	٨٢٣.٦٠
B	B1	٥٣٢	٥.٥٥٢	١	٨٧٠.٠٠	٩	٨٦٢.٤٢
	B2	٢١٠	٢.٣٤٠	٩	٨٦٢.٤٢	١٢	٨٥٨.٨٠
	B3	١٤٠	١.٥٦٠	١٢	٨٥٨.٨٠	١٤	٨٤٧.٧٠
	B4	٧٠	٠.٧٨٠	١٤	٨٤٧.٧٠	١٥	٨٤٠.٧٠
	B5	١٣٥	١.٥٥٥	١٥	٨٤٠.٧٠	١٧	٨٣٥.٩٠
C	C1	٤٠٨	٢.٥١٠	١	٨٧٩.٨٥	٧	٨٦٠.٠٠
	C2	٢٨٠	١.٦٣١	٧	٨٦٠.٠٠	١١	٨٥٠.١٠
	C3	٢٧٦	٢٩.٢٤١	١١	٨٥٠.١٠	١٥	٨٤٥.٩٠
	C4	٣٦٠	٢.٧٧٠	١٥	٨٤٥.٩٠	٢٠	٨٣٤.٦٠
	C5	٢١٠	١.٢٣٢	٢٠	٨٣٤.٦٠	٢٣	٨٣٥.٦٠
D	D1	٤٩١	٧.٥٣٣	١	٨٣٥.٩٠	٨	٨٢٧.٤٠
	D2	٣٦٥	٢.٢٢٢	٨	٨٢٧.٤٠	١٣	٨٢٢.٤٠
E	E1	٢٤٠	٠.٣٥٠	١	٨٢٣.٧٠	٤	٨٢٢.٤٠
F	F1	١٣٠	١.٣٩٠	١	٨٢٢.٣٩	٣	٨١٨.٩٦
	F2	٤١٥	٤.٤٥١	٣	٨١٩.٩٦	٩	٨١٦.٣٨
	F3	١٤٢	١.٥٩٢	٩	٨١٦.٣٨	١١	٨١٢.٠٢
	F4	٢٤٨	٣.٥٥١	١١	٨١٢.٠٢	١٦	٨١١.٠٠
	F5	١٨٥	٤.٣٢٠	١٦	٨١١.٠٠	١٩	٨٠٦.٩٤
	F6	٣٦٨	٣.٥٥٠	١٩	٨٠٦.٩٤	٢٣	٨٠٦.٧١
	F7	١٣٣	٢.٣٨٠	٢٣	٨٠٦.٧١	٢٦	٨٠٥.٤١
G	G1	١٠٠	٣.٢١٢	١	٨٥٩.٣٦	٣	٨٤٨.٤٩
	G2	٢٠٨	٢.٧١٠	٣	٨٤٨.٤٩	٦	٨٣٢.٣٩
	G3	٢٠٧	٤.٤٥١	٦	٨٣٢.٣٩	٩	٨٢٥.٥٠

Main line	Branched line	Manhole #	(MSL)	H. Distance (m)	Notes
A	A1	1	892.10		
				٤٢	
		2	٨٩٠.٠٠		
				٧٠	
		٣	٨٨٤.٥٠٠		
				٧٠	
		٤	٨٧٩.٠٠		
				٧٠	
		٥	٨٧٥.٥٠٠		
				٧٠	
		٦	٨٧٢.٠٠		
				٧٠	
		٧	٨٧٠.٠٠		
				٤١	
		٨	٨٧٠.٠٠		
	A2				
		٨	٨٧٠.٠٠		
				٢٧	
		٩	٨٧٠.٠٠		
				٧٠	
		١٠	٨٧١.٠٠		
				٧٠	
		11	٨٧١.٠٠		
				٧٠	
		12	٨٧٠.٠٠		
				٥٣	
		13	٨٦٨.٠٠		
	A3				
		13	٨٦٨.٠٠		
				١٤	
		14	٨٦٨.٠٠		
				٧٠	
		15	٨٦٦.٠٠		

