HYDROLOGICAL ASPECTS OF JUNE 28, 2017 FLOOD AT IIT ROORKEE

A DISSERTATION

Submitted in partial fulfillment of the requirements for the award of the degree

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in

DISASTER MITIGATION AND MANAGEMENT

By

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CENTRE OF EXCELLENCE IN DISASTER MITIGATION AND MANAGEMENT

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CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the dissertation entitled **"HYDROLOGICAL ASPECTS OF JUNE 28, 2017 FLOOD AT IIT ROORKEE"** in partial fulfillment of the requirement for the award of degree of **MASTER OF TECHNOLOGY** in **DISASTER MITIGATION AND MANAGEMENT,** and submitted in the Centre of Excellence in Disaster Mitigation and Management of Indian Institute of Technology Roorkee, is a record of my own work carried out during a period from July 2017 up to May 2018 under the supervision of **Dr N.K.Goel**, Professor in the Department of Hydrology, Indian Institute of Technology Roorkee, Roorkee, India.

The matter embodied in this dissertation has not been submitted by me for the award of any other degree.

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ABSTRACT

The design of efficient storm water drains is a matter of great concern for urban planners and engineers. Efficient drainage is required for disposal of extreme flows as well as normal flows. The present study focuses to provide solution for urban flood issues in a small urban catchment. An unusual storm event, which occurred on June 28, 2017 at Indian Institute of Technology Roorkee campus, and the entire Roorkee area, caused heavy damages to the property in the campus and a very large area in the campus and the city got flooded. This storm has been critically analyzed in terms of its severity using frequency analysis approach. Using spatial data sets available in public domain and surveyed map, the capacity of existing drainage system has been assessed using rational formula and Storm Water Management Model.

For this purpose the following steps were developed and the work was carried out.

a. Demarcation of the campus boundary on the map and demarcating the existing drains on the same map.
b. Demarcation of all the external drains having inlet inside the campus and computing their catchment area at various inlet points.
c. Development of IDF curves for IIT Roorkee.
d. Computation of time of concentration at each catchment.
e. Computation of peak discharges at various points of the stormwater drain.
f. Adequacy of the existing drains of the campus.

After the assessment of the efficiency of existing stormwater drainage system of IIT Roorkee campus, recommendations are made so that the runoff generated due to high intensity rainfall could be controlled in future.

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LIST OF ABBREVIATIONS

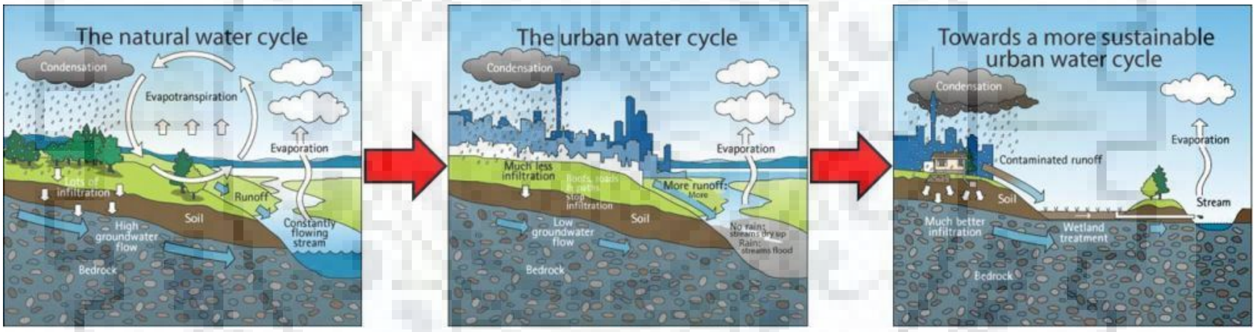
ASTER	Advanced Spaceborne Thermal Emission and Reflection Radiometer
BMP	Best Management Practices
CAD	Computer Aided Design
C_k	Coefficient of kurtosis
CoEDMM	Centre of Excellence in Disaster Mitigation and Management
Cs	Coefficient of skewness
C_v	Coefficient of variation
DEM	Digital Elevation Model
GLO	Generalized Logistic Distribution
GIS	Geographic Information System
IDF curves	Intensity Duration Frequency curves
IITR	Indian Institute of Technology Roorkee
INR	Indian Rupee
K-S Test	Kolmogorov- Smirnov Test
LID	Low Impact Development
LU/LC	Land use/ Land cover
MOM	Method of Moments
NDMA	National Disaster Management Authority
NIH	National Institute of Hydrology
NRCS	Natural Resource Conservation Service
PT III	Pearson Type III
LPT III	Log Pearson Type III
PWM	Probability Weighted Moments
SCS-CN	Soil Conservation Service- Curve Number
SRTM	Shuttle Radar Topography Mission
SWMM	Storm Water Management Model
TR-55	Technical Release 55
UNDP	United Nations Development Programme
USGS	United States Geological Survey
USEPA	United States Environmental Protection Agency
WS	Watershed

CHAPTER 1
INTRODUCTION

1.1 URBAN FLOODS

Flood is an overflow of a large amount of water beyond its normal limits, especially over what is normally dry land (Chow, 1956). Flooding has been a recurrent phenomenon in India from time immemorial. Each year floods of varying magnitude affect some parts of the country or the other. Due to increased urbanization, there has been increasing trend of floods especially in urban settlements commonly termed as urban floods.

Urban flood is the inundation of land or property in a built environment, particularly in more densely populated area, caused by the rainfall which exceeds the capacity of drainage systems, such as storm sewers.



Source: Auckland City Council (2010)

Figure 1.1 Water movement in different environments

1.2 CAUSES OF URBAN FLOOD

Urban planners and engineers from all over the world are facing the challenge to reduce the risk in cities due to urban floods and trying to develop possible solutions for more sustainable environment. Urban floods can be caused due to natural factors or anthropogenic factors. Natural factors that can result in urban flooding are **(i) heavy rainfall, (ii) silting of channels, and (iii) lack of natural water bodies** whereas anthropogenic factors which cause urban flood are **(i) population pressure, (ii) deforestation, (iii) Un-authorized colonies, (iv) urbanization, (v) poor water and sewage management, (vi) lack of attention to the natural hydrological systems, and (vii) lack of flood control measures.**

Due to the increasing frequency of water- related disasters that occur due to heavy rainstorm event in urban areas, urban flooding has become an important issue. The risk of urban flood increases due to the growing trend of urbanization (Duan et al., 2016). As per the report by NDMA, most of the Indian cities are facing flooding at frequent intervals. Total geographical area stated to be flood prone is 48 million ha out of 329 million ha, which makes India highly vulnerable to floods. On an average, India loses 1600 human lives and 18.05 billion INR each year due to various losses, such as damage to agriculture, houses and other public utilities, caused by the flood (National Disaster Management Authority, 2008).

Globally, more people reside in urban areas than in rural areas, with 54 percent of the world's population residing in urban areas in 2014. In 1950, 30 percent of the world's population was urban, and by 2050, 66 percent of the world's population is projected to be urban. (United Nations, Department of Economic and Social Affairs, 2014). By 2021 the population of India is expected to increase by 600 million; about 55% of the population will be living in metro cities, spread among more than 100 metro cities (UNDP, 2004).

During the past few years, many researchers have worked on the issue of urban flooding; some of these are (Gupta, 2007; Liu & Cheng, 2014; Ranger et al., 2011; Tingsanchali, 2012).Various Indian cities have been severely affected due to increasing trend of urban flood disasters. The most notable among them are Hyderabad, Ahmadabad, Delhi, Chennai, Kolkata, Mumbai etc. The detailed list of affected cities is given in Table 1.1 (National Disaster Management Guidelines, 2010).

After the Mumbai floods (2005), it was decided that urban flooding would be considered as a separate disaster, delinking it from floods, as the causes of urban floods and the strategies to deal with them are also totally different. After Mumbai floods a separate guideline on urban flood management was formulated by National Disaster Management Authority in 2010.

In most of the cases of urban floods, the major cause is encroachment as they lead to decrease in the capacity of natural drains. It is mainly due to improper solid waste disposal, debris in the drains, silt deposition etc. which reduces their capacities to carry runoff during heavy rainstorm.

As per the report by National Disaster Management Authority, the list of cities in India which got affected due to urban flood in the past few years is given in the Table 1.1.

Table 1.1 Significant Urban flood events in India since 2000

Year	Event
2000	Mumbai, Chennai, Bangalore, Kolkata, Hyderabad
2001	Ahmadabad, Bhubaneswar, Thane, Mumbai
2002	Delhi
2003	Delhi, Ahmadabad, Vadodara
2004	Chennai
2005	About 10 cities; Mumbai was the worst affected
2006	Number of affected cities rose to 22. Surat was the worst affected. Vishakhapatnam airport was inundated for more than 10 days., **Roorkee**
2007	Number of affected cities rose to 35. Kolkata was the worst affected.
2008	Jamshedpur, Mumbai, Hyderabad were worst affected.
2009	Delhi, Mumbai
2010	Delhi, Guwahati, Ahmadabad, Leh, Mumbai, **Roorkee**
2011	Guwahati
2013	Surat, Kolkata, Delhi, Bangalore, **Roorkee**
2014	Srinagar
2015	Srinagar, Mumbai, Chennai, **Roorkee**
2016	Delhi
2017	Mumbai, Ahmadabad, **Roorkee**

Urban flood is due to increasing trend of urbanization, as it leads to increase in flood peaks from 1.8 to 8 times and flood volumes up to 6 times (National Disaster Management Guidelines, 2010). Haphazard urbanization due to increasing population and their demand, encroachment, and under designing of the stormwater drainage network system, there is increase in runoff, and reduction in time of concentration which cause urban floods (Goonetilleke et al., 2005; Teemusk & Mander, 2011; Tingsanchali, 2012).

Due to increased capacity of flow through drains and impervious surfaces, there is manifold increase in the maximum flow and its frequency. Therefore for an efficient management of urban flood risk, there is a need for storm water drainage system which is capable of routing storm waters to moving waters effectively.

Impervious surfaces should be connected to these drains so that the stormwater can be transported easily to natural drains. This can be achieved through the design of efficient drainage system by calculating runoff generated in an area over a period of time during an event (Tucci, 2008). (Burns et al., 2012) focused on the protection of natural hydrologic processes by restoring the pre-development hydrograph of a watershed in an urban catchment. But due to limitation of land it cannot be achieved to a great extent, yet a cushioned hydrograph peak can reduce the flood risk at downstream areas to some extent.

Due to several factors such as migration of population from rural to urban areas in search of better employment opportunities and better living conditions, encroachment of city drains, change in climatic pattern etc., the need and the requirement to take an action in these disaster related aspects have become an important issue.

These issues may not cause sudden disaster but over time their effects would be immense and destroy the whole natural ecosystem of our environment.

Urban development increases the local hydrological and hydrometeorological conditions of the city and on the other hand increased population concentration increases the vulnerability.

In urbanized areas, huge amounts of impervious surface increment results in increase in runoff coefficient, hence runoff increases eventually.

1.3 NEED FOR THE STUDY

In recent years, the campus of Indian Institute of Technology Roorkee is facing drainage congestion with the stagnation of rainwater at different locations causing inconvenience to the community.

Improper stormwater management and change in land use are obstructing the natural infiltration capacity of the soil. Increased intensities and frequencies of rainfall due to micro-climate change, improper drainage system design, improper futuristic structural planning for peak flood alteration, lack of groundwater recharge facilities such as rain gardens, and lack of sufficient avenues for wastewater disposal and its reuse leads to more frequent urban floods.

Due to the development of infrastructures like hostels, lecture complexes etc. impervious area is increased in recent years, which is resulting in higher runoff creating flooding problems in the low lying areas of the campus. Hence, there is a need to assess, monitor, and predict the rainfall-runoff characteristics of IIT Roorkee campus to estimate the design storm and design flood under the current trend of land use/land cover change.

Roorkee faced flooding situation during the years of 2006, 2010, 2015 and more recently on June 28, 2017 when 236.6 mm of rainfall occurred in 4 hrs. The impact of flooding on various zones of the campus was an alarm for rethinking in the direction of analyzing the drainage adequacy of the campus. This rainfall event caused huge damage to property of the campus and affected the normal movement of traffic within the campus. Figure 1.2 shows the flooded hostels and affected areas of the campus.

Figure 1.2 Affected areas of IIT Roorkee on June 28, 2017

1.4 AIM AND OBJECTIVES

The study aims at successfully computing the drainage capacity of existing stormwater drain and its efficiency to handle the runoff during high intensity rainfall.

The specific objectives of the study are as follows:

a. To analyze trends of extreme rainfall at IIT Roorkee campus;
b. Critical analysis of June 28, 2017 rainstorm;
c. To simulate and analyze the flooding in IIT Roorkee campus due to the recent extreme rainfall event of June 28, 2017; and

d. To assess the adequacy of existing drainage system of IIT Roorkee campus to handle flows resulting from design storms using codal practices based on the rational formula and verifying it using Storm Water Management Model.

1.5 CHAPTERIZATION

This study is an important step towards the development of plans and management of urban flooding of the area. Since more of infrastructures are coming up in the campus, therefore, it is necessary to have an efficient runoff management system. The organization of chapters is briefly described below:

Chapter 1 provides an overview about the flood, causes of floods, aim and objectives of the study, and brief about the recent urban floods experienced by Indian cities.

Chapter 2 provides the review of the study carried out by various researchers and suitable model to deal with stormwater management issues.

Chapter 3 discusses in brief about the study area and data used for further study process.

Chapter 4 is about an overview of the method to conduct stormwater management analysis. It explains the steps involved while conducting the storm water management which could be applied to any small urban subcatchment in general.

Chapter 5 provides the final results and conclusions related to the study. It also focuses on steps of how the analysis was done using empirical formulae as well as through software.

Chapter 6 provides conclusions derived from the analysis and recommendations for future work.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

This chapter presents a brief review of works done on analysis of Roorkee rainfall data and application of Storm Water Management Model (SWMM) in urban flood modelling. The major focus is to understand the possible solutions that could be adopted to mitigate the affect of urban floods in an area.

2.1.1 Studies related to Roorkee

Mathur & Goel, 1989 analyzed the unusualness of the storm that occurred in July 1988 at Roorkee. They compared it with the most severe storms of the same station during 1979-1982 to determine that whether 1988 storm was an outlier or not. They suggested that there is a need to develop a methodology for frequency analysis of extreme precipitation series in presence of outliers.

Kartika, 2006 analyzed the urban flood drainage planning of IIT Roorkee campus and used Storm Water Management Model to determine the runoff during heavy storms of various durations. It was concluded that drainage system of the campus is well designed for rainfall of 50 years return period.

Bhushan, 2012 studied the drainage issues for Adarsh Nagar area of Roorkee and assessed runoff using SWMM. It was concluded that the drainage system is unable to handle the runoff generated from rainfall having 25 years return period. The drains are under designed.

2.1.2 Studies based on application of Storm Water Management Model (SWMM)

These studies discussed below analyse the urban flood issues. Most of the studies make use of Storm Water Management Model to handle urban floods. Studies have been selected so as to cover some Indian case studies, and some studies of foreign countries have also been analyzed based on the most cited papers in this particular type of research.

Tsihrintzis & Hamid, 1998 used SWMM for the calibration and verification during relatively short duration, single event in South Florida. The study attempts to provide the modelers with the way to select appropriate input parameters to be used in planning studies and test the applicability of SWMM model on small urban catchments. They finally concluded that the model is very effective and can be used as a perfect decision-making tool and can be used to analyze existing best management practices (BMPs) and to improve and/or update the systems for more effective performance.

Hsu et al., 2000 used SWMM for the analysis of inundation on urban areas due to an overflow of storm sewers. The simulated results can be applied to establish flood-mitigation measures. Due to urbanization, the overland flow is affected due to various anthropogenic factors such as land use and storm drainage system. They adopted SWMM for computing the water flow in storm sewer systems and hydrographs at manholes during the inundation in Taipei. The analysis may help the concerned authorities to prevent flood damages by redesigning and enlarging the stormwater sewer capacities in the inundated zones.

Karamouz et al., 2010 proposed best management practices that should be adopted while dealing with urban floods that consider anthropogenic and climate change effects. The case study deals with the area of Tehran metropolitan (Iran). They emphasized on how climate change alters the local weather characteristics by high variations in rainfall, temperature and runoff result. The reliability of urban drainage system can be increased by considering climate change impact at micro scale. They concluded that climate change resulted in increased rainfall of high intensity which led to the review of existing drainage systems.

Suriya & Mudgal, 2012 studied the impact of urbanization in the southern part of Chennai, a coastal city located in the North Eastern corner of Tamil Nadu State, India. Decrease in land resources for agriculture due to increased human settlements, industrial growth and infrastructure development is leading to congestion of stormwater drainage during monsoon season. They concluded that intensive development in flood-prone areas leads to increase in both volume and runoff.

Huong & Pathirana, 2013 studied the impact of climate change and urban development in increasing flood risk in the city of Can Tho city, Vietnam. Changes in hydrological and hydrometeorological conditions increase flood hazard, as well as to urban concentrations increase the vulnerability to any disaster. The study tries to integrate the effect of internal as well as external factors that may act as the drivers of the flood in future. The major analysis of this study is that they considered both, the internal pressures on the urban system as well as the external ones. Particularly the consideration of the impact of urban development on urban rainfall intensities is an important feature.

Swathi et al., 2013 explored the SWMM model for the BITS Pilani campus, Hyderabad, India. They divided the entire catchment into 18 sub-catchments. They analyzed that SWMM is very well suited for urban catchments and the campus storm network system has been well planned and has sufficient carrying capacity to cater the simulated rainfall event of the year 2006.

Weilin et al., 2014 studied the serious urban flooding problem due to rainstorm water logging in Guancheng district, Dongguan city. The main reason of flooding came out to be the small diameter of the pipes at the nodes. Finally, they analyzed that one possible solution may be to increase the pervious surface near the bottleneck sections which will improve the rainwater infiltration capacity of the area. They also compared the simulated result with actual flooding scenario which proved the reliability of the SWMM model. Therefore SWMM can be used by the management departments for solving the rainwater storm of the city and ultimately reducing the losses caused by the rainstorm to the area.

