URBAN FLOOD MODELLING: A CASE STUDY OF ROORKEE CITY

A DISSERTATION

Submitted in partial fulfilment of the requirements for the award of the degree
of
MASTER OF TECHNOLOGY
in
HYDROLOGY

By

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JUNE, 2008

CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in this dissertation entitled **"URBAN FLOOD MODELLING: A CASE STUDY OF ROORKEE CITY"** in partial fulfillment of the requirements for the award of the degree of **Master of Technology** in Hydrology, and submitted in the Department of Hydrology, Indian Institute of Technology Roorkee, Roorkee is an authentic record of my own work carried out during a period from July 2007 to June 2008 under the supervision of Dr. D. S. Arya, Assistant Professor, Department of Hydrology, Indian Institute of Technology Roorkee, Roorkee.

The matter presented in this thesis has not been submitted by me for the award of any other degree of this or any other Institute.

Date: 26/06/2008

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This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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I am greatly indebted to Dr. D. S. Arya, my supervising professor; I express my sincere gratitude for his guidance throughout my studies. He helped me to stay focused, to continually move forward toward my goals, as well as improve my literary writing and presentation skills. He provided expert advice and keen supervision. I wish to express my gratitude to him.

Also I am indebted to all the faculty members of Department of Hydrology, the Head of the Department, Dr. N. K. Geol, Dr. B. S. Mathur, Dr. D. K. Srivastava, Dr. D. C. Singhal, Dr. R. Singh, Dr. H. Joshi, Dr. M. Perumal and Dr. M. K. Jain, and all the non-teaching staff of the department.

Also I would like to acknowledge Mr. Satish Kumar, research scholar, Department of Hydrology for his valuable support and advices during this study. I am also thankful to Mr. D. P. Sharma who helped me in obtaining the maps of the study area and Mr. Sanjeev who helped me in the field inspection and local data collection.

I am profusely thankful to my wife, Nazik Ali; she was enthusiastically taking care of the children when I burnt the midnight oil. She took all additional responsibilities and spared me to accomplish the work. I am forever grateful for the steadfast love, support, and inspiration she has given to me each and every day. I am also thankful to my children (Musaab, Mohammed and Abdullah) who might have missed me during my studies at IIT Roorkee. This work is dedicated to all of them. Without the blessing of my parents (Eisa, father and Rahma, mother) and love and kindness of my sister Um Salama, I am sure I would not have achieved my goals – thank you very much for the guidance and wisdom extended all through the years building my professional and personal lives.

Thank you all very much, once again!!!

Elhadi

Urban development may lead to changes in local hydrology and water environment. Increasing levels of impervious surfaces in urban areas result in a higher volume of runoff with higher peak discharge, shorter travel time and more severe pollutant loadings.

In this study, Storm Water Management Model (SWMM) developed by the Environmental Protection Agency (of the United States) was used to simulate the runoff and transport storm water through drainage networks by performing hydrologic and hydraulic analyses of storm water. SWMM is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality primarily in urban areas. The City of Rookee was chosen as the study area. Rainfall data (SRRG) was available for the period (1977-2007) from the hydrometeorological observatory of the Department of Hydrology, IIT Roorkee. The rainfall data was analysed to calculate design storm for different durations and return periods. Annual maximum 24 hours rainfall was also used in simulation. Gumbel's Extreme Value distribution was used to find out 5 yrs, 10 yrs, 15yrs and 25 yrs return period rainfall. Hourly distribution factors provided by Indian Meteorological Department (IMD) and Central Water Commission (CWC) were used for the distribution of rainfall within 24 hours. The design return period was taken as 5 years. For I-D-F the design intensity for 30 minutes duration was taken as 101.1 mm/hr and for 24 hr maximum rainfall was 173.11 mm/day.

GIS was used to prepare study area maps and to delineate the sub-catchments. Digital Elevation Model (DEM) and Digital Surface Model (DSM) were generated using spot elevations. According to the natural drainage pattern, the study area was divided into three sub-catchments having different flow directions, 1) IIT, Roorkee campus and surroundings area up to Solani River, 2) from BSM College up to the railway station southwards and 3) from BSM College north wards up to Solani River. The selected design storm was used for storm water simulation using SWMM. Because of no flooding observed in sub-catchments 1 & 2, only sub-catchment 3 was analysed further. Simulation shows that the part of the sub-catchment 3 (in Ambertalab area) remains flooded (95 cm at node J3 and 80 cm at node J4) for about 1.5 hours.

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List of Abbreviations

Abbreviation	Definition
CWC	Central Water Commission
DEM	Digital Elevation Model
DSM	Digital Surface Model
EPA	Environmental Protection Agency of the USA
GIS	Geographical information System
GPS	Global Positioning System
IMD	Indian Meteorological department
IRI	Irrigation Research Institute
IRS	Indian Remote Sense
Pan	Panchromatic
SRTM	Shuttle Radar Topography Mission
SWMM	Storm Water Management Model

CHAPTER-I

INTRODUCTION

1.1 General

Floods are natural events that have always been an integral part of the geologic history of the earth. Flooding occurs along the rivers, streams, and lakes and coastal areas, in alluvial fan, in ground-failure areas such as subsidence, in areas influenced by structural measures, and in areas having inadequate drainage system. Urbanization influences the rainfall-runoff process in urban catchments. As a result of increasing impervious area the magnitude of runoff volume is increasing and time to peak is decreasing. With the continuous change in urbanization and climate, rainfall showing a changing trends and urban flood control has become a very severe problem.

An urban area is, by definition, an area of concentrated human activity, which is characterized by extensive impervious areas and manmade watercourses. The result is an increase in runoff volume and flow that may result in flooding watercourse and habitat destruction. Urban flooding may create considerable infrastructural and large economic losses in terms of production. Most of the urban flood occurs due to ill designed and/or insufficient drainage system. Depositions of silt in drains aggravate the flooding problems in urban areas. In many cases it has been observed that the growth of the city is haphazard having improper planning and drainage facilities.

Urban Hydrology might be defined as the study of the hydrological processes occurring within the urban environment (Putra, 2007). The engineering objectives when dealing with urban hydrology is to provide control on peak flows and maximum depths at all locations within the drainage system. The hydrologic problems that must be solved to address these objectives are the prediction of runoff peaks, volumes, and hydrograph anywhere in the drainage area. These problems are often separated into those involving the surface drainage system, for which rainfall must be converted into an overland flow hydrograph, and those involving the channel or sewer system, which often may be handled through conventional flow routing technique. Estimation of base flow in urban drainage system also requires special consideration because water may enter the channel both as infiltration (seepage into a conduit from ground water) and as domestic sewage.

Simulation and modelling of urban floods are essential to understand the bottlenecks in the drainage system and also to estimate the extent of flooding. Several mathematical models are widely used to model the dynamics of rainfall-runoff and flood generation process. In this study, use of SWMM model was envisaged to simulate the storm water and GIS was used to study the drainage pattern and for pre and post processing of data and results.

1.2 Introduction to Urban Flooding Modelling

Floods are triggered by many causes. Heavy rainfall, tropical storms, snow or ice melt, dam break, mudslide, insufficient capacity of transportation and storage are all among the major flooding origins. Geographically, there are three main types of flooding: *Riverine flooding* happens when extreme rainfall attacks a river basin, *Urban flooding* is triggered when surface runoff exceeds the capacity of drainage systems, which happens when heavy rainfall pours on sewers with the limited capacity, or even medium rainfall falls on poorly planned or operated drainage systems, and *Coastal flooding* takes place when heavy rainfall on inland encounters storm surges from the sea (Linmei, 2003).

Urban flood Modelling is complicated and there are many factors that should be considered while modelling the urban floods. The following physical processes should be taken into account while modelling urban flood:

- Rainfall-runoff processes;
- Flow in separate, or combined sewer systems;
- Flow along drains and surface streams;
- Flow or ponding in other open surface spaces;
- Flooding in basement or other underground structures, and
- Flow exchange between different parts of the drainage systems.

An urban flood model may have two sub-models: 1) *Hydrological routine* which include many components like rainfall, sub-catchments characteristics, runoff hydrographs etc. and 2) *Hydraulic routine* which include surface drains, underground sewer, junctions etc.

1.3 Flood management

Floods not only happen in river basins, the risk of urban flooding is also increasing due to rapid urbanization. Unlike river floods, urban flooding happens more frequently and causes large amount of accumulated damage, though the damage per event is relatively smaller compared with the severe consequences caused by river flooding. In addition, urban flooding has brutal impacts on municipality's activities when it happens. Therefore, more attention should be paid to it.

Some researchers and representatives of people who have been flooded think that minor adjustments to the way urban drainage is managed are not enough and that a major change in attitude is necessary. They argue that piped systems can never economically be designed to cope with all storms and that there is an unrealistic expectation that flooding should never happen. Instead, people should accept that flooding in some places is normal. Rather than attempting to prevent flooding by building bigger pipes, planners should aim to manage excess water on the surface and direct floodwater to areas where it will do the least damage, such as 'sacrificial' storage areas in parks and car parks.

1.4 Examples of Major Urban Flooding in Recent Years

On 26 July, 2005 Mumbai received a record- breaking rain of 94 cm in one day. The disaster pulled India's financial capital to a grinding halt. The flooding and subsequent mudslides wrecked havoc upon all who live in the city. (Deaths 1000; affected 20 Million; Missing 100; Evacuated 52,000; Financial Loss US$ 1 Billion) Fig (1.1).

In year 2003, a heavy rainfall coupled with high water levels in River Gash caused a large damages to Kassala city, Sudan. During the flood period 79% of the city was flooded leaving 80% of the population homeless Fig (1.2).

On July 23, 2001, a total of 620 mm rainfall was recorded in a spar of only 10 hours at Islamabad. The water level of Lai Nullah and its tributaries remarkably rose and all houses and some road bridges along the way were swept away. The flood has been the largest and heaviest among the recorded floods, and thus can be taken as a national disaster in Pakistan.

Figure (1.1) Mumbai floods, India

Figure (1.2) Kassala floods, Sudan.

1.5 Geneses of urban floods

Following are the main causes of urban flood as reported in the literature and observed:

- Rainfall over short period of time, or an ice or debris jam causes a river or stream to overflow and flood the surrounding area.
- Urban floods occur within six hours of a rain event or after a dam or levee failure.
- Flood occurs in urban areas when prolonged rainfall over several days is intense.
- As land is converted from fields or woodlands to roads and parking lots, it loses its ability to absorb rainfall these combined effects causes urban floods. Urbanisation increases runoff to six times over that would occur on natural terrain.
- Urban floods occur when the sewer system capacity is insufficient to drain the storm water as general.
- Topography, soil conditions, and ground cover also play important roles. Most urban flooding is caused by slow-moving thunderstorms repeatedly moving over the same area, or heavy rains from hurricanes and tropical storms. Urban floods on the other hand, can be slow or fast-rising, but generally develop over a period of hours or days.

1.6 Objectives of the Study

The broad objective of the study is to carryout urban flood modelling. The city of Roorkee has been selected as a case study. Following are the explicit objectives:

i. Study of drainage pattern using GIS in the City of Roorkee,

ii. Design storm analysis, and

iii. Study of urban floods by storm water modelling using SWMM.

1.7 Methodology

i. For the first objective, SRTM data shall be used and the available remote sensing data in the Department of Hydrology and from Google Earth will be used to prepare the drainage pattern.

ii. In the second objective, SRRG data for the last 10 years shall be analysed for the design storm to update the earlier work. Annual 24 hours maximum rainfall values shall also be considered for the design storm. Gumbel's Extreme Value distribution will be used to calculate the various return period rainfalls.

iii. Using the results of (i) and (ii) to study the urban floods, storm water will be simulated for the selected design storm using SWMM software.

CHAPTER-II

URBAN FLOOD MODELLING: A REVIEW OF METHODS AND MODELS

2.1 - Literature Review: Urban flood modelling

Urban development may lead to changes in hydrology and eco-hydrology of urban conglomerations. Increasing levels of impervious surfaces in urban areas result in a higher volume of runoff with higher peak discharge, shorter travel time and more severe pollutant loadings. Urban imperviousness is very important parameter to manage for urban watershed and the related water environment. Lee et al. (2003) explained how directly connected impervious area (DCIA) can affect runoff volume. The British Lloyd-Davies rational Method assumes that the (DCIA) contributes 100% runoff for whole urban catchments. The oldest and still widely used method for storm drainage design is the Rational Method that was first introduced by the Irish engineer Mulvane (1850). The American Kuichling (1880) and the British Lloyd Davies (1906). While the American Rational Method uses the runoff coefficient according to runoff characteristics and total land area, Lloyd Davies method only consider 100% of runoff from the DCIA.

Bennis and Crobeddu (2007) developed a new runoff simulation model based on the improved rational hydrograph method. It represents the urban catchments as a linear system where the impulse response function is rectangular-shaped, with duration equal to the time of concentration. The runoff is computed with a convolution product between the rainfall intensity and the impulse response function of the catchments. It explicitly considered the contribution of pervious and impervious areas, time of variability of rainfall, the initial abstraction and infiltration.

In a study carried by Ole et al. (2004), a combination of GIS and 1D hydrodynamic model has been used to constitute a cost efficient system for planning and management of drainage system suffering from urban flooding. The result was easily understandable flood inundation maps (Dhaka and Bangkok cities). Also, Ole et. al. (2004) concluded that only a few projects have dealt with modelling of urban flooding in developing countries. Some of the few case studies dealing with modelling of urban flooding which includes both the pipe network system and extended surface flooding are: Bangkok; Dhaka City; Fukuoka and Tokyo (Japan); Harris Gully (USA); Indore (India) and Playa de Gandia (Spain). These studies modelled urban flooding as a one-dimensional (1D) problem.

Haping and Jianzhong (1999) developed an Advanced Urban Flood Dynamic Simulation Model (UFDSM). It is a new type model to calculate and forecast urban storm water and flooding water, which has been applied primarily in some cities in China. Compared with traditional method, UFDSM/GIS is very efficient in data input, output, storing, searching, displaying and analysing.

Hsu et al. (2000) developed an urban inundation model, combining a storm sewer model SWMM, two-dimensional (2D) diffusive overland-flow model and operations of pumping stations to simulate inundation in urban areas caused by the surcharge of storm sewers and outlet pumping stations. SWMM is employed to solve the storm sewer flow component and to provide the surcharged flow hydrographs for surface runoff exceeding the capacity of the storm sewers. The 2D diffusive overland-flow model considering the non-inertia equation with Alternative Direction Explicit numerical scheme is then used to calculate the detailed inundation zones and depths due to the surcharged water on overland surface. The combined model is suitable for analysis of inundation on urban areas due to overflow of storm sewers and flooding caused by failure of pumping stations.

The integration of SWMM model with GIS for Kansas City and Missouri resulted in a more effective way of gathering and storing data, creating data files, and displaying modelling results. Finally, with the City maintaining the modelling information within their GIS, future master plans will benefit from the up-to-date information (Jennifer, 1994).

SWMM was used with HEC-FDA software Thomas, (2002) to calculate expected annual flood damages associated with existing conditions and proposed alternatives to flood control projects. The Southwest Louisville Flooding (Ohio River, U.S.A) Study provided a special challenge because of the large number of structures in the study area (68,000), the size of the study area (32.0 sq. mi), and the large amount of data being generated from the urban hydraulics software (SWMM), which modelled a combined sewer area with over 216 miles of pipe with a diameter of 18" or greater and over 4,800 sewer manholes.

In study by Kartika, E., 2006, in IITR campus, SWMM was used to simulate rainfall-runoff processes and evaluated the drainage system. The model worked successfully and gave good result. In study by J. R., Putra, 2007, SWMM was used to evaluate drainage system of Sekanak catchment, the study showed great potential of SWMM to evaluate the efficiency of the drainage system.

An urban storm water model usually requires extensive spatial data because of the complexity of urban surfaces, flow paths, and conduits. Many of these features are geographic in nature; therefore Geographic Information Systems (GIS) is required in urban flood modelling. A good overview of the concepts of GIS and database technology and their application in the field of natural systems hydrology was given by Singh and Fiorentino (David et al., 2001). GIS has a long history of use in water resources field. However, application of GIS in urban storm systems has been limited because of the need for large, extensive, and detailed spatial and temporal database. Use of GIS in urban storm water modelling is a growing technology designed for storing, manipulating, analysing, and displaying data in a geographical context. It can be characterized as a software package that efficiently relates geographic information to attribute data stored in a database. To make GIS a more prominent feature in urban storm water modelling, urban storm-water engineers have to work with GIS specialists and eventually be trained by them.

Urban areas always present some risk of flooding when rainfall occurs (Satterthwaite, 2008). Buildings, roads, infrastructure and other paved areas prevent rainfall from infiltrating into the soil – and so produce more runoff. Heavy and/or prolonged rainfall produces very large volumes of surface water in any city, which can easily overwhelm drainage systems. In well-developed cities, this is rarely a problem because good provision for storm and surface drainage is easily built into the urban fabric, with complementary measures to protect against flooding

2.2 Literature Review: Hydrodynamic modelling

In study by Cevsa and Miguel, (2007), Environmental Protection Agency storm water management model (SWMM) calibrated with measured rainfall and surface runoff flow data, used to check the accuracy of the results obtained by an algorithm developed using the Mac-Cormack explicit finite difference method to solve the kinematic and diffusion wave governing equations for both overland and open channel flow. The results obtained from SWMM are in good agreement with the results obtained from applying the MacCormack algorithm.

H. J., Fouad and M. H., Rabi (2002) developed a newly accuracy-based dynamic time step estimate for one dimensional overland flow kinematic wave, its function of the mesh size, watershed slope, roughness, excess rainfall and time of concentration. The new criteria were developed by comparing the consistent formulation of the Glarkin-Cracnk Nicholson numerical solution of the kinematic wave equation to the characteristic mesh-based analytical solution. This method is aimed at solving the problem of stability criteria in solving kinematic wave overland flow solution.

Distributed parameter models are classified according to (1) the description of the runoff processes and (2) the dimensionality of the flow description (Hawes et al. 2006). In the first case, models are classified as deterministic, stochastic, or mixed, depending on the degree of certainty with which the runoff processes are described in the model. In the second case, models are classified as either one-dimensional (1D) or two-dimensional (2D). For the simulation of runoff in very small watersheds, both types of model typically employ the kinematic wave approximation to the Saint Venant flow equations. This method involves numerically solving the continuity or mass balance equation using a uniform flow approximation to compute flow velocity. 1D runoff models are based on the assumption that runoff from a watershed can be treated as a set of 1Dflows and that these flows may be integrated to provide a simulated hydrograph at the outlet of the watershed. This concept is generally implemented in a distributed parameter model in one of two ways. The first involves defining the model elements such that they form cascades, whereas the second involves the use of a flow routing algorithm to determine a single outflow direction for each element based on the local topography. In contrast to the 1D models described above, 2D models route flow implicitly and are mathematically more complex. However, 2D models simulate the spatial distribution of overland flow more realistically and with greater accuracy than 1D models do.

2.3 Review of Urban storm water models

There are literally hundreds of models developed by academic institutions, regulatory authorities, governmental department and engineering consultants that are capable of simulating water quality and quantity in an urban catchment. The models chosen below represent a wide range of capabilities, spatial and temporal resolutions. A more detailed description of the capabilities of these models is provided below.

DR_3M-QUA: it is developed by US geological Survey. It is Distributed Routing Rainfall-Runoff model (DR_3M), used for water quality and quantity routing (Zoppuo, 2001).

HSPF: Hydrological Simulation Program-Fortran (HSPF) was developed in mid 1970s by US EPA to model broad range of hydrologic and water quality processes in agricultural and rural watershed. Urban watershed can also be simulated. It is considered the most comprehensive and flexible model of watershed hydrology and water quality available it is a continuous watershed hydrology and water quality simulation package.

Mike-SWMM: this package combines *MIKE 11* and the well known *SWMM* model to strengthen Mike 11 in one-dimensional unsteady flow modelling, which solves the shallow water wave equations using implicit finite difference scheme, replacing the temperamental *EXTRAN* module in *SWMM*. The combined model can perform hydrologic, hydraulic and water quality analysis of storm water and waste water drainage systems.

QQS: the Quality-Quantity Simulator (QQS) can perform continuous or single event simulation using five-minute time intervals. In can simulate flows in pipes and channel using an implicit finite difference approximation of the kinematic wave equations, storage routing, backwater analysis and pipes under pressure. Quality routing through channels and pipes, storage and receiving water is performed using plug flow.

SWMM:, SWMM was developed by a consortium of American engineers for the US Environmental Protection Agency (EPA). It has been applied globally for storm water planning, design and rehabilitation purposes. It is a mathematical model capable of representing urban storm runoff including sewage storage and treatment and combined sewer overflow phenomenon. It will be used in this study and explained in detail later.

Wallingford Model: Wallingford model is a suite of models developed at the Hydraulics Research Institute, Wallingford, United Kingdom. The Wallingford Procedure describes the hydraulic design and analysis of pipe networks for both new schemes and existing systems. It can accommodate both storm water sewers and combined sewers. The whole package provides a range of methods from which a series of calculation techniques can be selected to suit the conditions of any particular design scheme (Zoppuo, 2001, Linmei, 2003).

STORM: It was developed in Hydrologic Engineering Centre in 1977 by the US Corps. of Engineers, storage, treatment, overflow model is capable of simulating runoff and pollutant loads from urban and rural watersheds in response to precipitation. It is a continuous model and it can be used for single events. There is no attempt to route runoff along the catchment. Three methods are available for calculating the hourly runoff; coefficient method, soil-complex-cover method and unit hydrograph method. The runoff is a linear relationship between runoff and precipitation minus rainfall (Zoppuo, 2001). Comparison of functionality and accessibility of some storm water model was given in table (2.1).

2.4- Hydrologic Methods Used for Storm Water Modelling

This model is used in estimating quantities of storm water runoff from urban drainage areas and other small watersheds. This model is based on conservation of mass, usually satisfy continuity equation. This accounts for various hydrologic processes that produce runoff from urban areas. These include rainfall, abstraction, evaporation, infiltration, detention storage and nonlinear reservoir routing of overland flow. Details information about hydrologic models was given in this chapter.

Table (2.1) functionality and accessibility of some urban storm water models

(Source; Zoppou 2001)

Model name	Functionality			Accessibility	
	Planning	Operation	Design	Public	commercial
DR3-QUAL	√		√	√	
HSPF	√		√	√	
MIKE-SWMM	√	√	√		√
QQS	√		√	?	
STORM	√			√	
SWMM	√		√	√	
Wallingford	√	√	√		√

2.4.1-Rainfall-Runoff Processes

Runoff is that fraction of the rainfall which moves over the surface or through the soil towards water features. For most purpose, runoff refers to surface runoff only. Runoff occurs when the rainfall exceeds the demands of interception, evaporation, infiltration, and surface storage. Runoff occurs after the intensity of the rainfall exceeds the rate at which water can infiltrate the soil. Infiltration is the entry of water into the soil surface. The movement of the water downward through the soil profiles is called percolation. When the intensity of the rainfall exceeds the infiltration rate, the excess rainfall begins to pond on the soil surface. This water fills the small depressions caused by irregularities in the soil surface. The volume of water that is temporarily held in these depressions is called surface storage. When the rainfall ceases, the water held in surface storage will either infiltrate into the soil or evaporate. The volume of water that exceeds the volume of surface storage becomes surface runoff. Runoff may begin as relatively uniform layer of water moving over the soil surface called sheet flow. Gradually the water begins to concentrate into small channels.

They are two groups of factors affecting runoff processes, namely *storm characteristics* and *watershed characteristics*. The storm characteristics that influence runoff are intensity, duration, and areal extend of the storm. Watershed characteristics that influence runoff include size, shape, and topography, soils, and vegetation. As the size of a watershed increases, total runoff volumes and peak runoff rates increase. Watersheds that are long and narrow will generally have reduced peak runoff rates compared to more compact watersheds of the same size, because it takes longer for runoff from the most remote point of the watershed to reach the outlet. The time required for runoff from the most remote point of the watershed to reach the outlet is called the time of concentration. If the storm has duration less than the time of concentration of the watershed, peak flows will be less than would occur for a storm of the same intensity with duration equalling or exceeding the time of concentration (T_c). Topography of the watershed also influences runoff rates and volumes.

Watersheds with an extensive network of steep channels will produce greater runoff rates than watershed with few channels or one having mild slope. Watershed with deep, permeable soils will produce less runoff than watersheds with thin soil overlying less permeable materials. Vegetation influences the rate and volume of runoff because it retards the flow of the runoff over the soil surface. There are many methods used for runoff computation, relevant ones are discussed further in the following section.

2.4.2-Rational Formula

The rational formula for estimating peak runoff rates dates from the 1850s in Ireland and was introduced in the United State by Emil Kuichling in 1880. Since then it has become the most widely used method for designing drainage facilities for small urban and rural watersheds (up to 300 ha. (Kartika, 2006). Peak flow is found from:

$$Q_p = K_c C^* I^* A \qquad (2.1)$$

When US customary units are used, then conversion factor Kc = 1.008 to convert acre inch/hr to ft^3/sec is routinely ignored. This conversion is the basis for the term rational method.

Where,

Q_p = the peak runoff rate (cusecs or cumecs)

I = Rainfall intensity is that for the time of concentration of the total area drained.

Kc = conversion factor (1.0 acre in/hr = 1.008 cfs, and 1.0 ha mm/hr = 0.00278 cms)

The rational for the method lies in concept that application of steady, uniform rainfall intensity will cause runoff to reach it is maximum rate when all parts of the watershed are contributing to the outflow at the point of design. That condition is met after elapsed time t_c the time of concentration, which usually is taken as the time for a drop of water to flow from the most remote part of the watershed to the point of design. At this time, the runoff rate matches the net rain rate.

Rainfall intensity

It has been observed that shorter the duration of critical rainfall, the greater would be the expected average intensity during the period. For example, during a 30 minute rainfall, some 5 minute period, or any period less than 30 minutes length, will have average rainfall intensity greater than that of the whole storm. The critical duration of the storm will be which produces maximum runoff. This duration is equal to the time of concentration, since shorter periods do not allow the whole area to contribute water, and longer duration will give smaller average rainfall intensity. The problem thus reduces to one of establishing a relation between time of rainfall duration and probable or expected rainfall intensity. For the design purpose high intensities are of importance.

Time of Concentration

Time of concentration is defined as the time required for rain falling at the hydraulically most remote location in the watershed to reach the outlet (Larry, 2001). It is determined from the hydraulic characteristics of the principal flow path, which typically is divided into two parts, overland flow and flow in defined channels; the times of flow for each segment are added to obtain t_c. Many formulae have been used for computation of tc. Like,

1. Kinematic wave time of concentration formula

$$t_c = \frac{0.93 l^{0.6} N^{0.6}}{i^{0.4} S^{0.3}} \qquad (2.2)$$

2. Kirpich formula

$$t_c = 0.0078 * \left[\frac{l^{0.77}}{S^{0.385}} \right] \qquad (2.3)$$

S = Slope ft/ft, i = rainfall intensity in/hr, n = Manning roughness and l = length of flow path ft.

3. National Resources Conservation Service (NRCS) formula.

$$t_c = 1.67 t_l \qquad (2.4)$$

$$t_l = l^{0.8} \frac{(S+1)^{0.7}}{1900 w_s^{0.5}} \qquad (2.5)$$

Where,

tc = time of concentration

l = travel length (ft)

S = slope ft/ft

t_l = watershed lag time

w_s = average watershed slope (%)

Calculation of flow time in storm drains can readily be estimated by knowing the type of pipe or channel, slope size, and discharge. The estimation of inlet time is frequently based on judgement; reported values vary from 5-30 minutes. Densely developed areas with impervious tracts immediately adjacent to the inlet might be assigned inlet periods of 5 min, but minimum value of 10-20 min is more usual. For a critical area, the time of concentration is based on two components: these components are inlet time i. e., the time required for the rain water to flow over the land surface and enter the drain at various inlets and the time of flow. The time of flow is the time required for water to flow through the drain from the starting point up to the critical section under examination. In this study, time of concentration for overland flow in the sub-catchment is worked out using Kirpich Formula.

The Runoff Coefficient 'C'

The runoff coefficient C in the rational formula is the portion of precipitation that makes its way to the drain. Its value depends on a large number of factors such as permeability of the surface, type of ground cover, shape and size of catchment, the topography, the geology, antecedent moisture condition, recurrence interval, land use, and amount of urban development, rainfall intensity, surface and channel roughness, and duration of storm. It is also to be remembered that runoff coefficient tends to become larger as rain fall continues due to filling of depressions in impervious surfaces and saturation of the upper layers of exposed soil. The value of 'C' commonly adopted for use in Rational Formula is given in the following table.