Main line	Branched line	Manhole #	(MSL)	H. Distance (m)	Notes
A	A3			٧٠	
		١٦	٨٦٤.٥٠		
				٧٠	
		١٧	٨٦٤.٤٠		
				٧٠	
		١٨	٨٦٤.٢٠		
				٢٧	
		١٩	٨٦٤.٧٠		
	A٤				
		19	٨٦٤.٧٠		
				٤٣	
		20	٨٦٣.٤٠٠		
				٣٩	
		21	٨٦٤.٠٠		
				٤٢	
		٢٢	٨٦٢.٦٠		
				٧٠	
		٢٣	٨٦١.٨٠		
				٧٠	
		٢٤	٨٥٩.٢٠		
	A5				
		24	٨٥٩.٢٠		
				٧٠	
		25	٨٥٦.٨٠		
				٧٠	
		26	٨٥٢.٥٠		
				٧٠	
		٢٧	٨٥٠.٠٠		
				٧٠	
		٢٨	٨٤٨.٤٠		
				٢٨	
		٢٩	٨٤٧.٩٠		

Main line	Branched line	Manhole #	(MSL)	H. Distance (m)	Notes
A	A6	٢٩	٨٤٧.٩٠		
				٧٠	
		٣٠	٨٤٥.٣٠		
	A7	٣٠	٨٤٥.٣٠		
		٣١	٨٣٩.٦٠		
	A8				
		31	839.60		
				25	
		32	836.7		
				46	
		33	833.50		
				36	
		34	831.10		
				٣٢	
		35	827.50		
				٧٠	
		36	823.60		
B	B1				
		1	٨٧٠.٠٠		
				٧٠	
		2	٨٦٩.٠٠		
				٧٠	
		3	٨٦٧.٧٠		
				٧٠	
		4	٨٦٦.٩٠		
				٧٠	
		5	٨٦٦.٠٠		
				٧٠	
		6	٨٦٥.٢٠		
				٧٠	

Main line	Branched line	Manhole #	(MSL)	H. Distance (m)	Notes
B		٧	٨٦٤.٠٠		
				٧٠	
		٨	٨٦٣.٠٠		
				٤٢	
		٩	٨٦٢.٤٠		
	B2				
		9	٨٦٢.٤٠		
				٧٠	
		10	٨٦٠.٤٠		
				٧٠	
		١٢	٨٥٩.٥٠		
				٧٠	
		١٣	٨٥٦.٨٠		
	B3				
		13	٨٥٦.٨٠		
				٧٠	
		14	٨٥٣.٥٠		
				٧٠	
		15	٨٤٧.٧٠		
	B4				
		16	847.70		
				70	
		17	840.70		
	B5				
		17	٨٤٠.٧٠		
				٧٠	
		١٨	٨٣٧.٨٠		
				٦٥	
		١٩	٨٣٥.٩٠		

Main line	Branched line	Manhole #	(MSL)	H. Distance (m)	Notes
		1	٨٧٩.٨٠		
				٧٠	
		2	٨٧٧.٢٠		
				٧٠	
		3	٨٧٤.٢٠		
				٧٠	
	C1	4	٨٧٠.٥٠		
				٧٠	
		5	٨٦٧.١٠		
				٧٠	
		6	٨٦٣.٥٠		
				٥٨	
		7	٨٦٠.٠٠		
		7	٨٦٠.٠٠		
				70	
		8	٨٥٧.٥٠		
C				70	
	C2	9	٨٥٥.٦٠		
				٧٠	
		10	٨٥٢.٠٠		
				٧٠	
		11	٨٥٠.١٠		
		11	٨٥٠.١٠		
				٧٠	
		12	٨٤٧.٩٠		
				٧٠	
	C3	13	٨٤٥.٥٠		
				٧٠	
		14	٨٤٥.٠٠		
				٦٥	
		١٥	٨٤٥.٩٠		

