بسم الله الرحمن الرحيم

DESIGN OF STORM WATER DRAINAGE SYSTEM FOR THE HEBRON CITY

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Palestine Polytechnic University

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CERTIFICATION

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College of Engineering and Technology

Department of Civil and Architectural Engineering

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Prepared By:

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In accordance with the recommendations of the project supervisors, 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 Department for the degree of Bachelor of Science in Engineering in Surveying and Geomatics Engineering.

Project Supervisor:

Examining Committee:

Department Chairman:

إهداء

الى أبي العزيز وأمي الحبيبة.......

الى كل الشرفاء في زمن السماسرة والمتعملقين في زمن التقزم والكادحين في زمن الاستغلال والقهر.

الى الذين سطروا بدمائهم خيوط التحرير والانقاذ,و الذين يقبسون من بطولة أسلافهم العظام،

بطولة تدك بعزماتها المؤمنة، صروح الجاهلية الباغية التي تنكرت لمنهج الحق، لتقيم على أنقاضها

دولة الفضيلة.

الى رفاق الدرب جميعا"......

الى تلك الواحة في ذاك العالم الهالك.(فلسط)...

إليهم أهدي هذا العمل

مروان الزغير

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MARWAN AL ZUGHAYAR

ABSTRACT

DESIGN OF STORM WATER DRAINAGE SYSTEM FOR

THE HEBRON CITY

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Supervisors: Dr. Majed Abu sharkh

The wide expansion and accelerated development of the Hebron 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 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

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(ألم تر أن الله أنزل من السمآء ماء فسلكه ينابيع في الأرض ثم يخرج به
زرعا مختلفا الوانه ثم يهيج فتراه مصفرا ثم يجعله حطما ان في ذلك
لذكرى لأولى الألباب)

صدق الله العظيم

الزمر:الآية **٢١**

CHAPTER ONE

INTRODUCTION

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CHAPTER ONE

INTRODUCTION

1.1GENERAL 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 the Hebron 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 the Hebron 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 Hebron city.

1.2 PROBLEM STATEMENT

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.

Hebron 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 the Hebron city easily observed most of this rainfall and provided the primary source for recharging the ground water aquifer.

Most of the areas in the Hebron 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 the Hebron 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 OF THE PROJECT

The overall objective of this study is to investigate water drainage system in the Hebron 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 the Hebron city.
2- Determine the sub catchments and catchments of the study area with the help of aerial photogram metric map and Geographical Information System (GIS).
3- Design of a new storm water drainage network for the center of the city.
4- Development of several plans for the construction of the proposed storm water network and prepare bill of quantities.
5- 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 the Hebron city faces annually from cumulated water in the streets.
2. Make visit to Metrological 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 Hebron city were obtained.
4. The layout of the proposed storm water drainage system is going.
5. The necessary hydraulic calculations and other design requirements will be carried out in the coming months.

6. Prepare bill of Quantities to the new storm water drainage system.
7. Prepare the necessary report, maps and drawings for the project.

1.5 PHASES OF THE PROJECT

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

Table 1.1: Phases of The Project With Their Expected Duration

Phase No.	Title	Duration								
		2003			2004					
		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 Phase: 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.

3. Evaluation of the contour maps and matching it with actual ground level in the project area.

1.5.2 Second Phase: Design of storm water collection system

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

1. Establish 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 Phase: Writing the report

After finishing the design calculations of the main trunks and preparing the different drawings,1 will prepare the needed bill of quantities and the cost estimation of its items, then writing finalizing the report of the project.

1.6 ORGANIZATION OF THE REPORT

The study report has been prepared in accordance with the objectives and scope 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 Project Area” describes geography of the project area, topography, climate, land use, road network, and the summary of the chapter.

The third 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 fourth chapter entitled “Design and Planning Criteria” describes introduction, catchment areas, rainfall characteristics, runoff flow, design parameters, and the summary of the chapte.

The fifth chapter entitled “Analyses and 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 sixth chapter entitled “ Bill of Quantities” deals with the item of the projects estimated quantity of each item.

The seventh chapter entitled “Conclusions” discusses the conclusions of the study.

1.7 SUMMARY

In this chapter, the problem of storm water drainage in the Hebron 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 PROJECT AREA

2.1 INTRODUCTION

2.2 GEOGRAPHY

2.3 CLIMATE

2.4 LAND USE AND ROAD NETWORK

2.5 SUMMARY

CHAPTER TWO

THE PROJECT AREA

2.1 INTRODUCTION

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

2.2 GEOGRAPHY

Hebron is situated some 35 Km south of Jerusalem, as shown in the project location plan, Fig.2.1. The project area comprises the future storm water drainage system for the Hebron 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 119320 habitant. This population is expected to grow substantially up to the year 2028 planning horizon of this project (Arabteck Jardaneh, 1999).

The topography within the municipal boundary is extremely hilly. The main part of the city is situated within a wadi system falling from north to south with steeper sided wadis leading into this, particularly from the west. The ground elevations within this area range from +1000 m with respect to sea level in the north, to around +800 m with respect to sea level in the south-east, where the main industrial area is situated.