Advantage of rational Method; Simple, Quick and give good estimate. *The disadvantage* of the Method; runoff coefficient tend to be more opinion than fact, time of concentration also hard to estimate and more over factors are very dependent on local conditions

2.4.3- SCS Curve Number (CN) Method

The U.S. Soil Conservation Service (SCS) in 1975 developed three procedures for estimating runoff volume and peak rates of discharge from urban areas. They are known collectively as Technical Release-55 (TR-55) and individually as the graphical method, chart method, and tabular method. The method was edited in 1986 to incorporate more versions for accurate estimation of runoff in urban areas. (TR-55) presents simplified procedures to calculate storm runoff volume, peak rate of discharge, hydrographs, and storage volumes required for floodwater reservoirs. These procedures are applicable to small watersheds, especially urbanizing watersheds, in the United States. TR-55 provides a number of techniques that are useful for modelling small watersheds. It utilizes the SCS runoff equation to predict the peak rate of runoff as well as the total volume. TR-55 also provides a simplified "tabular method" for the generation of complete runoff hydrographs. (The SCS is now known as the National Resource Conservation Service, or NRCS: the former name is used here because of its widespread use in the literature and well known).

Table (2.2) Values of Runoff Coefficient 'C' (Source: IRC: SP: 50-1999)

Sl. No.	Description of Surface	Value of 'C'
i)	Watertight surface (concrete or bitumen), steep bare rock	0.9
ii)	Moderately steep built up area with about 70% impervious	0.8
iii)	Flat built up area with about 60% area impervious	0.55
iv)	Unpaved area along roads	0.3
v)	Green area (loamy)	0.3
vi)	Green area (sandy)	0.2
vii)	Lawns and parks	0.15

The basic equation for computing the excess rainfall or direct runoff from a storm water by SCS method is given as follows:

$$P_e = \frac{(P - I_a)^2}{P - I_a + S} \qquad (2.6)$$

Where

P_e = excess rainfall in inches (always less than or equal to the depth of precipitation P)

I_a = initial abstraction before ponding (in), for which no runoff will occur

S = potential Maximum storage (in)

By study of results from many small experimental watersheds, an empirical relation was developed.

$$I_a = 0.2S \qquad (2.7)$$

On this basis

$$P_e = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad (2.8)$$

$$S = z\left[\frac{1000}{CN} - 10\right] \qquad (2.9)$$

z = conversion factor (z = 1 for cfs, z = 25.4 for metric system)

Limitations of SCS CN Method

- Since daily rainfall data were used in the development of the equation, the time distribution and duration of storms were not considered. If all other factors are constant, all storms having the same rainfall magnitude but different duration or intensity will produce equal amount of direct runoff volume. Whereas rainfall intensity does have an effect on the hydrologic response of the watershed.
- The equation tends to over predict runoff volume for a discontinuous storm, because it does not account for the recovery of soil storage caused by infiltration during periods of no rain.
- The CN procedure does not work well in areas where large proportion of flow is subsurface, rather than direct runoff.
- Since the SCS curve numbers were developed from annual maximum one-day runoff data, the CN procedure is less accurate when dealing with small runoff events.

The SCS runoff equation is widely used in estimating direct runoff because of its simplicity, flexibility, and versatility. The hydrologic data used to estimate CN are normally available in most ungauged watersheds. Since CN is the only parameter required, the accuracy of runoff prediction is entirely dependent on the accuracy of CN.

2.5- Hydrodynamic Modelling Methods of Urban Storm Water

This model used to calculate the flow rate and water levels for description of propagation of flow in time and space. Kinematic, Diffusion Approximation or full Dynamic wave modelling is used to simulate flow in storm drainage systems and flows as general. Flow propagation along river channel or an urban watershed drainage system is an unsteady non-uniform flow, unsteady because it varies in time, non-uniform because flow properties such as water surface profile (elevation), velocity, and discharge are not constant along the channel. This model solves continuity equation Eq. (2.10) as well as momentum Eq. (2.11) or energy equation as coupled system of equation. This model describe the spatial variability of the process, it allows full consideration to backwater effects and pressurized flow by solving the full Saint Venant's equations.

Continuity Equation:

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}=q \qquad (2.10)$$

Where,

A = the cross sectional area of the Channel

Q = discharge at the section

t = time

x = Distance

q = lateral inflow per unit length.

Momentum Equation:

$$g^* \frac{\partial h}{\partial x} + v^* \frac{\partial v}{\partial x} + \frac{\partial v}{\partial t} = g^*(s_0 - s_f) \qquad (2.11)$$

Where,

v = velocity

h = water depth

g = acceleration due to gravity

S_0 = bed slope

S_f = friction slope

x = distance

t = time

Flow through the sewer network is considered as one dimensional flow (1D flow). One Dimensional distributed routing method has been classified as:

i. Kinematic Wave Routing, which governs the flow when the inertial and pressure forces are not important, when the gravitational forces of the flow are balanced by the frictional resistance force. It is useful when the slope are steep and back water effect negligible Eq. (2.12)

$$S_f = S_o \qquad (2.12)$$

ii. Diffusion Wave routing, is used when the pressure force is important and inertial force remaining unimportant Eq. (2.13). Both kinematics and diffusion wave models are helpful in describing downstream wave propagation when the slope of the channel is greater than (0.01 %) and there are no wave propagating upstream due to disturbance such as tide, turbulence inflow, and reservoir operation.

$$\frac{\partial h}{\partial x} = (s_0 - s_f) \qquad (2.13)$$

iii. Dynamic wave model, when both inertial and pressure forces are important such as in mild-sloped rivers or sewer network, and backwater effect, from downstream disturbances are not negligible, then both the inertial and pressure force terms in the momentum equation are needed, under this circumstances the dynamic wave routing method is required, which involves numerical solution of the full Saint Venant's equation Eq. (2.11).

2.5.1-Numerical Solution of St. Venant's Equation

These partial differential equations {Eq. (2.10) & (2.11)}, were developed a century ago, have only been recently applied to general hydrologic engineering problems, because, it was not possible to solve these equations efficiently without high speed digital computers. Basically there are two types of numerical solution methods or finite difference techniques used in solving the differential equations encountered in the kinematics wave approximation of the Saint Venant's equation; they are the explicit and implicit scheme.

Explicit Scheme: Explicit methods applied to the governing equations usually resulting in linear algebraic equations from which the unknowns can be evaluated directly or sequentially without iterative computations. This method is simpler but can be unstable, which means that small values of Δx and Δt are required for convergence of the numerical procedure.

*Implicit Scheme:*Implicit finite-difference method advances the solution of the St. Venant's Equation from one time line to the next simultaneously for all points along the time line. It involves nonlinear algebraic finite difference equations which involve iterations and stable for large computation steps with little loss of accuracy and hence works much faster than the explicit method. However, mathematically this method is more complicated. A major difference between implicit and explicit methods is that implicit method are conditionally stable for all time steps, where as explicit method are numerically stable only for time steps less than a critical value determined by courant condition.

In urban flooding simulation, the hydrological processes are separated conceptually from the hydraulic of the drainage system. The computation of the surface runoff from rainfall can be carried out by standard surface runoff model e.g. time/area, kinematics or linear reservoir models. Rainfall from each sub-catchment is then used as input for the dynamic model, simulation flows in the pipe and street system. The runoff from the catchments is entered in the model either on the street or directly in the sewer depending on the layout of the drainage system.

2.6- Recommendations of IRC

Indian Road Congress (IRC: SP: 50-1999) gives some of the current practice being followed in some metropolitan cities in India as follows:-

i. Bombay: the runoff coefficient adopted in fully developed area is 1.0. In less developed areas the coefficient is worked out which may range between 0.58 to 1.0. The critical intensity of rainfall is considered 50 mm/hr and the frequency of the storm 2 times a year.

ii. Madras: the intensity of rainfall adopted is 25 mm/hr. this roughly corresponds to rainfall intensity of 60 minutes duration with a frequency of 1 in 1.25 years.

iii. Delhi: the average value of runoff which is adopted for different category of drains is as follows:-

a) Internal drains (0.177 m^3/ha)	1 cft/acre
b) Intercepting drains (0.132 m^3/ha)	0.75 cusec/acre
c) Main drain (0.88 m^3/ha)	0.5 cusec/acre

the above values have been worked out on the following assumption; rainfall intensity of 30 minutes duration at the rate of 2.5`` (62.5mm) per hour occurs once in two years. Time of concentration 30 min minutes and the average runoff coefficient adopted is 0.6.

2.7-A Review of Storm Water Management Model (SWMM)

2.7.1 Introduction

The Environmental Protection Agency Storm Water Management Model (EPA SWMM) is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. The runoff component of SWMM operates on a collection of sub-catchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, channels, storage/treatment devices, pumps, and regulators. SWMM tracks the quantity and quality of runoff generated within each sub-catchment, and the flow rate, flow depth, and quality of water in each pipe and channel during a simulation period comprised of multiple time steps. The storm water management model (SWMM), was originally developed for the EPA between 1969 and 1971 and was the first comprehensive model of its type for urban runoff analysis. SWMM main window was given Fig (2.1) and physical features in Fig (2.2).

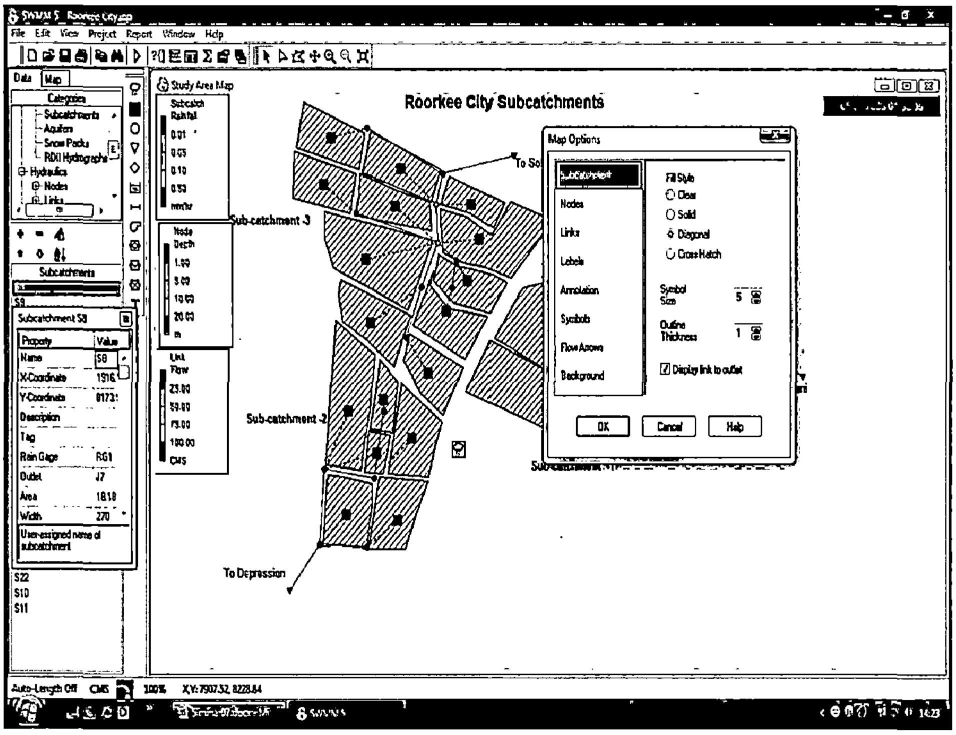

Figure (2.1) The EPA SWMM main window

Application History

SWMM is a comprehensive computer model for analysis of quantity and quality problems associated with urban runoff. It has an impressive longevity. It has been used in scores of U.S. cities (Kansas and Missouri) as well as extensively in Canada, Europe, Australia and elsewhere. The model has been used for very complex hydraulic analysis for combined sewer overflow mitigation as well as for many storm water management planning studies and pollution abatement projects, and there are many instances of successful calibration and verification. SWMM simulate runoff and transport of storm through drainage network by performing hydraulic and hydrologic model analysis of storm water in the drainage system.

SWMM Conceptual Model

SWMM conceptualizes a drainage system as a series of water flows between several major compartments. These compartments and the SWMM objects they contain include:

i. The Atmosphere compartment, SWMM uses Rain Gage objects to represent rainfall inputs to the system.

ii. The Land Surface compartment, which is represented through one or more Sub-catchment objects. It receives precipitation

iii. The Groundwater compartment receives infiltration from the Land Surface compartment and transfers a portion of this inflow to the Transport compartment. This compartment is modelled using Aquifer objects.

iv. The Transport compartment contains a network of conveyance elements (channels, pipes, pumps, and regulators) and storage/treatment units that transport water to outfalls. Inflows to this compartment can come from surface runoff, groundwater interflow, sanitary dry weather flow, or from user-defined hydrographs. The components of the Transport compartment are modelled with Node and Link objects.

Not all compartments need appear in a particular SWMM model. For example, one could model just the transport compartment, using pre-defined hydrographs as inputs.

Hydrological Modelling Features **SWMM**

SWMM accounts for various hydrologic processes that produce runoff from urban areas. These include:

i. Time-varying rainfall.
ii. Evaporation of standing surface water.
iii. Snow accumulation and melting.
iv. Rainfall interception from depression storage.
v. Infiltration of rainfall into unsaturated soil layers.
vi. Percolation of infiltrated water into groundwater layers.
vii. Interflow between groundwater and the drainage system.
viii. Nonlinear reservoir routing of overland flow.
ix. Spatial variability in all of these processes is achieved by dividing a study area into a collection of smaller, homogeneous sub-catchments.

Hydraulic Modelling Features

SWMM also contains a flexible set of hydraulic modelling capabilities used to route runoff and external inflows through the drainage system network of pipes, channels, storage/ treatment units and diversion structures. These include the ability to:

Handle networks of unlimited size.

i. Use a wide variety of standard closed and open conduit shapes as well as natural channels.
ii. Model special elements such as storage/treatment units, flow dividers, pumps, weirs, and orifices.
iii. Utilize either kinematics wave or full dynamic wave flow routing methods. Model various flow regimes, such as backwater, surcharging, reverse flow, and surface ponding.
iv. Apply user-defined dynamic control rules to simulate the operation of pumps, orifice openings, and weir crest levels.

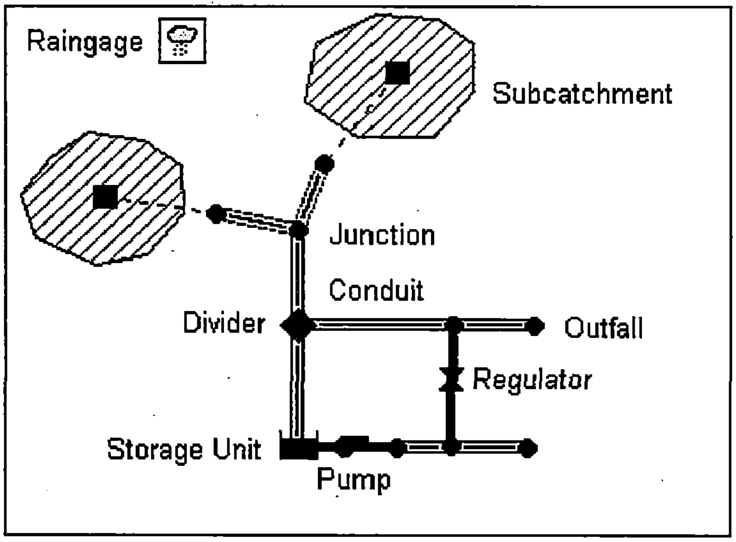

Figure (2.2) Example of physical objects used to model a drainage system

SWMM Requirement for Urban Flood Modelling

i. Study area map (including, area, imperviousness, slope, roughness, depression storage and infiltration).
ii. Rain gauge network (rainfall series).
iii. Catchments & Sub-catchments delineation.
iv. Sewer networks and locations.
v. Nodes (junctions, outfalls, dividers and storage units).
vi. Links (conduits, pumps, orifices, weirs and outlets).
vii. Transects & Controls.
viii. Observed Data for model calibration & verification (observed hydrographs).

SWMM Salient Features

i. Freely available in public domain
ii. The model performs well in urbanized areas with impervious drainage, although it has been used elsewhere. Worldwide application
iii. It incorporates infiltration models – Horton, Green-ampt and SCS CN method.
iv. Flow routing both kinematics and dynamic wave routing.
v. Very good graphical user interface.

Limitations

i. Two dimensional flows cannot be modeled.
ii. It cannot directly read/write the data from/to any GIS format.

Assumptions

i. Overland flow and channel flows are assumed to be one dimensional.

ii. Always the pipe flows are assumed to have backwater effects and are modeled with full dynamic wave.

iii. Flow from surface to the channels takes place only at the specific nodes defined.

Typical Application of SWMM

Since its inception, SWMM has been used in thousands of sewer and storm water studies throughout the world. Typical applications include:

i. Design and sizing of drainage system components for flood control.

ii. Sizing of detention facilities and their appurtenances for flood control and water quality protection.

iii. Flood plain mapping of natural channel systems.

iv. Designing control strategies for minimizing combined sewer overflows.

v. Evaluating the impact of inflow and infiltration on sanitary sewer overflows.

vi. Generating non-point source pollutant loadings for waste load allocation studies.

2.7.2 Overview of Computational Method used in SWMM

Surface Runoff

The conceptual view of surface runoff used by SWMM is illustrated in Fig (2.3) below. Each sub-catchment surface is treated as a nonlinear reservoir (NLR). Inflow comes from precipitation and any designated upstream sub-catchments. There are several outflows, including infiltration, evaporation, and surface runoff. The capacity of this "reservoir" is the maximum depression storage, which is the maximum surface storage provided by ponding, surface wetting, and interception. Surface runoff per unit area, Q, occurs only when the depth of water in the "reservoir" exceeds the maximum depression storage, dp, in which case the outflow is given by Manning's equation. Depth of water over the sub-catchment (d in meter) is continuously updated with time (t in seconds) by coupling the continuity equation and Manning's equation and solving numerically a water balance equation over the sub-catchment Eq. (2.14).

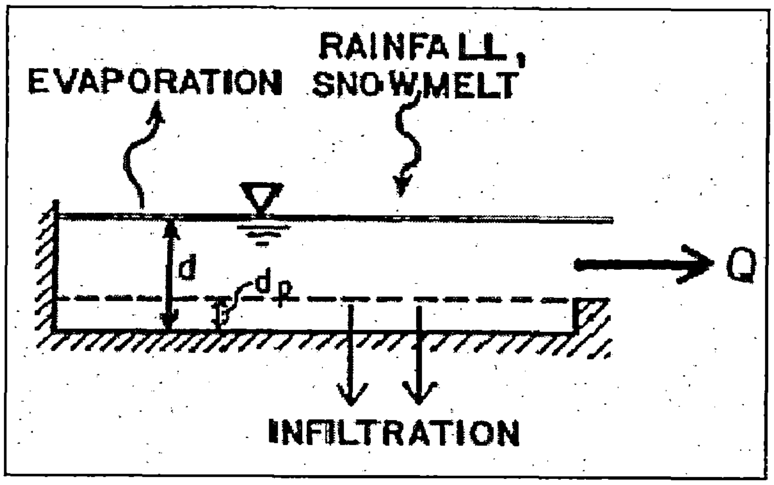

Figure (2.3) Shows Conceptual view of surface runoff computation.

$$\frac{d_j - d_{j-1}}{\Delta t} = I_j + W \frac{S_0^{1/2}}{10000nA} \times [d_{j-1} + 1/2(d_j - d_{j-1})]^{5/3} \qquad (2.14)$$

Where,

d = is the depth of water in the sub-catchment (m)

W = sub-catchment width (m)

n = Manning's Coefficient for overland flow resistance

S_0 = average slope of ground (m/m)

A = area of sub-catchment (ha)

I = rainfall intensity (m/s)

The excessive depths (d) in NLR model are determined at each time step with the Newton-Raphson iterative method and the corresponding flow rates at the outlet of the reservoir are computed with Manning's.

Infiltration

Infiltration is the process of rainfall penetrating the ground surface into the unsaturated soil zone of pervious sub-catchments areas. SWMM offers three choices for modelling infiltration:

Horton's Equation

This method is based on empirical observations showing that infiltration decreases exponentially from an initial maximum rate to some minimum rate over the course of a long rainfall event. Input parameters required by this method include the maximum and minimum infiltration rates, a decay coefficient that describes how fast the rate decreases over time, and a time it takes a fully saturated soil to completely dry.

Green-Ampt Method

This method for modelling infiltration assumes that a sharp wetting front exists in the soil column, separating soil with some initial moisture content below from saturated soil above. The input parameters required are the initial moisture deficit of the soil, the soil's hydraulic conductivity, and the suction head at the wetting front.

Curve Number Method

This approach is adopted from the NRCS (SCS) Curve Number method for estimating runoff. It assumes that the total infiltration capacity of a soil can be found from the soil's tabulated Curve Number. During a rain event this capacity is depleted as a function of cumulative rainfall and remaining capacity. The input parameters for this method are the curve number, the soil's hydraulic conductivity (used to estimate a minimum separation time for distinct rain events), and a time it takes a fully saturated soil to completely dry.

Flow routing

Steady State

Steady Flow routing represents the simplest type of routing possible (actually no routing) by assuming that within each computational time step flow is uniform and steady. Thus it simply translates inflow hydrographs at the upstream end of the conduit to the downstream end, with no delay or change in shape. The Manning equation is used to relate flow rate to flow area (or depth). This type of routing cannot account for channel storage, backwater effects, entrance/exit losses, flow reversal or pressurized flow. It can only be used with dendritic conveyance networks, where each node has only a single outflow link (unless the node is a divider in which case two outflow links are required). This form of routing is insensitive to the time step employed and is really only appropriate for preliminary analysis using long-term continuous simulations.

Kinematic Wave Routing

This routing method solves the continuity equation along with a simplified form of the momentum equation in each conduit. The latter requires that the slope of the water surface equal the slope of the conduit. The maximum flow that can be conveyed through a conduit is the full-flow Manning equation value. Any flow in excess of this entering the inlet node is either lost from the system or can pond atop the inlet node and be re-introduced into the conduit as capacity becomes available. Kinematic wave routing allows flow and area to vary both spatially and temporally within a conduit. This can result in attenuated and delayed outflow hydrographs as inflow is routed through the channel. However this form of routing cannot account for backwater effects, entrance/exit losses, flow reversal, or

pressurized flow, and is also restricted to dendritic network layouts. It can usually maintain numerical stability with moderately large time steps, on the order of 5 to 15 minutes. If the aforementioned effects are not expected to be significant then this alternative can be an accurate and efficient routing method, especially for long-term simulations.

Dynamic Wave Routing

Dynamic Wave routing solves the complete one-dimensional Saint Venant's flow equations and therefore produces the most theoretically accurate results. These equations consist of the continuity and momentum equations for conduits and a volume continuity equation at nodes. With this form of routing it is possible to represent pressurized flow when a closed conduit becomes full, such that flows can exceed the full-flow Manning equation value. Flooding occurs when the water depth at a node exceeds the maximum available depth, and the excess flow is either lost from the system or can pond atop the node and re-enter the drainage system. Dynamic wave routing can account for channel storage, backwater, entrance/exit losses, flow reversal, and pressurized flow. Because it couples together the solution for both water levels at nodes and flow in conduits it can be applied to any general network layout, even those containing multiple downstream diversions and loops. It is the method of choice for systems subjected to significant backwater effects due to downstream flow restrictions and with flow regulation via weirs and orifices. This generality comes at a price of having to use much smaller time steps, on the order of a minute or less (SWMM will automatically reduce the user-defined maximum time step as needed to maintain numerical stability).

Surface Ponding

Normally in flow routing, when the flow into a junction exceeds the capacity of the system to transport it further downstream, the excess volume overflows the system and is lost. An option exists to have instead the excess volume be stored atop the junction, in a ponded fashion, and be reintroduced into the system as capacity permits. Under Kinematic Wave flow routing, the ponded water is stored simply as an excess volume. For Dynamic Wave routing, which is influenced by the water depths maintained at nodes, the excess volume is assumed to pond over the node with a constant surface area. This amount of surface area is an input parameter supplied for the junction. Alternatively, the user may wish to represent the surface overflow system explicitly. In open channel systems this can include road overflows at bridges or culvert crossings as well as additional floodplain storage areas. In closed conduit systems, surface overflows may be conveyed down streets, alleys, or other surface routes to the next available storm water inlet or open channel. Overflows may also be impounded in surface depressions such as parking lots, back yards or other areas.

2.7.3 Computational Steps of Hydrology and Hydraulics

The drainage area is divided into several sub-catchments and associated with the drainage channels receiving water from the tributary sub-catchments.

1. rainfall is added to the sub-catchment according to the specified hyetograph:

   $$d_1 = d_t + r_t * \Delta t \qquad (2.15)$$

   Where,

   d_1 = the water depth after rainfall

   dt = the water depth of sub-catchment at time t

   rt = the intensity of rainfall in time interval Δt

2. infiltration f_s is computed using Green-Ampt Model, Horton or Curve-Number

   For Green-Ampt:

   $F_s = (S * IMD)/(i/Ks - 1)$, for $I > Ks$

   $f_p = Ks\ (1 + (S * IMD)/F)$, for $I \leq Ks$, $F \geq Fs$

   Where,

   f = actual infiltration rate

   f_p = infiltration capacity (mm/s)

   I = rainfall intensity (mm/s)

   F = cumulative infiltration volume in the event (mm)

   F_s = cumulative infiltration volume to cause surface saturated (mm)

   IMD = initial moisture deficit for the event (fraction)

   S = average capillary suction at wetting front (mm of water)

   Ks = saturated hydraulic conductivity of the soil

3. If the resulting water depth of sub-catchment d_l is large than the specified detention depth d_d and outflow rate is computed using Manning's equation

   $$v = \frac{1.49}{n} (d_l - d_d)^{2/3} S^{1/2} \qquad (2.16)$$

   And

   $$Q_s = v * w * (d_l - d_d) \qquad (2.17)$$

   Where

   V = the velocity

   n Manning's Roughness Coefficient

S = the ground slope ratio

W = the width (length / longitudinal of water travel in catchment)

Q_s = the outflow rate

4. the continuity equation is solved to determine water depths of the sub-catchments resulting from rainfall, infiltration, and outflow, thus;

$$d_{t+\Delta t} = d_1 - \frac{Qs}{A} * \Delta t \qquad (2.18)$$

Where A is the surface area of the sub-catchment

5. Step 1-4 are repeated until computations for all sub-catchments are completed.
6. inflow (Q_{in}) to the catchment is computed as a summation of outflow from tributary sub-catchments ($Q_{s,i}$) and flow rate of immediate upstream channels ($Q_{g,i}$)

$$Q_{in} = \sum Q_{s,i} + \sum Q_{g,i} \qquad (2.19)$$

7. The inflow is added to raise the existing water depth of the channels according to it is geometry. Thus,

$$Y_1 = Y_t + \frac{Qm}{As} \Delta t \qquad (2.20)$$

Where, Y_1 and Y_t water depth of the gutter

A_s = is the mean water surface area between Y_1 and Y_t

8. The outflow is calculated for the channel using Manning's equation

$$v = \frac{1.49}{n} (R)^{2/3} S_i^{1/2}$$

$$Q_g = v * Ac$$

Where Q_g = is the flow in the gutter

Ac = is the cross-section area of channel at Y_1

Si = the invert slope of channel

R = hydraulic radius

9. The continuity equation is solved to determine the water depth of the channel resulting from the inflow and out flow

$$Y_{t+} \Delta_t = Y_1 + (Q_{in} - Q_g) \frac{\Delta t}{As} \Delta t \qquad (2.21)$$

For Kinematic Wave Routing, continuity equation and simplified form of momentum are solved for each channel

Continuity Equation:

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}=q$$

Momentum Equation

$S_f = S_o$ or $Q = \alpha A^{\beta}$

A = the cross sectional area of the section

Q = discharge at the section

t = time

x = Distance

q = lateral inflow per unit length.

S_f = friction slope

S_o = bed slope

α and β = are kinematic wave parameters

The above equations are combined together to produce an equation with Q as the only unknown variable.

10. Step 6-9 are repeated until all the gutters are finished
11. The flows reaching the point concerned are added to produce a hydrograph coordinate along time axis.
12. The processes from 1 to 11 are repeated in succeeding time periods until the complete hydrograph is computed.