Zope et al., 2016 studied the impact of land use land cover and urbanization on floods in Mumbai. Change in land use led to the great impact on runoff in the area. They concluded that lower return periods led to a maximum change in peak discharge/volume of runoff compared to higher return periods for change in land use conditions which means that frequency of runoff increased.

Zhu & Chen, 2017 studied the effect of Low Impact Development practices on flood reduction in a typical residential area of Guangzhou, China under different rainfall scenarios. SWMM model was used to analyze the flooding hotspots and their comprehensive characteristics. SWMM helps to get a clear idea about the performance of various low impact development practices feasible in an area. Therefore a rainfall- runoff model is constructed based on SWMM model since it is a very useful model for guiding the formulation of appropriate measures. They concluded that although low impact development practices are very effective in rainfall of short duration, lower intensities but may tend to be less effective in case of increased rainfall duration, and intensity. Therefore they are not the effective solutions for high intensity rainfall areas.

Yu et al., 2017 used SWMM model to simulate flood scenario which is driven by designed rainfall either from existing intensity-duration-frequency curves or future ones subjected to climate change conditions in a small typical urban catchment of Singapore. For the drainage design SWMM was used for the identification of the best conduit sizes. SWMM is basically used for the simulation of urban drainage flows. It consists of the rainfall-runoff module and routing module for calculating the runoff through the urban drainage system. The area was divided into 24 sub-catchments and routing was done through 24 conduits. They found that analysis after considering the future climate change scenarios (since climate change may affect the rainfall variability and ultimately the drainage design) may lead to providing better results which may lead to better decision-making strategies.

A document on Urban Hydrology for Small Watersheds, TR-55 (USDA, 1986) provides the process to calculate stormwater runoff volume using SCS-CN method. Runoff calculation is important to estimate the return period of the storm which ultimately helps to understand its unusualness.

Since the study make use of DEM to delineate the watershed. Selection of the DEM becomes an important task. Baral et al., 2016 studied the comparison of Cartosat, ASTER and SRTM DEMs of different terrain and concluded that SRTM DEM is best suited for all kinds of terrain as the mean and standard deviation of various elevation points on SRTM DEM are more close to topographical maps as compared to Cartosat and ASTER DEMs, although all three have same spatial resolution of 30 mts.

2.2 CONCLUSIONS

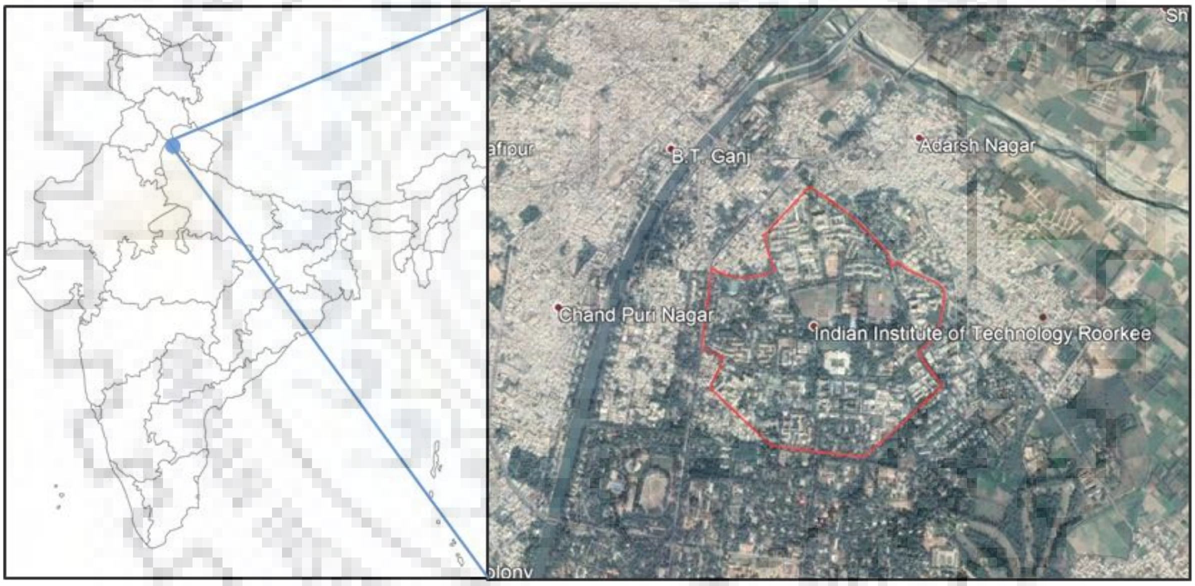
The impact of urbanization leads to imperviousness and ultimately runoff is generated in the area. Use of low impact development practices can be helpful to a great extent. Also delineation of the catchment area is an important aspect since it determines the actual flow through the entire area. This catchment delineation can be further processed in models such as Storm Water Management Model. The efficiency of Storm Water Management Model has been verified in various studies by the researchers, and its suitability for small as well as large catchment areas has been well established.

CHAPTER 3

STUDY AREA AND DATA USED

3.1 LOCALE

Roorkee town (latitude 29°50'00''N to 29°55'00''N and longitude 77°50'00''E to 77°55'00''E) is situated close to the foothills of Himalayas on the right bank of river Solani, which is a tributary of river Ganga (Figure 3.1). It is situated at a distance of 172 km north-east of the Indian capital, Delhi. The upper Ganga canal flows through the center of the city dividing it into old Roorkee towards the west and IIT Roorkee and cantonment area towards the east. The average elevation of the city is 268 meters (879 feet) above mean sea level.



Source: https://www.google.co.in

Figure 3.1 Location map of the study area

3.2 CLIMATE

The area experiences moderate sub-tropical to humid climate with three distinct seasons viz. summer followed by rainy and winter seasons. The summers start from late March and go till mid-June with minimum and maximum temperature values on an average 20°C and 45°C respectively (Singh et al., 2016). The area receives an annual rainfall of about 1200 mm. Majority of the rainfall occurs during the period June 15 to September 30.

3.3 ABOUT INDIAN INSTITUTE OF TECHNOLOGY ROORKEE

The main campus of IIT Roorkee has an area of 365 acres (1,480,000 sq. mts.). It is situated on National Highways 58 and 73. Nestled within this, are several heritage buildings, modern academic departments, hostels, hospital, schools, banks, community centers, several activity centers, residences, apartments, and other buildings.

3.3.1 Drainage system of the campus

The drainage network in IIT Roorkee campus for stormwater is mostly an open drainage system. Majority of the drains of the campus are concrete lined and have culverts of different dimensions. In general, the water in the campus moves towards eastern direction as the natural slope of the campus is towards Khanjarpur which ultimately connects to Solani River. Channels of smaller dimensions carry runoff from the residential houses, other academic buildings and open grounds to the main stormwater drain which is covered and carries all the storm flow from the catchment to Solani River (Figure 3.2). In the absence of adequate carrying capacity, even during short duration medium intensity rainfall people often face water stagnation and flooding problems at various locations of the campus.

Over the years there is a drastic change in impervious areas of IIT Roorkee campus and adjoining settlements has been observed, as shown in Google image of the year 2002 and 2015 (Figure 3.3).

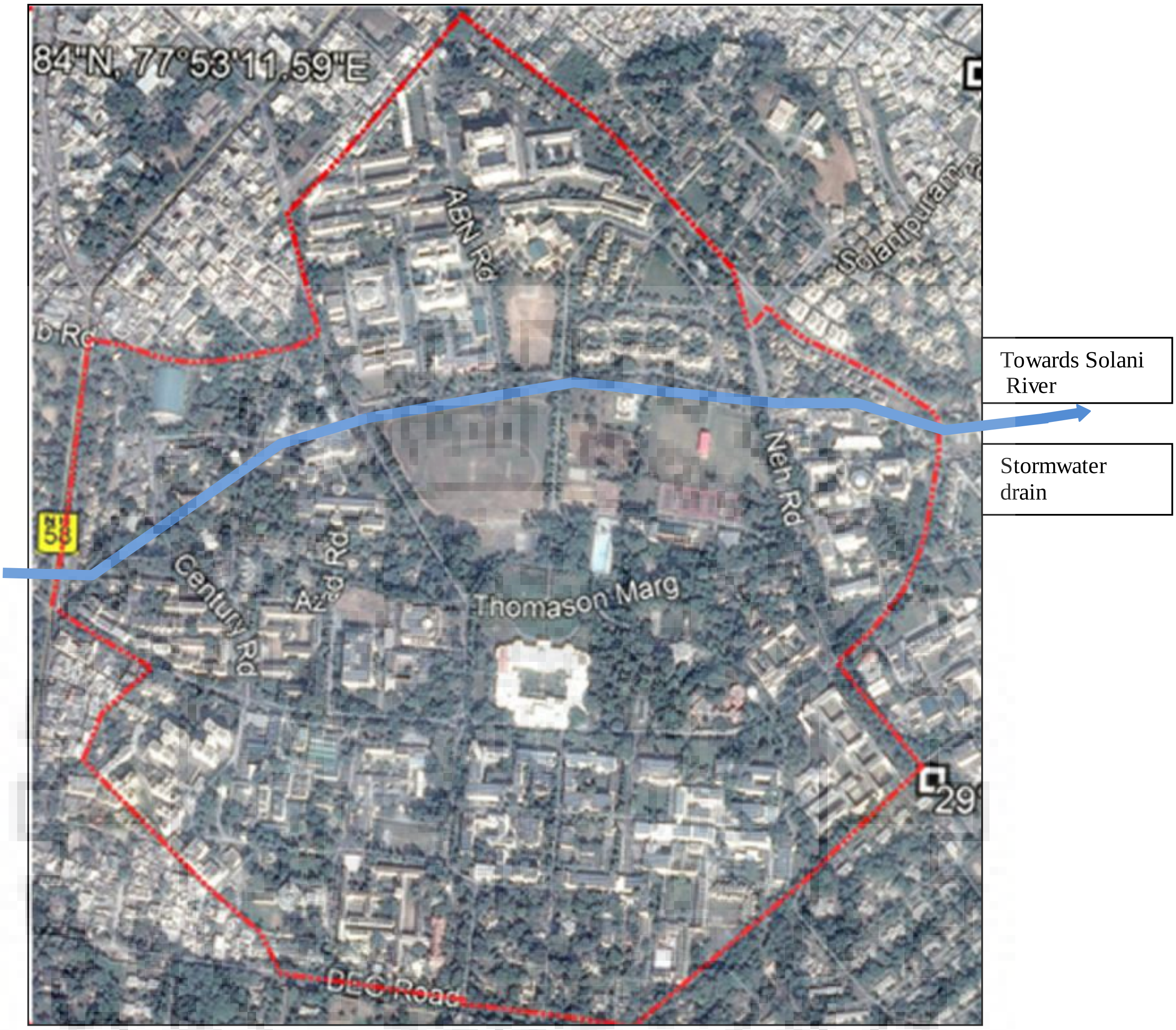


Figure 3.2 Location of stormwater drain in the campus

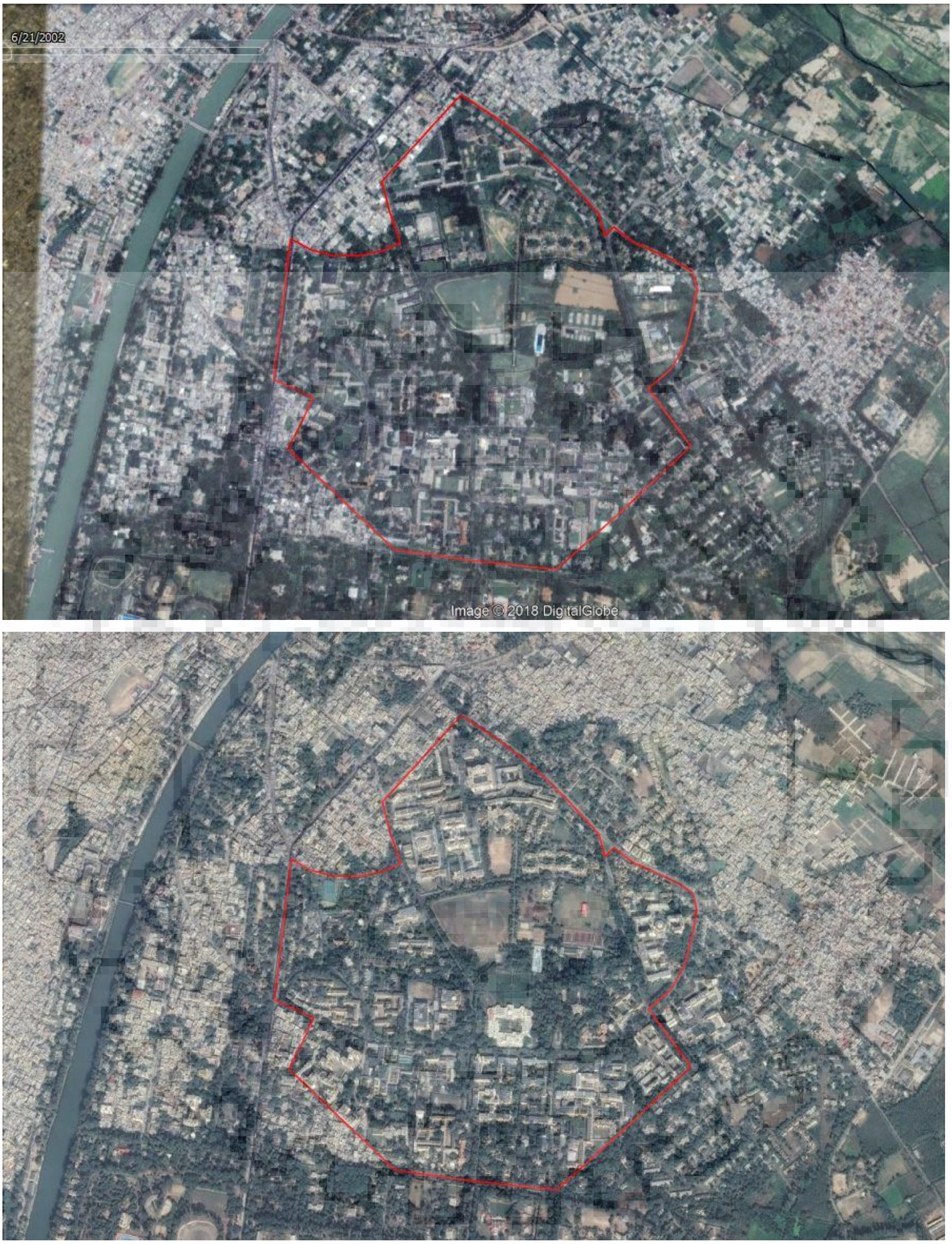


Figure 3.3 Land use change over years (2002 and 2015)

3.4 DATA USED

For the analysis rainfall data surveyed map of the campus and the DEM data have been used as follows:

3.4.1 Rainfall data

a. One hundred sixteen years of annual maximum rainfall data from 1901 to 2016 (Appendix A) for frequency analysis was obtained from Department of Hydrology, IIT Roorkee; and
b. 24 hr Self Recording Rain Gauge (tipping bucket type) data of June 28, 2017 was obtained from Department of Hydrology, IIT Roorkee (Appendix E).

3.4.2 Maps of the campus

A computer-aided design (CAD) drawing map of the campus showing elevation contour lines (at 20 cm interval) roads, buildings, drains along with their sections was obtained from construction department of IIT Roorkee.

3.4.3 DEM data

Digital Elevation Model was obtained from USGS website has been used to delineate the watershed. For the analysis, SRTM-DEM data having 30 mts. resolution is used.
(Accessed date: 17/11/2017).

CHAPTER 4
METHODOLOGY

4.1 GENERAL

The present chapter deals with the steps involved with the storm water analysis of IIT Roorkee campus for the rainfall event of June 28, 2017. The steps of methodology are given below.

i. Collection of site specific data;
ii. Development of hyetograph, Intensity duration and Depth duration curve of June 28, 2017;
iii. Frequency analysis of 116 years data of annual maximum daily rainfall data of Roorkee;
iv. Development of Intensity Duration Frequency curves of Roorkee;
v. Determination of return period at different durations of June 28, 2017;
vi. Computation of catchment areas at different inlet points of the campus;
vii. Computation of contributing areas at various points of the main drain of the campus;
viii. Computation of the peak discharges using rational formula for June 28, 2017 event;
ix. Computation of the existing capacity of drains at various points using Manning's formula;
x. Setting up of Storm Water Management Model (SWMM);
xi. Computation of capacities using SWMM;
xii. Comparison of the drain capacities using rational formula and SWMM model;
xiii. Computation of duration of flooding at various points due to inadequate capacities of the drain and other consequences; and
xiv. Design capacities of the main drain of the campus for a return period of 200 years.

The details of above steps are explained in the following sections.

4.2 COLLECTION OF SITE SPECIFIC DATA

For primary data collection, a reconnaissance survey was done during and after the rainstorm of June 28, 2017. During the survey following analysis was done.

a. Inside and outside survey of the drains that carry water to the main drain of IIT Roorkee campus during the storm;
b. Maintenance issues related to the drains;
c. Identifying the most vulnerable zones during the storm inside IIT campus; and
d. Losses occurred during and after the rainfall event.

After the complete reconnaissance survey of the site, data from secondary sources were collected and integrated with primary data to end up at some effective results. For this, the data from USGS and Google earth were used. Detailed surveyed data of the study area was obtained from the construction department of IIT Roorkee.

This data was further processed using software such as ArcGIS, AutoCAD, Google Earth and Sketchup to obtain:

a. Slope map, DEM, LULC map;
b. Drainage pattern and link inside and outside of the campus;
c. Delineated watershed that is contributing water inside the campus; and
d. 3D view of the campus with demarcation of flooded zones.

4.3 DEVELOPMENT OF HYETOGRAPH, DEPTH DURATION AND INTENSITY DURATION CURVE OF JUNE 28, 2017

Incremental rainfall is computed using graph obtained from self recording rain gauge from Department of Hydrology. The cumulative rainfall is converted to incremental rainfall and intensity duration graph, i.e. hyetograph is made. From the data maximum intensity duration and maximum depth duration curves are made.

4.4 FREQUENCY ANALYSIS OF 116 YEARS DATA OF ANNUAL MAXIMUM DAILY RAINFALL DATA OF ROORKEE

Past 116 years annual maximum rainfall data was used for the frequency analysis, the steps are explained below.