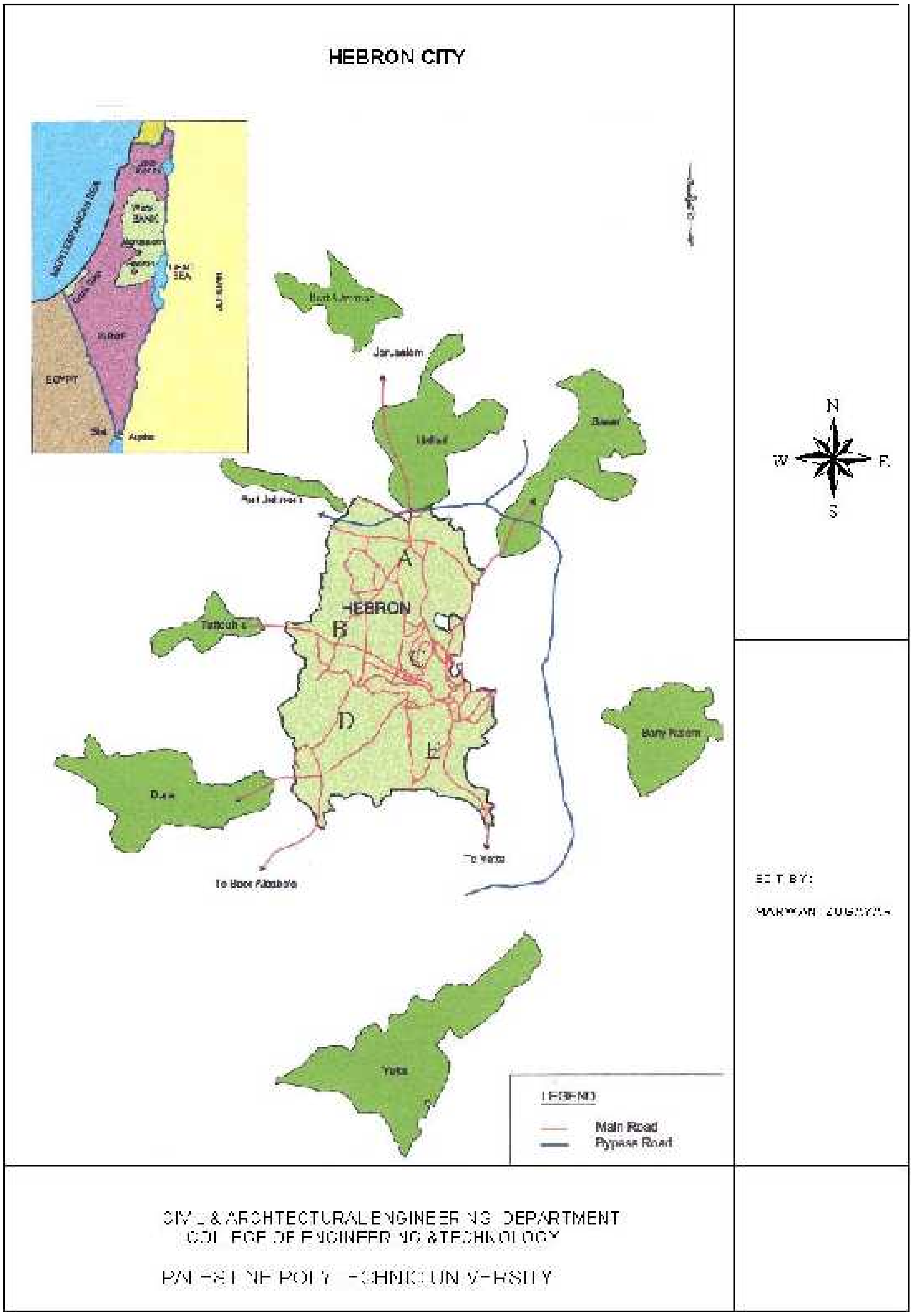


Fig.2.1-Location Map of the Hebron CITY

The commercial center of the city is situated in the east central part of the area with dense housing on the adjacent hillsides. There is also extensive commercial development along the main roads leading from the city center to Yatta and Jerusalem (Arabteck Jardaneh, 1999).

2.3 CLIMATE

Hebron 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 climateological data presented in the following paragraphs were obtained from the survey carried out by Meteorological Station which is located at the elevation of +985 m.

2.3.1 Rainfall

The average annual rainfall in the Hebron city for the last 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 1995 / 1996 – 1999 / 2000

As Given From Hebron Metrological Station

Year / Month	1995 / 1996		1996 / 1997		1997 / 1998		1998 / 1999		1999 / 2000		5 – years Mean
	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	Monthly Rainfall (mm)	No. of Raining days	Average Rainfall (mm)
September	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
October	0.4	1	29.7	4	6.9	6	7.8	1	22.9	5	13.55
November	43.0	6	18.8	5	3.9	3	16	1	1.3	3	16.6
December	77.7	7	48.6	10	145.1	11	16.7	5	220.6	11	101.74
January	122.1	15	132.4	9	114.0	11	81.4	10	145.7	10	119.12
February	37.8	9	200	90	56.0	9	72.9	5	92.2	9	91.78
March	191.3	11	88.9	15	115.7	2	22.6	7	24.8	3	88.66
April	17.2	6	10.5	3	1.0	1	44.4	3	5.1	1	15.64
May	0.0	0.0	132.6	2	2.0	2	0.0	0.0	56.9	2	14.3
Total	489.5	55	541.5	57	444.6	45	261.8	32	569.5	44	461.39

2.3.2 Temperature

The temperature range is characterized by considerable variations between summer and winter. The mean temperature values at the Hebron Metrological 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, Hebron has low values of relative humidity as compared to those in the plain. As shown in Table 2.3 the relative humidity in Hebron 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 Metrological 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 (11996 –2000)

As Given From Hebron Metrological Station

Year / Month	1996		1997		1998		1999		2000		5–years Mean Temperature	
	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
January	5	10	7	12	5	10	6	12	7	11	6	11
February	6	12	2	8	5	12	6	13	6	10	5	11
March	6	13	4	11	5	13	8	16	7	12	6	14
April	9	18	8	17	12	21	10	19	9	19	10	18
May	15	25	16	25	15	24	16	25	15	23	15	25
June	15	26	16	26	16	26	16	25	15	25	16	26
July	18	28	18	27	20	29	18	28	19	28	18	28
August	17	28	16	26	20	30	19	29	20	29	18	28
September	16	26	15	25	18	27	17	27	16	27	17	26
October	13	22	16	23	16	24	15	23	14	22	15	23
November	11	18	13	18	13	20	12	18	12	18	11	18
December	6	12	8	14	8	13	8	14	9	15	8	14

Table 2.3- Mean Monthly Relative Humidity (%) During the period (1996 – 2000)

As Given From Hebron Metrological Station

Year / Month	1996	1997	1998	1999	2000	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

2.4 LAND USE AND ROAD NETWORK

2.4.1 Land Use

The land area of the Hebron city is approximately 32.43 dounms. There is no clear city plan defining land use in the various zones of Hebron. 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. The central market place will remain in its present location and industries will be concentrated in the southern part of the city only.

2.4.2 Roads

The plan of the existing and planned roads in the Hebron city has been obtained from the Hebron municipality .The main roads traverse the city from north to south in the same direction as the main drainage channels.

2.5 SUMMARY

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

CHAPTER THREE

DESIGN OF STORM DRAINAGE SYSTEM

CHAPTER THREE

DESIGN OF STORM WATER DRAINAGE SYSTEM

3.1 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.

3.2 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 .

3.2.1 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,

$$Q = C.i.A \tag{3.1}$$

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, acres.