2.8 Storm Water Drainage Design

The design of the drainages system involves (i) estimation of the total discharge that the system will require to drain off and (ii) fixing the slope and dimensions of the drain to have adequate capacity to carry the discharge and afford proper maintenance.

The discharge is dependent upon intensity and duration of precipitation characteristics of the area and time required for such flow to reach the drain. The storm water flow for this purpose may be determined by using the rational method, hydrograph method, and rainfall runoff correlation studies, digital computer models and empirical formulae. The empirical formulae that are available for estimating the storm water runoff can be used only when comparable conditions to those for which the equations are derived initially can be assured. Of the different methods available, the rational method is most commonly used and serves the purpose of design.

Any method requires that existing rainfall data of the concerned area is analysed to permit a suitable forecast. Urban storm drains are not designed for the peak flow of rare occurrences such as 100 years or 50 years – as is the case with design of important structures such as bridges or weirs. However, it is necessary to provide sufficient capacity to prevent too frequent a flooding of the drainage area. There may be some water accumulation on the roads when the rainfall exceeds the design value which has to be permitted. The frequency of occurrence which can be permitted varies from place to place, depending upon the importance of the place and the expectancy of the public. Flooding at any time, however, causes inconvenience to people but they accept it once in a while considering the savings affected in drainage costs. The areas such as important junctions, areas having basement, substations etc. should be considered as important areas and higher frequency of flooding should be adopted in the design.

2.8.1 Hydraulic Design of Drains

Design of drain section

Capacity of the drain is normally designed using manning's formula

$$Q = v * A$$

$$v = \frac{1.49}{n} R^{2/3} S^{1/2}$$

Where,

Q= discharge in cumecs

V= mean velocity in m/s

N= Manning's roughness coefficient

R= hydraulic mean radius which is area of flow cross section divided by wetted perimeter.

S= gradient of drain bed

A= Area of flow cross section in m^2

Table (2.3) average values of Manning's coefficient for various surfaces.

(Source: IRC: SP: 50-1999)

No.	Type of Surface	Value of n
i)	Brick pitched drain	0.017
ii)	Plastered brick surface	0.015
iii)	Plastered brick surface with neat cement finish	0.013
iv)	Concrete pipes up to 600 mm diameter	0.015
v)	Concrete pipes above mm diameter	0.013
vi)	Dry rubble masonry	0.033
vii)	Dressed ashler surface	0.015
viii)	Dry stone pitching	0.020
ix)	Kutcha drain	0.025

Table (2.4) Manning's Roughness Coefficients "n" for Channels (Source: Chow, 1959)

Stream bed characteristics		Typical "n" value
concrete		0.012
gravel bottom with sides	concrete	0.02
	mortared stone	0.023
	riprap	0.033
Natural stream channel		
clean, straight stream		0.03
clean, winding stream		0.03
- with weeds and pools		0.04
-with heavy brush and timber		0.05
flood plains		0.1
pasture		0.035
field crops		0.04
light brush and weeds		0.05
dense brush		0.07
dense trees		0.1

While deciding the drain sections it is not sufficient that they are sufficient to carry the required discharge. Minimum and maximum velocities, minimum free board, maximum section of drain, channel shape, economic sections (for lined drains), cross slope in bed and silt pit required special attention of the designer.

STUDY OF DRAINAGE PATTERN AND PREPARATION OF SPATIAL DATA

3.1 Locale of study area

The selected study area is the city of Roorkee which is a small but pleasant and peaceful place with the grand spectacle of Himalayan ranges stretching in the east and north-east. The glamorous part of the Roorkee town is the *Upper Ganges Canal* which flows north-south and divides the city into two parts. Ganges canal with its raised embankment flanked with huge masonry lions is a special attraction of the city. Roorkee comprises of two administrative units:

i. Roorkee Municipal Board and
ii. Roorkee Cantonment Board.

Because of the engineering background and institutions located in the City, Roorkee has emerged as a famous centre for manufacturing engineering and scientific instruments mainly for survey and drawing purpose.

3.1.1 Location

Roorkee lies at $29^{\circ} 52'$ N Latitude and $77^{\circ} 53'$ E Longitude, with a total area of about 16.84 Km^2 and total population of about 115000 capita (2001census). It lies on the right bank of the River Solani and on the left bank and right bank of upper Ganges Canal and 274 meters above the mean sea level. It is situated 172 kilometres to the north of Delhi, Capital of India, on the Delhi -Dehradun-Messourie highway and 55 kilometres Roorkee-haridwar-Rishikesh road takes off from Roorkee, which further leads to Badrinath, Kedarnath, Gangotri, Yamunotri (all Hindu pilgrim places). The Mount Himalayas in the north side, the Ganges River in the east side and the Yamuna River in the west have acted as natural boundaries for the region. Because of these, this region has become fertile 'Indo-Gangetic plain'. Presently, Roorkee is in Haridwar district of Uttarakhand State. The neighbouring districts are Saharanpur in the west, Muzaffarnagar in south, Dehradun in north and Bijnore in the east side Fig. (3.1).

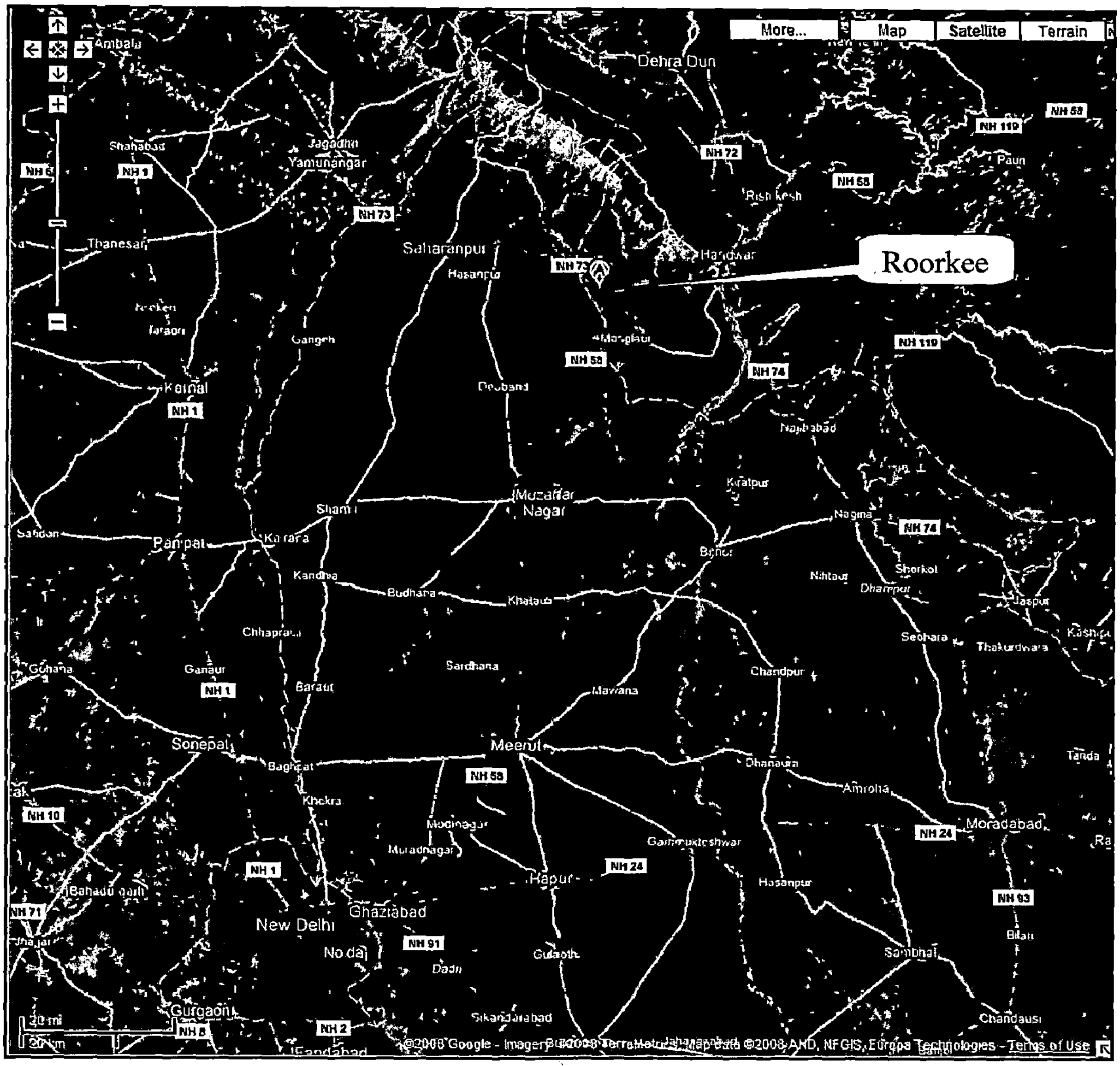

Figure (3.1) location Map of Roorke City

3.1.2 Climate

The climate of Roorkee resembles that of the state generally, though modified by its northern position and proximity to the hills. Being sub mountain district, with higher latitude than any other portion of the plains, it has a longer cold weather. Because of having longer cold season, presence of Upper Ganga canal, Solani River and healthy vegetation, the climate is somewhat on the humid side. Winter season may be considered to begin from November and continue up to month of February. The coldest months in general are December and January when the minimum temperature is around 2° C. A rise of temperature experienced from the beginning of March, which indicate the starting of summer. Summer season, witnesses minimum humidity around 10% during day time and a maximum temperature up to around 40° C. Rainy season, Though rains occur almost throughout the year, but 15th July to 15th September is the general duration in which monsoon rains are prominent. Annual average rainfall is around 110 cm. A typical climate of the town, as explained above, is shown in Fig (3.2).

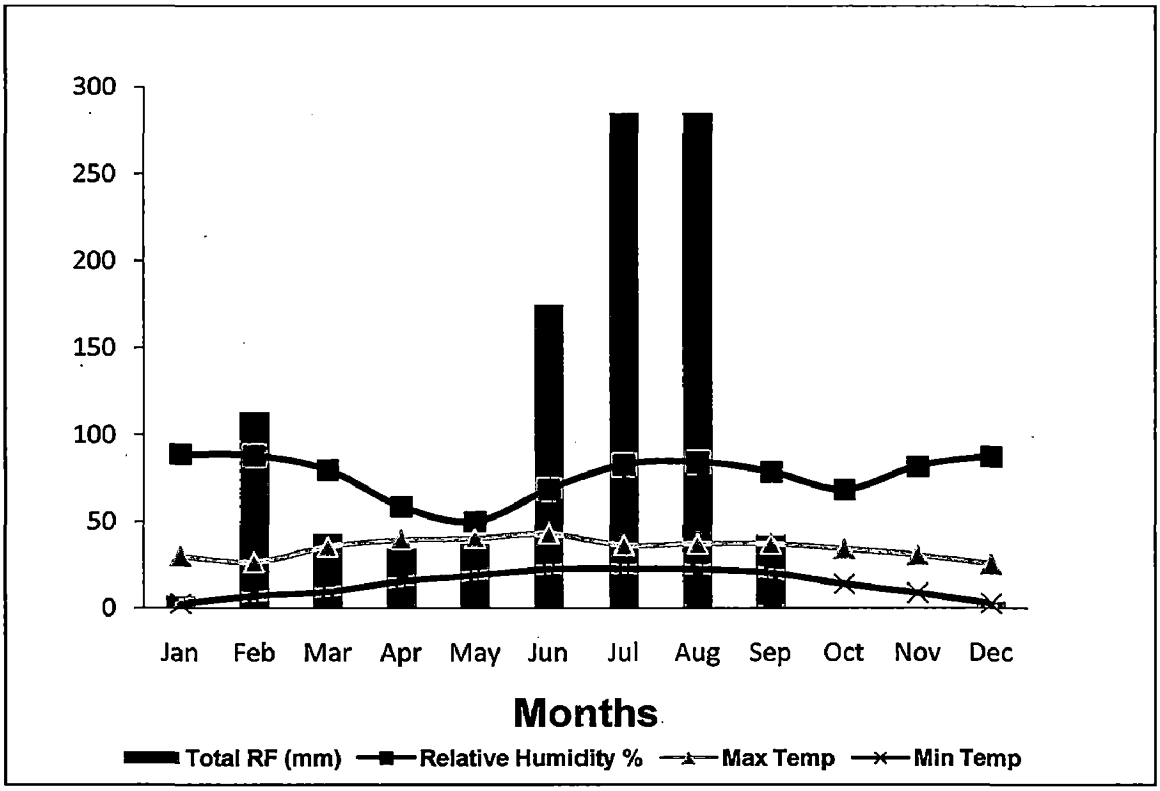

Figure (3.2) variations of temperature, rainfall and relative humidity during year 2007 in Roorkee city

3.2 Data Collection and Field Investigations

For storm water simulation, field investigations and inspection were conducted. The information regarding capacity, elevations, and condition of existing runoff channels and/or conduits, topography, size and shape of drainage area, extent and type of areal development, profiles, cross-sections, roughness data on pertinent existing streams and water courses, and locations of possible ponding areas were collected. Adequate information regarding soil conditions, including types, perviousness, and vegetation cover was also collected. Outfall and downstream flow conditions, including high-water occurrences and frequencies were also obtained from the field and with

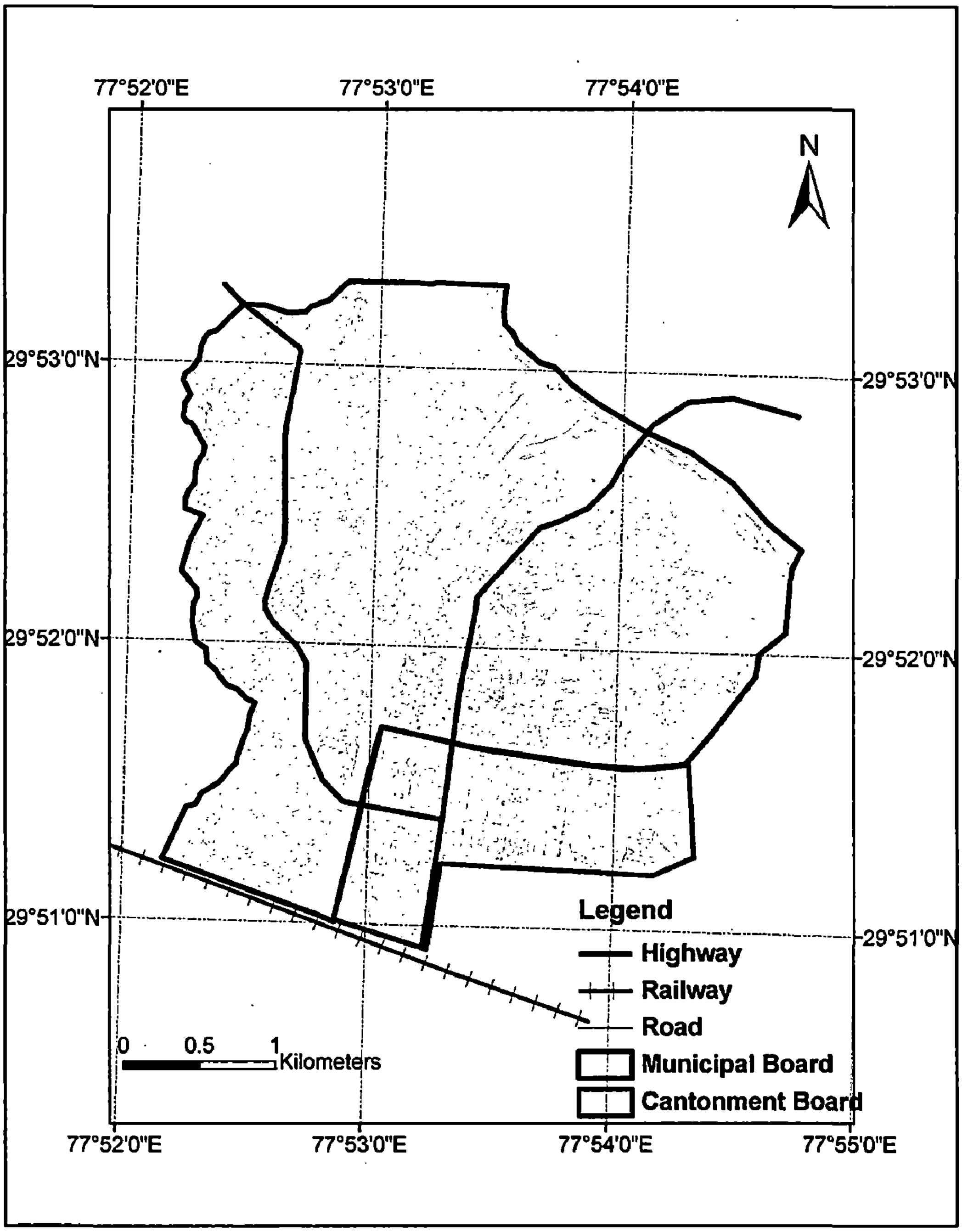

Figure (3.3) Administrative Boards of Roorke City

discussion with inhabitants of the areas. Maps and charts showing necessary detailed topography and other essential features of the areas about drains were obtained from the City authorities and other Institutes. Satellite imagery and contour maps were also obtained for delineation of sub-catchment and development of higher resolution Digital Elevation Models (DEM) for determination of volume, depth and areal extent of flooding.

3.3 Delineation of catchment and drainage networks

3.3.1 Digital Elevation Model (DEM) and Digital Surface Model (DSM) generation

SRTM data was analysed using spatial analyst of ArcGIS and the flow directions, flow accumulation and watershed boundaries were obtained Fig (3.5). To generate the DEM & DSM of higher resolution for simulation, a new point shape file was created having the spot elevations obtained from the drainage map (from Irrigation Research Institute IRI) and values collected using GPS. Using spatial analyst extension of Arc-map, the surface was generated using inverse distance weighted method Fig (3.4).

This SWMM does not support GIS and has no import functionality of GIS data. Hence, a surface was generated from the digital elevation model wherein the surrounding buildings were elevated to suitable height, considering that the water shall spread only along the roads. This data was used later for computation of flood extent.

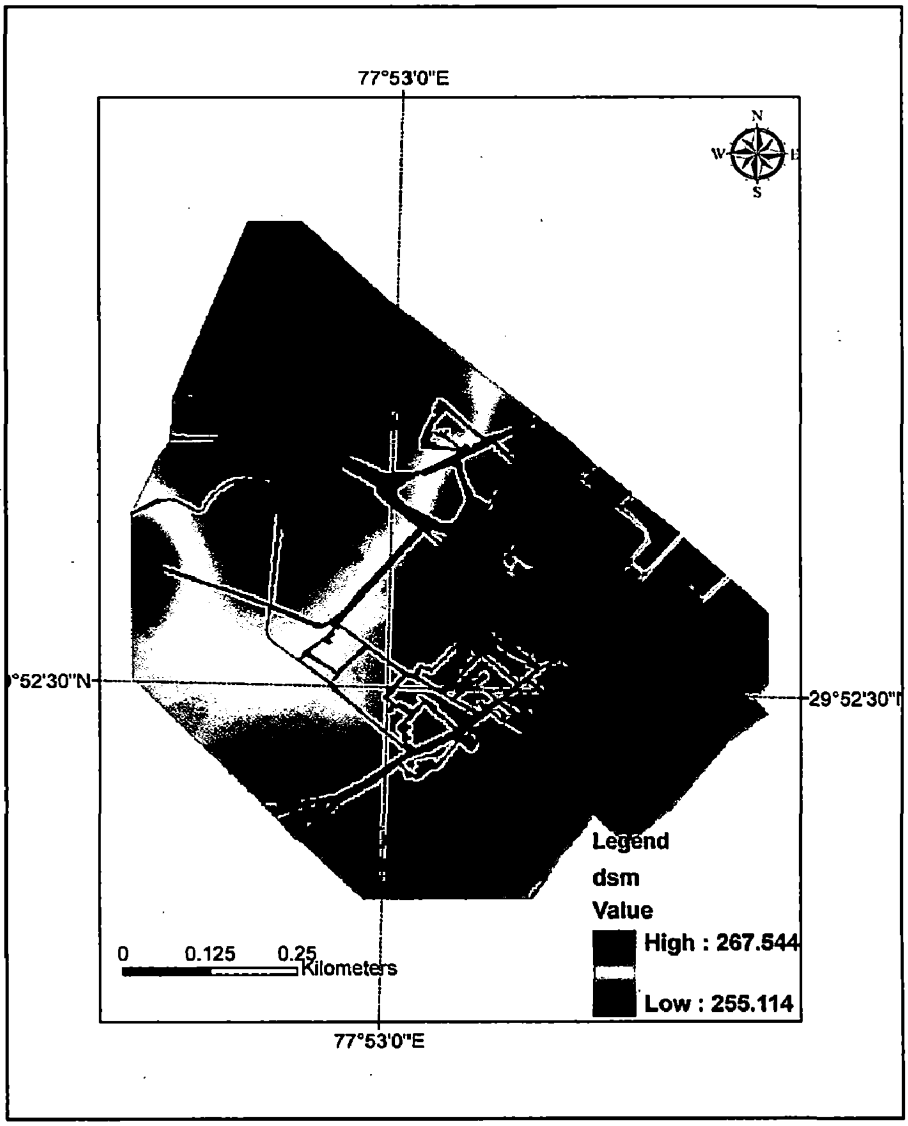

Figure (3.4):- Surface map of simulation area

3.3.2 Delineation of Sub-catchments

The natural drainage pattern was extracted from the DEM data and field investigations using ArcGIS hydrologic analysis, which shows that the study area is comprises of three sub-catchments of different flow directions namely 1) east Gang Canal IIT, Roorkee campus and surroundings areas this drains to Solani River, 2) west Gang Canal, from BSM College up to the railway station this drains on depression southwards and, 3) from BSM College north wards up to Solani River and this drains to Solani Fig (3.5).

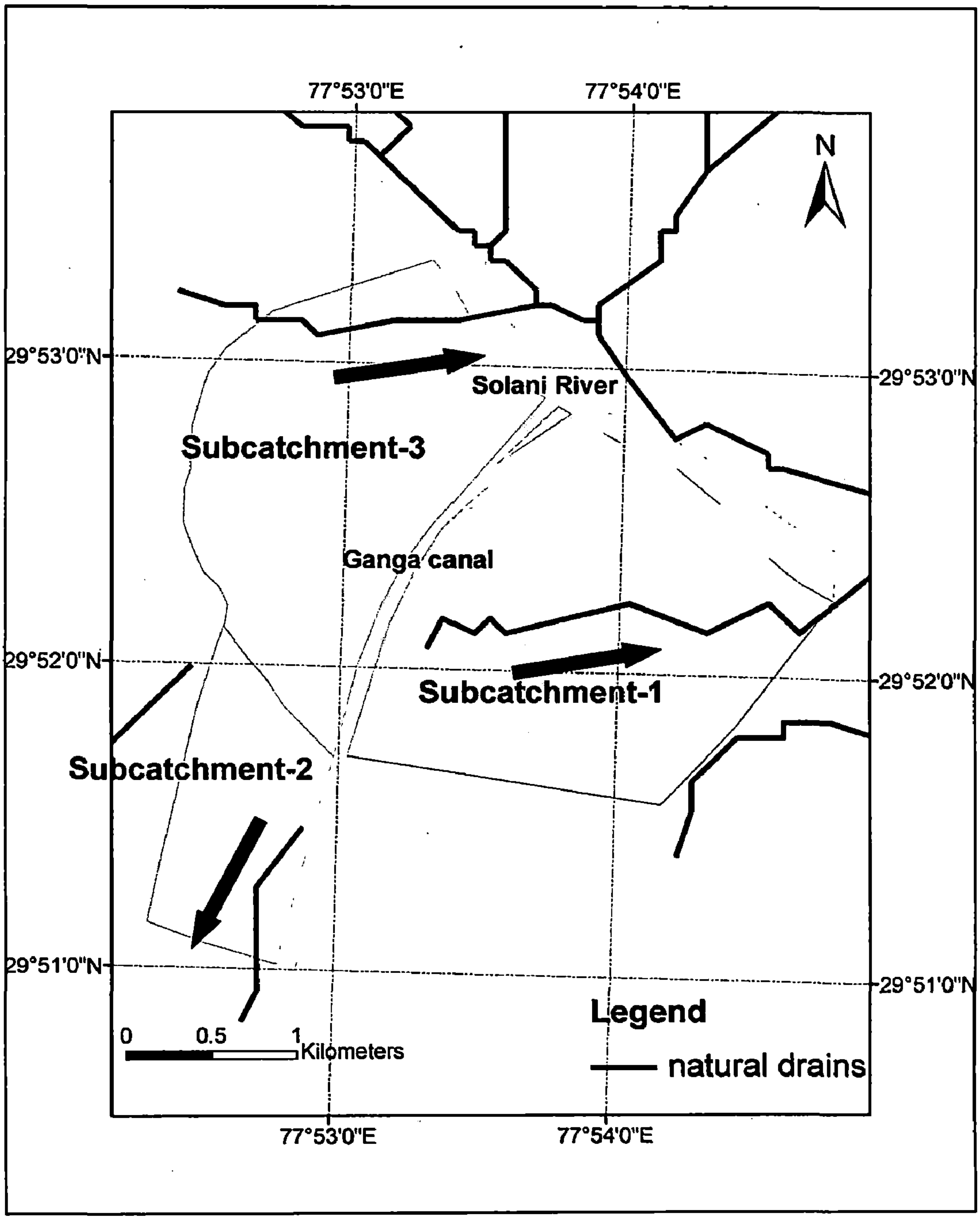

Figure (3.5) Map of Sub-catchment of the city

3.3.3 Delineation and digitization of drainage network

Pan data of IRS (C), available at the Department, was georeferenced with the help of Landsat data Fig (3.6). Drainage networks were delineated from the SRTM data. This was verified with the Pan data and with field investigations. The drainage networks were digitized and stored in line shape files. Roads, Railway, and Highway in the study area were also digitized with help of line shape files.

Drainage map of part of the study area was obtained from the Irrigation Research Institute Roorkee and is shown in Fig (3.7). Spot elevations were also given in the map; the map was scanned and imported to Arc-map. Supplementary survey was done using GPS for determination of the locations of nodes and for finer adjustment; the data was added as a new layer to GIS Software.