- Computations of statistical parameters i.e. mean, standard deviation, C_s, C_k;
- Check for randomness using turning point test and Anderson correlogram test;
- Check for stationarity using Kendall's rank correlation test;
- Computation of return periods using MOM and PWM techniques;
- Check of Goodness of fit tests for various distributions using Chi-square test, K-S test and D-index test;
- L-moment ratio diagram is generated using various parameters; and
- Estimation of rainfall quantiles of various return periods.

4.4.1 Computation of statistical parameters

From the annual maximum precipitation data of n years, mean, standard deviation (σ), coefficient of variance (C_v), coefficient of skewness (C_s), and coefficient of kurtosis (C_k) are computed using the 4.1, 4.2, 4.3, 4.4 and 4.5 respectively.

$$\mu = \frac{\sum_{i=1}^{n} X_i}{n} \qquad \ldots(4.1)$$

$$\sigma = \sqrt{\frac{\sum_{i=1}^{n}(X_i - \overline{X})^2}{n-1}} \qquad \ldots(4.2)$$

$$C_v(\sigma^2) = \frac{1}{(n-1)}\left[\sum_{i=1}^{n}(X_i - \overline{X})^2\right] \qquad \ldots(4.3)$$

$$C_s = \frac{n}{(n-1)(n-2)}\left[\frac{\sum_{i=1}^{n}\left(X_i - \overline{X}\right)^3}{\sigma^3}\right] \qquad \ldots(4.4)$$

$$C_k = \frac{n^2}{(n-1)(n-2)(n-3)}\left[\frac{\sum_{i=1}^{n}(X_i - \overline{X})^4}{\sigma^4}\right] \qquad \ldots(4.5)$$

Where X_i is the i^{th} variate and n is the total number of observations.

4.4.2 Tests carried out for frequency analysis

4.4.2.1 Turning point test

It is done to check the randomness of the series. In any sequence X_t, t=1, 2…n, a turning point 'p' occurs at t = i, if, X_i is greater than X_{i-1} or X_{i+1}

$$E(p) = \frac{2(n-2)}{3} \qquad \ldots(4.6a)$$

$$V(p) = \frac{16n - 29}{90} \quad \text{…(4.6b)}$$

$$Z = \frac{p - E(p)}{\sqrt{V(p)}} \quad \text{…(4.6c)}$$

Here, n is the number of observations.

4.4.2.2 Anderson correlogram test

Assuming n pairs of observations on two variables x and y. The correlation coefficient between x and y is given by

$$r = \frac{\sum(x_i - \bar{x})(y_i - \bar{y})}{\left[\sqrt{\sum(x_i - \bar{x})^2}\right]\left[\sqrt{\sum(y_i - \bar{y})^2}\right]} \quad \text{…(4.7)}$$

Where, the summations are over the n observations.

4.4.2.3 Kendall's rank correlation test

The test commonly known as 'τ' test is basically used for trend identification. For a sequence X_1, X_2…..X_n, determine the number of times say 'p', in all pairs of observations (X_i, X_j, j>1) i.e. $X_j>X_i$. Maximum number of such pairs occurs for an increasing sequence. This is rising trend where succeeding values are greater than preceding ones and p is given by (n-1) + (n-2) + … +1= n (n-1)/2.

$$\tau = \frac{4p}{n(n-1)} - 1 \quad \text{…(4.8a)}$$

$$E(\tau) = 0 \quad \text{…(4.8b)}$$

$$V(\tau) = \frac{2(2n+5)}{9n(n-1)} \quad \text{…(4.8c)}$$

$$Z = \frac{\tau - E(\tau)}{\sqrt{V(\tau)}} \quad \text{…(4.8d)}$$

4.4.3 Method to compute Return periods

4.4.3.1 Method of Moment (MOM)

Normal distribution

T year flood X_T is given by

$$X_T = \overline{X} + K_T * S_X \qquad \ldots(4.9)$$

Where,

$\overline{X}$ = sample mean

S_x = sample standard deviation

K_T = frequency factor, $C_s = 0$

Log Normal distribution

T year flood X_T is given by

$$X_T = exp(\overline{Y} + K_T * S_X) \qquad \ldots(4.10)$$

Where,

$\overline{Y}$ = mean of log (base e) transformed series

S_x = standard deviation of log transformed series

K_T = frequency factor, $C_s = 0$

Pearson Type III distribution

T year flood X_T is given by

$$X_T = \overline{X} + K_T * S_X \qquad \ldots(4.11)$$

Where,

K_T = frequency factor corresponding to C_s of original series.

Log Pearson Type III distribution

T year flood X_T is given by

$$X_T = exp(\overline{Y} + K_T * S_X) \qquad \ldots(4.12)$$

Where,

K_T = frequency factor corresponding to C_s of log transformed series.

Gumbel distribution

T year flood X_T is given by

$$X_T = u + \alpha * ln\left(ln\left(\frac{T}{T-1}\right)\right) \qquad \ldots(4.13)$$

Where,

$$\overline{X} = u + 0.5772\alpha \qquad \ldots(4.13a)$$

$$S_X = \frac{\pi * \alpha}{\sqrt{6}} \qquad \ldots(4.13b)$$

u = location parameter

α = scale parameter

Log Gumbel distribution

T year flood X_T is given by

$$\boldsymbol{X_T = exp\left(u + \alpha * ln\left(ln\left(\frac{T}{T-1}\right)\right)\right)} \quad \ldots(4.14)$$

Where,

$$\overline{X} = u + 0.5772\alpha \quad \ldots(4.14a)$$

$$S_X = \frac{\pi * \alpha}{\sqrt{6}} \quad \ldots(4.14b)$$

u = location parameter

α = scale parameter

4.4.3.2 Probability Weighted Moment (PWM)

Gumbel distribution

T year flood X_T is given by

$$\boldsymbol{X_T = (u + \alpha * y_T)} \quad \ldots(4.15)$$

Where,

$$u = M_{100} - 0.5772\alpha \quad \ldots(4.15a)$$

$$\alpha = \frac{M_{100} - 2M_{101}}{ln(2)} \quad \ldots(4.15b)$$

$$y_T = -ln\left(-ln\left(1 - \frac{1}{T}\right)\right) \quad \ldots(4.15c)$$

GEV distribution

T year flood X_T is given by

$$\boldsymbol{X_T = (u + \alpha\{1 - (-lnF)^k\} * k)} \quad \ldots(4.16)$$

Where,

$$u = b_0 + \alpha(\Gamma(1+k) - 1)/k \quad \ldots(4.16a)$$

$$\alpha = \frac{(2b_1 - b_0)k}{\Gamma(1+k)(1-2^{-k})} \quad \ldots(4.16b)$$

$$k = 0.759C + 2.9554C^2 \quad \ldots(4.16c)$$

$$C = \frac{(2b_1 - b_0)}{(3b_2 - b_0)} - \frac{ln(2)}{ln(3)} \quad \ldots(4.16d)$$

4.4.4 Goodness of fit tests

4.4.4.1 Chi-square test

It is a method to assess goodness of fit between observed and the expected values.

$$\chi^2\, comp = \sum_{j=1}^{m} \left(O_j - E_j\right)^2 / (E_j) \qquad \text{...(4.17)}$$

m = number of classes

O_j = observed frequency of j[th] class

E_j = expected frequency of j[th] class

Observed/ Expected frequency minimum 4-6 in each class.

4.4.4.2 K-S test

The value of test statistic 'D' is calculated as

$$D = Maximum\,|F_0(X) - F_r(X)| \qquad \text{...(4.18)}$$

$F_0(X)$ = observed cumulative frequency distribution of n observations.

$F_r(X)$ = theoretical frequency distribution

The critical value of D is obtained from the table.

4.4.4.3 D-index test

It is given by

$$D_{index} = (1/\bar{X}) \sum_{i=1}^{6} Abs(X_i - Y_i) \qquad \text{...(4.19)}$$

Where, X_i and Y_i are the i[th] highest observed and computed values for the distribution. The distribution giving the least D-index is considered to be the best fit distribution.

4.4.5 L-moment Ratio

L moments are linear combinations of PWMs

$$\lambda_1 = \beta_0 \qquad \text{...(4.20)}$$

$$\lambda_2 = 2\beta_1 - \beta_0 \qquad \text{...(4.21)}$$

$$\lambda_3 = 6\beta_2 - 6\beta_1 + \beta_0 \qquad \text{...(4.22)}$$

$$\lambda_4 = 20\beta_3 - 30\beta_2 - 12\beta_1 - \beta_0 \qquad \text{...(4.23)}$$

In particular,
$$\beta_0 = \frac{\sum_{i=1}^{n} X_i}{n} \qquad \ldots(4.24)$$

$$\beta_1 = \frac{\sum_{i=1}^{n} X_i * F_i}{n} \qquad \ldots(4.25)$$

$$\beta_2 = \frac{\sum_{i=1}^{n} X_i * F_i^2}{n} \qquad \ldots(4.26)$$

$$\beta_3 = \frac{\sum_{i=1}^{n} X_i * F_i^3}{n} \qquad \ldots(4.27)$$

Where,
$$F_i = \frac{(i - 0.35)}{n} \qquad \ldots(4.28)$$

i = rank of the data arranged in ascending order; $LC_v = \lambda_2/\lambda_1$; $LC_s = \lambda_3/\lambda_2$; measure of symmetry; and $LC_k = \lambda_4/\lambda_2$; measure of peakedness. Brief diagram of steps is given below.

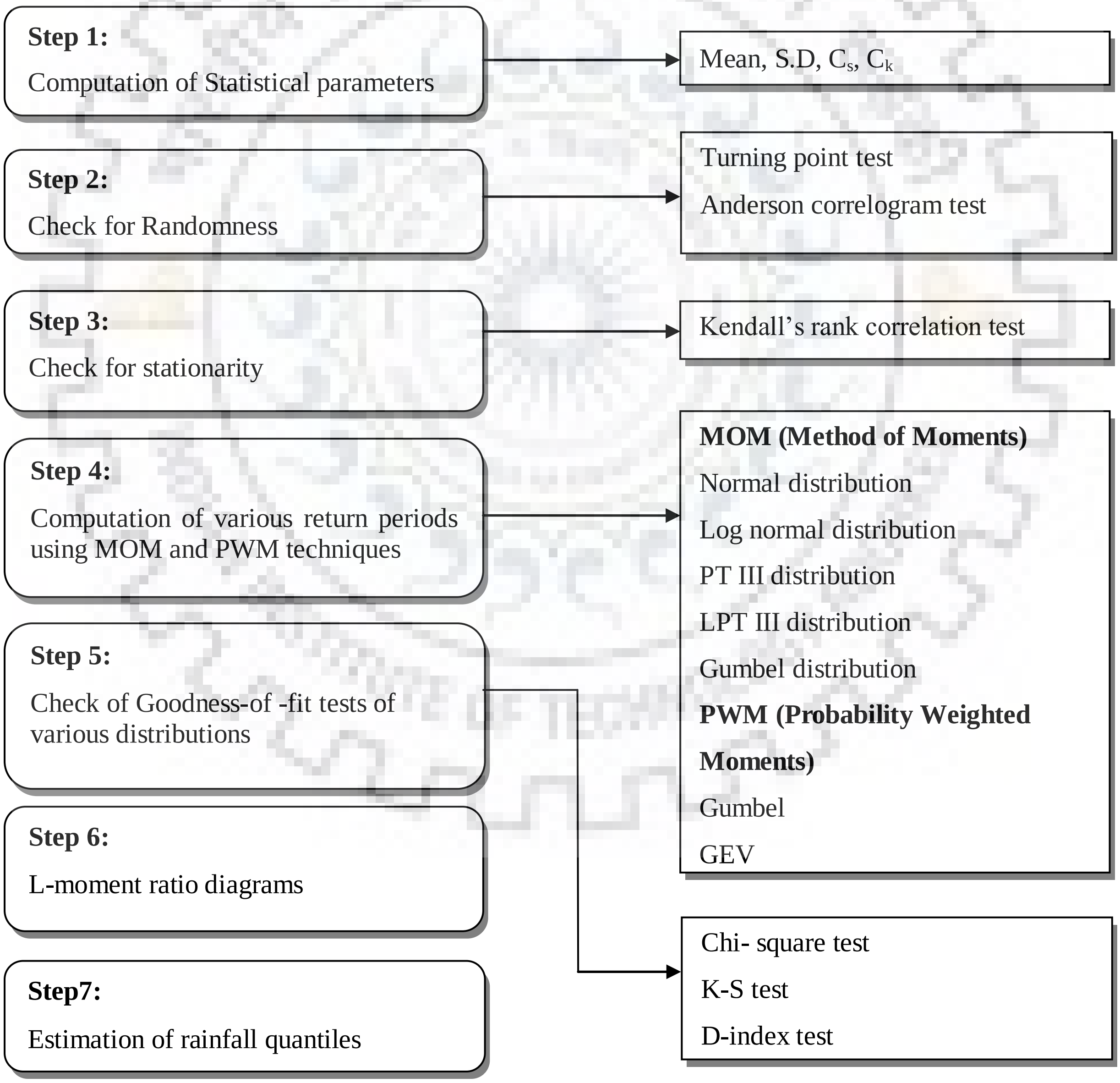


Figure 4.1 Steps to carry out frequency analysis

4.5 DEVELOPMENT OF INTENSITY DURATION FREQUENCY CURVES BASED ON THE ANALYSIS OF 116 YEARS DATA

Using the past 116 years data, the Intensity Duration Frequency curves for Roorkee has been developed. For this the equation 4.29 is used to get the intensity at smaller durations.

$$\frac{i}{I} = \frac{T + C}{t + c} \qquad \ldots(4.29)$$

Where t is smaller duration, i is the rainfall intensity of t hour, T is storm period, I is intensity at T, C and c are constants with value 1.

4.6 DETERMINATION OF RETURN PERIOD AT DIFFERENT DURATIONS OF JUNE 28, 2017

In 1931, Sherman developed the relationship,

$$I = \frac{KT^a}{(t + c)^b} \qquad \ldots(4.30)$$

Where I is the intensity in cm/hr., t is the duration in hours, T is return period, K, a, b, and c are constants depending on geographical location. Values of K, a, b and c are 6.0, 0.22, 0.5 and 0.8 respectively. The above values have been determined by Ram Babu et al., 1979.

The above equation was used to compute the return period at every hour during the event of June 28, 2017.

4.7 COMPUTATION OF CATCHMENT AREAS AT DIFFERENT INLET POINTS OF THE CAMPUS

Watershed delineation in ArcGIS is done using digital elevation model of the area. For the present study, SRTM-DEM data of 30-m resolution was downloaded from USGS (https://earthexplorer.usgs.gov/). Further processing was done in ArcMap 10.3.1 using the following steps.

Fill: It fills the sinks of DEM and results in depression less DEM.

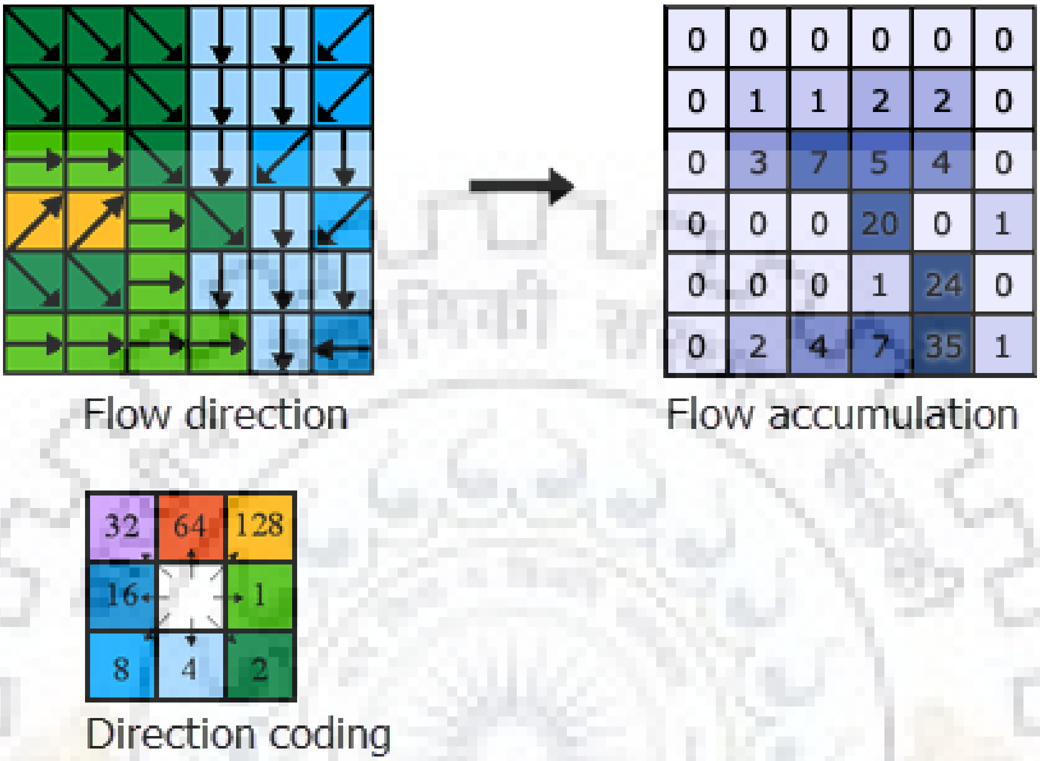
Flow direction: It helps to determine the direction of flow from each cell towards the steepest adjacent cell.

Flow accumulation: It calculates accumulated flow in each cell, determined by the accumulated weight of all the cells that flow in each down slope cell.

Basin: It divides a large DEM into smaller basin areas.

Snap pour point: It snaps the pour point to the nearest area of high flow.

Watershed: it delineates the watershed based on snapped stream gage pour point.



Source: http://pro.arcgis.com

Figure 4.2 Diagrams representing flow direction and accumulation process in GIS

4.8 COMPUTATION OF CONTRIBUTING AREAS AT VARIOUS POINTS OF THE MAIN DRAIN OF THE CAMPUS

Area and perimeter of the delineated watersheds was computed, AutoCAD and GIS have been used for this purpose. Also the impervious surfaces, vegetation cover, bare soil etc. have been calculated in AutoCAD.