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.

3.2.2 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:

$$C = \frac{\sum Ci.Ai}{\sum Ai} \qquad (3.2)$$

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 3.1 and Table 3.2).

Table 3.1-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

Table 3.2- 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

3.2.3 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.

$$t_c = t_i + t_f \qquad (3.3)$$

Where t_c: time of concentration.

t_i: inlet time.

t_f: flow time.

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

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.

3.2.4 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.

3.3 HYDRAULIC CONSIDERATION

3.3.1 Introduction

Wastewater systems and (storm water) 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 open-channel flow and 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 (Metcalf,1982).

3.3.2 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:

$$V = C\sqrt{RS} \tag{3.4}$$

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

C: the Chezy coefficient; $C = \dfrac{100\sqrt{R}}{m+\sqrt{R}}$, 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

$$H = a \frac{L \times V^2}{D \times 2g} \tag{3.5}$$

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:

$$V = \frac{1}{n} R^{2/3} S^{1/2} \tag{3.6}$$

Where n: the Manning's roughness coefficient [1/n (k_{str}) = 75 $m/s^{1/3}$].

R: the hydraulic radius = area /wetted perimeter (R= A/P).

- 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 (3.3).

Table 3.3: 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

3.3.3 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 .The hydraulic characteristics are similar as for closed channels, but the velocity of flow is reduced by increased air friction in the pipe with increasing water level, particularly near the top of the pipe. The velocity of flow and the flow rate are reduced at filling rates between 60% and 100%; the water level in the pipe is unstable at filling rates above 90% or 95%.

3.4 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.

3.4.1 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.

3.4.2 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.

3.4.3 Layout Plan

Proper storm sewer layout plan 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).

3.4.4 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, and for open channel depend on the type of profile that selected.

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 3.4.

Table 3.4: 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

Note: for a velocity of 0.75m/s the slopes shown above should be multiplied by1.56.

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

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. But for the open channel it is at the ground surface.

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.

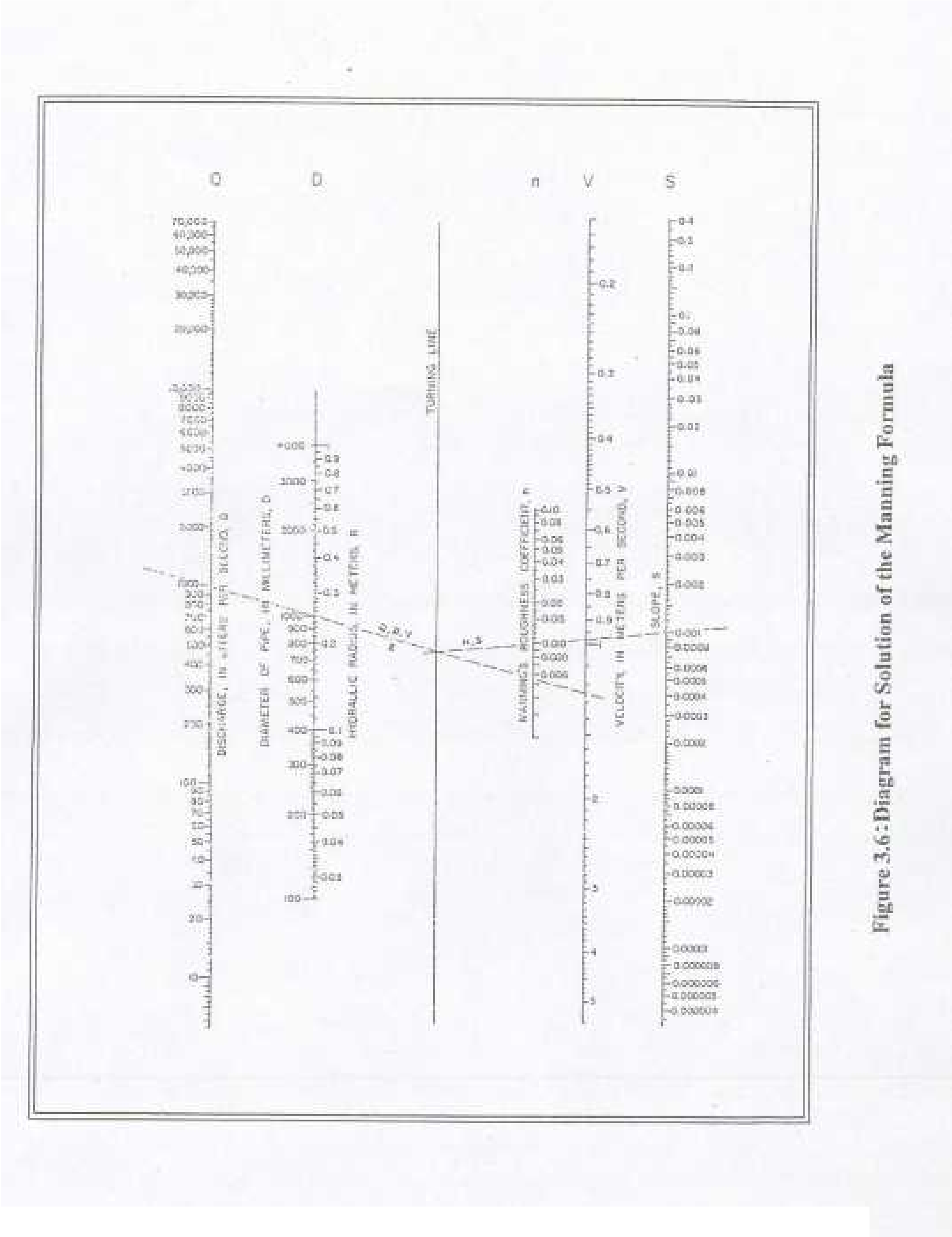
7. Design Equations and Procedures: Storm water sewers are mostly designed to flow partially full. Once the peak, average, and minimum flow estimates and made general layout and topographic features for each line are established, the design engineer begins to size the sewers. Design equations proposed 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:

$$V=\frac{1}{n}R^{2/3}S^{1/2} \tag{3.6}$$

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

$$Q = C.i.A \tag{3.1}$$

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 3.1. Hydraulic elements of circular pipes under partially-full flow conditions are provided in Figure 3.2.. It may be noted that the value of n decreases with the depth of flows Figure 3.1. 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. Use of equations 3.1 and 3.6 and Figures 3.1 and 3.2 and tables Appendix-B, one can design the drainage system.