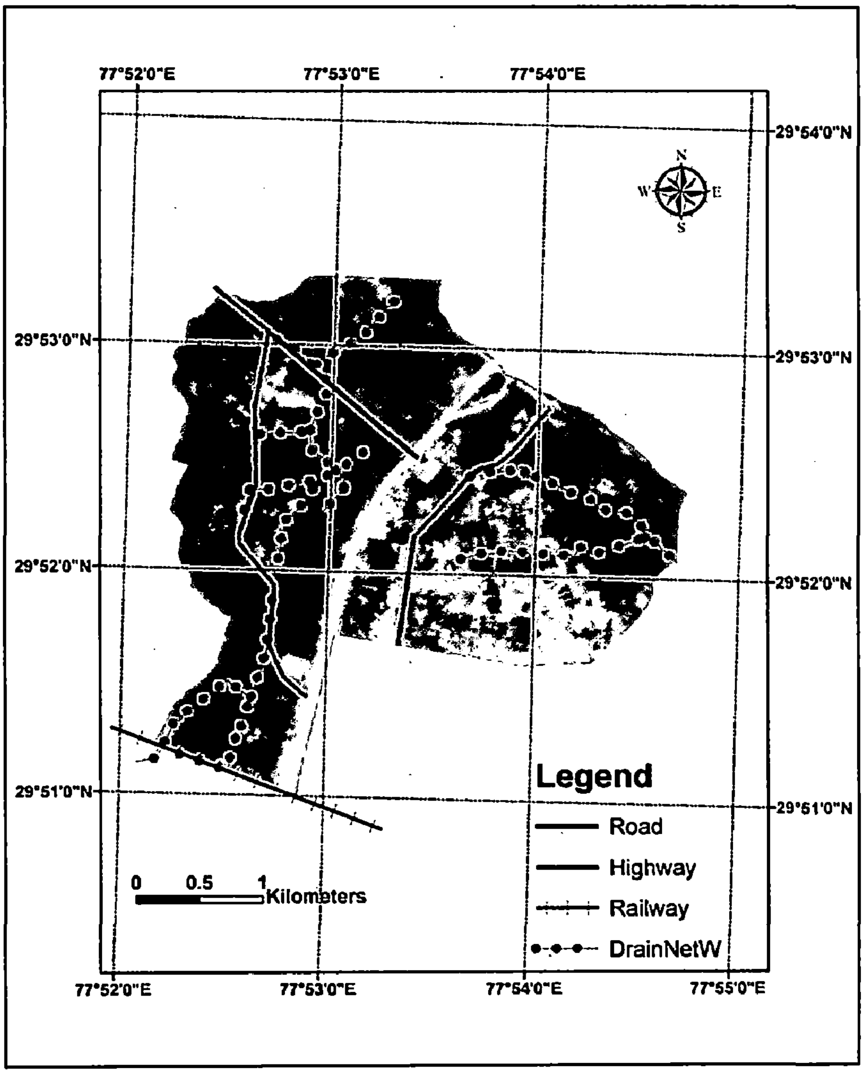

Figure (3.6) Drainage Network and digitized Highway, Roads and railway in Roorkee City

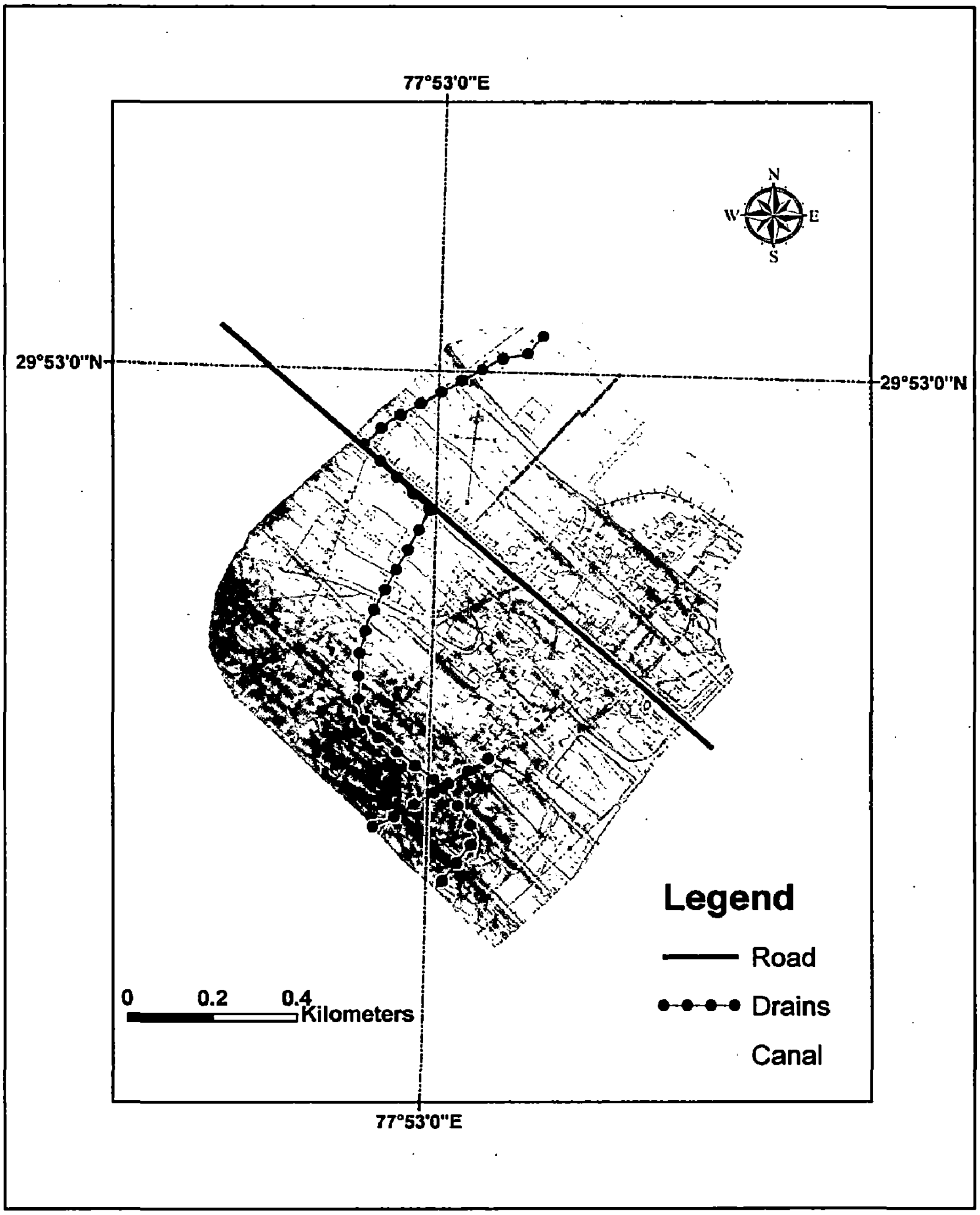

Figure (3.7) Part of drainage network obtained from IRI, Roorkee

3.4 Conclusion of drainage pattern study

The study of the drainage pattern obtained from SRTM data (though having a course resolution of around 83 m) indicates a water divide in old city i.e right side of the Upper Ganges Canal. Therefore two sub-catchments were delineated in this area. One is draining through the thickly populated city core and another one in the south east direction Fig (3.5). The left side area of the canal drains at a different point in downstream of river Solani, thus, these areas were considered as a separate catchment for further analysis. The arrows in Fig (3.8) show the direction of flow of natural drainage. Statistics of the sub-catchment are given in the following table (3.1).

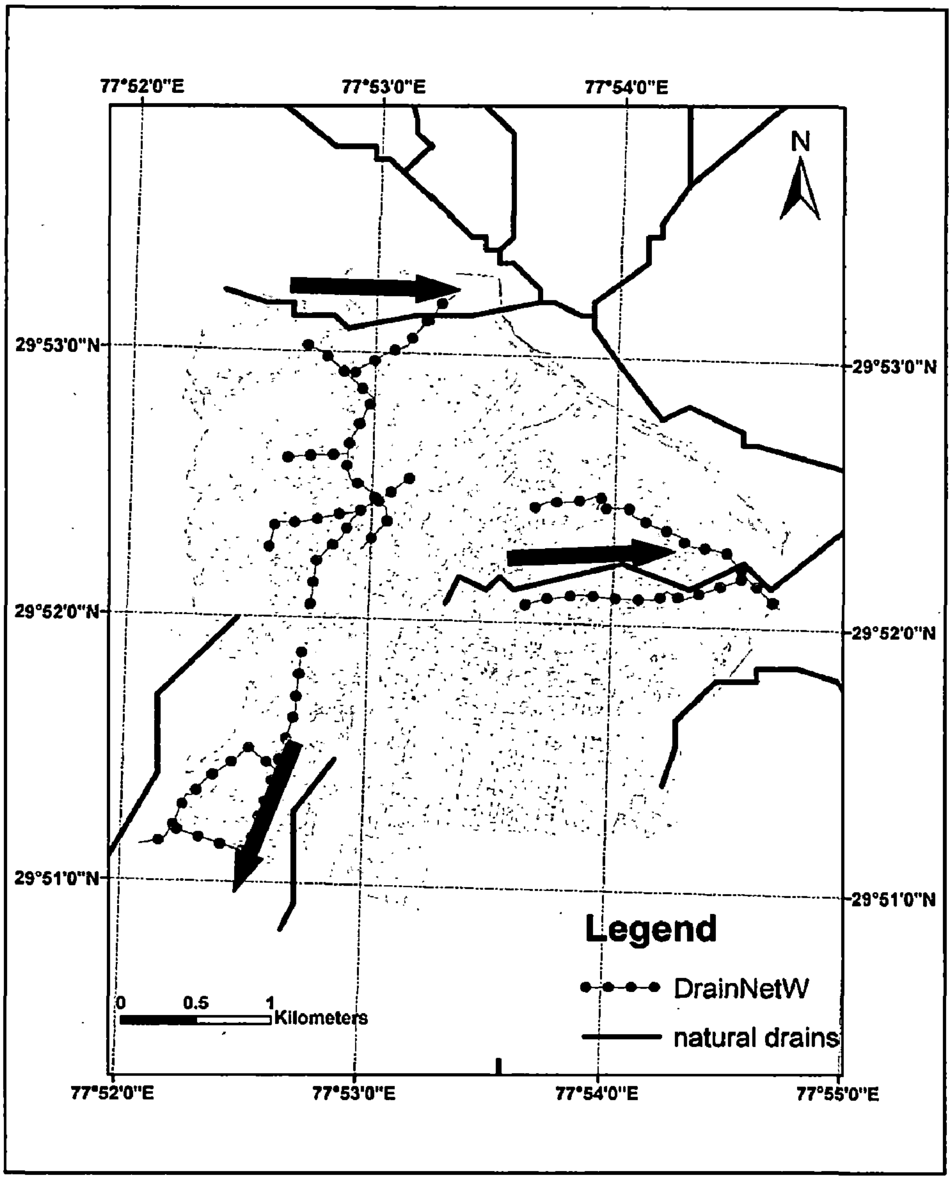

Figure (3.8) Natural drainage pattern and digitized drainage network of study area

Table (3.1) Statistics of sub-catchments

Sub-catchment	Area (ha)	Total length of drains (m)	Average Slope (%)
Sub-catchment-1	410	2700	0.65
Sub-catchment-2	130	2100	0.4
Sub-catchment-3	343	4000	0.5

DESIGN STORM ANALYSIS

4.1 General

The design storm for urban drainage is usually based on rainfall of 2 to 5 years recurrence interval. Where potential damage or functional requirements need a more severe criterion, a greater design recurrence interval should be used. Rainfall intensity should be determined from the best available intensity-duration-frequency curves. In this study, design storm analysis was conducted using rainfall records of the Observatory of the Department of Hydrology. The Design storm of various return periods are calculated using SRRG data and 24 hours maximum rainfall during the period 1977 to 2007. The Gumble's extreme value distribution is used for calculating rainfall depth of various return periods. The details are provided in subsequent sections.

4.2 Gumbel's Method of Extreme Value

This extreme value distribution was introduced by Gumbel (1941) and it is commonly known as Gumbel's Distribution. It is one of the most widely used probability distributions function for extreme values in hydrological and meteorological studies for prediction of flood peaks, maximum rainfalls, maximum wind speed, etc. Gumbel defined a flood as the maximum of the 365 daily flows and the annual series of flood flows constitute a series of largest values of flows. According to this theory of extreme events, the probability of occurrence of an event equal to or larger than a value x_o is;

$$P(X>=x_o) = 1- e^{(-e^{-y})} \quad (4.1)$$

In which y is dimensionless variable given by

$$y = \alpha(x-a) \quad (4.2)$$

$$a = \bar{x}-.045005\sigma_x \quad (4.3)$$

$$\alpha = 1.2825/\sigma_x \quad (4.4)$$

Where

$\bar{x}$ = mean and σ_x = standard deviation of the variates X.

The values of X with a recurrence interval T is used as

$$X_T = \bar{x}+K\,\sigma_{x-1} \quad (4.5)$$

Where

σ_{x-1} = standard deviation of the sample of size N

K = frequency factor expressed as

$$K = \frac{y_T - \bar{y}_n}{S_n} \qquad (4.6)$$

$$y_T = -\ln[\ln(T/(T-1))] \qquad (4.7)$$

$\bar{y}_n$= reduced mean, a function of sample size N and is given in table (4.1) for

$N \rightarrow \alpha$, $\bar{y}_n \rightarrow 0.577$.

S_n = reduced standard deviation, a function of sample size N given in table (4.2 for

$N \rightarrow \alpha$, $S_n \rightarrow 1.2825$.

These equations are used under the following procedure to estimate the maximum rainfall depth corresponding to a given return period,

1. Assemble the rainfall value and note the sample size N. Here the annual maximum daily rainfall is the variates X. find $\bar{x}$ and σ_{x-1} for the given data
2. Using table (4.1) and table (4.2) determine $\bar{y}_n$ and S_n appropriate to given N.
3. Find y_T for a given T using formula (4.7)
4. Find K using formula (4.6)
5. Then using formula (4.5) determine X_T.

4.2 Design Storm Analysis Using SRRG Data

Gautam (1997) analysed SRRG data of the Hydrometeorological Observatory of the Department of Hydrology during 1979 to 1996. This work was updated using the SRRG data from 1997 to 2006. During the period 1997-2006, a total 43 storms were used for analysis. From the above 43 storms the representative storm for each year was found out for the selected duration (Appendix- I & II). Maximum depth for durations 15, 30, 45, 60, 120, 180 minutes were worked and are presented in table 4.3a. The intensities were calculated and are presented in table 4.3b. Thus, the total length of rainfall data analysed for intensity-duration-frequency analysis was of 28 years.

Table (4.1) Reduced mean Y_n in Gumbel's Extreme Value Distribution (Subramanya, 2004)

N	0	1	2	3	4	5	6	7	8	9
10	0.4952	0.4996	0.5035	0.507	0.51	0.5128	0.5157	0.5181	0.5202	0.522
20	0.5326	0.5252	0.5268	0.5283	0.5296	0.5309	0.532	0.5332	0.5343	0.5353
30	0.5362	0.5371	0.538	0.5388	0.5396	0.5402	0.541	0.5418	0.5424	0.543
40	0.5436	0.5442	0.5448	0.5453	0.5458	0.5463	0.5468	0.5473	0.5477	0.5481
50	0.5485	0.5489	0.5493	0.5497	0.5501	0.5504	0.5508	0.5511	0.5515	0.5518
60	0.5521	0.5524	0.5527	0.553	0.4433	0.5535	0.5538	0.554	0.5543	0.5545
70	0.5548	0.555	0.5552	0.5555	0.5557	0.5559	0.5561	0.5563	0.5565	0.5567
80	0.5569	0.557	0.5572	0.5574	0.5576	0.5578	0.558	0.5581	0.5583	0.5585
90	0.5586	0.5587	0.5589	0.5591	0.5592	0.5593	0.5595	0.5596	0.5598	0.5599
100	0.56									

N= Return Period in Years

Table (4.2) Reduced Standard Deviation S_n in Gumbel's Extreme Value Distribution (Subramanya, 2004)

N	0	1	2	3	4	5	6	7	8	9
10	0.9496	0.9676	0.9833	0.9971	1.0095	1.0206	1.0316	1.0411	1.0493	1.0565
20	1.0628	1.0696	1.0754	1.0811	1.0964	1.0915	1.0961	1.1004	1.1047	1.1086
30	1.1124	1.1159	1.1193	1.1226	1.1255	1.1285	1.1313	1.1339	1.1363	1.1388
40	1.1413	1.1436	1.1458	1.148	1.1499	1.1519	1.1538	1.1557	1.1574	1.159
50	1.1607	1.1623	1.1638	1.1658	1.1667	1.1681	1.1696	1.1708	1.1721	1.1734
60	1.1747	1.1759	1.177	1.1782	1.793	1.1803	1.1814	1.1824	1.1834	1.1844
70	1.1854	1.1863	1.1873	1.1881	1.189	1.1898	1.1906	1.1915	1.1923	1.193
80	1.1938	1.1945	1.1953	1.1959	1.1967	1.1973	1.198	1.1987	1.1994	1.2001
90	1.2007	1.2013	1.202	1.2026	1.2032	1.2038	1.2044	1.2049	1.2055	1.206
100	1.2065									

N= Return Period in Years

Table (4.3) a Rainfall Depth Used for Computation of Design Storm (SRRG data)

Serial No.	Years	Max Depth (mm)					
		15 min	30 min	45 min	60 min	120 min	180 min
1	2006	30	50	60	70	90	122
2	2005	40	50	55	46	42	56
3	2004	20	30	42	56	72	40
4	2003	26	40	46	52	19	21
5	2002	16	20	30	40	56	68
6	2001	20	40	43	46	72	81
7	2000	20	40	52	60	90	95
8	1999	18	34	24	26	28	23
9	1998	20	40	50	52	66	80
10	1997	20	30	40	41	46	53
11	1996	30	54	68.00	81	113.00	118.00
12	1995	35	60	63	57.5	96.30	104.30
13	1994	23	41	58.00	66	86.50	116.50
14	1993	13	23	32.85	40	55.65	56.15
15	1992	24	46	59.00	75.5	114.00	126.40
16	1991	20	36.75	49.5	55.5	72.30	77.00
17	1990	35	66	80.00	93.9	107.40	118.35
18	1989	25.2	31.2	35.90	40.3	43.85	54.50
19	1988	21	38	55.00	67.5	94.05	116.85
20	1987	20.75	39.25	53.75	62.5	74.50	76.85
21	1986	19.75	29.75	33.95	34.25	34.30	37.65
22	1985	21.5	26	28.8	31.3	31.60	49.00
23	1984	10	15.5	20.00	17	21.00	21.68
24	1983	20	40	50.00	60	101.00	128.50
25	1982	20	36	47.50	54.5	67.60	70.20
26	1981	30	60	90	118.5	51.15	63.00
27	1980	20.7	35.2	40.55	46	74.00	85.60
28	1979	31.5	56.5	62.1	70	93.10	85.70

Table (4.3) b Intensities Corresponding to Maximum Depths (SRRG data).

Serial No.	Years	Maximum Intensity (mm/hr)					
		15 min	30 min	45 min	60 min	120 min	180 min
1	2006	120	100	80	70	45	40.67
2	2005	160	100	73.33	46	21	18.67
3	2004	80	60	56	56	36	13.25
4	2003	104	80	61.33	52	8.5	7
5	2002	64	40	40	40	28	22.67
6	2001	80	80	57.33	46	36	27
7	2000	80	80	69.33	60	45	31.67
8	1999	72	68	32	26	14	7.67
9	1998	80	80	66.67	52	33.5	26.67
10	1997	80	60	53.33	41	23	17.67
11	1996	120	108	90.67	81	56.50	39.33
12	1995	140	120	84	57.5	48.15	34.77
13	1994	92	82	77.33	66	43.25	38.83
14	1993	52	46	43.8	40	27.83	18.72
15	1992	96	92	78.67	75.5	57.00	42.13
16	1991	80	73.5	66	55.5	36.15	25.67
17	1990	140	132	106.67	93.9	53.70	39.45
18	1989	100.8	62.4	47.87	40.3	21.93	18.17
19	1988	84	76	73.33	67.5	47.03	38.95
20	1987	83	78.5	71.67	62.5	37.25	25.62
21	1986	79	59.5	45.27	34.25	17.15	12.55
22	1985	86	52	38.4	31.3	15.80	16.33
23	1984	40	31	26.67	17	10.50	7.23
24	1983	80	80	66.67	60	50.50	42.83
25	1982	80	72	63.33	54.5	33.80	23.40
26	1981	120	120	120	118.5	25.58	21.00
27	1980	82.8	70.4	54.07	46	37.00	28.53
28	1979	126	113	82.8	70	46.55	28.57

The Gumble's extreme value method was applied on rainfall depth series (table 4.3 a) and the results of various duration and return period were estimated. The compiled results are provided in Table 4.4a. The corresponding intensities are given in table 4.4b.

Table (4.4) a Rainfall depths for different durations and return periods, using Gumbel's Extreme Value Distribution (SRRG data)

Duration in minutes	Mean of rainfall series	Standard Deviation of series	Rainfall Depth in mm					
			5 years	10 years	15 years	25 years	50 years	100 years
			K=1.499	K=2.25	K=2.67	K=3.2	K=3.9	K=4.6
15	23.23	6.81	29.27	33.90	36.51	39.74	44.08	48.38
30	39.58	12.37	50.55	58.96	63.70	69.58	77.46	85.28
45	48.93	16.01	63.12	74.00	80.14	87.74	97.94	108.05
60	55.72	20.97	74.32	88.56	96.59	106.55	119.90	133.16
120	66.10	28.50	91.37	110.73	121.65	135.19	153.34	171.35
180	76.62	33.33	106.17	128.81	141.59	157.42	178.64	199.71

Table (4.4) b Rainfall intensities corresponding to the maximum depths as given in table (4.4) a.

Duration in minutes	Rainfall intensities in mm/hr					
	5 years	10 years	15 years	25 years	50 years	100 years
15	117.08	135.58	146.02	158.96	176.31	193.52
30	101.10	117.91	127.40	139.15	154.91	170.55
45	84.16	98.67	106.85	116.99	130.58	144.07
60	74.32	88.56	96.59	106.55	119.90	133.16
120	45.69	55.37	60.83	67.60	76.67	85.68
180	35.39	42.94	47.20	52.47	59.55	66.57

For N=28 years, from table (4.1) and (4.2) $Y_n = 0.5343$ and $S_n = 1.1047$

The Intensity-Duration-Frequency curves (I-D-F) were plotted using tables 4.4 and are shown in fig (4.1). The Intensity–Duration curve (I-D) was also plotted using table (4.3) b and is shown in figure 4.2.

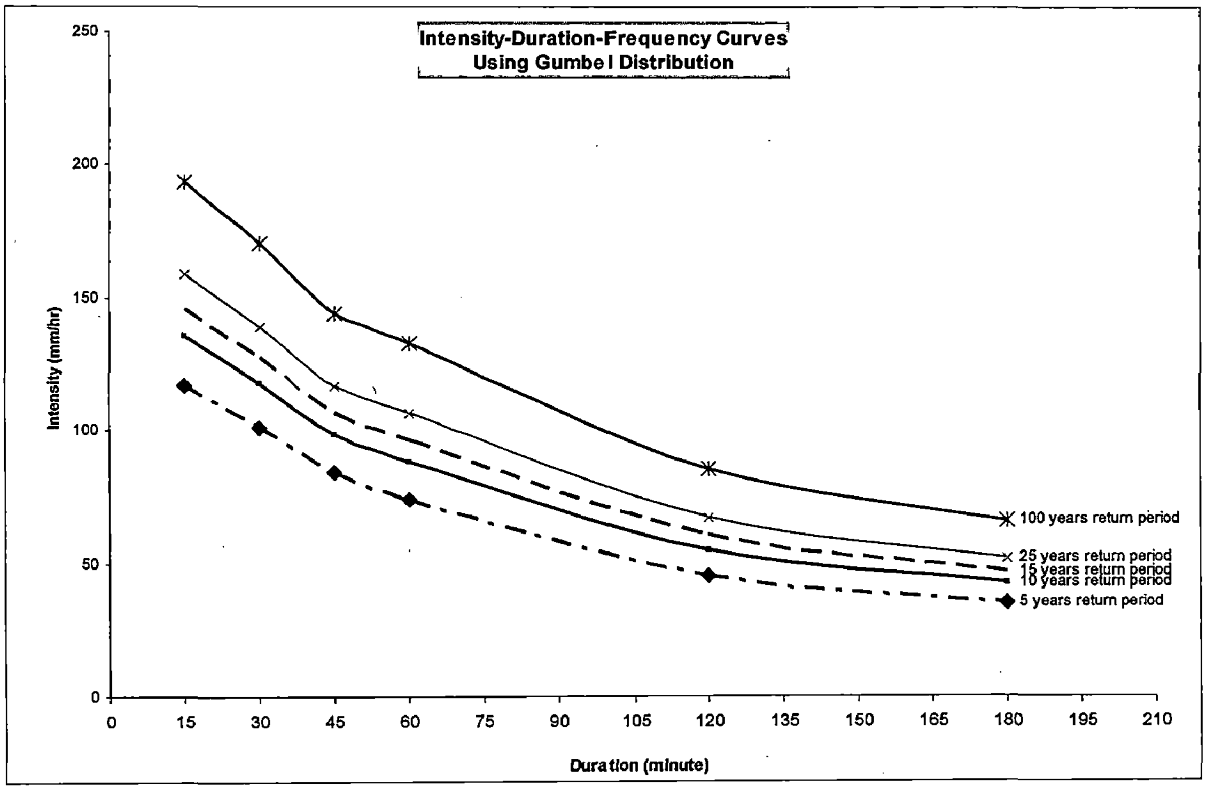

Figure (4.1) Intensity-Duration-Frequency Curves (using SRRG data during 1979-2006)

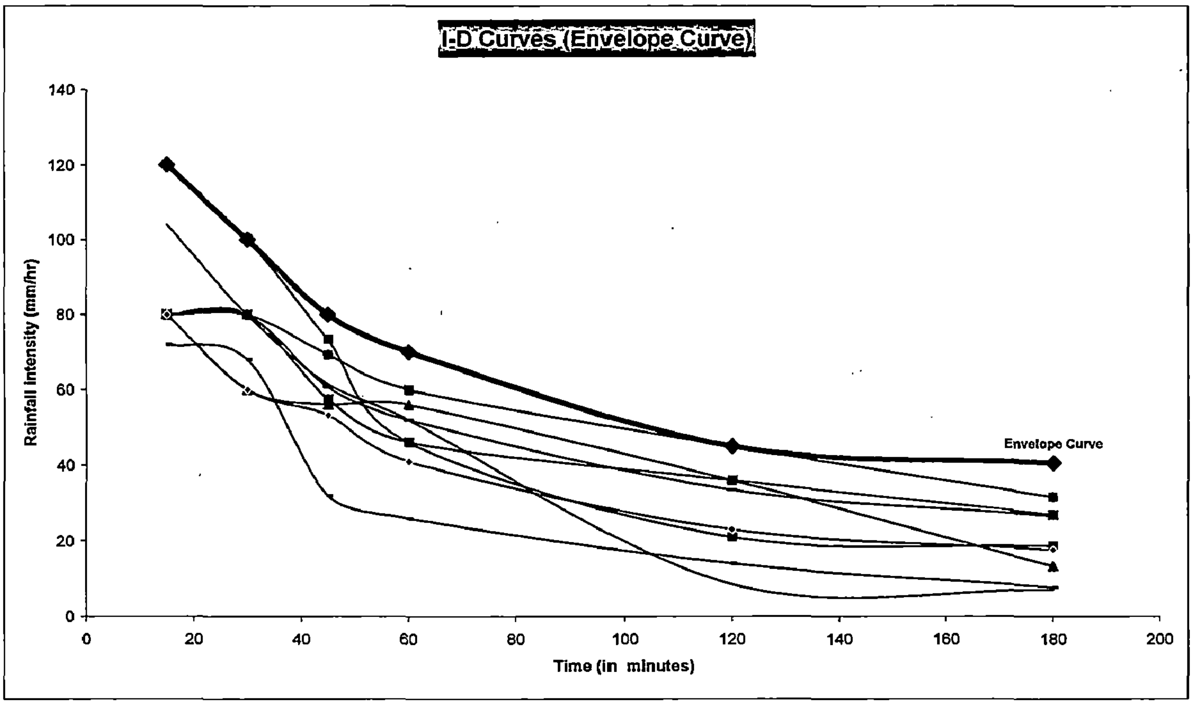

Figure (4.2) Intensity Duration Curve (using SRRG data during 1997-2006).

4.4-Design Storm Analysis Using Annual Rainfall Data

Annual daily maximum rainfall data for 30 years (1977-2007) were obtained from the observatory of the Department (table 4.5). The series was analysed using Gumbel's Extreme Value distributions to find the maximum rainfall depth for specific return periods. The results are tabulated in Table 4.6.

Twenty four hour distribution factors provided by Central Water Commission (CWC) and Indian Meteorological Department (IMD) were used to find out the distribution of rainfall within the 24 hours. The distributions are provided in tables (4.7) and (4.8).

In this study 5 years return period rainfall depths were used for storm water modelling. The depth calculated from SRRG data and maximum daily data with CWC and IMD distribution factors were used as the design storm for the city of Roorkee.