Main line	Branched line	Manhole #	(MSL)	H. Distance (m)	Notes
	C4	15	845.90		
				62	
		١٦	844.20		
				51	
		١٧	840.00		
				55	
		١٨	838.50		
				52	
		١٩	836.00		
				٧٠	
		٢٠	834.50		
	C5				
		20	834.50		
				٧٠	
		21	833.90		
				٧٠	
		22	834.50		
				٧٠	
		23	٨٣٥.٦٠		
	D1	1	٨٣٥.٦٠		
				٤٧	
		2	٨٣٤.٧٠		
				٥١	
		3	٨٣٣.٧٠		
				٧٠	
		4	٨٣٢.٠٠		
				٧٠	
		5	٨٣١.٠٠		
				٧٠	
		6	٨٣٠.٠٠		
				٧٠	
		٧	٨٢٩.٠٠		

Main line	Branched line	Manhole #	(MSL)	H. Distance (m)	Notes
				٧٠	
		٨	٨٢٨.٠٠		
				٤٣	
		٩	٨٢٧.٤٠٠		
	D2				
		٩	827.40		
				٧٠	
		١٠	٨٢٤.٠٠		
				٦٠	
		١١	٨٢١.٠٠		
				٦٥	
		١٢	٨٢٠.٠٠		
				٧٠	
		١٣	٨١٩.٠٠		
				٦٥	
		15	٨٢١.٠٠		
				٣٥	
		16	٨٢٢.٤٠		
	E				
		1	823.60		
				٧٠	
		2	822.90		
				٧٠	
		3	822.50		
				٥٠	
		4	822.50		
				٥٠	
		5	822.40		

Main line	Branched line	Manhole #	(MSL)	H. Distance (m)	Notes
F	F1	1	822.40		
				٦٠	
		2	820.50		
				٧٠	
		3	818.90		
	F2				
		3	٨١٨.٩٠		
				٧٠	
		4	٨١٨.٠٠		
				٧٠	
		5	٨١٧.٠٠		
				٧٠	
		6	٨١٦.٠٠		
				٧٠	
		٧	٨١٦.٠٠		
				٧٠	
		٨	٨١٦.٠٠		
				٦٥	
		9	٨١٦.٣٠		
	F3				
		9	816.30		
		10	813.50		
		11	812.00		
	F4				
		11	812.00		
				٢٠	
		١٢	٨١١.٥٠		
				٦٨	

Main line	Branched line	Manhole #	(MSL)	H. Distance (m)	Notes
F		١٣	٨١١.٣٠		
				٧٠	
		١٤	٨١١.٢٠		
				٧٠	
		١٥	٨١١.١٠		
				٢٠	
		16	٨١١.٠٠		
	F5				
		16	811.00		
				٥٤	
		17	809.00		
				٧٠	
		18	٨٠٧.٠٠		
				٦١	
		19	٨٠٦.٩٠		
	F6				
		19	806.90		
				٧٠	
		20	٨٠٤.٥٠		
				٧٠	
		21	٨٠٦.٠٠		
				٧٠	
		22	٨٠٧.٠٠		
				٥٨	
		23	٨٠٦.٧٠		
	F7	23	٨٠٦.٧٠		
				٤٢	
		24	٨٠٥.٧٠		
				٧٠	
		٢٥	٨٠٥.٥٠		
				٢٨	
		٢٦	٨٠٥.٤٠		

Main line	Branched line	Manhole #	(MSL)	H. Distance (m)	Notes
G	G1	١	856.4		
				٧٠	
		٢	٨٥٠.٤		
	G2	٢	٨٥٠.٤		
				٧٠	
		٣	٨٤٣.٦		
				٧٠	
		٤	٨٣٨.٥		
				٦٨	
		٥	٨٣٤.٣		
	G3	٥	٨٣٤.٣		
				٧٠	
		٦	٨٣١.٥		
				٦٩	
		٧	٨٢٩.٨		
				٦٨	
		٨	٨٢٨.٣		

Calculation sheet A3

6.4 Calculations for some lines of sewers:

Line A:

>>A 1: (from MH 1 to MH 8)

>>Channel length = 433.000

>>Tributary area (A) = 6.128.

>> Runoff coefficient (C) = 0.3.

>> C.A = 1.838 ha.

>> ∑C.A= 1.838 ha.