Fig, 3.1: Diagram for Solution of the Manning Formula

3.4.5 Design Computations

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 3.5 is typical of the way in which data can be organized to facilitate computations for closed system; Table 3.6 is typical of the way in which data can be organized to facilitate computations for Open channel.

3.4.6 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 3.5: Typical Computation Sheet for Storm Water Design for Closed System

	Location						Rainfall Intensity						Street Data			D E S I G N								
Line NO	Street	Upper MH No	Lower MH No	Length	Increment Tributary Area	Tributary Sewer Line NO	Runoff coefficient(C)	C.A	ΣC.A	Concent Time(Tc)	Intensity (I)	Q	Upper MH Elevation	Lower MH Elevation	Street Slope	Sewer Diameter	Depth of Flow (h)	Capacity Full (Qf)	Partial capacity (Q)	Sewer Slope	Velocity Full (V1)	Velocity partial (V)	Time Line Flow (TL)	Time Line Comulative
				(m)	(ha)			(ha)	(ha)	min	(l/s.ha)	(l/s)	(m)	(m)	(%)	(cm)	(cm)	(l/s)	(l/s)	(%)	(m/s)	(m/s)	(min)	(min)
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	17	18	19	20	21	22	23	24	25

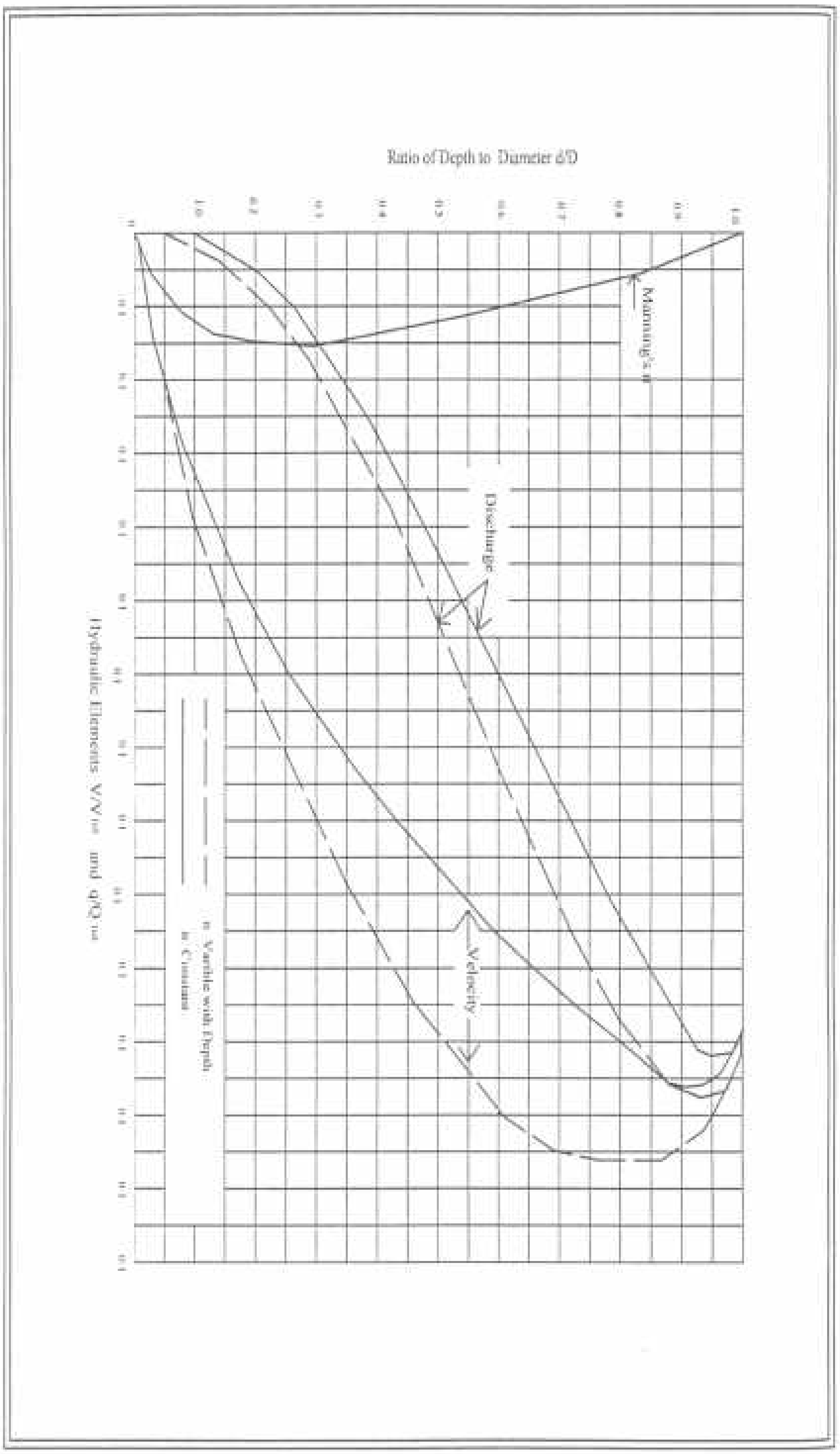


Figure 3.7: Hydraulic Properties of Circular Sewer

3.5 SUMMARY

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 and channels have been brought out. Finally the design and construction of community storm water drainage system has been briefly discussed.

CHAPTER FOUR

DESIGN AND PLANNING CRITEREA

CHAPTER FOUR

DESIGN AND PLANNING CRITEREA

4.1 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 the Hebron 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 catchment areas, rainfall characteristics, runoff flow, and the design parameters.

4.2 CATCHMENT AREAS

The Hebron city is divided into three main drainage catchment areas; the city is divided into regional catchment areas based on the topography of the area. Figure 4.1 shows the main three catchment areas of the city. The three main catchments are divided into many sub-catchment areas as illustrated in Figure 4.2. The sub-catchments and there areas have been assorted in Table 4.1. Figure 4.3 shows the catchments roads and houses.