Table (4.5) Maximum Annual 24 hour's rainfall depth

Sl. No. (m)	Years	Maximum Rainfall (mm)
1	1977	76.8
2	1978	155
3	1979	99.5
4	1980	90.6
5	1981	195.4
6	1982	76
7	1984	76
8	1985	93
9	1986	63.8
10	1987	80.5
11	1988	155
12	1989	241.5
13	1990	148
14	1991	138.2
15	1992	171
16	1993	97.5
17	1994	263
18	1995	134.6
19	1996	123.6
20	1997	125.0
21	1998	121.6
22	1999	40.1
23	2000	120.0
24	2001	121.6
25	2002	170.0
26	2003	90
27	2004	110.6
28	2005	115.8
29	2006	224
30	2007	77.5

Table (4.6) Maximum 24 hours Rainfall of Different Return Periods Using Gumbel's Extrenme Distribution

Sl. No.	Return Period	Maximum Daily Rainfall in (mm)
1	5 years	173.11
2	10 years	209.21
3	15 years	229.59
4	25 years	254.92
5	50 years	288.6

Table (4.7) Twenty four hours rainfall distribution using CWC distribution factor

Time	Distribution Factor	Rainfall in (mm)							
		5 years		10 years		15 years		25 years	
		Cum RF	Incr RF	Cum RF	Incr RF	Cum RF	Incr RF	Cum RF	Incr RF
1	0.17	29.45	29.45	35.57	35.57	39.03	39.03	43.34	43.34
2	0.27	46.77	17.32	56.49	20.92	61.99	22.96	68.83	25.49
3	0.36	62.36	15.59	75.32	18.83	82.65	20.66	91.77	22.94
4	0.43	74.48	12.13	89.96	14.64	98.72	16.07	109.62	17.84
5	0.48	83.15	8.66	100.42	10.46	110.20	11.48	122.36	12.75
6	0.53	91.81	8.66	110.88	10.46	121.68	11.48	135.11	12.75
7	0.58	100.47	8.66	121.34	10.46	133.16	11.48	147.85	12.75
8	0.63	109.13	8.66	131.80	10.46	144.64	11.48	160.60	12.75
9	0.67	116.06	6.93	140.17	8.37	153.83	9.18	170.80	10.20
10	0.70	121.25	5.20	146.45	6.28	160.71	6.89	178.44	7.65
11	0.73	126.45	5.20	152.72	6.28	167.60	6.89	186.09	7.65
12	0.76	131.65	5.20	159.00	6.28	174.49	6.89	193.74	7.65
13	0.79	136.84	5.20	165.28	6.28	181.38	6.89	201.39	7.65
14	0.82	142.04	5.20	171.55	6.28	188.26	6.89	209.03	7.65
15	0.84	145.50	3.46	175.74	4.18	192.86	4.59	214.13	5.10
16	0.86	148.97	3.46	179.92	4.18	197.45	4.59	219.23	5.10
17	0.88	152.43	3.46	184.10	4.18	202.04	4.59	224.33	5.10
18	0.90	155.90	3.46	188.29	4.18	206.63	4.59	229.43	5.10
19	0.92	159.36	3.46	192.47	4.18	211.22	4.59	234.53	5.10
20	0.94	162.83	3.46	196.66	4.18	215.81	4.59	239.62	5.10
21	0.96	166.29	3.46	200.84	4.18	220.41	4.59	244.72	5.10
22	0.98	169.76	3.46	205.03	4.18	225.00	4.59	249.82	5.10
23	0.99	171.49	1.73	207.12	2.09	227.29	2.30	252.37	2.55
24	1.00	173.11	1.73	209.21	2.09	229.59	2.30	254.92	2.55

Cum. RF = Cumulative Rainfall

Incr. RF = Incremental rainfall

Table (4.8) Twenty four hours rainfall distribution using IMD distribution factor

Time	Distribut-ion factor	Rainfall in (mm)							
		5 years		10 years		15 years		25 years	
		Cum RF	Incr RF	Cum RF	Incr RF	Cum RF	Incr RF	Cum RF	Incr RF
1	0.43	73.57	73.57	88.91	88.91	97.58	97.58	108.34	108.34
2	0.53	91.75	18.18	110.88	21.97	121.68	24.11	135.11	26.77
3	0.60	103.87	12.12	125.53	14.64	137.75	16.07	152.95	17.84
4	0.65	112.52	8.66	135.99	10.46	149.23	11.48	165.70	12.75
5	0.70	120.31	7.79	145.40	9.41	159.57	10.33	177.17	11.47
6	0.73	125.50	5.19	151.68	6.28	166.45	6.89	184.82	7.65
7	0.75	129.83	4.33	156.91	5.23	172.19	5.74	191.19	6.37
8	0.77	133.29	3.46	161.09	4.18	176.78	4.59	196.29	5.10
9	0.79	136.76	3.46	165.28	4.18	181.38	4.59	201.39	5.10
10	0.81	140.22	3.46	169.46	4.18	185.97	4.59	206.49	5.10
11	0.83	143.68	3.46	173.64	4.18	190.56	4.59	211.58	5.10
12	0.85	146.28	2.60	176.78	3.14	194.00	3.44	215.41	3.82
13	0.86	148.87	2.60	179.92	3.14	197.45	3.44	219.23	3.82
14	0.88	151.47	2.60	183.06	3.14	200.89	3.44	223.06	3.82
15	0.89	154.07	2.60	186.20	3.14	204.34	3.44	226.88	3.82
16	0.91	156.66	2.60	189.34	3.14	207.78	3.44	230.70	3.82
17	0.92	159.26	2.60	192.47	3.14	211.22	3.44	234.53	3.82
18	0.94	161.86	2.60	195.61	3.14	214.67	3.44	238.35	3.82
19	0.95	164.45	2.60	198.75	3.14	218.11	3.44	242.17	3.82
20	0.96	166.19	1.73	200.84	2.09	220.41	2.30	244.72	2.55
21	0.97	167.92	1.73	202.93	2.09	222.70	2.30	247.27	2.55
22	0.98	169.65	1.73	205.03	2.09	225.00	2.30	249.82	2.55
23	0.99	171.38	1.73	207.12	2.09	227.29	2.30	252.37	2.55
24	1.00	173.11	1.73	209.21	2.09	229.59	2.30	254.92	2.55

Cum. RF = Cumulative Rainfall

Incr. RF = Incremental rainfall

CHAPTER-V

URBAN FLOOD SIMULATION USING SWMM

Simulation and modelling of urban floods are essential to understand the bottlenecks in the drainage system and also to estimate the extent of flooding. Several mathematical models are widely used to model the dynamics of rainfall-runoff and flood generation process. In this study, SWMM was used to simulate the storm water. A review of the model is presented in Chapter II. The following sections discuss the application of SWMM to simulate the urban flood using the results of Chapter III and IV.

5.1 Input Data and Parameters for SWMM

5.1.1 Rain gauge station

Rain Gages supply precipitation data for one or more sub-catchment areas in a study region. The rainfall data can be either a user-defined time series or come from an external file. Several different popular rainfall file formats currently in use are supported, as well as a standard user-defined format.

The principal input properties of rain gages include:

1. rainfall data type (e.g., intensity, volume, or cumulative volume).
2. recording time interval (e.g., hourly, 15-minute, etc.).
3. source of rainfall data (input time series or external file).
4. name of rainfall data source.

There are two raingauge locations in the City. These are located in the Department of Hydrology, IIT Roorkee and at the Govt Inter College in the old city. The length of the data in GIC observatory is very small. Hence, the data available at the Department of Hydrology Observatory was used in the study.

Selected Design Storm: Rainfall analysis was done in chapter-IV. The Time of concentration for the longest drains for overland flow was calculated as follows using Kirpich formula,

l = 100 meter, S= 0.005

$$t_c = 0.0078*[\frac{l^{0.77}}{S^{0.385}}]$$

$$t_c = 0.0078*[\frac{(100*3.28)^{0.77}}{(0.005)^{0.385}}] = 5 \text{ min } 11 \text{ sec}$$

The average velocity of the conduits is calculated as 2.12 m/s and the total length of conduits is 2200 meters for sub-catchment 3. The time of concentration for channel flows is computed by total length/ average velocity i.e. 17 min 18 sec.

Total tc is the sum of overland flow time plus channel flow time which is 22 min 29 sec. So the time of concentration for the longest sub-catchment 3 is 22 min 29 sec. Hence, 30 minutes duration was taken as design duration for choosing the design storm intensity.

Five years return period was adopted for further analysis. For 5 yrs return period and 30 min duration the intensity of rainfall (SRRG data) is 101.1 mm/hr (table 4.4 b and Fig 4.1). Simulation shall also be carried out for the maximum 24 hrs rainfall of 5 years return period (i.e. 173.11 mm/day; table 4.6). Twenty four hour distribution factors provided by Central Water Commission and Indian Meteorological Department were used to distribute rainfall within the 24 hours (tables 4.7 & 4.8).

5.1.2 Details of sub-catchments

Sub-catchments are hydrologic units of land whose topography and drainage system elements drain to a single discharge point. The user is supposed to divide the study area into an appropriate number of sub-catchments, and for identifying the outlet point of each sub-catchment. Discharge outlet points can be either nodes of the drainage system or other Sub-catchments. Sub-catchments can be divided into pervious and impervious subareas. Surface runoff can infiltrate into the upper soil zone of the pervious subarea, but not through the impervious subarea. Impervious areas are themselves divided into two subareas - one that contains depression storage and another that does not. Runoff flow from one subarea into a sub-catchment can be routed to the other subarea, or both subareas can drain to the sub-catchment outlet.

The analysis (in Chapter-III) shows that the study area is comprises of three sub-catchments of different flow directions. The study of the drainage pattern obtained from SRTM data indicates a water divide in old city i.e. right side of the Upper Ganges canal. This has been verified by field investigations and survey. Therefore two sub-catchments were delineated in this part of the city. One is draining through the thickly populated city core and another one in the south direction Fig (5.1). The left side area of the canal drains to a different point in downstream of river Solani. Thus, these areas were considered as separate sub-catchments for further analysis. The schematic of catchment used in simulation is shown in.

Infiltration of rainfall from the pervious area of a sub-catchment into the unsaturated upper soil zone can be described using three different models: Horton, Green-Ampt and SCS Method. In this study Green-Ampt model is used. Necessary soil parameters for the model were taken from a study conducted by Kartika (2006) and are presented in table 5.2.

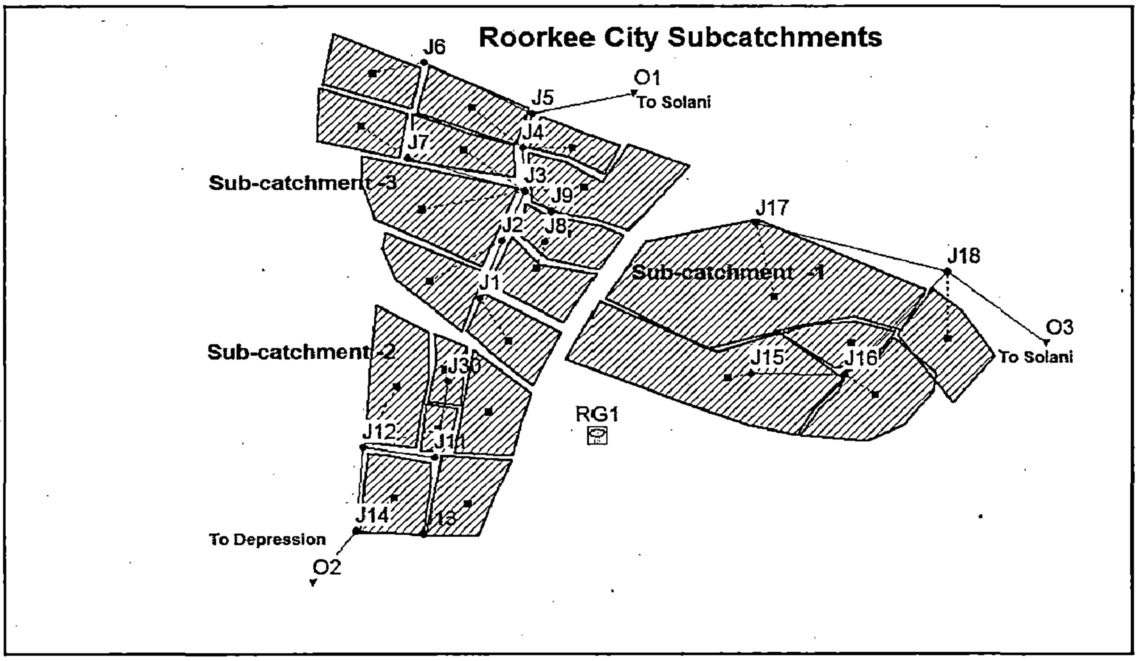

Figure (5.1) Schematic diagram for the sub-catchments in Roorkee city.

Table (5.1) Sub-catchments input parameters

Sub-catchment	Area (ha)	Width (m)	Impervious (%)
S1	12.51	310	60
S2	15.35	230	60
S3	43.2	550	60
S4	27.2	375	70
S5	17.3	260	70
S6	21.62	390	75
S7	31.4	540	60
S8	18.18	270	50
S9	9.92	225	70
S10	11.34	215	60
S11	23.05	350	45

Table (5.2) Soil properties at Roorkee City (Kartika, 2006)

Depth	Textural analysis			USDA
	Sand %	Silt %	Clay %	
0.00	66	12	22	**Sandy loam**
30.00	72.1	12.6	15.3	**Sandy loam**
60.00	85	4.6	10.6	Loamy sand
75.00	94.05	1.75	4.2	Sand
105.00	62.1	1.8	36.1	Clay
120.00	77.1	12.3	10.6	Loamy sand

Based on this data, sandy loam is considered as the representative soil type in the city. Various parameters used in Green-Ampt methods are then taken from the table 5.3

Table (5.3) properties of Sandy Loam soil as obtained from literature (Source: SWMM, 1993)

Soil texture class	K	ψ	Φ	FC	WP
Sand	4.74	1.93	0.437	0.062	0.024
Loamy sand	1.18	2.4	0.437	0.105	0.047
Sandy loam	**1.43**	**4.33**	**0.453**	**0.190**	**0.085**
loam	0.13	3.5	0.463	0.232	0.116
Silt loam	0.26	6.69	0.501	0.284	0.135
Sandy clay loam	0.06	8.66	0.398	0.244	0.136
Clay loam	0.04	8.27	0.464	0.310	0.187
Silty clay loam	0.04	10.63	0.471	0.342	0.210
Sandy clay	0.02	9.45	0.430	0.321	0.221
Silty caly	0.02	11.42	0.479	0.371	0.251
clay	0.01	12.6	0.475	0.378	0.265

K = saturated hydraulic conductivity
ψ = suction head
Φ = porosity, fraction
FC = field capacity, fraction
WP = welting point, fraction

5.1.3 Junction nodes

Junctions are drainage system nodes where links join together. Physically they can represent the confluence of natural surface channels, manholes in a sewer system, or pipe connection fittings. External inflows can enter the system at junctions. Excess water at a junction can become partially pressurized while connecting conduits are surcharged and can either be lost from the system or be allowed to pond atop the junction and subsequently drain back into the junction. Junction parameters and input data were given in tables (5.4) and (5.5).

Table (5.4) Junction Properties

Name	User-assigned junction name as shown in Fig (5.2)
Inflows	Assigned time series, dry weather to the junction. Assumed to be zero
Invert El.	Invert Elevation of the junction (meters)
Max. Depth	Maximum depth of the junction (i.e., from ground surface to invert, in meters)
Initial Depth	Depth of water at the junction at the start of the simulation (meters). Assume to be zero
Surcharge Depth	Addition depth of water beyond the maximum depth that is allowed before the junction floods. This parameter can be used to simulate bolted manhole covers.
Ponded Area	Area occupied by the ponded water at the junction after flooding occurs.

Table (5.5) Nodes input parameters

Node	Invert el. (m)	maximum depth (m)	latitude	longitude
J1	269.29	1.2	29 52 03	77 52 45
J2	267.24	1.2	29 52 23	77 52 56
J3	263.15	1.2	29 52 29	77 53 05
J4	261.35	1.2	29 52 41	77 52 56
J5	258.2	1.5	29 52 54	77 52 54
J6	260.8	1.5	29 53 03	77 52 41
J7	265.65	1.5	29 52 46	77 52 38
J8	266.73	1.3	29 52 15	77 52 58
J9	265.6	1.3	29 52 24	77 53 04
O1	254	Free outfall		

5.1.4 Outfall nodes

Outfalls are terminal nodes of the drainage system used to define final downstream boundaries under Dynamic Wave flow routing. For other types of flow routing they behave as a junction. Only a single link can be connected to an outfall node.

The boundary conditions at an outfall can be described by any one of the following stage relationships:

- the critical or normal flow depth in the connecting conduit

- a fixed stage elevation
- a tidal stage described in a table of tide height versus hour of the day
- a user-defined time series of stage versus time.

The principal input parameters required for outfalls are given in the following table.

Table (5.6) Outfall Properties

Name	User-assigned outfall name
Inflows	Assigned time series, dry weather to the outfall. If any
Invert El.	Invert Elevation of the outfall (meters)
Tide Gate	If any
Type	Type of outfall boundary condition:

5.1.5 Conduits

Conduits are conveyance system. Their cross-sectional shapes can be selected from a variety of standard open and closed geometries. Irregular natural cross-section shapes are also supported. For conduits properties see tables (5.7 & 5.8)

Table (5.7) Conduit Properties

Name	User assigned conduit name as shown in fig (F.2)
Inlet node	Name of node on the inlet end of the conduit (which is normally the end at higher elevation)
Outlet node	Name of node on the outlet end of the conduit (which is normally the end at lower elevation)
Shape	The geometric properties of the conduits cross section.
Length	Conduit length as Shown in table (5.8)
Roughness	Manning's roughness coefficient.
Inlet offset	Height of the conduit invert above the node invert at the upstream end of the conduit (meters)
Outlet offset	Height of the conduit invert above the node invert at the downstream end of the conduit (meters)
Initial Flow	Initial flow in the conduit if any.
Maximum Flow	Maximum flow allowed in the conduit under dynamic wave routing (flow units) use zero or leave it blank if not applicable

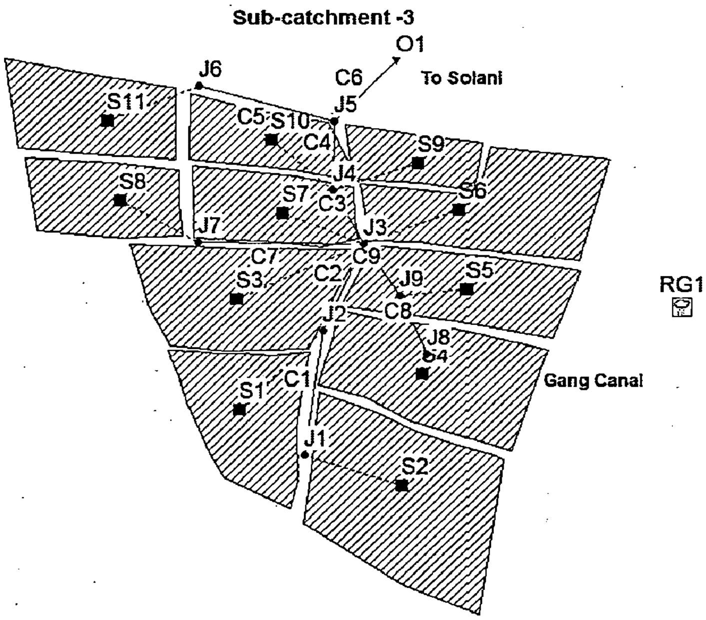

Figure (5.2) Details of the sub-catchment -3 used in simulation

Table (5.8) Conduits input parameters

Conduit	length (m)	width (m)	depth (m)	shape
C1	710	2	1.2	Rectangle
C2	580	2	1.2	Rectangle
C3	290	2	1.2	Rectangle
C4	620	2.5	1.2	Rectangle
C5	570	2	1.5	Trapezoidal
C6	780	3.2	1.5	Trapezoidal
C7	484	2	1.5	Rectangle
C8	365	2	1.3	Rectangle
C9	542	2	1.3	Rectangle

5.2 Output of the Model

Basic SWMM output consists of hydrographs and pollutographs (concentration vs. time) at any desired node in the drainage system. Depths and velocities along conduits and depths of nodes and many other parameters are also available. The output can be viewed either in tabular form or graphical from as per requirement of the user.

5.3 Simulation and Results

5.3.1 Pilot simulation study in the study area

In the preliminary run of the model for the entire city, no flooding was observed in the sub-catchment 1 and 2 using the design storm calculated from SRRG data and using annual maximum 24 hours for 5 years (as well 10 years) return period. Flooding was observed only in the sub-catchment 3 at node J3 and J4 (Fig (5.2). This area is the city core and is fully developed area and having maximum impervious areas. The drainage system is not sufficient to drain the generated storm runoff and detailed study was conducted for this sub-catchment.

In order to have a check for mass balance, the flood prone sub-catchment -3 was selected. Rational formula was used to compute the peak runoff from the sub-catchment and the results were compared with SWMM results. The results were found well within the acceptable range (table 5.9).

Table (5.9) comparison between peaks runoff computed using SWMM and Rational Formula for SRRG data.

Sub-catchment	Area (ha)	Runoff C	Q_p (SWMM)	Q_p Rational	Difference	% diff.
S1	12.51	0.998	3.35	3.49	-0.14	-4
S2	15.1	0.988	3.88	4.17	-0.29	-7
S3	43.2	0.981	10.85	11.86	-1.01	-9
S4	27.2	1.000	7.23	7.64	-0.41	-5
S5	17.3	1.000	4.65	4.8	-0.25	-5
S6	21.62	1.000	5.92	6.08	-0.28	-3
S7	31.4	0.991	8.17	8.71	-0.54	-5
S8	18.18	0.956	4.38	4.86	-0.48	-10
S9	9.92	1.000	2.71	2.78	-0.11	-2.5
S10	11.34	0.994	2.95	3.15	-0.2	-7
S11	23.05	0.940	5.4	6.06	-.66	-12

5.3.2 Simulation of urban flood in sub-catchment 3 using SRRG data

The detailed in-depth storm water simulation was conducted in the sub-catchment. The output of simulation for 5 years return period 30 minute duration rainfalls using SWMM are given in Appendix –III a.

SWMM uses Non-linear reservoir model (NLR) for computation of runoff from sub-catchment (Bennis, 2007). The NLR model conceptualizes the urban sub-catchment as a reservoir having the rainfall as input, rainfall abstractions, and runoff as output. The depth of water in the reservoir is found coupling the continuity equation and Manning's equation Sensitivity study was conducted to see how much Manning's coefficient affect the runoff generation process in the sub-catchments; the results were given in appendix IV. Fig (5.3 & 5.4) shows response of the parts of the sub-catchment (S1-S11) as shown in Fig 5.2. These hydrographs are used in SWMM as input to the corresponding nodes for hydraulic routing of the storm water.

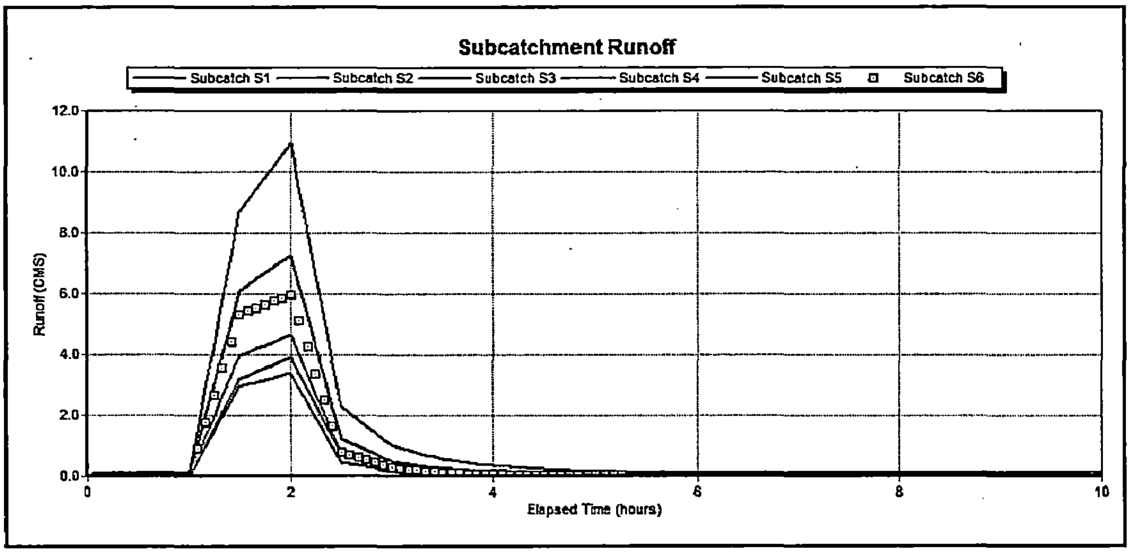

Figure (5.3) Sub-catchments runoff for S1 to S6

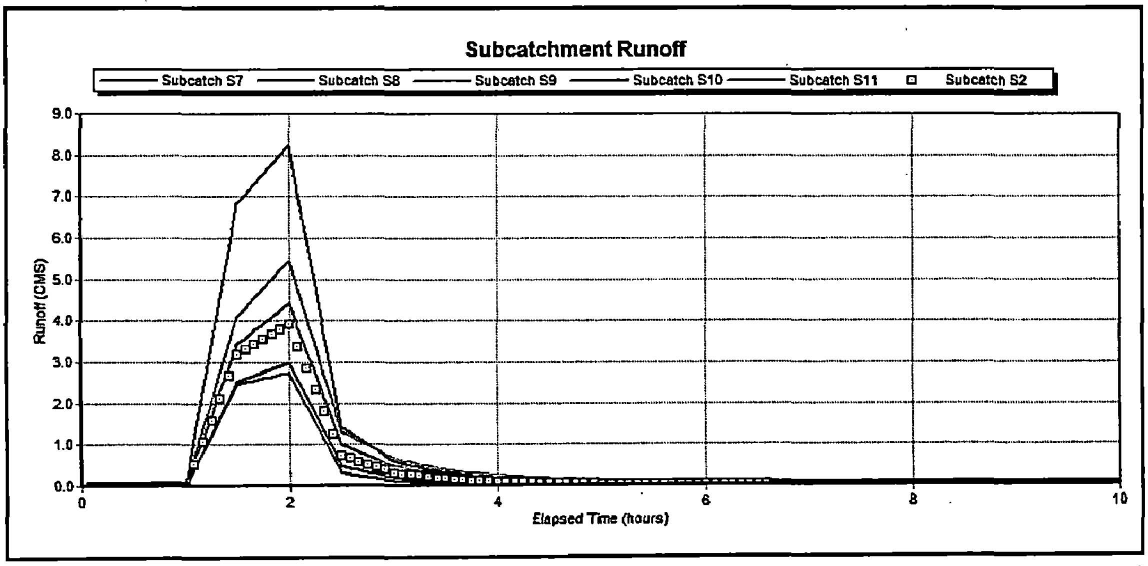

Figure (5.4) Sub-catchments runoff for S7, S8, S9, S10 S11 and S2

Total inflow to each node is the sum of upstream link flow and lateral flow (sum of directly connected sub-catchments). Fig (5.5) shows a representative hydrograph of lateral inflow at node J3 (wherein S3, S6 and S7 are contributing). Fig (5.6) shows the routed link flow from C1 to C4. It is evident from the figure that the node J3 is surcharged and there will be ponding atop the node. The total inflow to node J3 is shown in Fig (5.7).

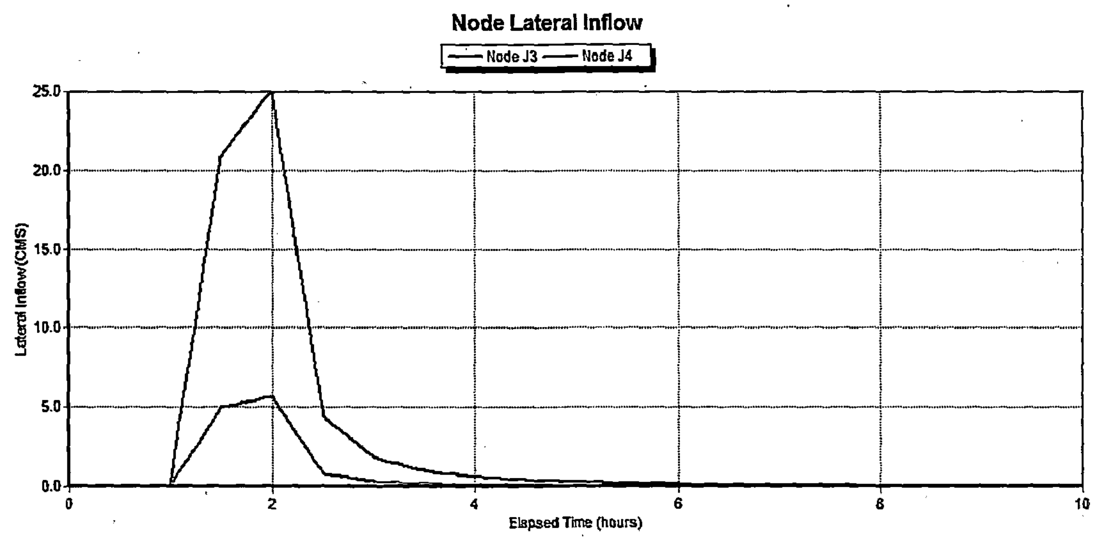

Figure (5.5) Lateral inflow to node J3 and J4

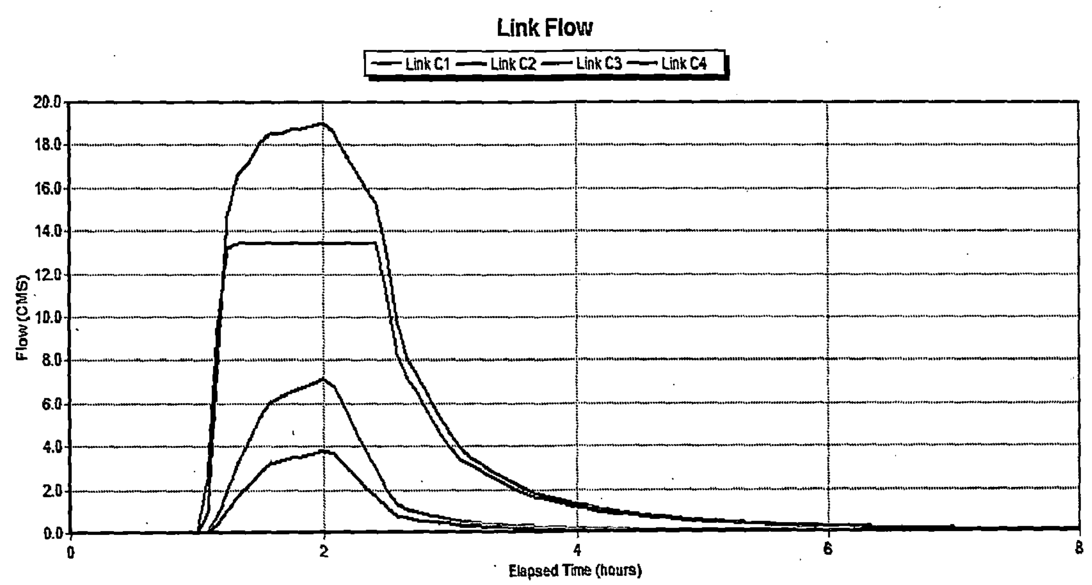

Figure (5.6) Conduits flow from C1-C4.