>> Concentration time (T_c) =ti + tf.

ti = 10 min.

tf = length of sewer/ velocity of flow.

Tc = 12.432

<< Rainfall intensity (i) = 169.385 l/s.ha.

<< Flow rate (Q) = i. ∑C.A. = 311.329 l/s.

<< Sewer slope(S) = 5.096 %.

*If:

-(G) < (S min) S=Smin.

-(G) > (Smax) S=Smax

-(Smin) < (G) < (Smax) S=G

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$ << To calculate Smax and Smin use :

<< Smax @ Vmax = 4 m/s Smin @ Vmin = 0.9 m/s

<< R = D/4

<< Diameter of sewer line determined by trial error method.

<< The constant n =0.015.

<< Maximum distance between any two manholes = 70 m.

According to table in appendix B with slope = 5.096 % , the data are :

(Q_f) = 330.700, (V_f) = 3.44 and D = 35 cm.

Thus, (Q/Q_f) = 0.941 .According to table in Appendix B ,the data are:

h/D = 0.836 ,(V/V_f) =1.050 .Where: h is the depth of flow in (cm),

V is the partial velocity in (m/sec)

D is the sewer diameter in (cm).

>>A 2: (from MH 8 to MH 13)

>>Channel length = 290.000.

>>Tributary area (A) = 3.398.

>> Runoff coefficient (C) = 0.3.

>> C.A = 1.019 ha.

>> ∑C.A= 2.857 ha.

>> Concentration time (T_c) =ti + tf.

ti = 10 min.

tf = length of sewer/ velocity of flow.

Tc = 15.548 min

<< Rainfall intensity (i) = 165.718 l/s.ha.

<< Flow rate (Q) = i. ∑C.A. = 473.456 l/s.

<< Sewer slope(S) = 0.679 %.

*If:

-(G) < (S min) S=Smin.

-(G) > (Smax) S=Smax

-(Smin) < (G) < (Smax) S=G

$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$ << To calculate Smax and Smin use :

<< Smax @ Vmax = 4 m/s Smin @ Vmin = 0.9 m/s

<< R = D/4

<< Diameter of sewer line determined by trial error method.

<< The constant n =0.015.

<< Maximum distance between any two manholes = 70 m.

According to table in appendix B with slope = 0.679 % , the data are :

(Q_f) = 506.100 l/sec, (V_f) = 1.790 m/sec and D = 60cm.

Thus, (Q/Q_f) = 0.935 .According to table in Appendix B ,the data are:

h/D = 0.827,(V/V_f) =1.060 .Where: h is the depth of flow in (cm),

V is the partial velocity in (m/sec)

D is the sewer diameter in (cm).

>>A 3: (from MH 13 to MH 19)

>>Channel length = 327.000 m.

>>Tributary area (A) = 3.978 ha.

>> Runoff coefficient (C) = 0.3.

>> C.A = 1.193 ha.

>> ∑C.A= 4.050 ha.

>> Concentration time (T_c) =ti + tf.

ti = 10 min.

tf = length of sewer/ velocity of flow.

Tc = 18.134 min

<< Rainfall intensity (i) = 162.734 l/s.ha.

<< Flow rate (Q) = i. ∑C.A. = 659.073 l/s.

<< Sewer slope(S) = 1.021 %.

*If:

-(G) < (S min) S=Smin.

-(G) > (Smax) S=Smax

-(Smin) < (G) < (Smax) S=G

$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$ << To calculate Smax and Smin use :

<< Smax @ Vmax = 4 m/s Smin @ Vmin = 0.9 m/s

<< R = D/4

<< Diameter of sewer line determined by trial error method.

<< The constant n =0.015.

<< Maximum distance between any two manholes = 70 m.

According to table in appendix B with slope = 1.021% , the data are :

(Q_f) = 921.900 l/sec, (V_f) = 2.450 m/sec and D = 70 cm.

Thus, (Q/Q_f) = 0.715 .According to table in Appendix B ,the data are:

h/D = 0.637 ,(V/V_f) =1.060 .Where: h is the depth of flow in (cm),

V is the partial velocity in (m/sec)

D is the sewer diameter in (cm).