Figure 4.1

Figure 4.2

Table 4.1- The Sub- Catchments Areas of the Hebron City

Number of Area	Tributary Sub- Catchment	Area (ha)
NW1	NW1	27.63
NW2	NW2	24.85
NW3	NW3	73.69
NW4	NW4	31.02
NW5	NW5 , NW1 , NW2	52.56
NW6	NW6	18.38
NW7	NW7 , NW3 , NW4	02.33
NW8	NW8 , NW1 , NW6	27.33
NW9	NW9 , NW5 , NW7	25.40
NW10	NW10	16.14
NW11	NW11	08.43
NW12	NW12 , NW9 , NW11	11.04
NW13	NW13	11.12
NW14	NW14 , NW12 , NW13	13.02
NW15	NW15 , NW10 , NE3	51.30
NW16	NW16	34.07
NW17	NW17 , NW14 , NW16	23.90
NW18	NW18	19.32
NW19	NW19	18.61
NW20	NW20 , NW15 , NW18	24.77
NW21	NW21 , NW20 , SW1	27.01
NW22	NW22 , NW18 , NW19	05.14
NW23	NW23 , NW17 , NW22	26.54
NE1	NE1	30.92
NE2	NE2	09.03
NE3	NE3 , NE2 , NW8	07.58
NE4	NE4	04.46
NE5	NE5, NE4	32.34
NE6	NE6	28.41
NE7	NE7	40.08
NE8	NE8, NE5 , NW21	26.45
SW1	SW1	17.35
SW2	SW2	51.34
SW3	SW3	12.79
SW4	SW4	12.08
SW5	SW5 , SW3 , SW4	04.31
SW6	SW6	08.09
SW7	SW7	08.13
SW8	SW8 , SW2 , SW10	04.31
SW9	SW9	06.92
SW10	SW10 , SW7 , SW9	07.17
SW11	SW11 , SW5 , SW8	01.38

Table 4.1- Cont.

Number of Area	Tributary Sub- Catchment	Area (ha)
SW12	SW12 , SW6 , SW11	14.82
SW13	SW13	25.95
SW14	SW14 , SW12 , SW13	33.69
SW15	SW15	23.17
SW16	SW16 , SW14 , SW15	05.09
SW17	SW17	24.53
SW18	SW18	10.90
SW19	SW19 , SW17 , SW18	17.16
SE1	SE1	03.37
SE2	SE2	10.22
SE3	SE3	11.64
SE4	SE4 , SE3	06.99
SE5	SE5	24.47
SE6	SE6 , SE1 , SE5 , NE8	9.24
SE7	SE7 , SE6 , NE6	15.81
SE8	SE8 , SE2 , SE7	16.13
SE9	SE9 , SE4 , NE7	29.68
SE10	SE10	06.00
SE11	SE11 , SE8 , SE9	12.55
SE12	SE12 , SE10	17.96
SE13	SE13 , SE11 , SE12	17.05
SE14	SE14	19.82
SE15	SE15	52.50
SE16	SE16 , SE13 , SE14	39.26
SE17	SE17	06.97
SE18	SE18 , SE16 , SE17	05.52
SE19	SE19	08.82
SE20	SE20	05.54
SE21	SE21 , SE16 , SE19	40.96
SE22	SE22 , SE20 , SE21	27.03
SE23	SE23 , SE19 , SE22	27.77
SE24	SE24	10.04
SE25	SE25 , SE23 , SE24	13.98

Figure 4.3

4.3 RAINFALL CHARACTERISTICS

4.3.1 General Condition

There is no significant variation in annual rainfall along the Hebron city from the north to the south, the average annual rainfall in the Hebron 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.

4.3.2 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 the Hebron city, the mean annual rainfall rate at Hebron 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 the Hebron city. The calculation is presented in Table 4.2.

Table 4.2: 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:

$$i = b*m^{T} \tag{4.1}$$

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 4.2 and by the help of Excel software. This allowing tracing the curve of Figure 4.4 for the Hebron city.

Figure 4.4 presents the rainfall intensity-duration curve for the 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.

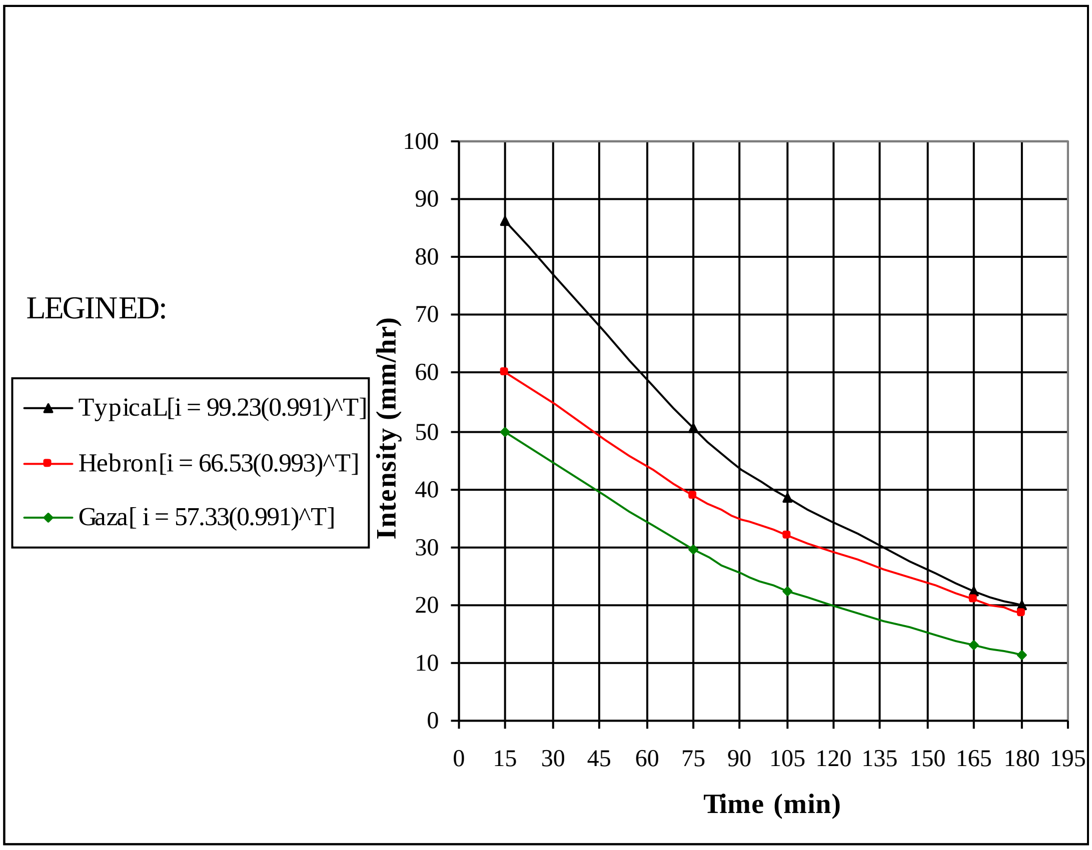


Figure 4.4 - The rainfall Intensity- Duration Curve

4.4 RUNOFF FLOW

4.4.1 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 4.3.