4.9 COMPUTATION OF THE PEAK DISCHARGES USING RATIONAL FORMULA FOR JUNE 28, 2017 EVENT

The runoff generated gradually increases from zero to constant value in a rainfall of very long duration. It increases as more flow reaches from remote areas to the outlet point and becomes constant at the time of concentration (t_c), which is defined by ‘the time taken for a drop of water from the farthest point of the catchment to reach the outlet’.

The peak value of runoff is given by:

$$Q = \frac{1}{3.6} * C * i * A \qquad \ldots(4.31)$$

Q = peak discharge (m^3/s)

C = coefficient of runoff

i = intensity of rainfall (mm/h)

A = area of catchment (km^2)

Coefficient C depends on the surface of the catchment. Since the catchment has various surfaces, therefore, weighted equivalent runoff coefficient is given by

$$C_e = \frac{\sum_1^N [C_i * A_i]}{A} \qquad \ldots(4.32)$$

4.9.1 Kirpich Equation

It relates time of concentration of the length of travel and slope of the catchment as

$$t_c = 0.01947 * L^{0.77} * S^{-0.385} \qquad \ldots(4.33)$$

t_c= time of concentration (min)

L= maximum length of travel of water (m)

S= slope of catchment

4.10 COMPUTATION OF THE EXISTING CAPACITY OF DRAINS AT VARIOUS POINTS USING MANNING'S FORMULA

To determine the actual designed capacity of the drain for carrying the runoff, Manning's equation is used. It helps to calculate runoff holding capacity of drains by determining the rate of flow at a particular cross-section. Runoff at eight points on the stormwater drain is calculated. The location of various points is given in Figure 5.16.

$$V = \frac{1}{n} * R^{2/3} * S^{1/2} \qquad \ldots(4.34)$$

$$Q = A * V \qquad \ldots(4.35)$$

n= manning's roughness coefficient

R= hydraulic radius (m)

S=channel slope

Q= runoff (m^3/s)

A=area (m^2)

V= velocity (m/s)

4.11 SETTING UP OF STORM WATER MANAGEMENT MODEL

The EPA Storm Water Management Model developed in 1971 is a rainfall runoff model used for single event or long term simulation of runoff quantity and quality from urban areas. The SWMM, developed under the support of US EPA, is commonly applied for quality and quantity processes of runoff in urbanized area.(States, 2015)

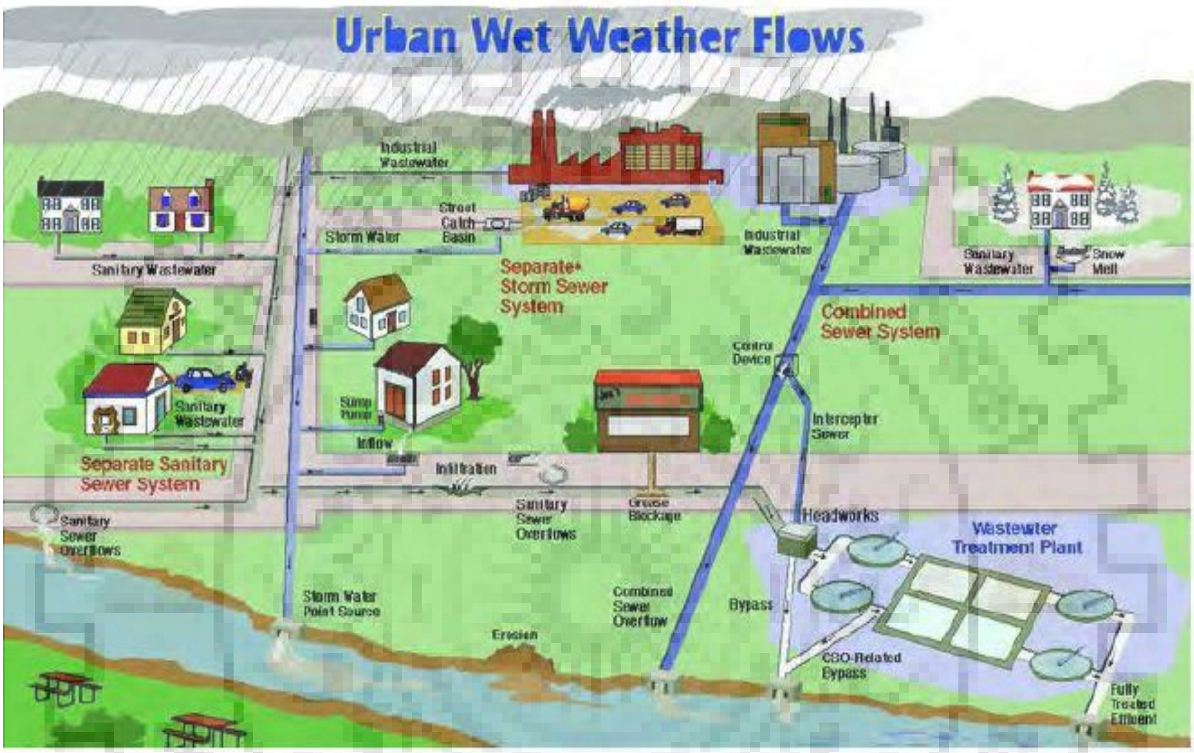


Figure 4.3 Pictorial representation of urban runoff

SWMM is used to assess the carrying capacity of existing drainage system and to design an efficient drainage system to carry a heavy rain storm safely. Since it is rainfall- runoff simulation model, it is capable to calculate the quantity in each pipe during simulation. It is used widely for planning, analysis and design related to stormwater runoff, combined sewers, sanitary sewers and other drainage systems in urban areas. The conceptual view of surface runoff used by SWMM is given in Figure 4.4. Surface runoff occurs only when the depth of water in the reservoir exceed the maximum storage, then the outflow is given by manning's equation. Each subcatchment is considered as a non linear reservoir. Inflow comes in the form of precipitation or any upstream subcatchment. Outflow may be due to infiltration, evaporation or surface runoff.

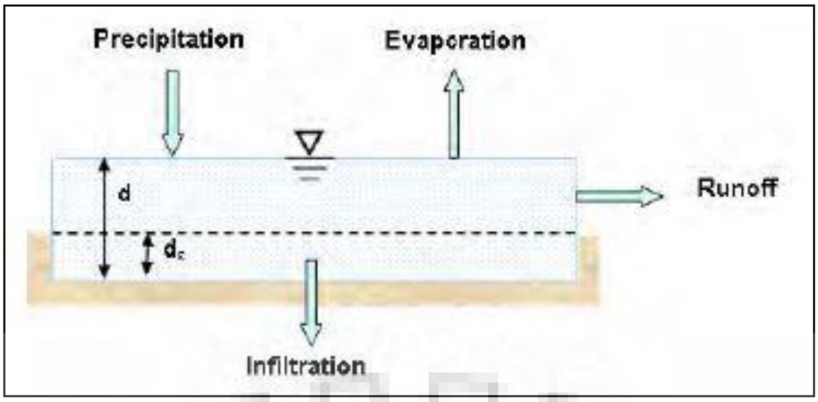


Figure 4.4 Conceptual view of surface runoff

SWMM conceptualizes drainage system as water and material flow between major environmental compartments which include;

a. Atmosphere compartment
b. Land Surface compartment
c. Groundwater compartment
d. Transport compartment

The atmosphere compartments make use of rain gauge as an input to the system. The subcatchment objects represent Land Surface compartment, aquifers denote the Groundwater compartment and network of channels, and pipes etc. denote the transport compartment. One particular model may have one or more than one compartments as input. (States, 2015)

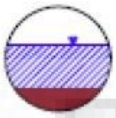
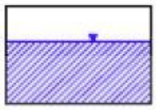
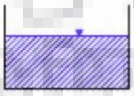
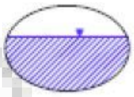
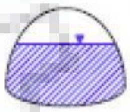
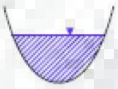
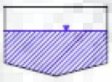
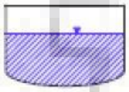
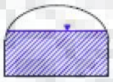
4.12 COMPUTATION OF CAPACITIES USING SWMM

It is used to determine the adequacy of existing drainage system. SWMM uses the Manning's roughness coefficient to express the relationship between flow rate (Q), cross-sectional area (A), hydraulic radius (R), and slope (S).

$$\boldsymbol{Q = \frac{1.49}{n} * A * R^{2/3} * S^{1/2}} \quad \text{…(4.36)}$$

Where, n is Manning's roughness coefficient. Slope S is interpreted through conduit slope. There are a number of cross-section shapes available for conduits as given in Table 4.1.

Table 4.1 Available cross section shapes for conduits

Name	Parameters	Shape	Name	Parameters	Shape
Circular	Full height		Circular force main	Full height, Roughness	
Filled circular	Full height		Rectangular closed	Full height, Width	
Rectangular-open	Full height, Width		Trapezoidal	Full height, Base width, Side slopes	
Triangular	Full height, Top width		Horizontal ellipse	Full height, Max. width	
Vertical ellipse	Full height, Max. width		Arch	Full height, Max. width	
Parabolic	Full height, Top width		Power	Full height, Top width, Exponent	
Rectangular-Triangular	Full height, Top width, Triangle height		Rectangular round	Full height, Top width, Bottom radius	
Modified basket handle	Full height, Bottom Width, Top radius		Egg	Full height	
Horseshoe	Full height		Gothic	Full height	
Catenary	Full height		Semi-elliptical	Full height	
Basket handle	Full height		Semi- circular	Full height	
Irregular natural channel	Transect coordinates		Custom closed shape	Full height, Shape curve coordinates	

Since the present study uses SCS-CN method, details are given in Table 4.2. These curve numbers are used to calculate the weighted curve number of individual subcatchments of the model. This is adopted from NRCS (SCS) Curve number method for runoff estimation.

Table 4.2 SCS Curve Numbers

Land Use Description	**Hydrologic Soil Group**			
	A	**B**	**C**	**D**
Cultivated land				
Without conservation treatment	72	81	88	91
With conservation	62	71	78	81
Pasture or range land				
Poor condition	68	79	86	89
Good condition	39	61	74	80
Meadow				
Good condition	30	58	71	78
Wood or forest land				
Thin stand, poor cover, no mulch,	45	66	77	83
Good cover	25	55	70	77
Open spaces, lawns, parks, golf courses, cemeteries, etc.				
Good condition; grass cover on 75% or more of the area	39	61	74	80
Fair condition; grass cover on 50-75% of the area	49	69	79	84
Commercial and business areas (85% impervious)	89	92	94	95
Industrial districts (72% impervious)	81	88	91	93
Residential				
Average lot size (%impervious)				
1/8 acre or less (65)	77	85	90	92
1/4 acre (38)	61	75	83	87
1/3 acre (30)	57	72	81	88
1/2 acre (25)	54	70	80	85
1 acre (20)	51	68	79	84
Paved parking lots, roofs, driveways, etc.	98	98	98	98
Street and roads				
Paved with curbs and storm sewers	98	98	98	98
Gravel	76	85	89	91
Dirt	72	82	87	89

4.13 COMPARISON OF THE DRAIN CAPACITIES USING RATIONAL FORMULA AND SWMM MODEL

The existing drainage capacity calculated using Manning's equation has been compared with the results obtained using rational formula and Storm Water Management Model. It is done to understand the efficiency of existing drain to handle such large flows.

4.14 COMPUTATION OF DURATION OF FLOODING AT VARIOUS POINTS DUE TO INADEQUATE CAPACITIES OF THE DRAIN AND OTHER CONSEQUENCES

This has been computed using SWMM Model. All the parameters such as invert level of the nodes; depth, shape, manning's roughness coefficient, length etc. of the conduit; area, width, imperviousness, curve number etc. of the subcatchments were used as an input in the model. After setting up the model simulation was run and the inundated zones were identified.

4.15 DESIGN CAPACITIES OF THE MAIN DRAIN OF THE CAMPUS FOR A RETURN PERIOD OF 200 YEARS

The runoff generated from 200 years return rainfall is computed using intensity from the IDF curves, and its value at various points on the nodes is calculated which could be used to redesign the drain in future.

The methodology and formulae used in storm water management analysis for a small urban catchment has been explained in detail in the above section. Figure 4.6 represents the workflow diagram of the study.

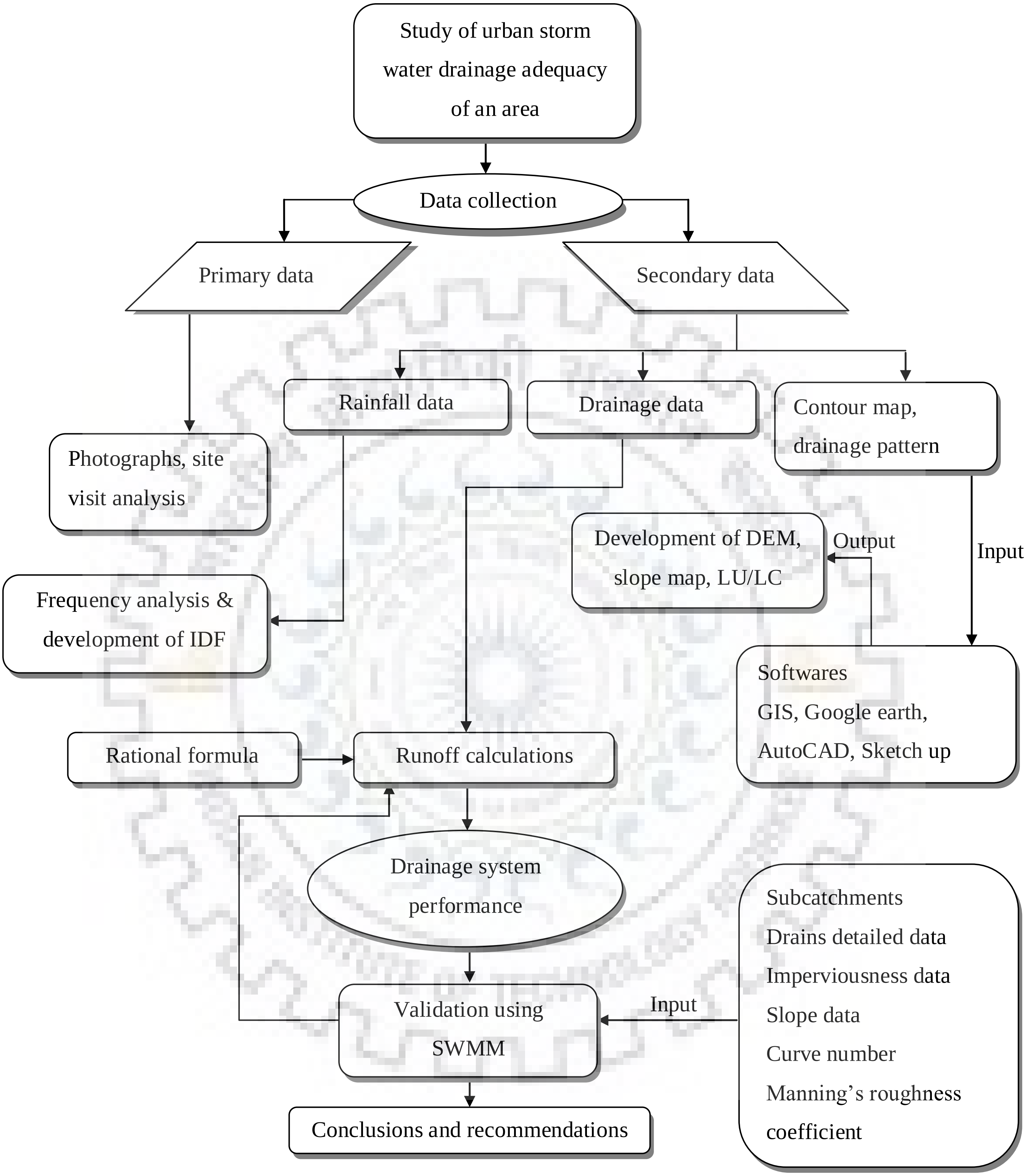


Figure 4.5 Steps for stormwater drain analysis

CHAPTER 5
RESULTS AND ANALYSIS

This chapter is about the results obtained during stormwater management analysis. The results are based upon the methodology explained in the previous chapter.

5.1 RAINFALL EVENT OF JUNE 28, 2017

5.1.1 Hyetograph of the storm

The rainfall of June 28, 2017 was majorly of 4 hrs duration. The depth of rainfall at time interval of 15 minutes is given in Table 5.1.

Table 5.1 Rainfall data of June 28, 2017 storm

Time (hh:mm)	Cumulative rainfall (mm)	Incremental rainfall (mm)
9:00	0	0
9:15	10	10
9:30	23	13
9:45	40	17
10:00	60	20
10:15	70	10
10:30	100	30
10:45	130	30
11:00	160	30
11:15	180	20
11:30	190	10
11:45	200	10
12:00	206	6
12:15	208	2
12:30	210	2
12:45	211.5	1.5
13:00	211.5	0

The rainstorm was of 4 hrs duration, the hyetograph is shown in Figure 5.1.