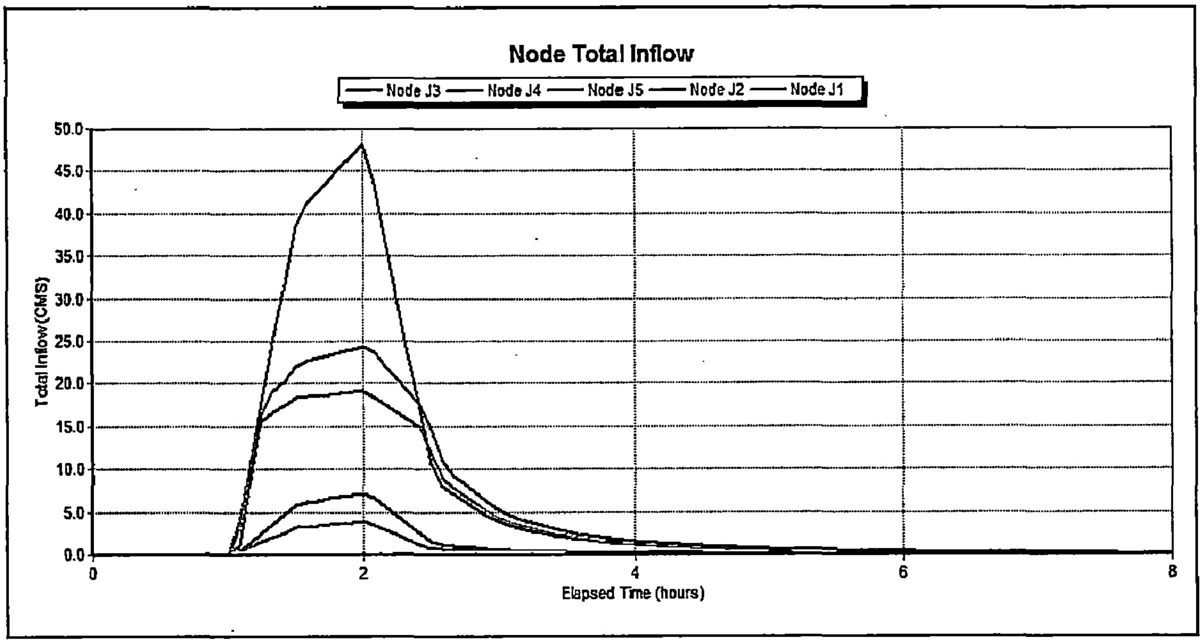

Figure (5.7) Total inflow at various nodes in the sub-catchment 3

If a downstream conduit doesn't have sufficient capacity to drain the total inflow at the node, the surcharged water accumulated atop the node as flood water. The flood hydrograph at the node is the difference between total inflow to the node and maximum capacity of draining conduits at the downstream. Flooding was observed at node J3 and node J4. Fig (5.8) shows the flood hydrograph at node J3 and J4.

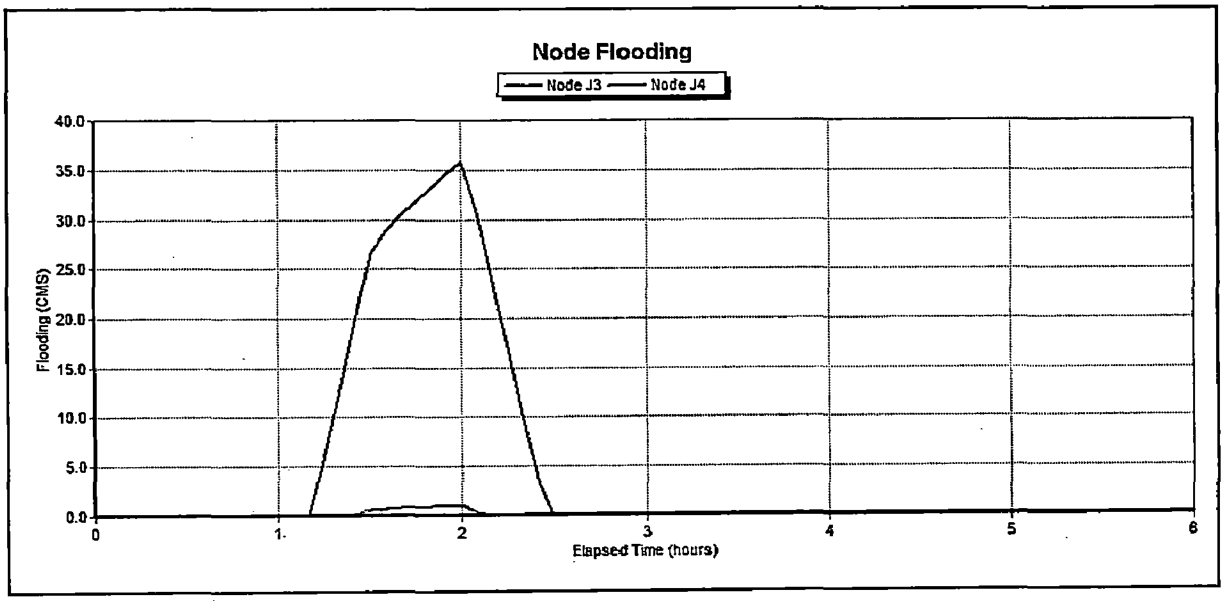

Figure (5.8) flood hydrograph at node J3 and node J4

SWMM is capable to show the water surface profile for any given or defined path along the drainage system. Fig (5.9) shows the water surface profile from junction J1 to J4. The profile clearly shows the surcharged conduit between J3 to J4.

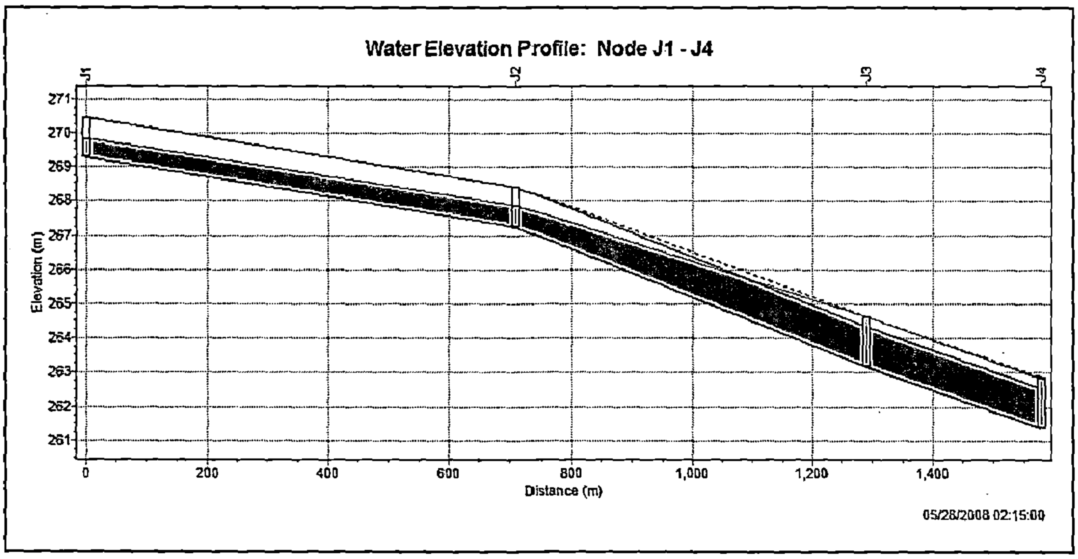

Figure (5.9) Water surface profile between node J1 and node J4

5.3.3 Simulation of Urban Flood in Sub-catchment 3 using 24 hours' maximum annual rainfall

Since CWC distribution factor doesn't gives any flooding; the results here were only shown for IMD distribution factor. In SWMM the rainfall is assumed to be uniformly distributed over the sub-catchment. Fig (5.10) shows the rainfall hyetograph for 5 yrs return period maximum 24 hours over some of the sub-catchments.

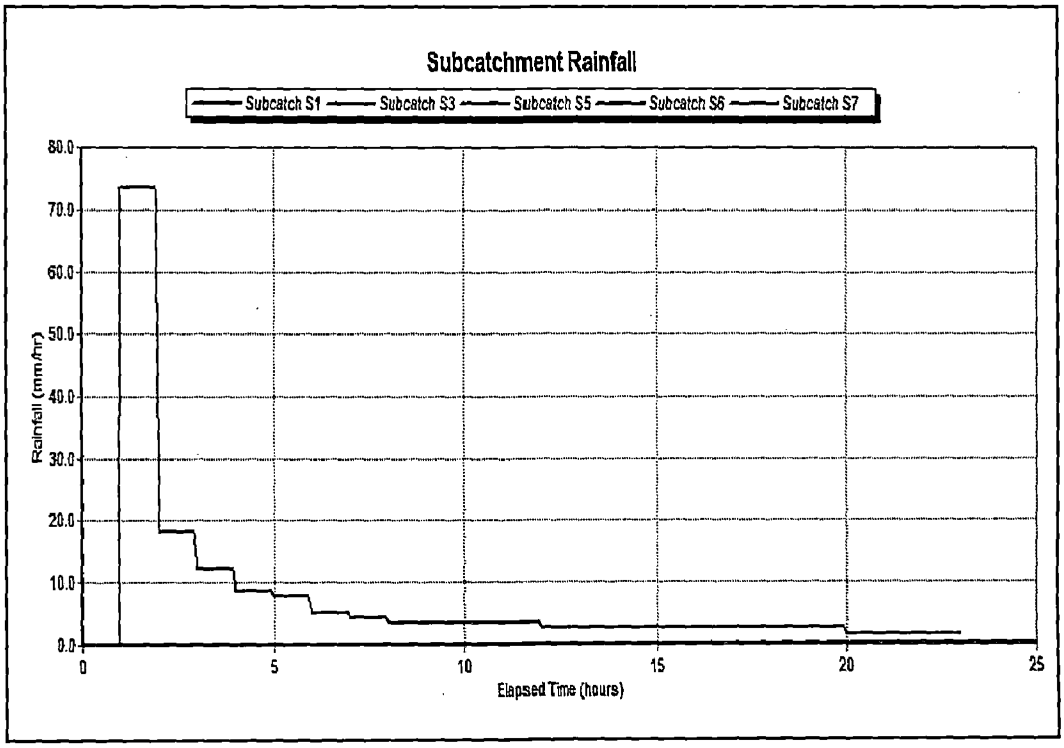

Figure (5.10) Sub-catchments rainfall.

The Figures (5.11 & 5.12) show the response of sub-areas S1 to S11 of sub-catchment -3. Lateral flow and conduit flow are shown in Figures (5.13 & 5.14). The total inflow hydrograph at junction J1 to J5 are shown in Fig (5.15). The flooding was observed at node J3 only (Fig 5.2), the flood hydrograph is shown in Fig (5.16). The detailed outputs for annual maximum 24 hrs rainfalls were given in Appendix III b & c.

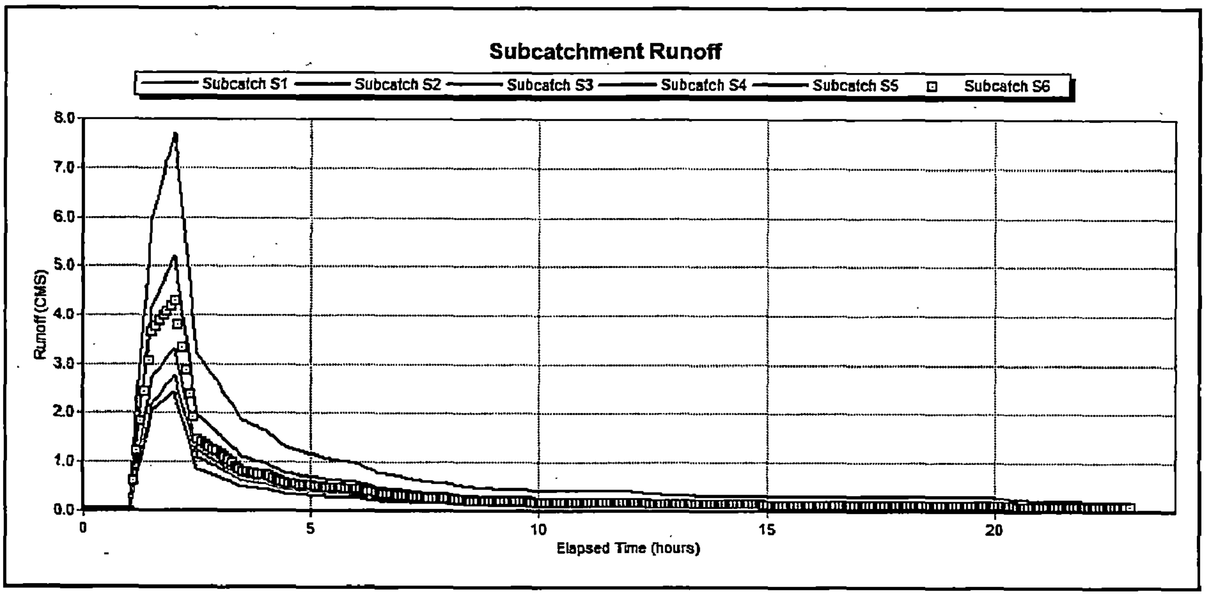

Figure (5.11) Sub-catchment runoff for S1 to S6

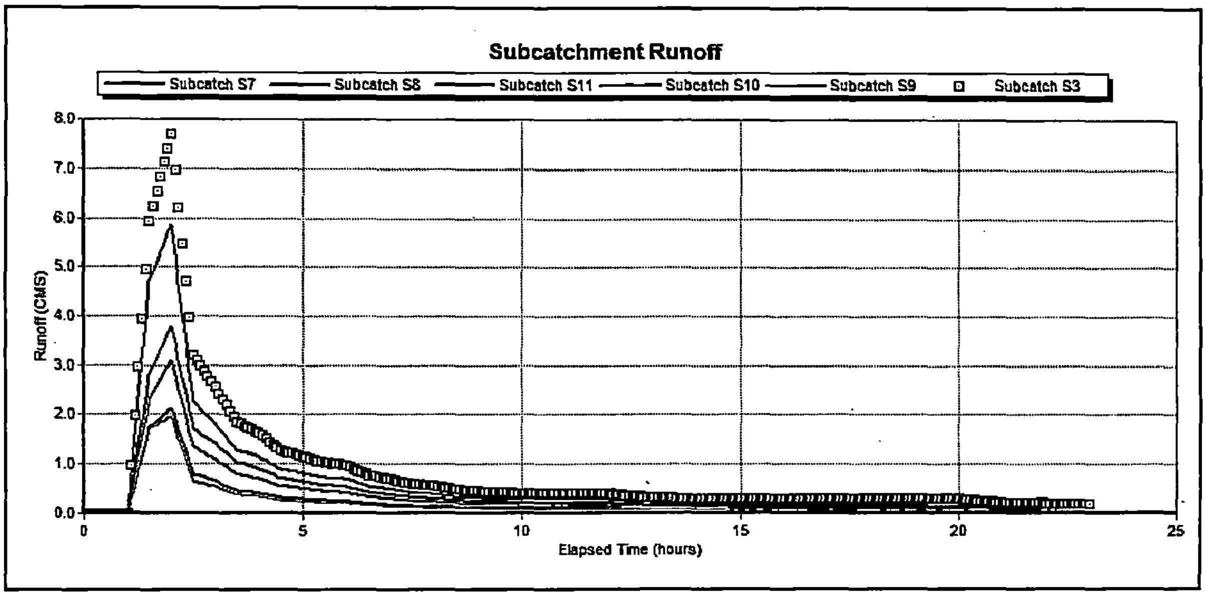

Figure (5.12) Sub-catchment runoff for S7, S8, S9, S10, S11, and S3

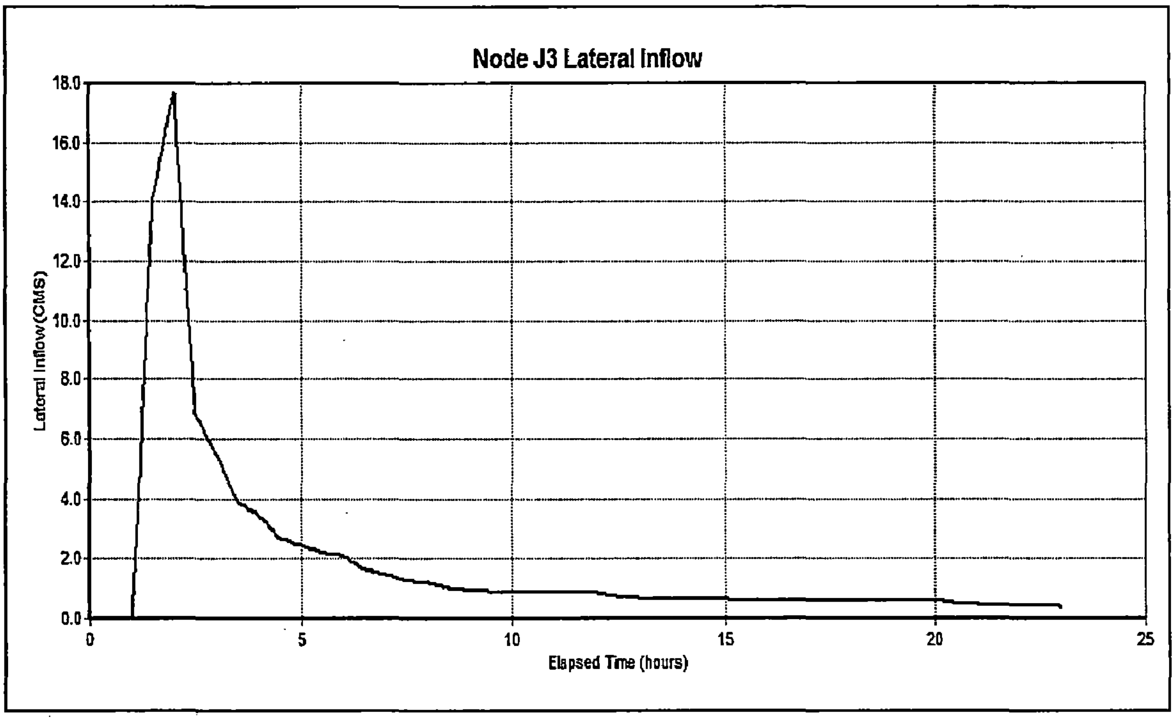

Figure (5.13) lateral inflow to node J3.

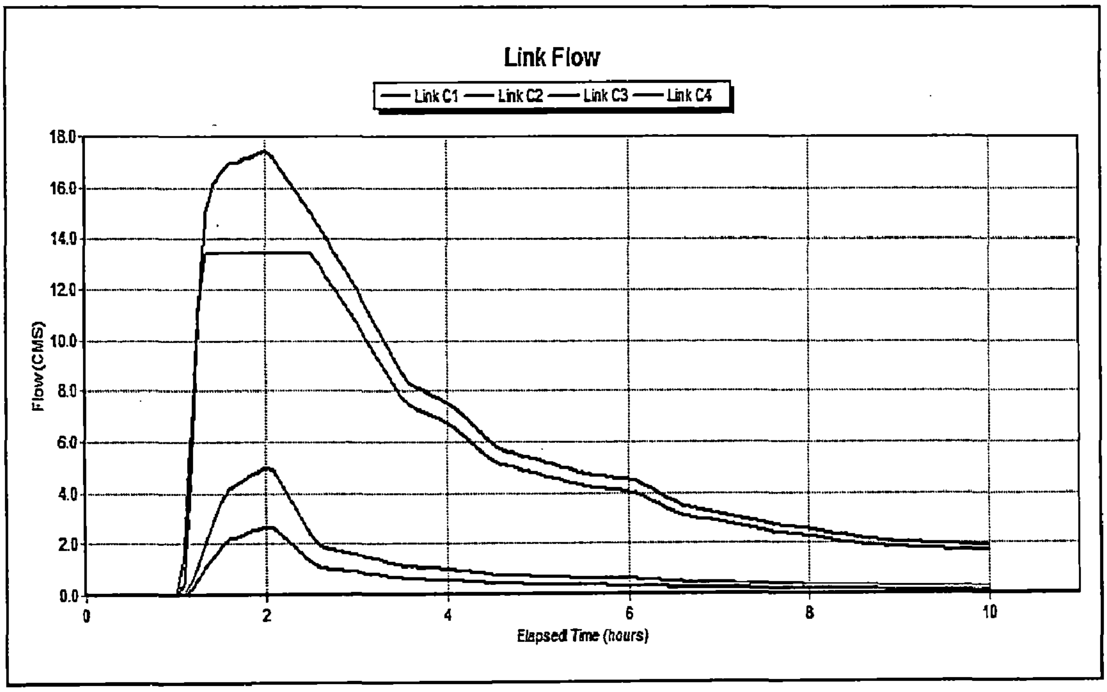

Figure (5.14) Conduits flow.

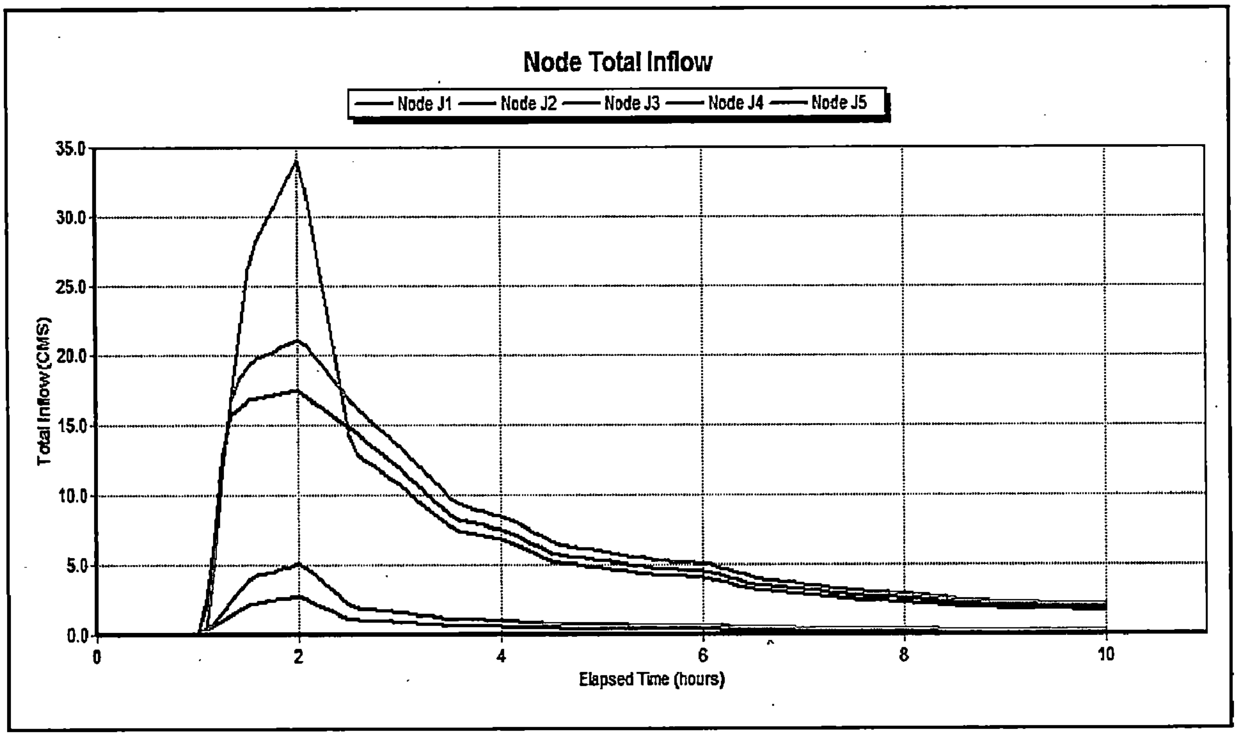

Figure (5.15) Inflow to nodes.

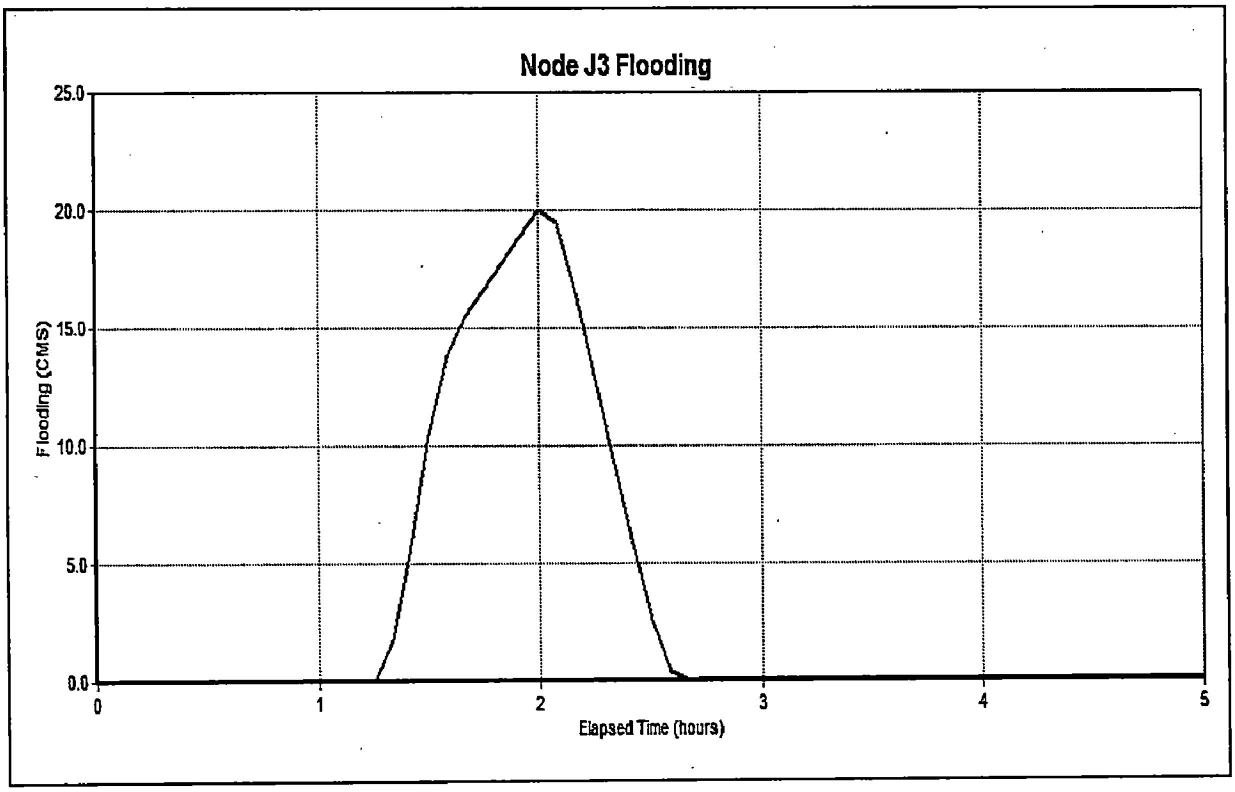

Figure (5.16) Total flood hydrograph at node J3

5.3.4 Comparison of the results of simulated Storm Water

Flooding was observed at two nodes (node J3 and node J4) using SRRG data, and one node J3 using annual maximum 24 hours data distributed using IMD distribution factor. However, no flooding was observed using CWC distribution factor. The volume of flooding using SRRG data is the maximum volume of flooding (10276.44 mm-ha) in sub-catchment 3. The volumes and flooding time are presented in table (5.10).