>>A 4: (from MH 19 to MH 2)

>>Channel length = 254.000 m.

>>Tributary area (A) = 3.330 ha.

>> Runoff coefficient (C) = 0.3.

>> C.A = 1.000 ha.

>> ∑C.A= 5.050 ha.

>> Concentration time (T_c) =ti + tf.

ti = 10 min.

tf = length of sewer/ velocity of flow.

Tc = 19.530min

<< Rainfall intensity (i) = 161.146 l/s.ha.

<< Flow rate (Q) = i. ∑C.A. = 813.790 l/s.

<< Sewer slope(S) = 2.10 %.

*If:

-(G) < (S min) S=Smin.

-(G) > (Smax) S=Smax

-(Smin) < (G) < (Smax) S=G

$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$ << To calculate Smax and Smin use :

<< Smax @ Vmax = 4 m/s Smin @ Vmin = 0.9 m/s

<< R = D/4

<< Diameter of sewer line determined by trial error method.

<< The constant n =0.015.

<< Maximum distance between any two manholes = 70 m.

According to table in appendix B with slope 2.10% , the data are :

(Q_f) = 1336.800 l/sec, (V_f) = 3.740 m/sec and D = 70 cm.

Thus, (Q/Q_f) = 0.608 .According to table in Appendix B ,the data are:

h/D = 0.5676 ,(V/V_f) =1.040 .Where: h is the depth of flow in (cm),

V is the partial velocity in (m/sec)

D is the sewer diameter in (cm).

Line B :

>>B1: (from MH 1 to MH 9)

>>Channel length = 532.000 m.

>>Tributary area (A) = 5.552 ha.

>> Runoff coefficient (C) = 0.3.

>> C.A = 1.666 ha.

>> ∑C.A= 1.666 ha.

>> Concentration time (T_c) =ti + tf.

ti = 10 min.

tf = length of sewer/ velocity of flow.

Tc = 14.779 min

<< Rainfall intensity (i) = 166.615 l/s.ha.

<< Flow rate (Q) = i. ∑C.A. = 277.581 l/s.

<< Sewer slope(S) = 1.425%.

*If:

-(G) < (S min) S=Smin.

-(G) > (Smax) S=Smax

-(Smin) < (G) < (Smax) S=G

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$ << To calculate Smax and Smin use :

<< Smax @ Vmax = 4 m/s Smin @ Vmin = 0.9 m/s

<< R = D/4

<< Diameter of sewer line determined by trial error method.

<< The constant n =0.015.

<< Maximum distance between any two manholes = 70 m.

According to table in appendix B with slope = 1.425% , the data are :

(Q_f) = 339.900 l/sec, (V_f) = 2.14 m/sec and D = 45 cm.

Thus, (Q/Q_f) = 0.817 .According to table in Appendix B ,the data are:

h/D = 0.711 ,(V/V_f) =1.080 .Where: h is the depth of flow in (cm),

V is the partial velocity in (m/sec)

D is the sewer diameter in (cm).

Line C :

>>C1: (from MH 1 to MH 7)

>>Channel length = 408.000 m.

>>Tributary area (A) = 2.510 ha.

>> Runoff coefficient (C) = 0.3.

>> C.A = 0.753 ha.

>> ∑C.A= 0.753 ha.

>> Concentration time (T_c) =ti + tf.

ti = 10 min.

tf = length of sewer/ velocity of flow.

Tc = 12.933 min

<< Rainfall intensity (i) = 168.800 l/s.ha.

<< Flow rate (Q) = i. ∑C.A. = 127.106 l/s.

<< Sewer slope(S) = 4.868%.

*If:

-(G) < (S min) S=Smin.

-(G) > (Smax) S=Smax

-(Smin) < (G) < (Smax) S=G

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$ << To calculate Smax and Smin use :

<< Smax @ Vmax = 4 m/s Smin @ Vmin = 0.9 m/s

<< R = D/4

<< Diameter of sewer line determined by trial error method.