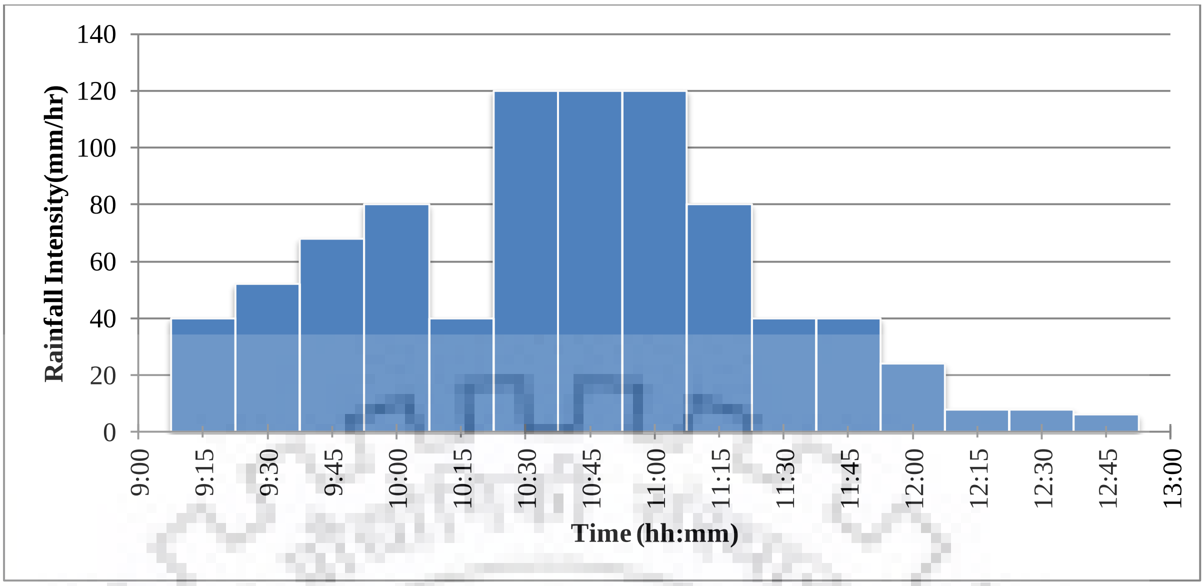


Figure 5.1 Hyetograph of rainfall event of June 28, 2017

The depth and intensity at interval of 15 minutes is shown in Table 5.2.

Table 5.2 Maximum Intensity and Maximum Depth duration of June 28, 2017 rainfall

Duration(min)	Depth(mm)	Intensity(mm/hr)
15	30	120
30	60	120
45	90	120
60	110	110
75	120	96
90	140	93
105	157	90
120	170	85
135	180	80
150	190	76
165	200	73
180	206	69
195	208	64
210	210	60
225	211.5	56
240	211.5	53

The depth duration and intensity duration curve is shown in Figure 5.2.

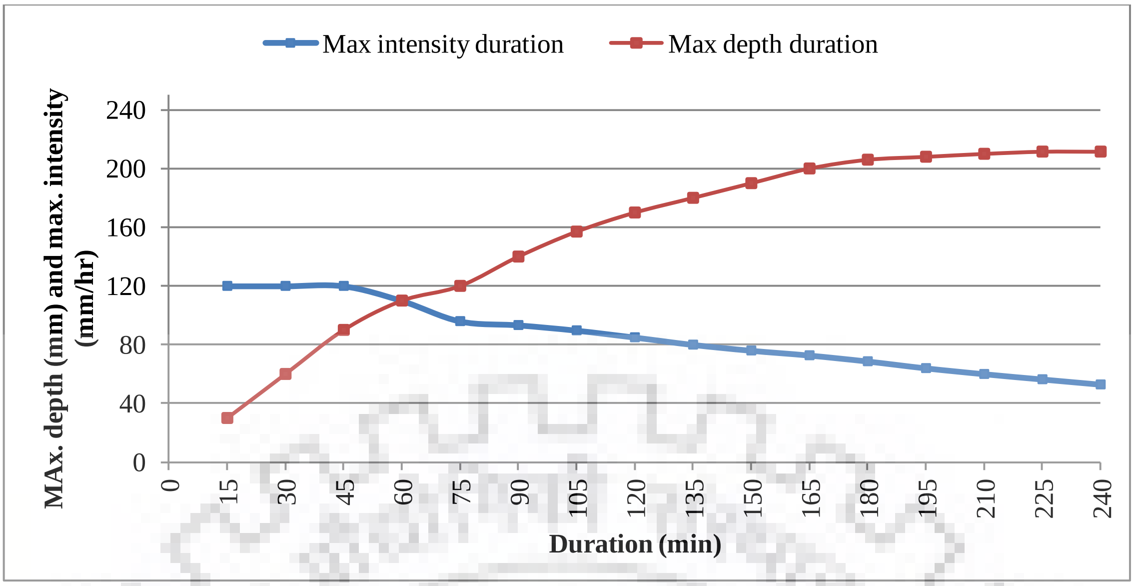


Figure 5.2 Maximum Intensity Duration and Maximum Depth Duration curves for storm of June 28, 2017

5.2 FREQUENCY ANALYSIS USING L-MOMENT APPROACH

The frequency of occurrence of extreme events can be determined by using probability distributions. In order to satisfy the condition of independent and identical distribution of random variable or say, rainfall events in this case, only the daily rainfall of highest magnitude of each year (1901-2016) was selected for frequency analysis.

In the present study, at-site frequency analysis of extreme rainfall events is performed using L-moments approach. L-moments approach is a recent development within statistics (Hosking 1990) and found to be superior to other methods used in similar studies carried out in the past (Bhuyan et al., 2016).

5.2.1 Results and analysis

Frequency analysis was performed excluding the event of the year 2017(236.6mm) to test whether this extreme event was an outlier or not. The values of L-coefficient of variation (LC_v), L-skewness (LC_s) and L-kurtosis (LC_k) are found to be 0.208690, 0.167564 and 0.207230 respectively. The L-moment diagram provided by Hosking (1990) (Rao, A.R & Hamed, 2000; Bisht et al., 2016) is used to identify the suitable rainfall frequency distribution (Figure 5.3) and Generalized Logistic Distribution (GLO) was found to be closest to the point defined by the values of L-skewness, i.e., LC_s = 0.167564 and L-kurtosis, i.e., LC_k = 0.207230.

Inverse form of the GLO distribution for a return period T and k≠0 can be expressed as,

$$X_T = \xi + \frac{\alpha}{k}(1 - (T-1)^{-k}) \qquad \ldots(5.1)$$

ξ, α, k are the location, scale and shape parameters respectively.

The location (ξ), scale (α), and shape (k) parameters of the GLO distribution are found to be 0.941517, 0.19889 and -0.167564 respectively.

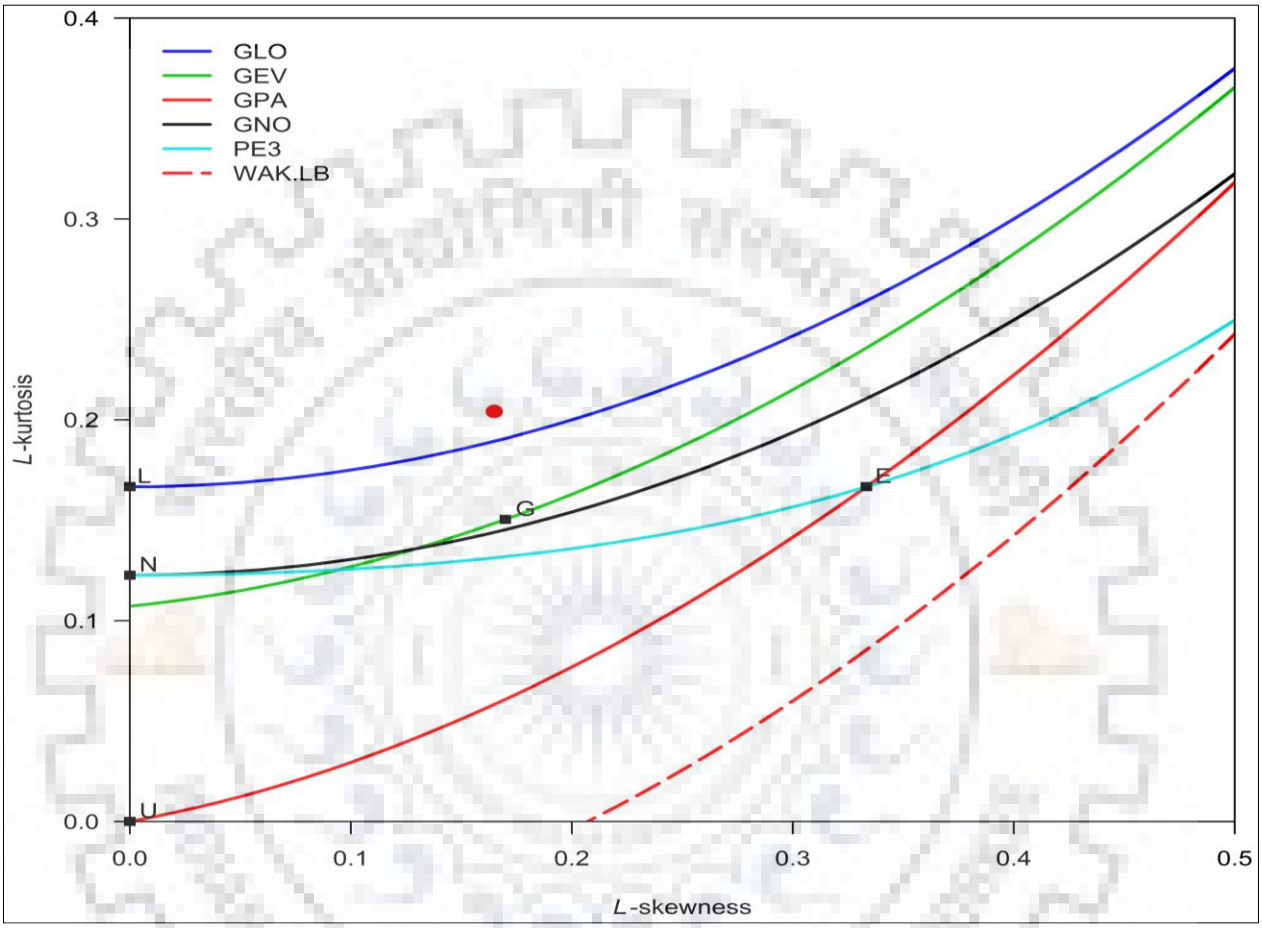


Figure 5.3 L-moment ratio diagram for IIT Roorkee for various distributions (excluding extreme event)

5.3 DETERMINATION OF RAINFALL FOR VARIOUS RETURN PERIODS

Equation of GLO is used to determine the precipitation at various return periods.

Table 5.3 Precipitation of various return periods for 24 hr duration

Return Period (years)	X_T (mm)	Return Period (years)	X_T (mm)
2	121.35	**50**	262.04
5	161.36	**100**	298.77
10	189.45	**500**	401.63
20	218.93	**1000**	455.08

The rainfall of June 28, 2017 experienced rainfall of 236.6mm in 24 hrs. The return period obtained using GLO (Equation 5.1) is about 30 years.

5.4 DEVELOPMENT OF INTENSITY DURATION FREQUENCY CURVES

a. Intensity of past 116 year daily data is converted into intensities of smaller durations, i.e. 1hr, 2hr, 6hr, 12 hrs, and 24 hrs., using Equation 5.2.
b. As observed, most of the storms in Roorkee last from 4-6 hrs. (Observed severe storm durations of past years), therefore, the value of T is taken as 5 hrs.

$$\frac{i}{I} = \frac{T+C}{t+c} \quad \ldots(5.2)$$

c. The intensity is obtained using Equation 5.3

$$I = \frac{KT^{a}}{(t+c)^{b}} \quad \ldots(5.3)$$

Table 5.4 Mean values of intensities of all smaller durations

Duration	1hr.	2hr.	6hr.	12hr.	24hr.
Mean (cm/hr)	1.61	1.07	0.46	0.25	0.13
S.D	0.62	0.41	0.18	0.10	0.05

Table 5.5 Values for intensities (mm/hr) corresponding to each duration and return period

Duration/R.P	**2**	**5**	**10**	**20**	**25**	**50**	**100**	**200**	**500**	**1000**
1	50.52	61.81	71.99	83.85	88.07	102.58	119.48	139.16	170.24	198.28
2	33.58	41.07	47.84	55.72	58.53	68.17	79.40	92.48	113.13	131.76
6	15.63	19.12	22.28	25.94	27.25	31.74	36.97	43.06	52.67	61.35
12	9.27	11.33	13.20	15.38	16.15	18.81	21.91	25.52	31.22	36.36
24	5.41	6.62	7.71	8.98	9.43	10.98	12.79	14.90	18.22	21.22

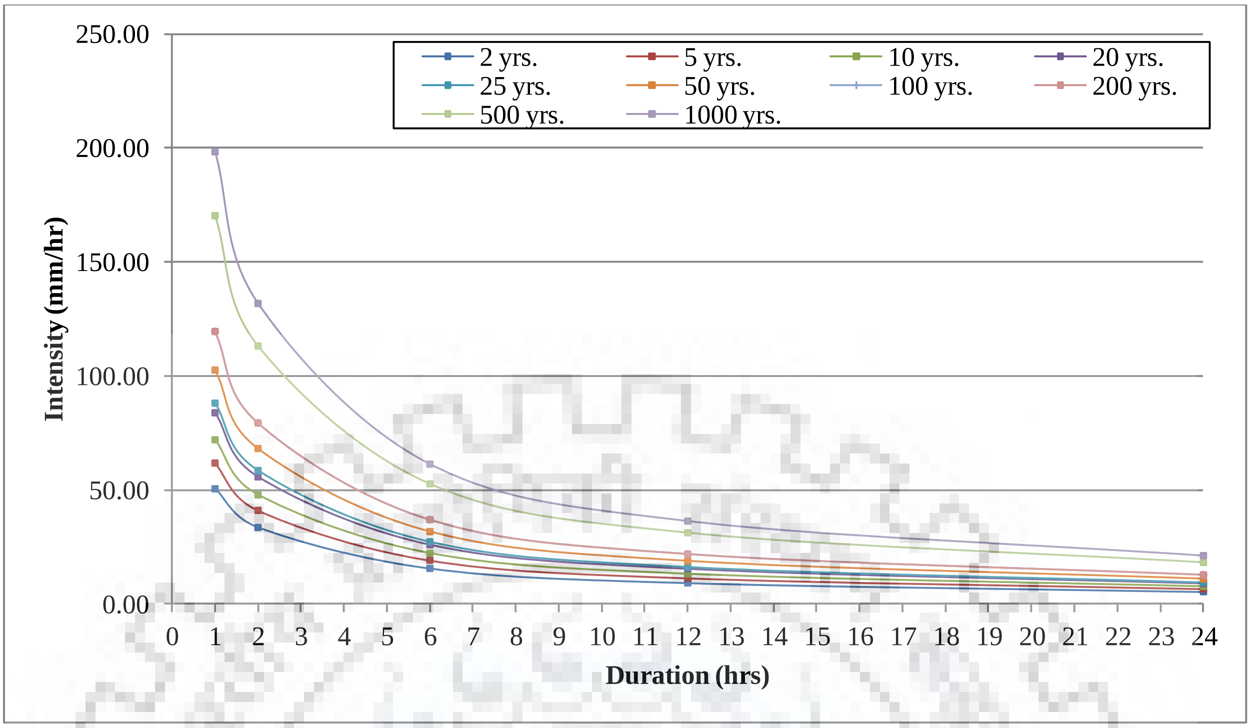


Figure 5.4 IDF plot for IIT Roorkee

The Return period at shorter interval for June 28, 2017 storm is evaluated using Equation 5.3. The table representing the values at each interval is given below.

Table 5.6 Return period at each duration for June 28, 2017

Duration (hr)	Intensity (cm/hr)	R.P.
0.25	12.00	8
0.50	12.00	23
0.75	12.00	53
1.00	11.00	69
1.25	9.60	65
1.50	9.33	93
1.75	8.97	119
2.00	8.50	136
2.25	8.00	146
2.50	7.60	159
2.75	7.27	174
3.00	6.87	176
3.25	6.40	164
3.50	6.00	155
3.75	5.64	146
4.00	5.29	134