Table (5.10) Comparison of results of SWMM simulation

Return period	Design Storm (mm/hr)	Volume of flooding mm/ha		Time flooded (min)	
		J3	J4	J3	J4
5 years 30 min (SRRG)	101.1	10276	185	73	43
5 yrs 24 hr IMD	*73.57	6001	0	76	0
5 yrs 24 hr CWC	*29.45	0	0	0	0

*Maximum in one hour

5.3.5 Computation of flood depth and extent

The total inflow hydrograph at node J3 Fig (5.7) and lateral inflow to node J4 Fig (5.5) were introduced to the corresponding nodes J3 and nodes J4 as time dependent inflow. The total inflow and lateral flows are given in table 5.11 and table 5.12. These were routed along the road as compound section (road plus drain Fig 5.17 & 5.18) to the node J4 considering that the ponded water will find its way only through the road. The road side were accordingly raised sufficiently to accommodate the ponded water (i.e. 2 m approximately). After the routing, the resulted profile of flood water between these nodes is given in Fig (5.19). This exercise was carried out for SRRG as well as annual 24 hrs rainfall (IMD) data.

Table (5.11) Total inflow to node J3

Hours	Total Inflow (CMS)
00:05:00	0
00:10:00	0
00:15:00	0
00:20:00	0
00:25:00	0
00:30:00	0
00:35:00	0
00:40:00	0
00:45:00	0
00:50:00	0
00:55:00	0
01:00:00	0.21
01:05:00	5.55
01:10:00	10.84
01:15:00	18.54
01:20:00	25.38
01:25:00	32.09
01:30:00	38.79
01:35:00	41.1
01:40:00	42.55
01:45:00	43.96
01:50:00	45.38
01:55:00	46.8
02:00:00	48.14
02:05:00	42.84
02:10:00	36.25
02:15:00	29.63
02:20:00	23.03
02:25:00	16.45
02:30:00	9.99
02:35:00	7.98
02:40:00	7.04
02:45:00	6.2
02:50:00	5.37
02:55:00	4.55
03:00:00	3.74

Table (5.12) Lateral inflow to node J4

Hours	Lateral Inflow (CMS)
00:05:00	0
00:10:00	0
00:15:00	0
00:20:00	0
00:25:00	0
00:30:00	0
00:35:00	0
00:40:00	0
00:45:00	0
00:50:00	0
00:55:00	0
01:00:00	0
01:05:00	0.96
01:10:00	1.65
01:15:00	2.47
01:20:00	3.33
01:25:00	4.12
01:30:00	4.93
01:35:00	5.05
01:40:00	5.18
01:45:00	5.31
01:50:00	5.44
01:55:00	5.57
02:00:00	5.68
02:05:00	4.82
02:10:00	4.03
02:15:00	3.18
02:20:00	2.39
02:25:00	1.61
02:30:00	0.79
02:35:00	0.71
02:40:00	0.63
02:45:00	0.54
02:50:00	0.45
02:55:00	0.37
03:00:00	0.3

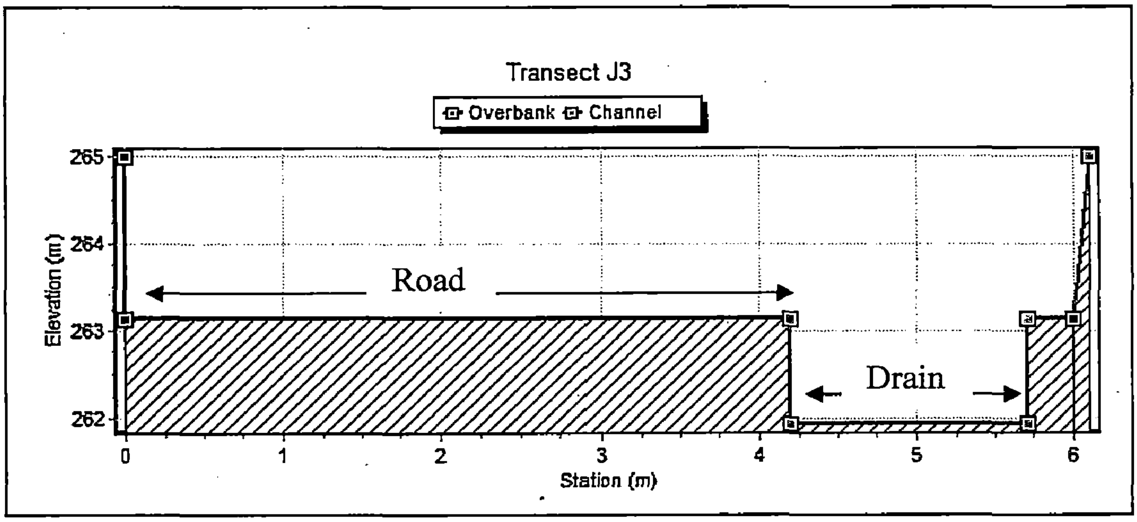

Figure (5.17) Compound section at node J3

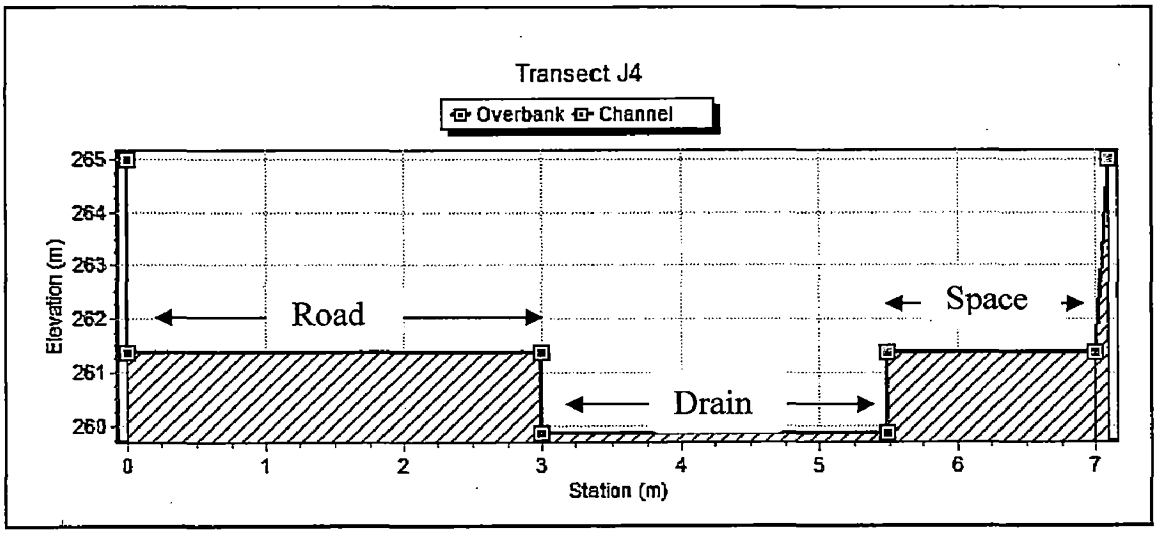

Figure (5.18) Compound section at node J4

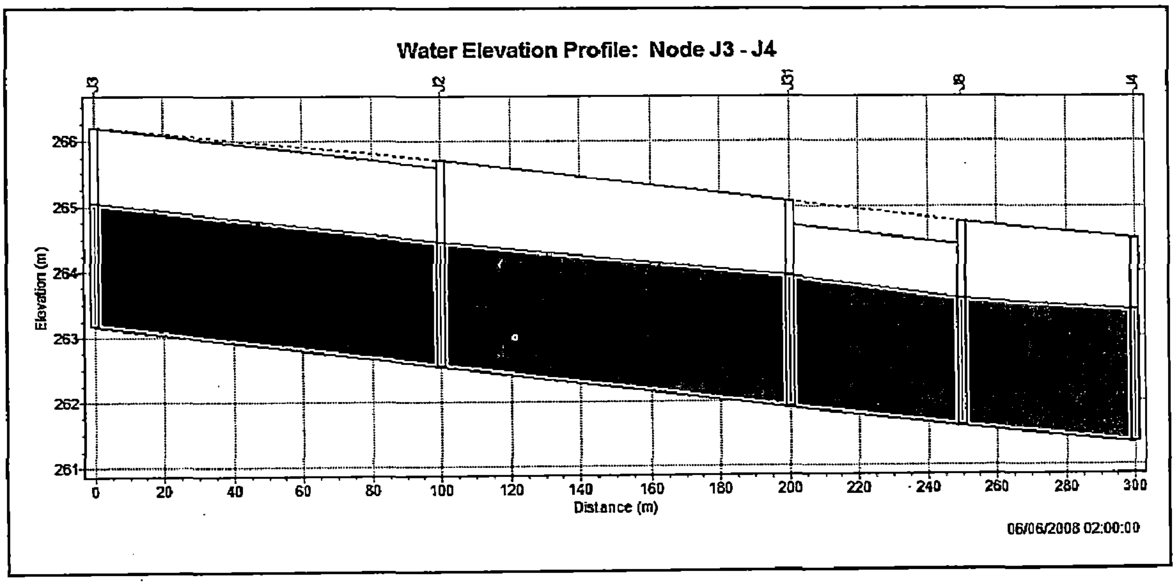

Figure (5.19) Flood water surface profile between node J3 and node J4 using compound sections (depths are taken from the bottom of the drains)

A comparison of the results of routing of the flood hydrograph along the compound section are given in table (5.13). The result of flooding using SRRG data were imported into GIS to determine the areal extent and the result are shown in Fig (5.20).

Table (5.13) result of SWMM routing of flow along compound section

Station	Cumulative Distance from node (m)	Total depth of water (m)	Depth of drain (m)	Depth of flooding (m)		Time flooded (min)	
				SRRG	Annual max. 24hr	SRRG	Annual max. 24hr
Node J3 Old station road (Amber talab)	0	*2.15	1.2	0.95	(0.71)	73	76
	100	2.14	1.2	0.94	(0.77)		
	200	2.10	1.2	0.90	(0.71)		
	250	2.10	1.2	0.90	(0.71)		
Node J4 (Avas vikas)	300	2.30	1.5	0.80	(0.01)	43	0

*Total depth was given for SRRG data only

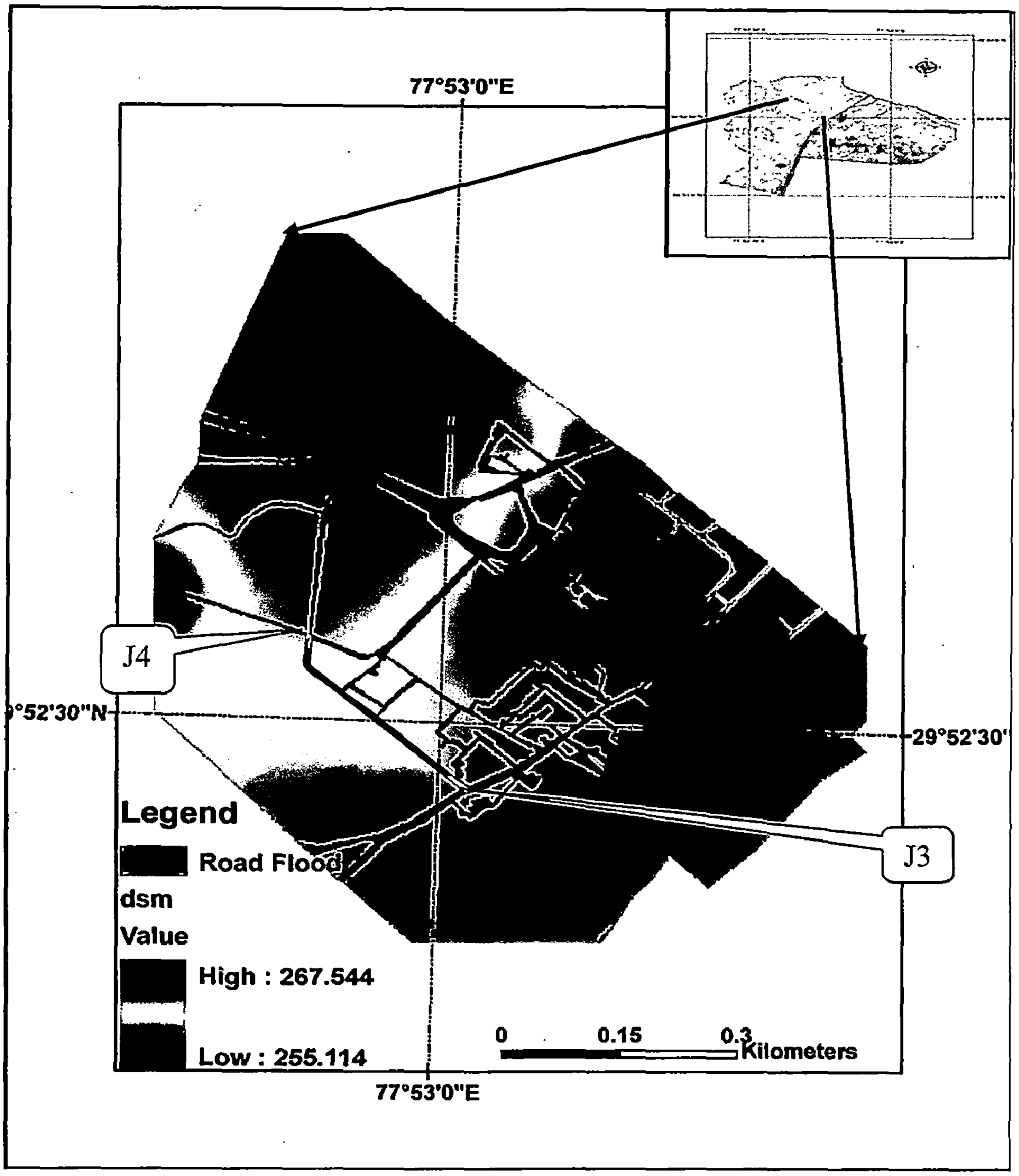

Fig (5.20) Extent of flooding along the road for SRRG data.

6.1 General

Floods are natural events that have always been an integral part of the geologic history of the earth. Floods not only happen in river basins, the risk of urban flooding is also increasing due to rapid urbanization. Unlike river floods, urban flooding happens more frequently and causes large amount of accumulated damage, though the damage per event is relatively smaller compared with the severe consequences caused by river flooding. In addition, urban flooding has brutal impacts on municipality's activities when it happens. In this context, a study was undertaken to carryout urban flood modelling and the city of Roorkee was chosen as the case study.

The Environmental Protection Agency Storm Water Management Model (EPA SWMM) is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. SWMM has many applications word over, specifically in sewer and storm water studies. SWMM was used to study the urban flooding in the City of Roorkee. GIS was also used for studying the drainage pattern in and around the City. It was also used as a pre and post data processing tool.

6.2 Findings of the Study

Following are the summary and conclusions of the present study:

i. SRTM DEM data Pan Data (IRS 1C) was analysed using spatial analyst of ArcGIS to find flow directions, flow accumulation and watershed boundaries. Higher resolution DEM & DSM were also prepared using spot elevations obtained from the drainage map (from IRI) and values collected using GPS.

ii. The study of the drainage pattern indicates a water divide in old city i.e right side of the Upper Ganges Canal. It was also verified with the field survey. Therefore two catchments were delineated in this area draining in different directions. One is draining through the thickly populated city core and another one in the south direction. The left side area of the canal drains to River Solani, but at a different point in downstream of the river.

iii. Design storm analysis was carried out for the City using SRRG data and 24 hours annual maximum data obtained from the Department of Hydrology. Gumbel's Extreme Value Distribution was used to find the design storm of various return periods. The design storms for 5 years return period was 101.1 mm/hr (for 30 minute duration) using SRRG data and 173.11 mm/day was using 24 hours annual maximum rainfall data.

iv. The study revealed that there was no flooding found in sub-catchment 1 and 2. However, there was flooding in the city core (sub-catchment 3) at node J3 (Old station road, Amber talab) and node J4 (near Avas vikas). The maximum volume of flooding was found 10276.44 mm/ha for SRRG data and 6001mm/ha for annual 24 hours rainfall.

v. This volume of flooding was routed along the roads as compound section (having drains plus road). The maximum depth of flooding was found to be 0.95 meter above the road at Old station road, Ambertalab and 0.80 meters before AvasVikas colony. Ponding in the area remains for about one and half hours.

vi. The EPA SWMM was successfully applied and the results were cross checked with alternate methods.

6.3 Future Scope of Work

i. Use of fine resolution DEM is suggested for more accurate delineation of flood drainage pattern and flood depth computations.

ii. Simulation of storm water was done only for major drains having width and depth greater than one meter, minor drains may also be considered in future studies.

iii. Calibration of infiltration parameters was done using soil properties; however observed runoff hydrograph might calibrate the model better.

iv. Drainage system of Roorkee requires a revisit to the design in order to accommodate the increased storm water attributed to urbanisation.

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Appendix I Selective Storms for Intensity Duration Curves the Analysis Up to 3 Hr Duration

Date	Starts Time (hr:min)	Cumulative Depth	15 min		30 min		45 min		60 min		120 min		180 min	
			Depth	Intensity	Depth	Intensity	Depth	Intensity	Depth	Intensity	Depth	Intensity	Depth	Intensity
09-07-2006	08:15	0.5												
	30	4	3.5											
	45	10	6											
	09:00	**20**	**10**	**40**	**16**	**32**	**22**	**29.33**	**26**	**26**	**31**	**15.5**	**35**	**11.67**
	:15	26	6											
	:30	30	4											
	:45	31	1											
	10:00	31.5	0.5											
	:15	33	1.5											
	:30	34	1											
	:45	34.5	0.5											
	11:00	35	0.5											
20-07-2006	08:00	2												
	:15	4	2											
	:30	5.5	1.5											
	:45	7	1.5											
	9:00	10	3											
	:15	20	10											
	:30	30	10											
	:45	40	10											
	10:00	**53**	**13**	**52**	**23**	**46**	**33**	**44**	**43**	**43**	**51**	**25.5**	**55**	**18.33**
	:15	54.5	1.5											
	:30	56	1.5											

	:45	56.5	0.5											
	11:00	57	0.5											
26-07-2006	00:00	0												
	00:15	28	28											
	:30	32	4											
	:45	36.5	4.5											
	01:00	37.5	1											
	:15	40	2.5											
	:30	50	10											
	:45	60	10											
	02:00	70	10											
	:15	90	20											
	:30	**120**	**30**	**120**	**50**	**100**	**60**	**80**	**70**	**70**	**90**	**45**	**122**	**40.67**
	:45	130	10											
	03:00	140	10											
	:15	150	10											
	:30	154	4											
	:45	157	3											
	04:00	158	1											
	:15	160	2											
	:30	161	1											
	:45	162	1											
	05:00	164	2											
	:15	166	2											
	:30	172	6											
	:45	175	3											
	06:00	180	5											
15-08-2006	14:15	0												
	:30	10	10											
	:45	**20**	**10**	**40**	**20**	**40**	**28**	**37.33**	**30**	**30**				
	15:00	28	8											

	:15	30	2											
26-05-2005	02:45	0												
	03:00	0.5	0.5											
	:15	10	9.5											
	:30	**30**	**20**	**80**	**34**	**68**	**43.5**	**58**	**46**	**46**				
	:45	44	14											
	04:00	46.5	2.5											
	:15	47.5	1											
11-07-2005	04:00	0												
	:15	10	10											
	:30	12	2											
	:45	12	0											
	05:00	12.5	0.5											
	:15	16	3.5											
	:30	24	8											
	:45	**36**	**12**	**44**	**20**	**40**	**24**	**32**	**27.5**	**27.5**	**40**	**20**	**46**	**15.33**
	06:00	40	4											
	:15	42	2											
	:30	43	1											
	:45	45	2											
	07:00	46	1											
	:15	47	1											
	:30	48	1											
	:45	50	2											
	08:00	51	1											
23-07-2005	08:00	0												
	:15	12	12	44	20	40	25	33.33						
	:30	20	8											
	:45	25	5											
13-08-2005	14:15	0												

	:30	20	20											
	:45	50	30	120	50	100	55	73.33						
	15:00	55	5											
16-08-2005	01:00	0	0											
	:15	2	2											
	:30	4	2											
	:45	14	10	40	14	28	18	24	20	20	29	14.5	34	11.33
	02:00	18	4											
	:15	22	4											
	:30	24	2											
	:45	28	4											
	03:00	29	1											
	:15	30	1											
	:30	31	1											
	:45	32	1											
	04:00	34	2											
	:15	34.5	0.5											
	:30	35	0.5											
	:45	35.5	0.5											
	05:00	36	0.5											
10-09-2005	09:30	0												
	:45	18	18	72	22	44	22.5	30	23	23	25	12.5	31	10
	10:00	22	4											
	:15	22.5	0.5											
	:30	23	0.5											
	:45	24	1											
	11:00	25	1											
	:15	26	1											
	:30	28	2											
	:45	30	2											
	12:00	32	2											

	:15	32.5	0.5											
	:30	34	1.5											
	:45	34.5	0.5											
	13:00	35	0.5											
	:15	36	1											
	:30	37	1											
	:45	37.5	0.5											
	14:00	38.5	1											
24-09-2005	03:15	10												
	:30	16	6											
	:45	22	6											
	04:00	40	18	72	24	48	28	37.33	34	34	42	21	56	18.67
	:15	44	4											
	:30	48	4											
	:45	50	2											
	05:00	53	3											
	:15	56	3											
	:30	60	4											
	:45	63	3											
	06:00	66	3											
	:15	70	4											
	:30	76	6											
	:45	86	10											
	07:00	90	4											
	:15	92	2											
	:30	94	2											
	:45	96	2											
	08:00	102	6											
	:15	103	1											
24-09-2005	09:15	0												
	:30	3	3											

	:45	6	3											
	10:00	10	4											
	:15	16	6											
	:30	22	6											
	:45	30	8	32	16	32	24	32	28	28	40	20	52	17.33
	11:00	38	8											
	:15	40	2											
	:30	43	3											
	:45	47	4											
	12:00	50	3											
	:15	52	2											
	:30	54	2											
	:45	57	3											
	13:00	60	3											
06-07-2004	05:00	0												
	:15	6	6											
	:30	16	10											
	:45	30	14											
	06:00	44	14											
	:15	60	16	64	30	60	42	56	56	56				
	:30	70	10											
	:45	86	16											
07-07-2004	00:00	0												
	:15	10	10											
	:30	20	10	40	20	40	30	40	34	34				
	:45	30	10											
	01:00	34	4											
23-08-2004	10:00	7												
	:15	10	3											
	:30	12	2											
	:45	12.5	0.5											

	11:00	13.5	1											
	:15	16	2.5											
	:30	17	1											
	:45	20	3											
	12:00	30	10	40	13	26	16	21.33	18	18	26.5	13.25	40	13.25
	:15	33	3											
	:30	35	2											
	:45	36	1											
	13:00	40	4											
05-07-2003	02:00	0												
	:15	10	10											
	:30	20	10											
	:45	40	20	80	30	60	40	53.33	50	50	72	36		
	03:00	50	10											
	:15	54	4											
	:30	60	6											
	:45	70	10											
	04:00	72	2											
21-07-2003	01:00	6												
	:15	7	1											
	:30	10	3											
	:45	15	5	20	10	20	13	17.33	16	16	19	8.5	21	7
	02:00	20	5											
	:15	23	3											
	:30	24	1											
	:45	24.5	0.5											
	03:00	25	0.5											
	:15	26	1											
	:30	26	0											
	:45	26.5	0.5											
	04:00	27	0.5											

	:15	27	0											
	:30	27.5	0.5											
	:45	27.5	0											
	05:00	28	0.5											
	:15	28	0											
	:30	28.25	0.25											
	:45	28.28	0.03											
	06:00	28.5	0.22											
	:15	29	0.5											
	:30	29.5	0.5											
	:45	29.75	0.25											
	07:00	30	0.25											
29-08-2003	21:30	4												
	:45	10	6											
	22:00	30	20											
	:15	50	20	80	40	80	46	61.33	52	52				
	:30	56	6											
	:45	57	1											
30-08-2003	07:45	17												
	08:00	20	3											
	:15	30	10											
	:30	56	26	104	36	72	46	61.33	49	49				
	:45	66	10											
03-08-2002	07:00	8												
	:15	10	2											
	:30	20	10											
	:45	30	10											
	08:00	34	4											
	:15	50	16	64	20	40	30	40	40	40	56	28	68	22.67
	:30	54	4											

	:45	60	6											
	09:00	64	4											
	:15	67	3											
	:30	70	3											
	:45	73	3											
	10:00	76	3											
6/9/2002 a	15:15	10												
	:30	12	2											
	:45	13	1											
	16:00	14	1											
	:15	17	3											
	:30	20	3											
	:45	22	2											
	17:00	25	3											
	:15	27	2											
	:30	30	3											
	:45	35	5											
	18:00	40	5	20	10	20	14	18.67	18	18	27	13.5	40	13.33
	:15	44	4											
	:30	48	4											
	:45	50	2											
	19:00	54	4											
	:15	56	2											
	:30	58	2											
	:45	60	2											
07-09-2002-b	01:00	84												
	:15	86	2											
	:30	88	2											
	:45	90	2											
	02:00	93	3											

	:15	96	3											
	:30	98	2											
	:45	100	2											
	03:00	104	4											
	:15	108	4											
	:30	110	2											
	:45	113	3											
	04:00	116	3											
	:15	120	4											
	:30	130	10	40	14	28	17	22.67	20	20	32	16	42	14
	:45	132	2											
	05:00	134	2											
	:15	136	2											
	:30	138	2											
	:45	139	1											
	06:00	140	1											
	:15	142	2											
	:30	145	3											
	:45	148	3											
	07:00	150	2											
07-09-2002-c	10:00	10												
	:15	11	1											
	:30	12	1											
	:45	14	2											
	11:00	16	2											
	:15	18	2											
	:30	20	2											
	:45	23	3											
	12:00	26	3											
	:15	27	1											

	:30	28	1											
	:45	29	1											
	13:00	30	1											
	:15	35	5											
	:30	40	5	20	10	20	12	16	15	15	24	12	32	10.67
	:45	42	2											
	14:00	45	3											
	:15	48	3											
	:30	50	2											
	:45	53	3											
	15:00	56	3											
	:15	58	2											
	:30	60	2											
	:45	62	2											
	16:00	64	2											
12-09-2002	13:15	10												
	:30	11	1											
	:45	12	1											
	14:00	13	1											
	:15	14	1											
	:30	15	1											
	:45	16	1											
	15:00	17	1											
	:15	18	1											
	:30	20	2											
	:45	22	2											
	16:00	25	3											
	:15	28	3	12	6	12	8	10.67	10	10	18	9	24	8
	:30	30	2											
	:45	31	1											
	17:00	32	1											

	:15	34	2									
	:30	37	3									
	:45	40	3									
	18:00	41	1									
	:15	42	1									
	:30	44	2									
	:45	46	2									
	19:00	48	2									
	:15	50	2									
	:30	51	1									
	:45	51	0									
	20:00	52	1									
17-06-2001	05:00	6										
	:15	10	4									
	:30	12	2									
	:45	14	2									
	06:00	20	6									
	:15	30	10	40	16	32	22	29.33	26	26	35	17.5
	:30	36	6									
	:45	40	4									
	07:00	41	1									
23-06-2001	:15	0										
	:30	20	20									
	:45	40	20	80	40	80	43	57.33				
	11:00	43	3									
16-07-2001	03:00	10										
	:15	20	10									
	:30	30	10									
	:45	40	10									
	04:00	50	10									
	:15	60	10									