<< The constant n =0.015.

<< Maximum distance between any two manholes = 70 m.

According to table in appendix B with slope = 4.868% , the data are :

(Q_f) = 135.500 l/sec, (V_f) = 2.720 m/sec and D = 25 cm.

Thus, (Q/Q_f) = 0.938 .According to table in Appendix B ,the data are:

h/D = 0.831 ,(V/V_f) =1.060 .Where: h is the depth of flow in (cm),

V is the partial velocity in (m/sec)

D is the sewer diameter in (cm).

>>C2: (from MH 7 to MH 11)

>>Channel length = 280.000 m.

>>Tributary area (A) = 1.631 ha.

>> Runoff coefficient (C) = 0.3.

>> C.A = 0.489 ha.

>> ∑C.A= 1.242 ha.

>> Concentration time (T_c) =ti + tf.

ti = 10 min.

tf = length of sewer/ velocity of flow.

Tc = 14.959 min

<< Rainfall intensity (i) = 166.405 l/s.ha.

<< Flow rate (Q) = i. ∑C.A. = 206.675 l/s.

<< Sewer slope(S) = 3.374%.

*If:

-(G) < (S min) S=Smin.

-(G) > (Smax) S=Smax

-(Smin) < (G) < (Smax) S=G

$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$ << To calculate Smax and Smin use :

<< Smax @ Vmax = 4 m/s Smin @ Vmin = 0.9 m/s

<< R = D/4

<< Diameter of sewer line determined by trial error method.

<< The constant n =0.015.

<< Maximum distance between any two manholes = 70 m.

According to table in appendix B with slope = 3.374% , the data are :

(Q_f) = 272.600 l/sec, (V_f) = 2.830 m/sec and D = 35 cm.

Thus, (Q/Q_f) = 0.759 .According to table in Appendix B ,the data are:

h/D = 0.667 ,(V/V_f) =1.070 .Where: h is the depth of flow in (cm),

V is the partial velocity in (m/sec)

D is the sewer diameter in (cm).

Line D :

>>D1: (from MH 1 to MH 8)

>>Channel length = 491.270 m.

>>Tributary area (A) = 7.533 ha.

>> Runoff coefficient (C) = 0.3.

>> C.A = 2.620 ha.

>> ∑C.A= 17.012 ha.

>> Concentration time (T_c) =ti + tf.

ti = 10 min.

tf = length of sewer/ velocity of flow.

Tc = 24.337 min

<< Rainfall intensity (i) = 155.796l/s.ha.

<< Flow rate (Q) = i. ∑C.A. = 2650.4 l/s.

<< Sewer slope(S) = 1.663%.

*If:

-(G) < (S min) S=Smin.

-(G) > (Smax) S=Smax

-(Smin) < (G) < (Smax) S=G

$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$ << To calculate Smax and Smin use :

<< Smax @ Vmax = 4 m/s Smin @ Vmin = 0.9 m/s

<< R = D/4

<< Diameter of sewer line determined by trial error method.

<< The constant n =0.015.

<< Maximum distance between any two manholes = 70 m.

According to table in appendix B with slope = 1.667% , the data are :

(Q_f) = 3028.900 l/sec, (V_f) = 3.860 m/sec and D = 100 cm.

Thus, (Q/Q_f) = 0.875 .According to table in Appendix B ,the data are:

h/D = 0.761 ,(V/V_f) =1.070 .Where: h is the depth of flow in (cm),

V is the partial velocity in (m/sec)

D is the sewer diameter in (cm).

>>D2: (from MH 8 to MH 13)

>>Channel length = 364.95 m.

>>Tributary area (A) = 2.222 ha.

>> Runoff coefficient (C) = 0.3.

>> C.A = 0.667 ha.

>> ∑C.A= 17.679 ha.

>> Concentration time (T_c) =ti + tf.

ti = 10 min.

tf = length of sewer/ velocity of flow.

Tc = 26.294 min

<< Rainfall intensity (i) = 153.669 l/s.ha.

<< Flow rate (Q) = i. ∑C.A. = 2716.714 l/s.