Table 4.3: 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 my project used runoff coeficients(C) = 0.2.

4.4.2 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

$$Q = C.i.A. \tag{3.1}$$

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

$$i = b*m^T \tag{4.1}$$

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 = 78.70 and m= 0.992.

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

$$V = \frac{1}{n} R^{2/3} S^{1/2} \tag{3.6}$$

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

4.5 DESIGN PARAMETERS

4.5.1 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. In open channel design the minimum and maximum velocity depend on the size and type of profile were we use.

4.5.2 Storm Water Channels 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.

4.5.3 Design Period

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

4.6 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 FIVE

ANALYSIS AND DESIGN

CHAPTER FIVE

ANALYSIS AND DESIGN

5.1 INTRODUCTION

In this project, an attempt is made to evaluate and design storm water drainage system for the the Hebron 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 and the final design and profiles of the suggested storm water drainage system.

5.2 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 the the Hebron 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 GIS and Arc View 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 channel sizes that can drain the required water runoff.
6. Revise the layout so as to optimize flow-carrying capacity at minimum cost. Channel lengths and sizes are kept as small as possible, channel 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 the Hebron city is illustrated in the Figures 5.A and 5.B. Table 5.1 shows the designing information for each line in the drainage network.

Figures 5.A

Figures 5.B

Table 5.1- Designing information for network lines

Line	Catchment	Area	Length	Total Area	Up.Point	Low.Point	Slope %
Line B	NW1	27.63	629.47 m	68.62 ha	982 m	945 m	5.88
	NW2	24.85					
	30.71% NW5	16.14					
Line B	Line b	68.62	744.12 m	95.97 ha	945 m	915 m	4.03
	52.04% NW5	27.35					
Line B	Line b	95.97	301.99m	228.41 ha	915 m	910 m	1.66
	NW3	73.69					
	NW4	31.02					
	NW7	2.33					
	NW9	25.4					
Line B	Line b	228.41	525.85m	3.06.09 ha	910 m	900 m	1.90
	NW11	8.43					
	NW12	11.04					
	NW13	11.12					
	NW14	13.02					
	NW16	34.07					
Line B	Line b	306.09	1015.69 m	347.49 ha	900 m	890 m	0.98
	NW17	23.90					
	65.9% NW23	17.50					
LineA	17.26% NW5	9.07	944.94 m	27.45 ha	982 m	935 m	4.97
	NW6	18.38					
Line A	Line a	27.45	563.97 m	85.70 ha	935 m	920 m	2.66
	NW8	27.33					
	NE1	30.92					
Line A	Line a	85.70	1054.92m	169.75 ha	920 m	890 m	2.84
	NW10	16.14					
	NE2	9.03					
	NE3	7.58					
	NW15	51.30					
Line D	NW18	19.32	593.57m	19.32 ha	945 m	905 m	6.74

Table 5.1- Cont.

Line	Catchment	Area	Length	Total Area	Up.Point	Low.Point	Slope %
Line D	Line d	19.32	795.28m	55.85 ha	905 m	888 m	2.14
	NW19	18.61					
	NW22	5.19					
	34.1% NW23	9.05					
	15.6% NW20	3.68					
Line C	Line A	169.75	143.06m	538.33 ha	890 m	888 m	1.40
	Line B	347.49					
	85.1% NW20	21.09					
Line E	Line C	538.33	587.85 m	638.54 ha	888 m	885 m	0.51
	Line D	55.85					
	SW1	17.35					
	NW21	27.01					
Line E	Line e	638.54	712.29 m	701.79 ha	884 m	870 m	1.97
	NE4	4.46					
	NE5	32.34					
	NE8	26.45					
Line E	Line e	701.79	137.75 m	767.28 ha	870 m	860 m	7.26
	SE1	3.37					
	SE5	24.47					
	SE6	9.24					
	NE6	28.41					
Line E	Line e	767.28	429.72 m	844.48 ha	860 m	855 m	1.16
	SE2	10.22					
	SE7	15.81					
	SE8	16.13					

Table 5.1- Cont.

Line	Catchment	Area	Length	Total Area	Up.Point	Low.Point	Slope %
Line P (16-17)	Line O (15-16)	844.48	306.59 m	916.39 ha	855 m	850 m	1.63
	NE7	40.09					
	SE4	6.99					
	SE3	11.64					
	SE9	29.68					
	SE11	12.55					
	SE10	6.00					
Line Q (17-18)	Line P (16-17)	916.39	871.27 m	1010.48 ha	850 m	835 m	1.72
	SE12	17.96					
	SE13	17.05					
	SE14	19.82					
	SE16	39.26					
Line R (18-19)	Line Q (17-18)	1010.48	313.89 m	1081.01 ha	835 m	830 m	1.60
	SE17	6.97					
	SE15	52.50					
	SE18	5.52					
	SE20	5.54					
Line S (19-20)	Line R (18-19)	1081.01	1529.317 m	1481.44 ha	830 m	810 m	1.31
	Main Catchment 2	312.79					
	Main Catchment 2	87.64					

5.3 DESIGN COMPUTATIONS

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

Design Example: Design a gravity flow storm water drainage channel

Design a gravity flow storm water drainage channel for the area [part of Hebron city line A (1)] shown in the accompanying Figure 5.1. The line is to be laid along part of Al-haras Street. 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 0.2.
2. Concentration time (T_c) use 15.370 minuets.
3. Rainfall intensity for the Hebron city equal 190.287 l/s.ha.
4. Runoff rate depending on the formula:

$$Q = C.i.A \tag{3.1}$$

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

Figures 5.1

Solution:

1. Lay out the storm water sewer. Draw a line to represent the proposed sewer (Figure 5.1).
2. Locate and number the upper and lower manhol of the line.
3. The necessary computations for the storm water sewer shown in Figure 5.1 are presented in Table 5.2. The data in the table are calculated as follow:

a. The entries in columns 2 through 5 are used to identify the point locations, their numbers and the length between them.