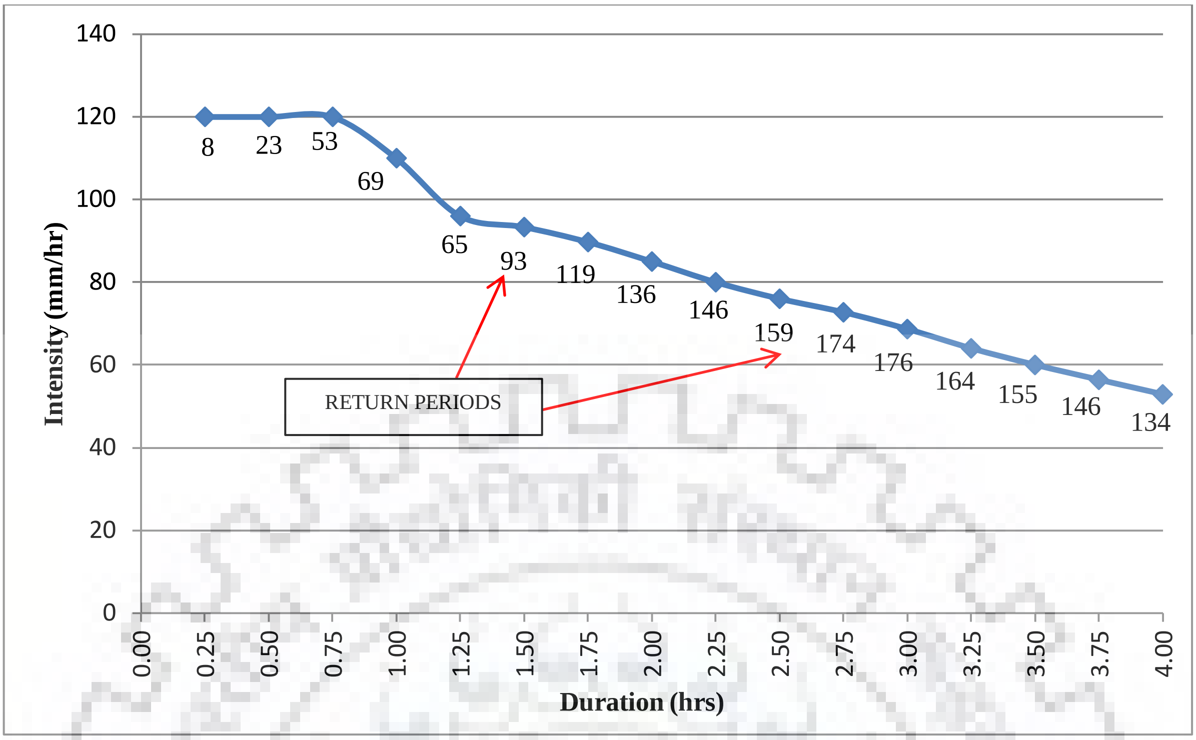


Figure 5.5 IDF for June 28, 2017

5.5 SITE SPECIFIC ANALYSIS

5.5.1 DEM and slope map of the study area

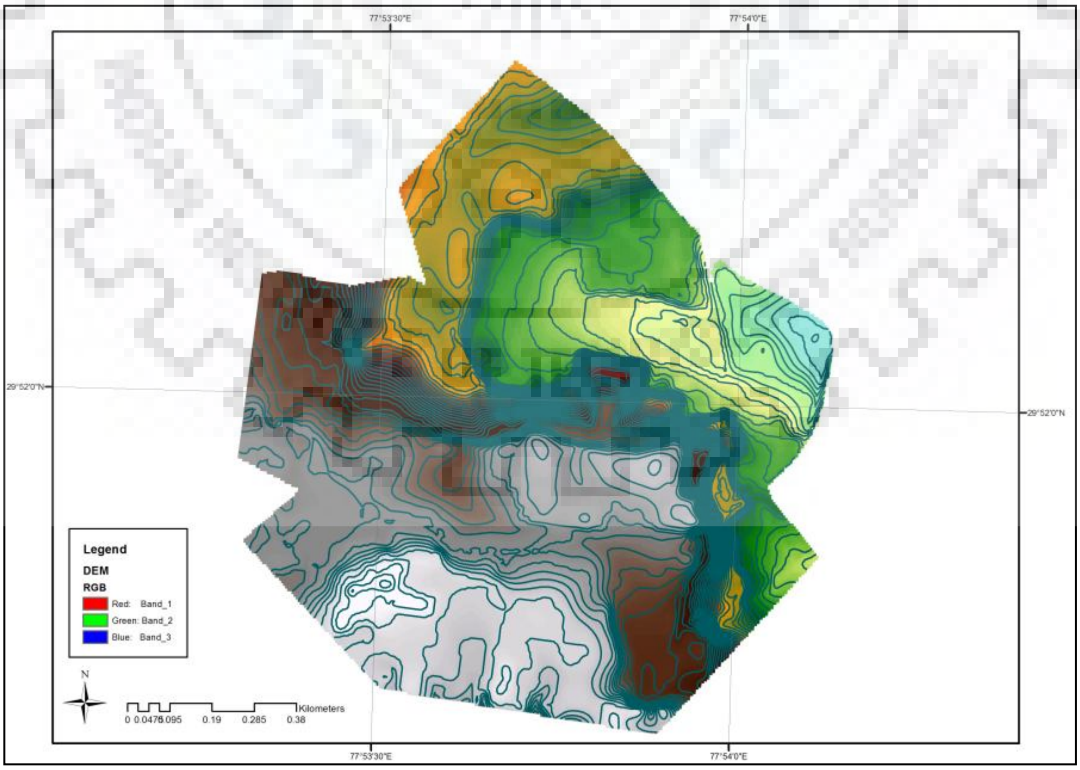


Figure 5.6 DEM of IIT Roorkee campus

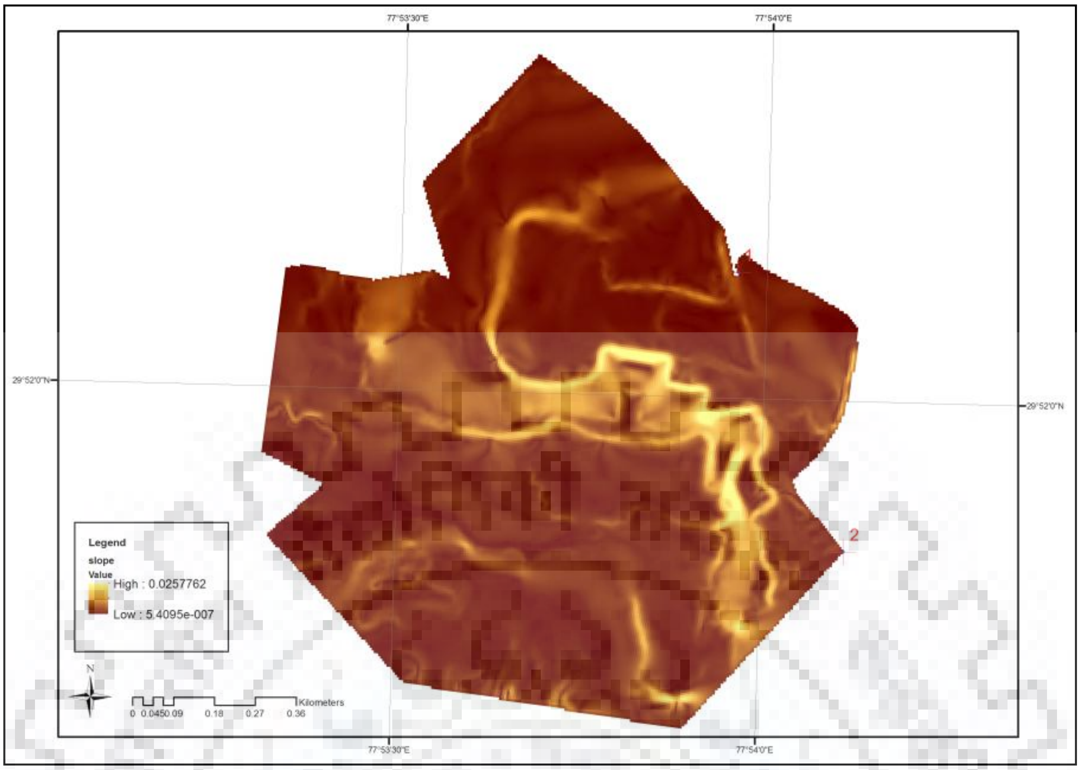


Figure 5.7 Slope map of IIT Roorkee campus

5.5.1.1 Inferences drawn from DEM and slope map

a. Inefficient drainage system along with improper solid waste management causes frequent flooding and water-logging in various areas.

b. The area has a **contoured topography that makes it vulnerable to flood during heavy rainstorms**. Most of the water flows towards the eastern zone of the campus but recent construction of structures led to decrease the pervious surfaces in the low lying regions and thus causing water logging in several places.

5.5.2 Development of Land use/Land cover map

Digitization of campus was done to understand the land use scenario of the area. It helps to identify the percentage of pervious and impervious areas in the campus. Drainage planning can be further enhanced using land use map of an area.

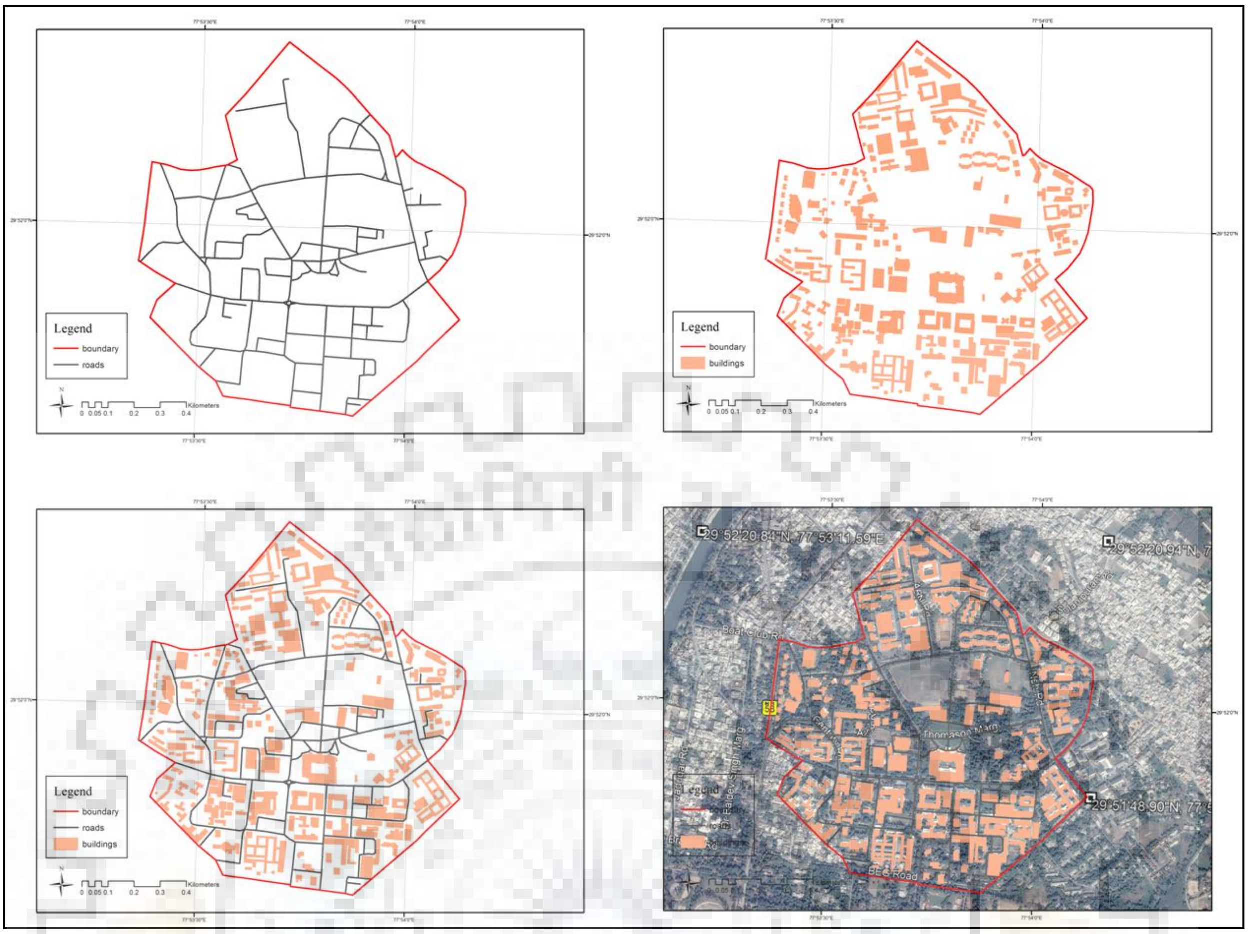


Figure 5.8 Digitized maps of IIT Roorkee campus areas

5.5.2.1 Inferences drawn from LULC

a. Although the built up surface is not much which may lead to high runoff during the storm events but improper planning causes obstruction to the natural flow of water. Since the slope of the campus is towards east i.e. towards Kasturba Bhawan, therefore water automatically follows natural path during the heavy storm and during this storm event water could not find clear passage thus caused flooding in hostels and various other places.

b. Construction of new buildings in the area which acted as a pervious space (before 2010) to infiltrate water is causing major harm to the structure as well as the residents. Below is the 3-dimensional view indicating the flooded areas of the campus and DEM to represent the actual cause of inundation.

The cause of inundation at various low lying areas during the high-intensity rainfall can be better understood through Figure 5.9.

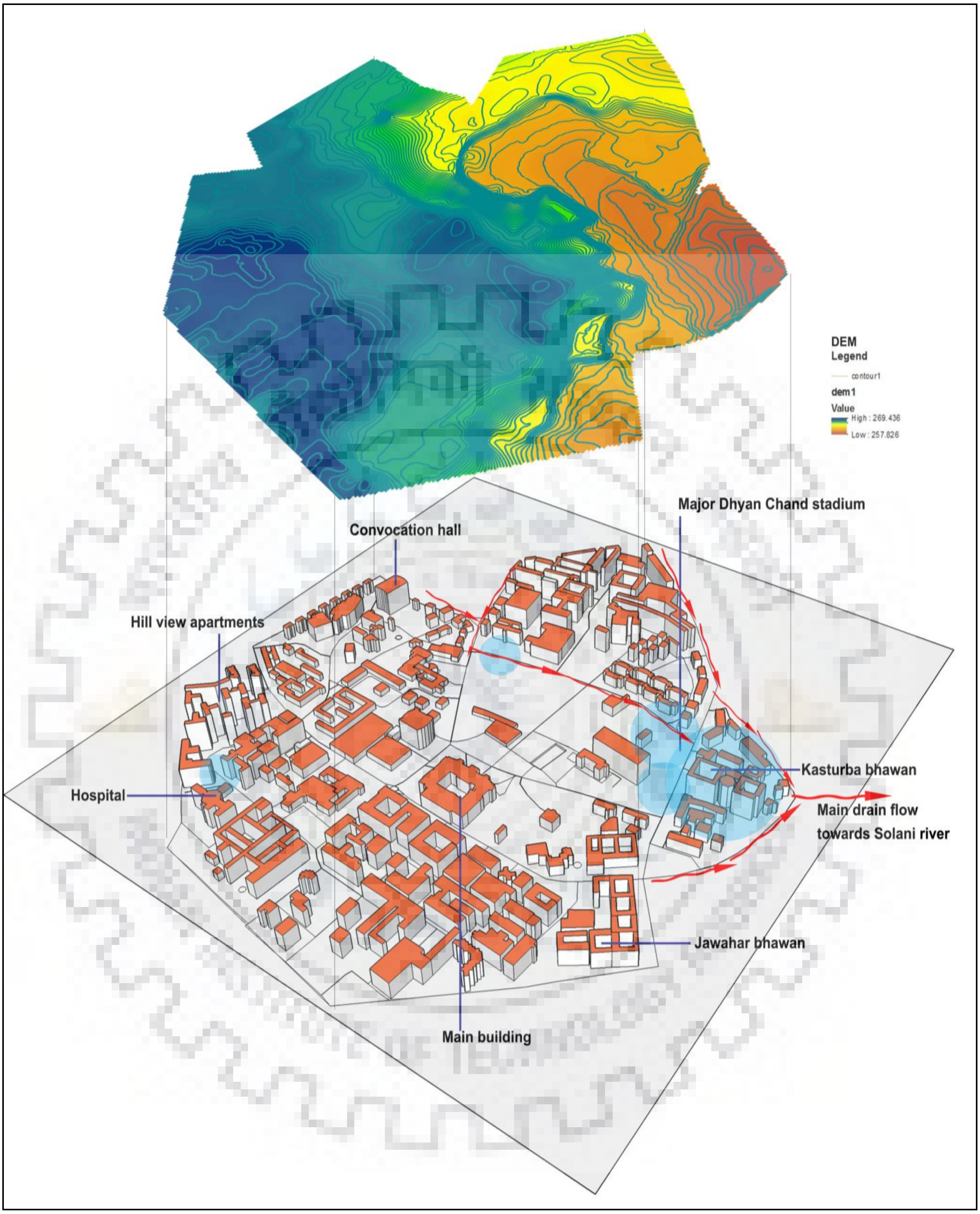


Figure 5.9 3-Dimensional view of flooded areas of IIT Roorkee

5.6 COMPUTATION OF CATCHMENT AREAS AT DIFFERENT INLET POINTS

The basin that consists of the watershed of the study area is obtained using SRTM-DEM of 30 meters resolution (Figure 5.10).

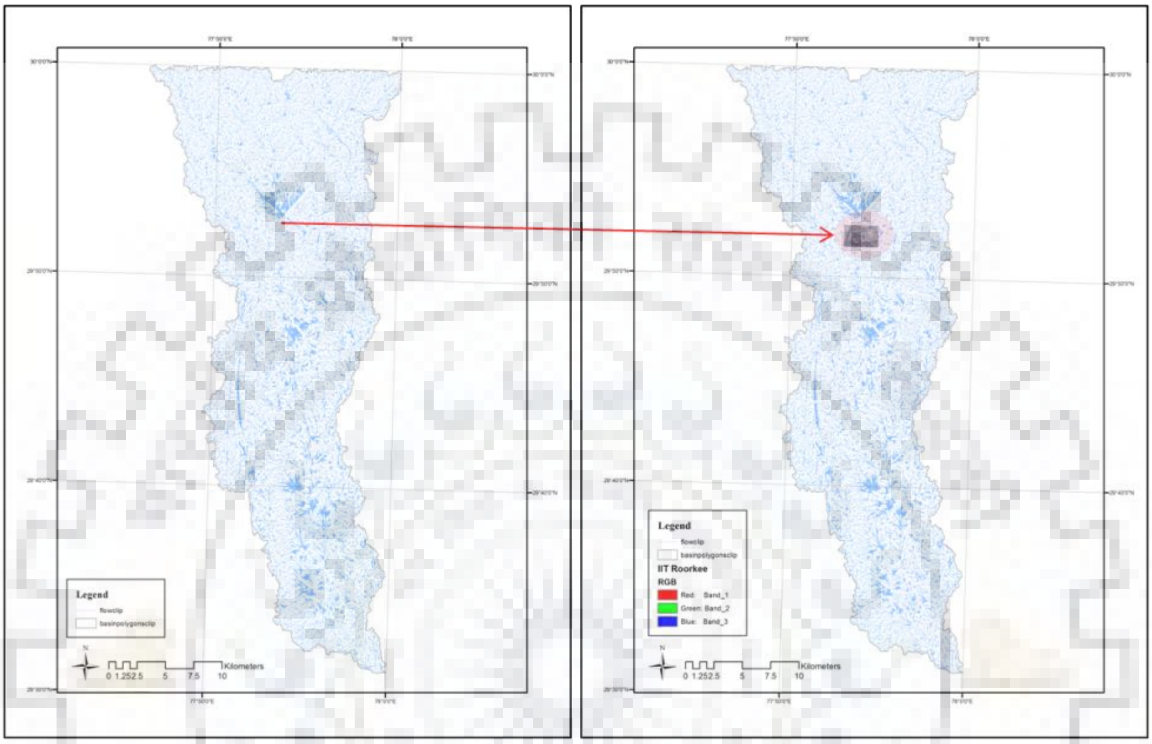


Figure 5.10 Basin of the study area

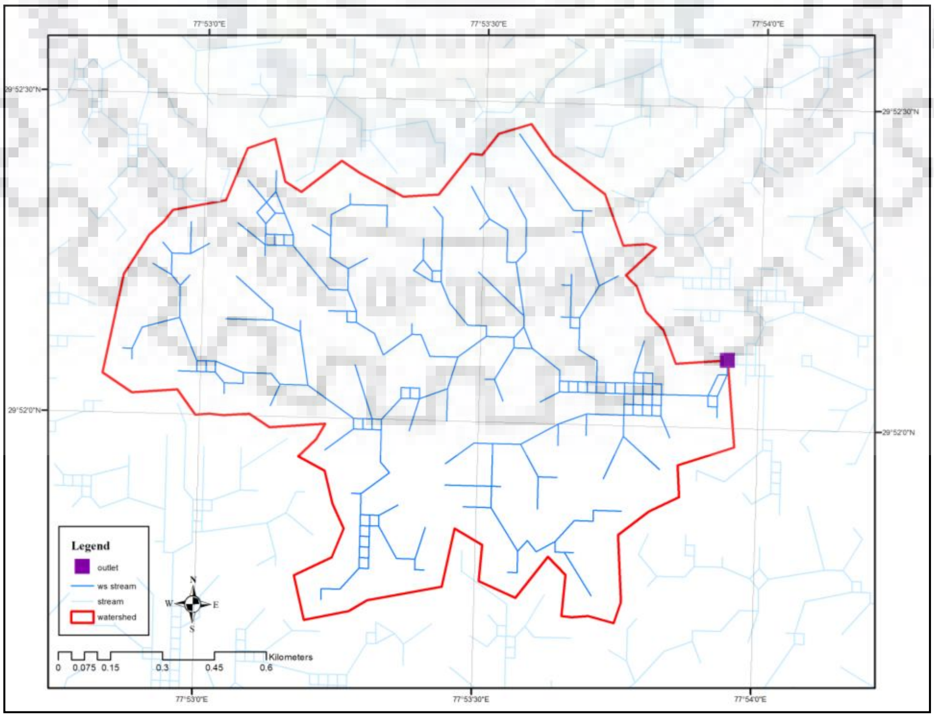


Figure 5.11 Stream flow in the watershed of the study area

Figure 5.11 shows the stream flow pattern in the watershed of the study area. It helps to analyze the direction of flow within the area.