<< Sewer slope(S) = 1.380%.

*If:

-(G) < (S min) S=Smin.

-(G) > (Smax) S=Smax

-(Smin) < (G) < (Smax) S=G

$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$ << To calculate Smax and Smin use :

<< Smax @ Vmax = 4 m/s Smin @ Vmin = 0.9 m/s

<< R = D/4

<< Diameter of sewer line determined by trial error method.

<< The constant n =0.015.

<< Maximum distance between any two manholes = 70 m.

According to table in appendix B with slope = 1.380% , the data are :

(Q_f) = 2789.800 l/sec, (V_f) = 3.61 m/sec and D = 100 cm.

Thus, (Q/Q_f) = 0.974 .According to table in Appendix B ,the data are:

h/D = 0.894,(V/V_f) =1.070 .Where: h is the depth of flow in (cm),

V is the partial velocity in (m/sec)

D is the sewer diameter in (cm).

Line E :

>>E1: (from MH 1 to MH 5)

>>Channel length = 240.200 m.

>>Tributary area (A) = 0.350 ha.

>> Runoff coefficient (C) = 0.3.

>> C.A = 0.105 ha.

>> ∑C.A= 9.979 ha.

>> Concentration time (T_c) =ti + tf.

ti = 10 min.

tf = length of sewer/ velocity of flow.

Tc = 22.504 min

<< Rainfall intensity (i) = 157.815 l/s.ha.

<< Flow rate (Q) = i. ∑C.A. = 1574.836 l/s.

<< Sewer slope(S) = 0.529%.

*If:

-(G) < (S min) S=Smin.

-(G) > (Smax) S=Smax

-(Smin) < (G) < (Smax) S=G

$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$ << To calculate Smax and Smin use :

<< Smax @ Vmax = 4 m/s Smin @ Vmin = 0.9 m/s

<< R = D/4

<< Diameter of sewer line determined by trial error method.

<< The constant n =0.015.

<< Maximum distance between any two manholes = 70 m.

According to table in appendix B with slope = 0.529% , the data are :

(Q_f) = 1684.000 l/sec, (V_f) = 3.350 m/sec and D = 80 cm.

Thus, (Q/Q_f) = 0.935 .According to table in Appendix B ,the data are:

h/D = 0.827,(V/V_f) =1.080 .Where: h is the depth of flow in (cm),

V is the partial velocity in (m/sec)

D is the sewer diameter in (cm).

>>F1: (from MH 1 to MH 3)

>>Channel length = 129.520 m.

>>Tributary area (A) = 1.39ha.

>> Runoff coefficient (C) = 0.3.

>> C.A = 0.417 ha.

>> ∑C.A= 28.075 ha.

>> Concentration time (T_c) =ti + tf.

ti = 10 min.

tf = length of sewer/ velocity of flow.

Tc = 27.985 min

<< Rainfall intensity (i) = 151.854 l/s.ha.

<< Flow rate (Q) = i. ∑C.A. = 4263.301 l/s.

<< Sewer slope(S) = 2.013%.

*If:

-(G) < (S min) S=Smin.

-(G) > (Smax) S=Smax

-(Smin) < (G) < (Smax) S=G

$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$ << To calculate Smax and Smin use :

<< Smax @ Vmax = 4 m/s Smin @ Vmin = 0.9 m/s

<< R = D/4

<< Diameter of sewer line determined by trial error method.

<< The constant n =0.015.

<< Maximum distance between any two manholes = 70 m.

According to table in appendix B with slope = 2.013% , the data are :

(Q_f) = 4284.400 l/sec, (V_f) = 4.51 m/sec and D = 110 cm.

Thus, (Q/Q_f) = 0.995 .According to table in Appendix B ,the data are:

h/D = 0.956,(V/V_f) =1.010 .Where: h is the depth of flow in (cm),

V is the partial velocity in (m/sec)

D is the sewer diameter in (cm).

>>F2: (from MH 3 to MH 9)

>>Channel length = 414.530 m.

>>Tributary area (A) = 4.451ha.

>> Runoff coefficient (C) = 0.3.