b. The entries in columns 6 and7 are used to identify the sewered area, column6 shows the partial sewered area in hectare, and column 7 shows the tributary sewer line number.

c. The entries in columns 8 through 13are used to identify rainfall intensity and to calculate the design flow. Runoff coefficient (C) is entered in column 8. The partial sewered area in hectare is multiblied by runoff coefficient (C) and the result is given in column 9.The cumulative multiplication of the sewered area in hectare is multiplied by runoff coefficient (C) are given in column 10. The concentration time is shown in column 11 and rainfall intensity is shown in column 12. Column 13 shows the cumulative runoff rate (Q) which obtained by multiply column (10) 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 5).

e. Sewer design information is summarized in columns 17 through 25. The required sewer diameter is chosen by trial and error as follow: beginning with any appropriate diameter ,and calculate new Q_{full} and V_{full} 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_{full}, where Q_p is the runoff rate design flow in l/s (column 13), from Appendix-B, the ratio of h/H 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 20 show the full capacity (Q_{full}) and partial capacity (Q_p) respectively. Column 21 shows the sewer slope. Column 22 and 23 show the full velocity (V_{full}) and partial velocity (V_p) respectively. Column 24 and 25 show the time of flow, column 24 shows time line flow (T_L) and it's calculated by dividing column 5 by column 23; column 25 shows cumulative time of the system network.

f. The distance between manhol chosen 70m max. The choice manhole will be based on knowledge of the street slopes and their relation

5.4 THE PROPOSED STORM WATER DRAINAGE SYSTEM

In the proposed study for the storm water drinage system the Hebron city, there are 5main chanel drainage lines. This sections deal with the results of the suggested storm water drainage network.

The appropriate channel lengths, and land slopes are found by doing the calculations given in the previous section. During and once the sewer design computations have been completed, alternative alignments have be examined, and the most cost– and energy–effective alignment has been selected. The final results for the appropriate for the proposed storm water drinage system, slopes and lengths of the channels are given in Table 5.2&5.3 The calculated velocities, flow rates, and depth of flow in channel are given in the same tables. The proposed storm water drinage system for the Hebron city is plotted in, lengths of all channels are shown. The proposed design of the storm water drainage system for each channel drainage line is plotted separately.

Figures 5.1

Figures 5.2

Figures 5.3

Figures 5.4

Figures 5.5

Figures 5.6

Figures 5.7

Figures 5.8

Figures 5.9

Figures 5.10

Figures 5.11

Figures 5.12

Figures 5.13

Figures 5.14

Figures 5.15

Figures 5.16

Figures 5.17

Figures 5.18

Figures 5.19

Figures 5.20

Figures 5.21

Figures 5.22

Figures 5.23

Figures 5.24

5.5 PROFILES OF DRAINAGE CHANNELS

The profiles of sewer area assist in the design and are used as the basis of construction drawings. The profiles are usually prepared for each sewer channel line at a horizontal and vertical scale. The profile shows the ground or street surface, manhols locations, elevation of important subsurface strata such as rock, locations of borings, all underground structures, basement elevations, and cross streets. A plan of the line and relevant other structures are usually shown on the same street (McGhee, 1991).

After all the calculation is completed and all the maps of the proposed storm water drainage system are prepared, detailed profiles for each sewer channel line are drawn. The profiles of sewer channels lines are shown in Figures 5.25to 5.549. These profiles have shown the ground elevation, the proposed sewer channels lines, manholes ,distance between manhols, depth of excavations, the sewer diameter and slopes the sewer channels lines.

Figures 5.25

Figures 5.26

Figures 5.27

Figures 5.28

Figures 5.29

Figures 5.30

Figures 5.31

Figures 5.32

Figures 5.33

Figures 5.34

Figures 5.35

Figures 5.36

Figures 5.37

Figures 5.38

Figures 5.39

Figures 5.40

Figures 5.41

Figures 5.42

Figures 5.43

Figuers 5.44

Figuers 5.45

Figuers 5.46

Figuers 5.47

Figuers 5.48

Figuers 5.49

5.6 SUMMARY

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

CHAPTER SIX

CONCLUSIONS

CHAPTER SIX

CONCLUSIONS

In this project, the trial is made to evaluate and design storm water drainage system for the Hebron city considering the water runoff, the wide expansion, accelerated development and growth of the Hebron 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 the Hebron 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 (gravity flow storm sewer system). Because of the topographical features of the area, the pumping is not needed.
3. It is found that there are three main catchments in the area, and each main catchment 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. In this project iused closed channel drainage system that is generally used for storm water and not for wastewater because, the storm water not has problems other than wastewater, whenever the water is flooded, (smell, and pollution….etc).
6. I used runoff coefficient (c) equal 0.2 because some of the area is commercial and open area.

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REFERENCES

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APPENDIX- A

TECHNICAL CALCULATIONS

TECHNICAL CALCULATION DETAILES

This appendix is prepared to explain the calculation follows to fill the table, for any misunderstanding in the following see chapter five, section (design example). These solutions are for some lines of sewer channels.

1) Line A 1

* From MH1 1 to MH6.
* Channel length = 290 m.
* Partial sewered area (A) = 9.426 ha.
* Runoff coefficient (C) = 0.2.
* C.A = 1.885 ha.
* ∑C.A= 1.885 ha.
* Concentration time (T_c) =ti+tf

ti=10min

tf=length of sewer/ velocity of flow

Tc=15.370min.

* Rainfall intensity (i) = 190.287 l/s.ha. use fig4.4
* Flow rate (Q) = i.∑C.A.

Q=358.714 l/s

- Sewer Slope = 4.14 %.
- If ground slope (G)<slope min(S) take S=Smin
- If G>Smax take S=Smax
- If Smin<G<Smax take S=G

- To calculate Smax &S min use manning equation $V=\frac{1}{n}R^{2/3}S^{1/2}$
- Smax at Vmax & Smin at Vmin.
- In this project V max=4m/s & Vmin=0.9m/s.
- R=D/4 for closed system.
- Diameter of sewer line determined by trial & error .
- Constant(n)=0.015.
- Maximum distance between two manhole 70m.
- Number of inlets= length of channel/distanse between inlets.

According to table in appendix b with slope (s) =4.00E-2 ,the data are:

Full flow (Q_f) = 421 l/s and Full velocity (V_f) = 3.350 m/s & D=40cm.