For watershed delineation pour points on the study area are marked and the sub-watersheds contributing to that pour point are generated for calculation of runoff generating from that region. Watersheds at four different pour points are generated out of which three are at the nodes of the main drain and fourth is the outfall point of the campus.

For delineating sub-watersheds followings procedure was followed in ArcMap 10.3.1:

Pour point – The points where accumulation is to be calculated.

Snap pour point- Snapping the pour point to nearby highest flow.

Watershed- Delineating watershed along the snapped pour point.

The area and perimeter of sub-watersheds are given in Table 5.7. It is used in the calculation of the runoff while using rational formula.

Table 5.7 Area and perimeter of watersheds

S.No	Watershed	Area (sq. mts.)	Perimeter (mts.)
1.	Watershed 1	732871	4388
2.	Watershed 2	927826	4974
3.	Watershed 3	1405654	5906
4.	Watershed 4	1538351	6445

Further details of sub watersheds are given in Figure 5.12 to 5.15.

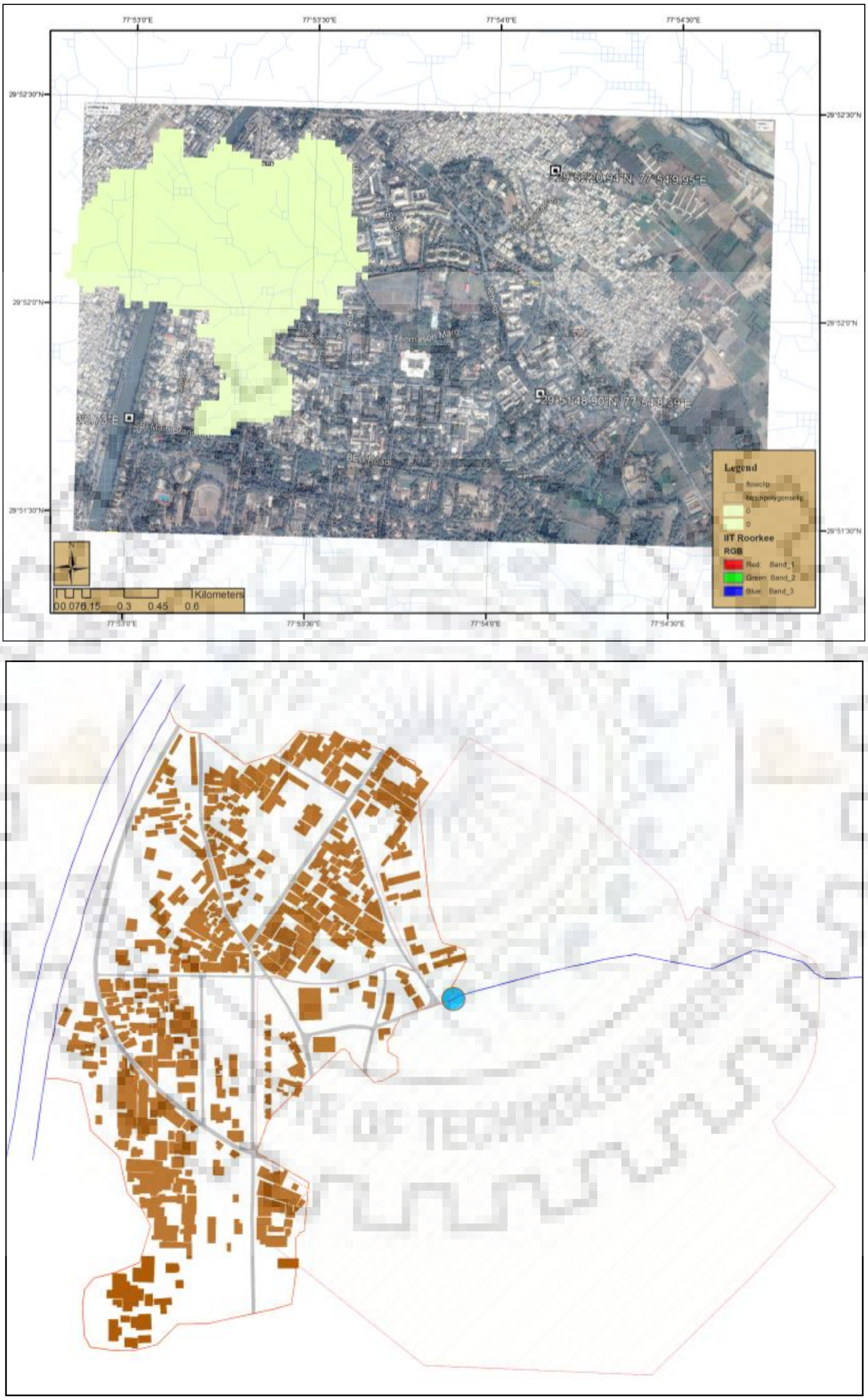


Figure 5.12 (a) Watershed 1 (b) impervious areas WS1

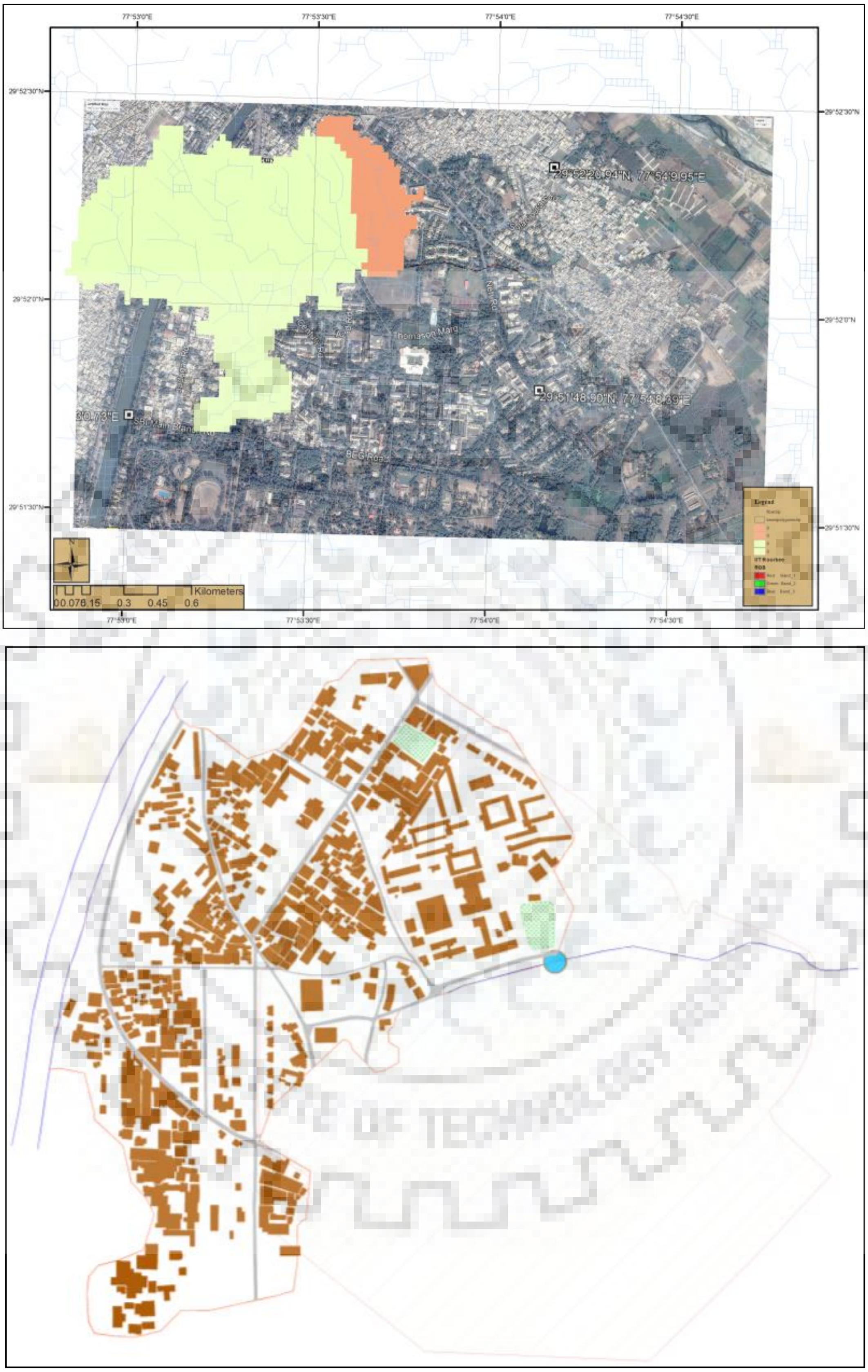
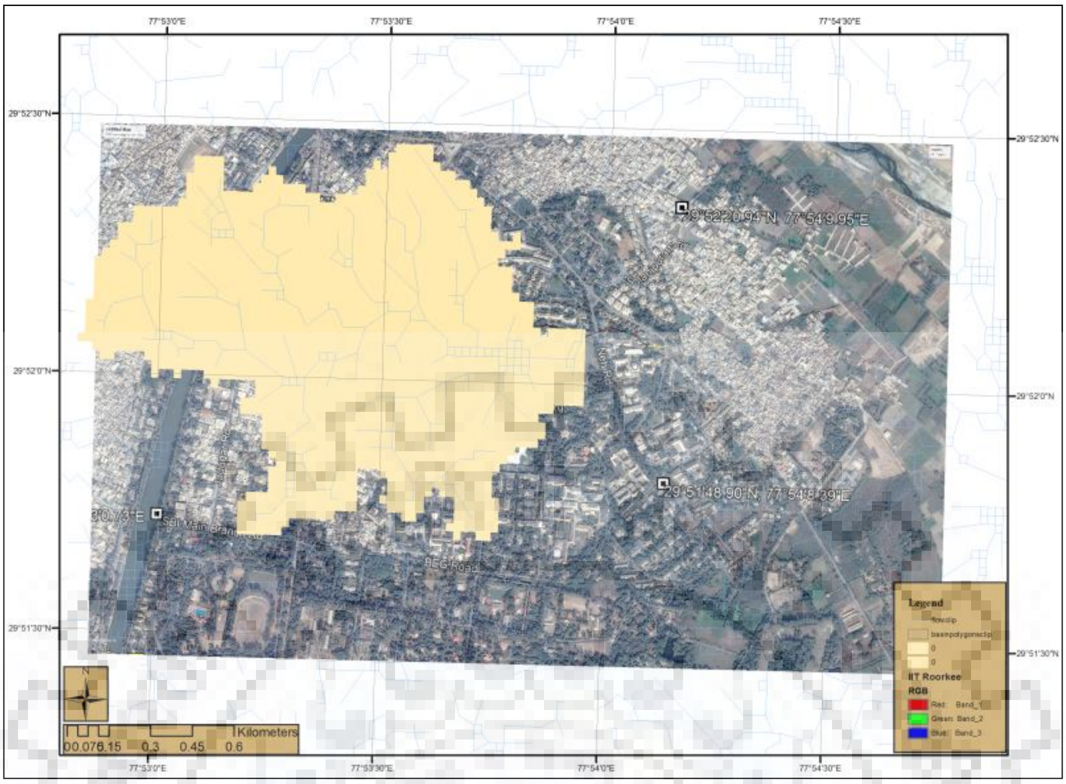


Figure 5.13 (a) Watershed 2 (b) impervious area WS2



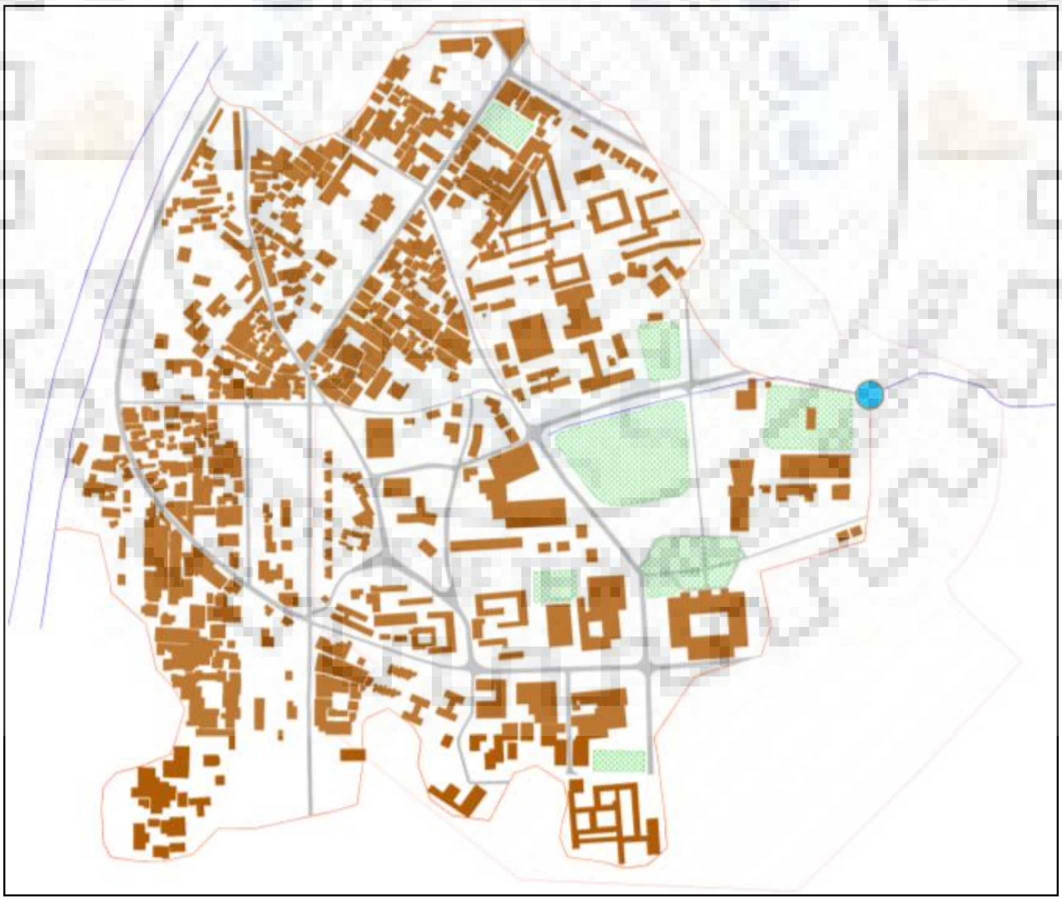


Figure 5.14 (a) Watershed 3 (b) impervious area WS3

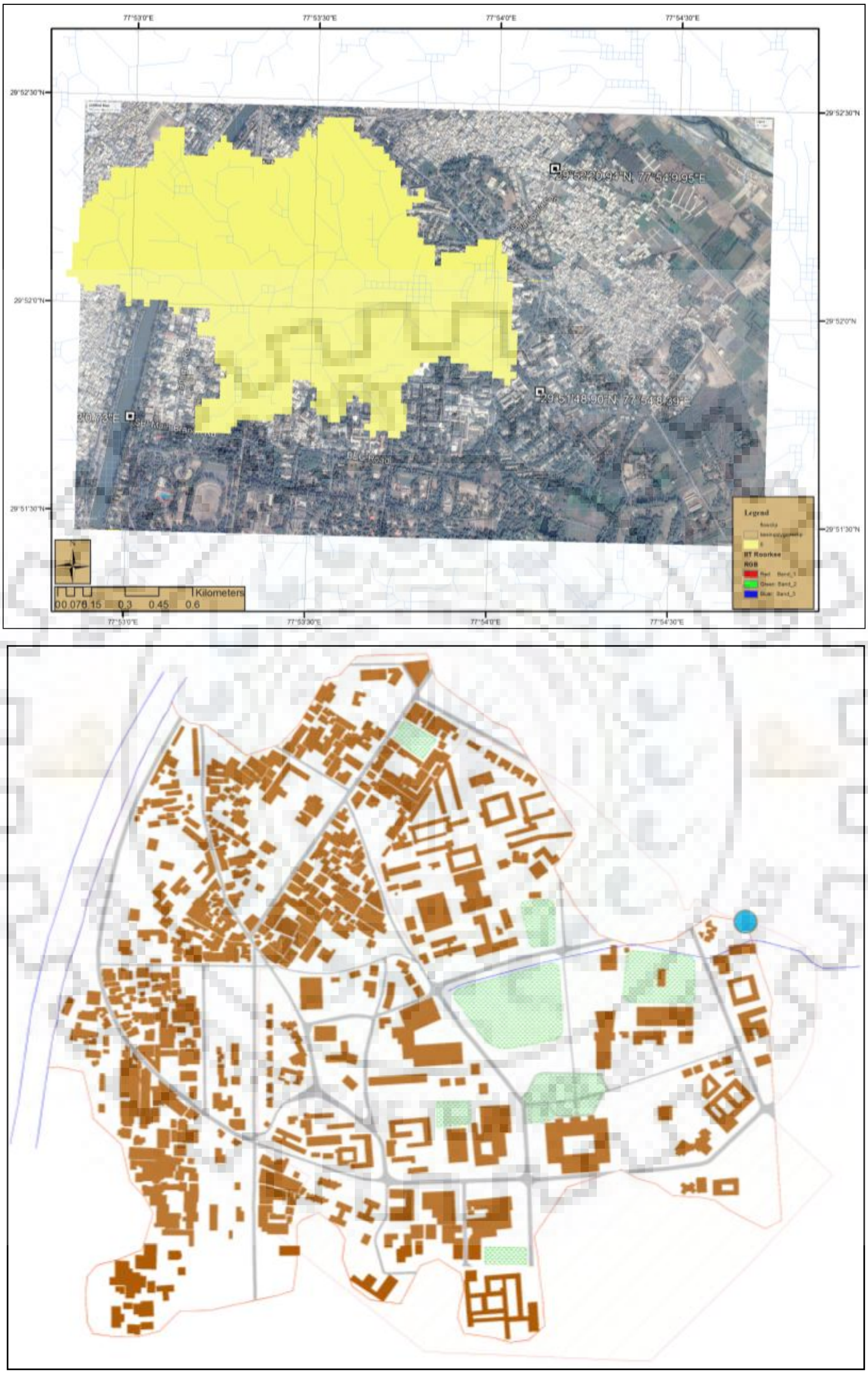


Figure 5.15 (a) Watershed 4 (b) impervious area WS4

The total impervious surface area of the subcatchments is also calculated to quantify the amount of runoff generated from watershed at specific nodes (Table 5.8).