	:30	76	16	64	26	52	36	48	46	46	72	36	81	27
	:45	80	4											
	05:00	82	2											
	:15	84	2											
	:30	86	2											
	:45	90	4											
	06:00	91	1											
13-08-2001	03:45	0												
	04:00	10	10											
	:15	14	4											
	:30	18	4											
	:45	20	2											
	05:00	21	1											
	:15	24	3											
	:30	27	3											
	:45	30	3											
	06:00	40	10	40	20	40	23	30.33	25	25	32	16		
	:15	50	10											
	:30	54	4											
12-08-2001	08:00	2												
	:15	20	18	72	28	56	34	11.33						
	:30	30	10											
	:45	36	6											
09-06-2000 a	09:15	10												
	:30	13	3											
	:45	17	4											
	10:00	20	3											
	:15	25	5											
	:30	30	5	20	10	20	13	17.33	17	17	27	13.5	34	11.33
	:45	33	3											

	11:00	36	3											
	:15	37	1											
	:30	39	2											
	:45	40	1											
	12:00	42	2											
	:15	44	2											
9/6/2000 b	23:00	14												
	:15	24	10											
	:30	36	12	48	22	44	25	33.33	28	28	35	17.5	43	14.33
	:45	39	3											
	00:00	42	3											
	:15	44	2											
	:30	45	1											
	:45	46	1											
	01:00	49	3											
	:15	51	2											
	:30	53	2											
	:45	56	3											
	02:00	57	1											
	:15	58	1											
14-07-2000	02:00	0												
	:15	20	20											
	:30	40	20	80	40	80	52	69.333	53	53				
	:45	52	12											
	03:00	53	1											
18-07-2000	02:00	5												
	:15	15	10											
	:30	35	20	80	30	60	40	53.33	50	50	56	28	62	20.67
	:45	45	10											
	03:00	55	10											
	:15	56	1											

22-08-2000	:30	58	2											
	:45	59	1											
	04:00	61	2											
	:15	63	2											
	:30	65	2											
	:45	66	1											
	05:00	67	1											
	:15	68	1											
	:30	69	1											
	:45	70	1											
	06:00	71	1											
	:15	72	1											
	:30	73	1											
	:45	74	1											
	07:00	75	1											
	:15	85	10											
	:30	87	2											
	:45	95	8											
	08:00	105	10											
	10:30	6												
	:45	16	10											
	11:00	36	20											
	:15	56	20	80	40	80	50	66.67	60	60	90	45	95	31.67
	:30	66	10											
	:45	76	10											
	12:00	86	10											
	:15	91	5											
	:30	96	5											
	:45	98	2											
	13:00	99	1											
	:15	100	1											

	:30	101	1											
	:45	103	2											
	14:00	105	2											
	:15	106	1											
23-07-1999	04:00	0												
	:15	4	4											
	:30	14	10	40	20	40	24	32	26	26	28	14		
	:45	24	10											
	05:00	26	2											
	:15	27	1											
	:30	27.5	0.5											
	:45	27.75	0.25											
	06:00	28	0.25											
	:15	33	5											
	:30	36	3											
12-08-1999	07:30	0												
	:45	18	18	72	34	68								
	08:00	34	16											
19-08-1999	10:00	2												
	:15	4	2											
	:30	10	6	24	11	22	16	21.33	18	18	20	10	23	7.67
	:45	15	5											
	11:00	20	5											
	:15	20.5	0.5											
	:30	21	0.5											
	:45	21.5	0.5											
	12:00	22	0.5											
	:15	23	1											
	:30	23.5	0.5											
	:45	24	0.5											
	01:00	25	1											

06-07-1998	01:30	0												
	:45	4	4											
	02:00	10	6											
	:15	30	20	80	40	80	46	61.33	50	50	66	33	80	26.67
	:30	50	20											
	:45	51	1											
	03:00	56	5											
	:15	60	4											
	:30	66	6											
	:45	67	1											
	04:00	67	0											
	:15	70	3											
	:30	80	10											
	:45	90	10											
03-08-1998	02:00	0												
	:15	20	20											
	:30	40	20	80	40	80	50	66.67	52	52	65	33.5		
	:45	50	10											
	03:00	52	2											
	:15	55	3											
	:30	57	2											
	:45	60	3											
	04:00	65	5											
02-06-1997	03:15	0												
	:30	2	2											
	:45	10	8											
	04:00	24	14	56	22	44	28	37.33	34	34	40	13.33		
	:15	30	6											
	:30	36	6											
	:45	37	1											
	05:00	37.5	0.5											

	:15	40	2.5											
24-07-1997	05:00	0												
	:15	10	10											
	:30	30	20	80	30	60	40	53.33	41	41				
	:45	40	10											
	06:00	41	1											
03-08-1997	14:00	32												
	:15	33	1											
	:30	36	3											
	:45	38	2											
	15:00	40	2											
	:15	50	10											
	:30	60	10	40	20	40	24	32	30	30	46	23	53	17.67
	:45	64	4											
	16:00	70	6											
	:15	76	6											
	:30	82	6											
	:45	83	1											
	17:00	85	2											

Appendix II Maximum depth and intensities for selective storm

Year	15 minute		30 minute		45 minute		60 minute		120 minute		180 minute	
	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity
2006	10	40	16	32	22	29.33	26	26	31	15.5	35	11.67
	13	52	23	46	33	44	43	43	51	25.5	55	18.33
	30	120	50	100	60	80	70	70	90	45	122	40.67
	10	40	20	40	28	37.33	30	30				
Selected storm	**30**	**120**	**50**	**100**	**60**	**80**	**70**	**70**	**90**	**45**	**122**	**40.67**

Year	15 minute		30 minute		45 minute		60 minute		120 minute		180 minute	
	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity
2005	20	80	34	68	43.5	58	46	46				
	12	44	20	40	24	32	27.5	27.5	40	20	46	15.33
	12	44	20	40	25	33.33						
	40	120	50	100	55	73.33						
	10	40	14	28	18	24	20	20	29	14.5	34	11.33
	18	72	22	44	22.5	30	23	23	25	12.5	31	10
	18	72	24	48	28	37.33	34	34	42	21	56	18.67
	8	32	16	32	24	32	28	28	40	20	52	17.33
Selected storm	**40**	**120**	**50**	**100**	**55**	**73.33**	**46**	**46**	**42**	**21**	**56**	**18.67**

Year	15 minute		30 minute		45 minute		60 minute		120 minute		180 minute	
	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity
2004	16	64	30	60	42	56	56	56				
	10	40	20	40	30	40	34	34				
	10	40	13	26	16	21.33	18	18	26.5	13.25	40	13.25
	20	80	30	60	40	53.33	50	50	72	36		
	20	80	30	60	42	56	56	56	72	36	40	13.25
Selected storm	**20**	**80**	**30**	**60**	**42**	**56**	**56**	**56**	**72**	**36**	**40**	**13.25**

Year	15 minute		30 minute		45 minute		60 minute		120 minute		180 minute	
	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity
2003	5	20	10	20	13	17.33	16	16	19	8.5	21	7
	20	80	40	80	46	61.33	52	52				
	26	104	36	72	46	61.33	49	49				
Selected storm	**26**	**104**	**40**	**80**	**46**	**61.33**	**52**	**52**	**19**	**8.5**	**21**	**7**

Year	15 minute		30 minute		45 minute		60 minute		120 minute		180 minute	
	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity
2002	16	64	20	40	30	40	40	40	56	28	68	22.67
	5	20	10	20	14	18.67	18	18	27	13.5	40	13.33
	10	40	14	28	17	22.67	20	20	32	16	42	14
	5	20	10	20	12	16	15	15	24	12	32	10.67
	3	12	6	12	8	10.67	10	10	18	9	24	8
Selected storm	**16**	**64**	**20**	**40**	**30**	**40**	**40**	**40**	**56**	**28**	**68**	**22.67**

Year	15 minute		30 minute		45 minute		60 minute		120 minute		180 minute	
	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity
2001	10	40	16	32	22	29.33	26	26	35	17.5		
	20	80	40	80	43	57.33						
	16	64	26	52	36	48	46	46	72	36	81	27
	10	40	20	40	23	30.33	25	25	32	16		
	18	72	28	56	34	11.33						
Selected storm	**20**	**80**	**40**	**80**	**43**	**57.33**	**46**	**46**	**72**	**36**	**81**	**27**

Year	15 minute		30 minute		45 minute		60 minute		120 minute		180 minute	
	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity
2000	5	20	10	20	13	17.33	17	17	27	13.5	34	11.33
	12	48	22	44	25	33.33	28	28	35	17.5	43	14.33
	20	80	40	80	52	69.333	53	53				
	20	80	30	60	40	53.33	50	50	56	28	62	20.67
	20	80	40	80	50	66.67	60	60	90	45	95	31.67
Selected storm	**20**	**80**	**40**	**80**	**52**	**69.333**	**60**	**60**	**90**	**45**	**95**	**31.67**

Year	15 minute		30 minute		45 minute		60 minute		120 minute		180 minute	
	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity
1999	10	40	20	40	24	32	26	26	28	14		
	18	72	34	68								
	6	24	11	22	16	21.33	18	18	20	10	23	7.67
Selected storm	**18**	**72**	**34**	**68**	**24**	**32**	**26**	**26**	**28**	**14**	**23**	**7.67**

Year	15 minute		30 minute		45 minute		60 minute		120 minute		180 minute	
	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity
1998	20	80	40	80	46	61.33	50	50	66	33	80	26.67
	20	80	40	80	50	66.67	52	52	65	33.5		
Selected storm	**20**	**80**	**40**	**80**	**50**	**66.67**	**52**	**52**	**66**	**33.5**	**80**	**26.67**

Year	15 minute		30 minute		45 minute		60 minute		120 minute		180 minute	
	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity	depth	intensity
1997	14	56	22	44	28	37.33	34	34	40	13.33		
	20	80	30	60	40	53.33	41	41				
	10	40	20	40	24	32	30	30	46	23	53	17.67
Selected storm	**20**	**80**	**30**	**60**	**40**	**53.33**	**41**	**41**	**46**	**23**	**53**	**17.67**

Appendix III

a / Output of SWMM output files (5 yrs return period 30 minute duration).

EPA STORM WATER MANAGEMENT MODEL - VERSION 5.0 (Build 5.0.005b)

Analysis Options
Flow Units .... CMS
Infiltration Method ...... GREEN_AMPT
Flow Routing Method ...... DYNWAVE
Starting Date MAY-28-2008 00:00:00
Ending Date MAY-28-2008 23:00:00
Report Time Step 00:05:00
Wet Time Step 00:30:00
Dry Time Step 01:00:00
Routing Time Step .... 300.00 sec

************************* Runoff Quantity Continuity *************************	Volume hectare-m	Depth mm
Total Precipitation	23.313	101.000
Evaporation Loss	0.000	0.000
Infiltration Loss	0.655	2.837
Surface Runoff	23.782	103.035
Final Surface Storage	0.010	0.045
Continuity Error (%)	-4.868	

************************* Flow Routing Continuity *************************	Volume hectare-m	Volume Mliters
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	23.786	237.862
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.000	0.000
External Outflow	13.382	133.825
Surface Flooding	10.461	104.615
Evaporation Loss	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	0.002	0.024
Continuity Error (%)	-0.253	

Subcatchment Runoff Summary

Subcatchment	Total Precip	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Runoff Coeff mm
S1	101.000	0.000	0.000	2.443	103.003	1.020
S2	101.000	0.000	0.000	2.973	102.891	1.019
S3	101.000	0.000	0.000	3.203	102.728	1.017
S4	101.000	0.000	0.000	2.065	104.427	1.034
S5	101.000	0.000	0.000	1.994	104.449	1.034
S6	101.000	0.000	0.000	1.439	105.060	1.040
S7	101.000	0.000	0.000	2.828	102.958	1.019
S8	101.000	0.000	0.000	4.122	101.114	1.001
S9	101.000	0.000	0.000	1.696	104.262	1.032
S10	101.000	0.000	0.000	2.718	102.989	1.020
S11	101.000	0.000	0.000	4.676	100.228	0.992
Totals	101.000	0.000	0.000	2.837	103.035	1.020

Node Depth Summary

Node	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days hr:min	Total Flooding mm/ha	Total Minutes Flooded
J1	0.20	0.67	269.96	0 02:00	0	0
J2	0.22	0.75	267.99	0 02:00	0	0
J3	0.61	1.50	264.65	0 01:14	10276.44	73
J4	0.61	1.50	262.85	0 01:24	185.40	43
J5	0.40	0.96	259.16	0 02:01	0	0
J6	0.22	0.72	261.52	0 02:00	0	0
J7	0.18	0.59	266.24	0 02:00	0	0
J8	0.29	1.06	267.79	0 02:00	0	0
J9	0.37	1.46	267.06	0 01:59	0	0
O1	0.39	0.96	254.96	0 02:01	0	0

Conduit Flow Summary

Conduit	Maximum Flow CMS	Time of Max Occurrence days hr:min	Maximum Velocity m/sec	Length Factor	Maximum /Design Flow	Total Minutes Surcharged
C1	3.88	0 02:00	2.73	1.00	0.45	0
C2	7.21	0 02:00	3.69	1.00	0.54	0
C3	13.16	0 02:28	5.48	1.00	1.04	79
C4	17.28	0 02:01	8.11	1.00	0.83	0
C5	5.42	0 02:00	3.22	1.00	0.38	0
C6	22.68	0 02:01	5.66	1.00	0.45	0
C7	4.41	0 02:00	2.11	1.00	0.29	0
C8	7.24	0 02:01	5.52	1.00	0.73	0
C9	11.83	0 02:00	4.55	1.00	0.99	9

Flow Classification Summary

Conduit	--- Fraction of Time in Flow Class ---- Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Avg. Froude Number	Avg. Flow Change
C1	0.02	0.00	0.00	0.38	0.60	0.00	0.00	0.75	0.0015
C2	0.02	0.00	0.00	0.82	0.16	0.00	0.00	0.49	0.0017
C3	0.02	0.00	0.00	0.26	0.72	0.00	0.00	1.24	0.0034
C4	0.02	0.00	0.00	0.28	0.70	0.00	0.00	1.26	0.0030
C5	0.02	0.00	0.00	0.82	0.16	0.00	0.00	0.58	0.0012
C6	0.02	0.00	0.00	0.27	0.71	0.00	0.00	1.42	0.0016
C7	0.02	0.00	0.00	0.98	0.00	0.00	0.00	0.34	0.0009
C8	0.02	0.00	0.00	0.75	0.23	0.00	0.00	0.70	0.0029
C9	0.02	0.00	0.00	0.78	0.21	0.00	0.00	0.60	0.0035

<u>Highest Continuity Errors</u>
Node J4 (-0.39%)
Node J2 (-0.23%)
Node J5 (-0.09%)
Node J9 (0.03%)
Node J3 (0.03%)

<u>Time-Step Critical Elements</u>
Link C3 (62.84%)
Link C4 (0.16%)

Routing Time Step Summary
****

Minimum Time Step : 24.40 sec
Average Time Step : 132.26 sec
Maximum Time Step : 300.00 sec
Percent in Steady State : 0.00
Average Iterations per Step : 2.08
Analysis begun on: Wed Jun 18 08:26:19 2008
Total elapsed time: < 1 sec

Appendix III
b / output of SWMM for annual 24 hrs data using IMD distribution

EPA STORM WATER MANAGEMENT MODEL - VERSION 5.0 (Build 5.0.005b)

Analysis Options
****
Flow Units .... CMS
Infiltration Method ...... GREEN_AMPT
Flow Routing Method ...... DYNWAVE
Starting Date MAY-28-2008 00:00:00
Ending Date MAY-28-2008 23:00:00
Report Time Step 00:05:00
Wet Time Step 00:30:00
Dry Time Step 01:00:00
Routing Time Step .... 300.00 sec

**************************	Volume	Depth
Runoff Quantity Continuity	hectare-m	mm
**************************	---------	-------
Total Precipitation	39.363	170.535
Evaporation Loss	0.000	0.000
Infiltration Loss	1.230	5.331
Surface Runoff	38.129	165.190
Final Surface Storage	0.701	3.035
Continuity Error (%)	-1.772	

**************************	Volume	Volume
Flow Routing Continuity	hectare-m	Mliters
**************************	---------	---------
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	37.948	379.485
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.000	0.000
External Outflow	31.857	318.572
Surface Flooding	6.002	60.016
Evaporation Loss	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	0.144	1.438
Continuity Error (%)	-0.143	

****
Subcatchment Runoff Summary
****

Subcatchment	Total Precip	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Runoff Coeff mm
S1	170.535	0.000	0.000	5.535	165.465	0.970
S2	170.535	0.000	0.000	5.535	164.918	0.967
S3	170.535	0.000	0.000	5.535	164.574	0.965
S4	170.535	0.000	0.000	4.151	166.975	0.979
S5	170.535	0.000	0.000	4.151	167.107	0.980
S6	170.535	0.000	0.000	3.459	168.301	0.987
S7	170.535	0.000	0.000	5.535	165.103	0.968
S8	170.535	0.000	0.000	6.919	162.444	0.953
S9	170.535	0.000	0.000	4.151	167.442	0.982
S10	170.535	0.000	0.000	5.535	165.225	0.969
S11	170.535	0.000	0.000	7.610	161.212	0.945
Totals	170.535	0.000	0.000	5.331	165.190	0.969

****
Node Depth Summary
****

Node	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days hr:min	Total Flooding mm/ha	Total Minutes Flooded
J1	0.13	0.52	269.81	0 02:00	0	0
J2	0.15	0.59	267.83	0 02:00	0	0
J3	0.52	1.50	264.65	0 01:19	6001.69	76
J4	0.55	1.49	262.84	0 01:59	0	0
J5	0.37	0.91	259.11	0 02:01	0	0
J6	0.15	0.55	261.35	0 02:00	0	0
J7	0.12	0.46	266.11	0 02:00	0	0
J8	0.19	0.80	267.53	0 02:00	0	0
J9	0.24	1.00	266.60	0 02:00	0	0
O1	0.36	0.91	254.91	0 02:02	0	0

****
Conduit Flow Summary
****

Conduit	Maximum Flow CMS	Time of Max Occurrence days hr:min	Maximum Velocity m/sec	Length Factor	Maximum /Design Flow	Total Minutes Surcharged
C1	2.74	0 02:00	2.47	1.00	0.32	0
C2	5.12	0 02:01	2.86	1.00	0.38	0
C3	13.11	0 02:32	5.46	1.00	1.04	96
C4	16.76	0 02:00	5.60	1.00	0.81	0
C5	3.77	0 02:00	2.57	1.00	0.26	0
C6	20.53	0 02:02	5.49	1.00	0.41	0
C7	3.08	0 02:00	1.87	1.00	0.20	0
C8	5.18	0 02:00	5.00	1.00	0.52	0
C9	8.46	0 02:00	3.68	1.00	0.71	0

****
Flow Classification Summary
****

Conduit	--- Fraction of Time in Flow Class ----							Avg.	Avg.
	Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Froude Number	Flow Change
C1	0.01	0.00	0.00	0.51	0.48	0.00	0.00	1.01	0.0003
C2	0.01	0.00	0.00	0.99	0.00	0.00	0.00	0.51	0.0004
C3	0.01	0.00	0.00	0.00	0.99	0.00	0.00	1.56	0.0011
C4	0.01	0.00	0.00	0.00	0.99	0.00	0.00	1.63	0.0009
C5	0.01	0.00	0.00	0.99	0.01	0.00	0.00	0.57	0.0003
C6	0.01	0.00	0.00	0.00	0.99	0.00	0.00	1.80	0.0004
C7	0.01	0.00	0.00	0.99	0.00	0.00	0.00	0.35	0.0002
C8	0.01	0.00	0.00	0.62	0.37	0.00	0.00	0.99	0.0006
C9	0.01	0.00	0.00	0.95	0.04	0.00	0.00	0.69	0.0008

Appendix III

c / output of SWMM for annual 24 hrs data using CWC distribution

EPA STORM WATER MANAGEMENT MODEL - VERSION 5.0 (Build 5.0.005b)

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****************
Analysis Options
****************
Flow Units ............... CMS
Infiltration Method ...... GREEN_AMPT
Flow Routing Method ...... DYNWAVE
Starting Date ............ MAY-28-2008 00:00:00
Ending Date .............. MAY-28-2008 23:00:00
Report Time Step ......... 00:05:00
Wet Time Step ............ 00:30:00
Dry Time Step ............ 01:00:00
Routing Time Step ........ 300.00 sec


**************************        Volume         Depth
Runoff Quantity Continuity     hectare-m            mm
**************************     ---------       -------
Total Precipitation ......        39.379       170.605
Evaporation Loss .........         0.000         0.000
Infiltration Loss ........         1.231         5.334
Surface Runoff ...........        37.445       162.226
Final Surface Storage ....         0.940         4.074
Continuity Error (%) .....        -0.603


**************************        Volume        Volume
Flow Routing Continuity        hectare-m       Mliters
**************************     ---------     ---------
Dry Weather Inflow .......         0.000         0.000
Wet Weather Inflow .......        37.127       371.275
Groundwater Inflow .......         0.000         0.000
RDII Inflow ..............         0.000         0.000
External Inflow ..........         0.000         0.000
External Outflow .........        36.944       369.441
Surface Flooding .........         0.000         0.000
Evaporation Loss .........         0.000         0.000
Initial Stored Volume ....         0.000         0.000
Final Stored Volume ......         0.224         2.244
Continuity Error (%) .....        -0.110
```

****
Subcatchment Runoff Summary
****

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Runoff Coeff
S1	170.605	0.000	0.000	5.539	163.097	0.956
S2	170.605	0.000	0.000	5.539	161.917	0.949
S3	170.605	0.000	0.000	5.539	161.351	0.946
S4	170.605	0.000	0.000	4.154	163.813	0.960
S5	170.605	0.000	0.000	4.154	164.046	0.962
S6	170.605	0.000	0.000	3.462	165.454	0.970
S7	170.605	0.000	0.000	5.539	162.258	0.951
S8	170.605	0.000	0.000	6.923	159.411	0.934
S9	170.605	0.000	0.000	4.154	164.958	0.967
S10	170.605	0.000	0.000	5.539	162.509	0.953
S11	170.605	0.000	0.000	7.616	158.187	0.927
Totals	170.605	0.000	0.000	5.334	162.226	0.951

****
Node Depth Summary
****

Node	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days hr:min	Total Flooding mm/ha	Total Minutes Flooded
J1	0.13	0.26	269.55	0 02:01	0	0
J2	0.14	0.29	267.53	0 02:02	0	0
J3	0.53	1.20	264.35	0 02:03	0	0
J4	0.59	1.31	262.66	0 02:04	0	0
J5	0.39	0.76	258.96	0 02:05	0	0
J6	0.14	0.27	261.07	0 02:00	0	0
J7	0.12	0.23	265.88	0 02:00	0	0
J8	0.18	0.39	267.12	0 02:00	0	0
J9	0.22	0.48	266.08	0 02:01	0	0
O1	0.39	0.76	254.76	0 02:06	0	0

****
Conduit Flow Summary
****

Conduit	Maximum Flow CMS	Time of Max Occurrence days hr:min	Maximum Velocity m/sec	Length Factor	Maximum /Design Flow	Total Minutes Surcharged
C1	0.98	0 02:01	2.73	1.00	0.11	0
C2	1.84	0 02:02	1.24	1.00	0.14	0
C3	12.20	0 02:04	5.09	1.00	0.97	0
C4	13.68	0 02:05	5.29	1.00	0.66	0
C5	1.28	0 02:01	3.93	1.00	0.09	0
C6	14.96	0 02:06	4.99	1.00	0.30	0
C7	1.06	0 02:00	0.89	1.00	0.07	0
C8	1.88	0 02:00	4.60	1.00	0.19	0
C9	3.08	0 02:01	1.84	1.00	0.26	0

****
Flow Classification Summary
****

	--- Fraction of Time in Flow Class ----							Avg.	Avg.
Conduit	Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Froude Number	Flow Change
C1	0.01	0.00	0.00	0.25	0.74	0.00	0.00	1.03	0.0001
C2	0.01	0.00	0.00	0.99	0.00	0.00	0.00	0.49	0.0001
C3	0.01	0.00	0.00	0.00	0.99	0.00	0.00	1.55	0.0009
C4	0.01	0.00	0.00	0.00	0.99	0.00	0.00	1.65	0.0006
C5	0.01	0.00	0.00	0.99	0.00	0.00	0.00	0.55	0.0001
C6	0.01	0.00	0.00	0.00	0.99	0.00	0.00	1.84	0.0003
C7	0.01	0.00	0.00	0.99	0.00	0.00	0.00	0.34	0.0001
C8	0.01	0.00	0.00	0.28	0.71	0.00	0.00	1.01	0.0002
C9	0.01	0.00	0.00	0.99	0.00	0.00	0.00	0.67	0.0002

Appendix IV Study of manning's "n"

The figure shows that with increase of Manning's "n" in impervious surfaces the generated surface runoff decreases significantly and vice versa. While for pervious there is no significant change in the Runoff volume. This is mainly due to contribution of impervious area to runoff generation.

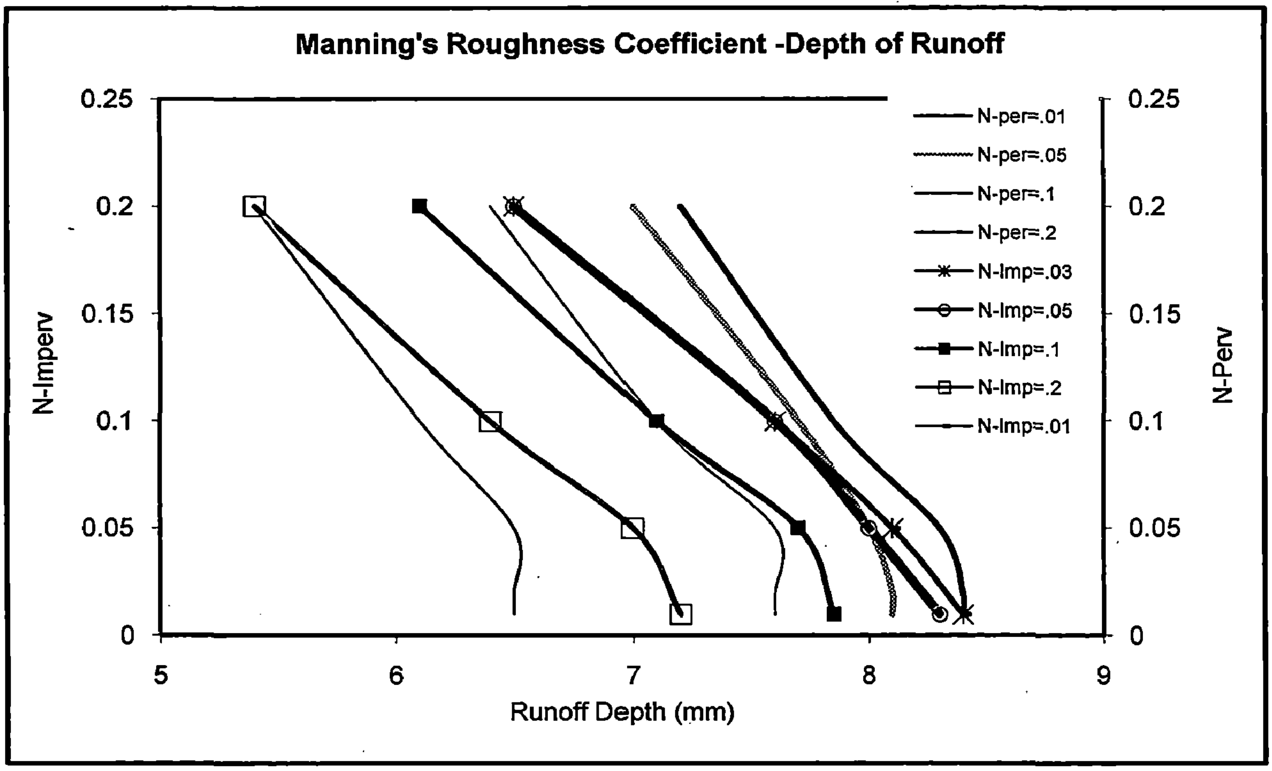

Figure shows the result of study for Manning's "n"

Table (4.13) shows the runoff for different values of Manning's Coefficient "n" for pervious and impervious surfaces in the simulation area.

"N" impervious	Runoff Depth (mm)					
	"N" pervious					
	0.01	0.02	0.03	0.05	0.1	0.2
0.01	8.4	8.4	8.4	8.3	7.85	7.2
0.05	8.1	8.1	8.1	8	7.7	7
0.1	7.6	7.6	7.6	7.6	7.1	6.4
0.2	6.5	6.5	6.5	6.5	6.1	5.4