>> C.A = 1.335 ha.

>> ∑C.A= 29.410 ha.

>> Concentration time (T_c) =ti + tf.

ti = 10 min.

tf = length of sewer/ velocity of flow.

Tc = 30.445 min

<< Rainfall intensity (i) = 149.253 l/s.ha.

<< Flow rate (Q) = i. ∑C.A. = 4389.531 l/s.

<< Sewer slope(S) = 0.622%.

*If:

-(G) < (S min) S=Smin.

-(G) > (Smax) S=Smax

-(Smin) < (G) < (Smax) S=G

$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$ << To calculate Smax and Smin use :

<< Smax @ Vmax = 4 m/s Smin @ Vmin = 0.9 m/s

<< R = D/4

<< Diameter of sewer line determined by trial error method.

<< The constant n =0.015.

<< Maximum distance between any two manholes = 70 m.

According to table in appendix B with slope = 0.622% , the data are :

(Q_f) = 4489.5 l/sec, (V_f) = 2.92 m/sec and D = 140 cm.

Thus, (Q/Q_f) = 0.978.According to table in Appendix B ,the data are:

h/D = 0.900,(V/V_f) =1.030 .Where: h is the depth of flow in (cm),

V is the partial velocity in (m/sec)

D is the sewer diameter in (cm).

>>F3: (from MH 9 to MH 11)

>>Channel length = 141.690 m.

>>Tributary area (A) = 1.592ha.

>> Runoff coefficient (C) = 0.3.

>> C.A = 0.478 ha.

>> ∑C.A= 29.888 ha.

>> Concentration time (T_c) =ti + tf.

ti = 10 min.

tf = length of sewer/ velocity of flow.

Tc = 31.221 min

<< Rainfall intensity (i) = 148.441 l/s.ha.

<< Flow rate (Q) = i. ∑C.A. = 4436.604 l/s.

<< Sewer slope(S) = 1.46%.

*If:

-(G) < (S min) S=Smin.

-(G) > (Smax) S=Smax

-(Smin) < (G) < (Smax) S=G

$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$ << To calculate Smax and Smin use :

<< Smax @ Vmax = 4 m/s Smin @ Vmin = 0.9 m/s

<< R = D/4

<< Diameter of sewer line determined by trial error method.

<< The constant n =0.015.

<< Maximum distance between any two manholes = 70 m.

According to table in appendix B with slope = 1.460% , the data are :

(Q_f) = 6869.700 l/sec, (V_f) = 4.46 m/sec and D = 140 cm.

Thus, (Q/Q_f) = 0.646 .According to table in Appendix B ,the data are:

h/D = 0.590,(V/V_f) =1.050 .Where: h is the depth of flow in (cm),

V is the partial velocity in (m/sec)

D is the sewer diameter in (cm).

6.5 SUMMARY

In this chapter, the layout of the proposed storm water drainage system for Dura city has been described. The detailed design computations have been given and discussed. The proposed storm water drainage system has been presented.

CHAPTER SEVEN

CONCLUSIONS & APPENDIXS

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CONCLUSIONS

In this project, the trial is made to evaluate and design storm water drainage system for the center of Dura city, considering the water runoff, the wide expansion, accelerated development and growth of the city. The result brought out many important conclusions. The main conclusions drawn from the present study are summarized below:

1. Most of the areas in Dura city do not have a natural drainage outlet. Heavy rainfall causes storm water to collect in low areas and flood streets and walk ways.

2. The flow in the proposed storm water drainage system is going by gravity Because of the topographical features of the area.

3. It is found that there are five main catchments in the area, and each main catchments consists of many sub- catchments.

4. In the design of the storm water drainage system, the slope of sewers followed the slope of the ground to decrease the cost of construction.

5. We used runoff coefficient (C) equal 0.3 because most of the area is residential areas.

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APPENDIX-B

HYDRAULIC TABELES AND DIAGRAM

APPENDIX I- Inlet Spacing

Street slope, %	0-1	3-5	5-10	10-30
Spacing, m	<40	40-60	60-80	80-100

APPENDIX II-