Thus, Q/Qf=0.852 According to table in appendix b,the data are:

h/D=0.738 ,V/Vf=1.07 h: depth of flow(cm),V: partial velocity(m/s),

D : sewer diameter(cm).

2) Line B10

* From MH30 to MH33.
* Channel length = 210 m.
* Partial sewered area (A) = 7.398 ha.
* Runoff coefficient (C) = 0.2.
* C.A = 1.480 ha.
* ∑C.A= 18.720 ha.
* Concentration time (T_c) =ti+∑tf

ti=10min

tf=length of sewer/ velocity of flow

Tc=22.479min

* Rainfall intensity (i) = 178.448 l/s.ha. use fig4.4

* Flow rate (Q) = i.∑C.A.

Q=3340.376 l/s

- Sewer Slope = 1.43 %.
- If ground slope (G)<slope min(S) take S=Smin
- If G>Smax take S=Smax
- If Smin<G<Smax take S=G
- To calculate Smax &S min use manning equation $V=\frac{1}{n}R^{2/3}S^{1/2}$
- Smax at Vmax & Smin at Vmin.
- In this project V max=4m/s & Vmin=0.9m/s.
- R=D/4 for closed system.
- Diameter of sewer line determined by trial & error .
- Constant(n)=0.015.
- Maximum distance between two manhole 70m
- Number of inlets= length of channel/distance between inlets.

According to table in appendix b with slope (s) =1.430E-2 ,the data are:

Full flow (Q_f) = 5655 l/s and Full velocity (V_f) = 4.25m/s & D=130cm.

Thus, Q/Qf=0.59 According to table in appendix b,the data are:

h/D=0.551 ,V/Vf=1.03 h: depth of flow(cm),V: partial velocity(m/s),

D : sewer diameter(cm).

3) Line C

* From MH1 to MH4.

* Channel length = 139 m.

* Partial sewered area (A) = 2.729ha.

* Runoff coefficient (C) = 0.2.

* C.A = 0.546 ha.

* ∑C.A= 50.427 ha.

* Concentration time (T_c) =ti+∑tf

ti=10min

tf=length of sewer/ velocity of flow

Tc=25min

* Rainfall intensity (i) = 174.422 l/s.ha. use fig4.4

* Flow rate (Q) = i.∑C.A.

Q=8795.570 l/s

- Sewer Slope = 1.3 %.
- If ground slope (G)<slope min(S) take S=Smin
- If G>Smax take S=Smax
- If Smin<G<Smax take S=G
- To calculate Smax &S min use manning equation $V=\frac{1}{n}R^{2/3}S^{1/2}$
- Smax at Vmax & Smin at Vmin.
- In this project V max=4m/s & Vmin=0.9m/s.
- R=D/4 for closed system.
- Diameter of sewer line determined by trial & error .

- Constant(n)=0.015.
- Maximum distance between two manhole 70m
- Number of inlets= length of channel/distance between inlets.

According to table in appendix b with slope (s) =1.30E-2 ,the data are:

Full flow (Q_f) = 9231 l/s and Full velocity (V_f) = 4.59m/s & D=160cm.

Thus, Q/Qf=0.95 According to table in appendix b,the data are:

h/D=0.849 ,V/Vf=1.05 h: depth of flow(cm),V: partial velocity(m/s),

D : sewer diameter(cm).

4) Line D4

* From MH10 to MH22.

* Channel length =792 m.

* Partial sewered area (A) = 20.00 ha.

* Runoff coefficient (C) = 0.2.

* C.A = 4 ha.

* ∑C.A= 9.143 ha.

* Concentration time (T_c) =ti+∑tf

ti=10min

tf=length of sewer/ velocity of flow

Tc=26.955min

* Rainfall intensity (i) = 171.366l/s.ha. use fig4.4

* Flow rate (Q) = i.∑C.A.

Q=1566.766 l/s

- Sewer Slope = 1.75 %.
- If ground slope (G)<slope min(S) take S=Smin
- If G>Smax take S=Smax
- If Smin<G<Smax take S=G
- To calculate Smax &S min use manning equation $V=\frac{1}{n}R^{2/3}S^{1/2}$
- Smax at Vmax & Smin at Vmin.
- In this project V max=4m/s & Vmin=0.9m/s.
- R=D/4 for closed system.
- Diameter of sewer line determined by trial & error .
- Constant(n)=0.015.
- Maximum distance between two manhole 70m
- Number of inlets= length of channel/distance between inlets.

According to table in appendix b with slope (s) =1.75E-2 ,the data are:

Full flow (Q_f) = 1734 l/s and Full velocity (V_f) = 3.450m/s & D=80cm.

Thus, Q/Qf=0.90 According to table in appendix b,the data are:

h/D=0.786 ,V/Vf=1.07 h: depth of flow(cm),V: partial velocity(m/s),

D : sewer diameter(cm).

5) Line E33

* From MH83 to MH85.

* Channel length =100 m.

* Partial sewered area (A) = 16.970ha.

* Runoff coefficient (C) = 0.2.

* C.A = 3.394 ha.

* ∑C.A= 104.439 ha.

* Concentration time (T_c) =ti+∑tf

ti=10min

tf=length of sewer/ velocity of flow

Tc=42.947min

* Rainfall intensity (i) = 148.298l/s.ha. use fig4.4

* Flow rate (Q) = i.∑C.A.

Q=15488 l/s

- Sewer Slope = 0.80 %.
- If ground slope (G)<slope min(S) take S=Smin
- If G>Smax take S=Smax
- If Smin<G<Smax take S=G
- To calculate Smax &S min use manning equation $V=\frac{1}{n}R^{2/3}S^{1/2}$
- Smax at Vmax & Smin at Vmin.
- In this project V max=4m/s & Vmin=0.9m/s.
- R=D/4 for closed system.
- Diameter of sewer line determined by trial & error .

- Constant(n)=0.015.
- Maximum distance between two manhole 70m
- Number of inlets= length of channel/distance between inlets.

According to table in appendix b with slope (s) =8.0E-3 ,the data are:

Full flow (Q_f) = 20946 l/s and Full velocity (V_f) = 4.10m/s & D=220cm.

Thus, Q/Qf=0.7390 According to table in appendix b,the data are:

h/D=0.653 ,V/Vf=1.07 h: depth of flow(cm),V: partial velocity(m/s),

D : sewer diameter(cm).

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-