Table 5.8 Area covered under all watersheds

	WS1	WS2	WS3	WS4
Area (km^2)	0.73	0.92	1.4	1.54
Impervious area (km^2)	0.24	0.30	0.43	0.46

5.6.3.1 Inferences drawn

a. After delineation, it was concluded that catchment is the basis for planning and designing the stormwater systems in any area irrespective of administrative boundaries. The drainage must follow actual watershed boundary rather than administrative boundaries (National Disaster Management Guidelines, 2007).

b. Runoff increases as the amount of impervious surface increases; therefore the use of permeable surfaces must be promoted.

5.7 COMPUTATION OF PEAK DISCHARGES USING RATIONAL FORMULA

The rational formula is used to compute the runoff generated on June 28, 2017 at various points on the main drain of the campus.

Table 5.9 Calculation using rational formula

	Node A	Node B	Node E
Tc (min)	30.12	35.66	42.58
depth at tc (mm)	60.24	71.32	85.17
Ic (mm/h)	120	120	120
Q (m^3/s)	8.03	11.65	17.27

5.8 COMPUTATION OF EXISTING DRAINAGE CAPACITY USING MANNING'S FORMULA

The Manning's equation is used to calculate the existing drainage capacity of the main drain of the campus at various points. It is done to assess that whether the drain is adequate for such a high intensity rainstorm.

Table 5.10 Calculation of runoff capacity of existing drainage system

Node	width(b)	Slab	height(h)	L(m)	R	slope change		ΔS/L	A(m^2)	V(m/s)	Q(m^3/s)
A	**1.45**	**0.2**	**1.05**	**283**	**0.429**	**260.813**	**260.043**	**0.003**	**1.523**	**2.119**	**3.23**
B	**1.45**	**0.2**	**1.05**	**21**	**0.429**	**260.043**	**259.618**	**0.020**	**1.523**	**5.779**	**8.80**
C	2.39	0.15	1.3	113	0.623	259.618	259.308	0.003	3.107	2.728	8.48
D	2.3	0.15	1.35	166	0.621	259.308	258.885	0.003	3.105	2.625	8.15
E	**2.3**	**0.2**	**1.4**	**64**	**0.631**	**258.885**	**258.323**	**0.009**	**3.220**	**4.926**	**15.86**
F	2.78	0.2	2	42	0.820	258.323	258.273	0.001	5.560	2.159	12.01
G	3.05	0.21	2	104	0.865	258.273	258.148	0.001	6.100	2.249	13.72
H	3	0.25	1.98	60	0.853	258.148	257.382	0.013	5.940	7.262	43.13

*For the above calculation the value of manning's roughness coefficient (n) is 0.014.

The runoff generated at A, B and E are validated using rational formula and Storm Water Management Model to check the efficiency of the drain.

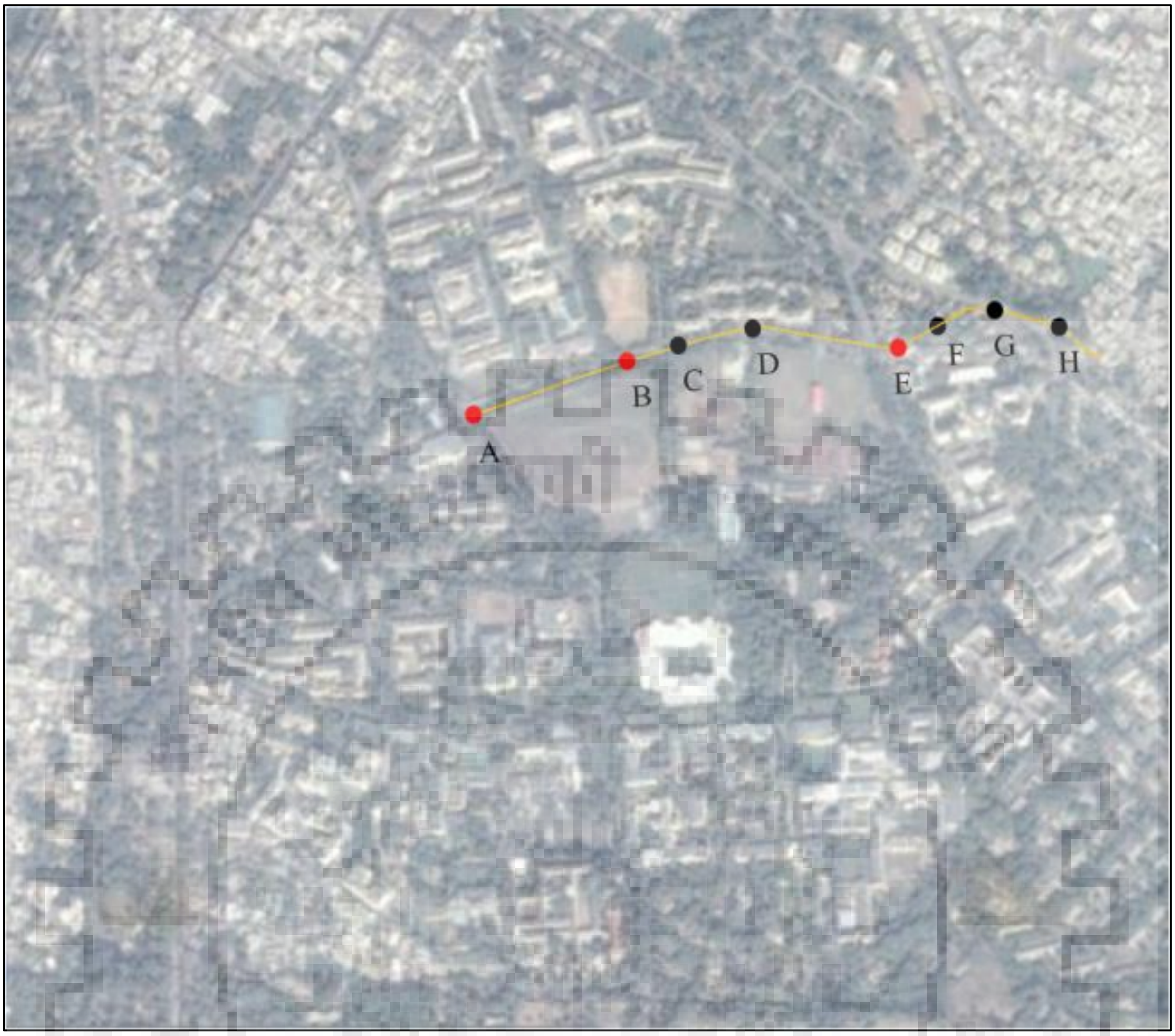


Figure 5.16 Location of nodes where runoff is calculated

5.9 STORM WATER MANAGEMENT MODEL

Rainfall of 176 years Return period on June 28, 2017 occurred at Roorkee inundating various drains in the campus of IIT Roorkee.

For analyzing the drainage effectiveness to handle such a heavy storm of 176 years return period SWMM has been used.

5.9.1 INPUT PARAMETERS

Input parameters used in SWMM for simulation of IITR campus is given in subsequent sections (States, 2015).

5.9.1.1 Subcatchments

Subcatchments are hydrologic units of land whose topography and drainage system elements direct surface runoff to a single discharge point. The subcatchments can be divided depending upon the outlet identified by the user. The outlet of subcatchment can be nodes or any other subcatchment.

Based upon the DEM, the subcatchments are further divided into pervious and impervious subareas. The properties of the subcatchments decide the amount of water unable to percolate and that results in runoff. Runoff flow from one subarea in a subcatchment can be routed to the other subarea, or both subareas can drain to the subcatchment outlet.

In the present study the infiltration of the subcatchments is described using curve number method.

For the analysis using the Storm Water Management Model, the campus area is divided into 23 subcatchments (the subcatchments are divided based upon the watershed area contributing to runoff inside the campus). The DEM and drainage plan of the area were used as a base map for division of sub catchments. The detailed input parameters (impervious area, weighted curve number, width etc.) of each subcatchment are given in the Table 5.11.

Table 5.11 Details of subcatchments used in SWMM model

Sub-catchments	Built	Green	Roads	Soil	Total area	Width of subcatchments	(% of impervious surface)	Weighted CN
1	17758	3834	4391	36487	62470	249.94	35.46	59.79
2	145113	-	15689	150918	311720	558.32	51.59	64.50
3	13659	-	-	24937	38596	196.46	35.39	58.91
4	25990	6334	1000	93611	126935	356.28	21.26	54.62
5	46986	-	8212	102381	157579	396.96	35.03	59.90
6	4980	-	683	22368	28031	167.43	20.20	55.17
7	5755	-	1679	13539	20973	144.82	35.45	60.61
8	3994	-	-	17056	21050	145.09	18.97	54.31
9	2810	-	-	6764	9574	97.85	29.35	57.22
10	16400	-	-	13539	29939	173.03	54.78	64.34
11	5981	2175	-	22235	30391	174.33	19.68	53.79
12		5386	288	56625	62299	249.60	0.46	48.36
13	14863	18855	627	61946	96291	310.31	16.09	51.68
14	8996	-	3998	31634	44628	211.25	29.12	59.03
15	8960	2974	2992	64023	78949	280.98	15.14	53.66
16	13502	4569	356	19134	37561	193.81	36.89	58.31
17	18800	1697	4284	25302	50083	223.79	46.09	63.36
18	10990	-	2010	18709	31709	178.07	41.00	61.81
19	46914	-	3278	104990	155182	393.93	32.34	58.50
20	12812	-	2320	6839	21971	148.23	68.87	70.50
21	19642	-	1017	19590	40249	200.62	51.33	63.90
22	19590	3210	4699	34314	61813	248.62	39.29	61.08
23	28996	-	1000	27965.5	55961	236.56	53.60	66.13

For determining curve number of individual subcatchments following method is used (USDA, 1986):

$$\boldsymbol{Q} = \frac{(\boldsymbol{P} - \boldsymbol{I_a})^2}{(\boldsymbol{P} - \boldsymbol{I_a}) + \boldsymbol{S}} \qquad \ldots(5.4a)$$

$$\boldsymbol{Q} = \frac{(\boldsymbol{P} - \mathbf{0.2}\boldsymbol{S})^2}{(\boldsymbol{P} + \mathbf{0.8}\boldsymbol{S})} \qquad \ldots(5.5b)$$

Q = runoff (mm)

P = rainfall (mm)

S = potential maximum retention after runoff begins (mm)

I_a = Initial abstraction (mm)

$I_a = 0.2S$ (for small watersheds)

$$S = \frac{1000}{CN} - 10 \qquad \text{…(5.6)}$$

CN values for residential, green areas, roads and soil are 77, 39, 98 and 49 respectively.

Figure 5.17 SWMM subcatchments

5.9.1.2 Junction nodes

The intersection point of link is called as junction. External inflows can also enter the system at junctions and they increase the overall inflow at that point. Junction becomes pressurized when the conduits get surcharged.

The parameters required for input at junctions are:

a. Invert (channel or manhole bottom) elevation
b. Height to ground surface

Table 5.12 Details of nodes used in SWMM model

J1	J2	J3	J4	J5	J6	J7	J8	J9	J10
260.813	260.043	259.618	259.308	258.885	258.323	258.273	258.148	257.382	262.58

J11	J12	J13	J14	J15	J16	J17	J18	J19	J20
262.839	262.345	262.335	261.65	261.353	260.613	260.418	260.418	260.383	260.353

J21	J22	J23	J24	J25	J26	J27	J28	J29	J30
261.283	261.923	262.463	263.478	262.343	261.668	262.398	261.82	265.808	262.998

J31	J32	J33	J34	J35	J36	J37	J38	J39	J40
262.925	262.768	263.078	262.953	262.728	260.915	259.605	259.573	261.342	260.445

J41	J42	J43	J44	J45	J46	J47	J48	J49	J50
259.9	259.425	261.587	261.512	260.157	264.077	263.149	260.187	266.115	267.905

J51	J52	J53	J54	J55	J56	J57	J58	J59	J60
267.64	267.625	267.39	265.73	265.69	266.95	267.605	268.125	267.85	267.82

J61	J62	J63	J64	J65	J66	J67	J68	J69	J70
267.175	266.395	267.07	266.835	266.5	255.771	266.95	265.485	265.259	266.426

J71	J72	J73	J74	J75	J76	J77	J78	J79	J80	J81
264.718	262.865	267.37	265.593	264.513	262.788	260.468	267.445	260.37	259.322	259.182

5.9.1.3 Conduits

Transportation of water from one node to another is done using conduits. There are various shapes that can be selected in SWMM based upon the site data.

The principal input parameters for conduits are:

a. Names of the inlet and outlet nodes
b. Offset height or elevation above the inlet and outlet node inverts
c. Conduit length
d. Manning's roughness
e. Cross-sectional geometry

Table 5.13 Details of conduits used in SWMM model

Conduits	Shape	Width	Height	Length	Manning's coefficient
C1	RC	1.45	1.05	276.00	0.014
C2	RC	1.45	1.05	210.74	0.014
C3	RC	2.39	1.30	113.75	0.014
C4	RC	2.30	1.35	172.16	0.014
C5	RC	2.30	1.40	46.54	0.014
C6	RC	2.78	2.00	42.80	0.014
C7	RC	3.05	2.00	101.80	0.014
C8	RC	3.00	1.98	63.08	0.014
C9	RO	0.33	0.92	25.11	0.010
C10	RO	0.25	0.21	153.60	0.010
C11	RO	0.48	0.40	9.60	0.010
C12	RO	0.46	0.38	76.55	0.010
C13	RO	0.47	0.40	82.79	0.010
C14	RO	0.47	0.40	153.00	0.010
C15	RO	0.38	0.45	67.41	0.010
C16	RO	1.00	0.38	9.27	0.010
C17	RO	1.00	0.48	9.53	0.010
C18	RO	0.34	0.60	281.09	0.010
C19	RO	0.34	0.34	117.00	0.010
C20	RO	0.43	0.40	31.65	0.010
C21	RO	0.65	0.57	129.63	0.010
C22	RO	0.42	0.88	97.91	0.010
C23	RO	0.60	0.68	79.44	0.010
C24	TR	0.35	0.41	48.27	0.010
C25	RO	0.46	0.65	80.46	0.010
C26	RO	0.48	0.50	46.22	0.010
C27	TR	0.20	0.52	13.59	0.010
C28	RO	0.38	0.60	10.75	0.010
C29	RO	0.39	0.20	47.30	0.010
C30	RO	0.79	0.36	135.40	0.010
C31	RO	0.50	0.27	47.03	0.010
C32	RO	0.50	0.27	2.39	0.010
C33	RO	0.46	0.44	101.40	0.010
C34	RO	0.43	0.44	70.06	0.010
C35	RO	0.43	0.52	72.91	0.010
C36	RO	0.46	0.76	4.95	0.010

Conduits	Shape	Width	Height	Length	Manning’s coefficient
C37	RO	0.46	0.26	28.19	0.010
C38	RO	0.45	0.44	68.23	0.010
C39	RO	0.98	0.88	30.08	0.010
C40	RO	0.85	1.30	63.68	0.010
C41	RO	0.85	1.40	140.84	0.010
C42	RO	0.85	1.14	39.53	0.010
C43	RO	0.23	0.40	140.27	0.010
C44	RO	1.23	0.63	108.38	0.010
C45	RO	0.35	0.25	91.61	0.010
C46	RO	0.22	0.45	15.56	0.010
C47	RO	0.30	0.25	86.64	0.010
C48	RO	0.35	0.38	86.32	0.010
C49	RO	0.60	0.30	123.90	0.010
C50	RO	0.35	0.43	170.50	0.010
C51	RO	0.47	0.24	38.29	0.010
C52	RO	0.47	0.20	99.47	0.010
C53	RO	0.25	0.22	58.49	0.010
C54	RO	0.25	0.25	160.53	0.010
C55	RO	0.45	0.40	132.52	0.010
C56	TR	0.48	0.17	123.80	0.010
C57	RO	1.37	1.05	63.76	0.010
C58	RO	1.25	0.86	107.86	0.010
C59	RO	0.55	0.25	42.58	0.010
C60	RO	0.67	0.30	147.33	0.010
C61	RO	1.00	0.71	165.60	0.010
C62	RO	1.00	0.60	92.03	0.010
C63	RO	0.96	1.00	100.20	0.010
C64	TR	0.40	0.60	110.60	0.010
C65	RO	1.15	0.90	142.70	0.010
C66	RO	1.00	0.96	104.20	0.010
C67	RO	0.95	0.95	114.70	0.010
C68	RO	0.95	1.16	6.90	0.010
C69	RO	1.25	1.13	18.99	0.010
C70	RO	0.64	0.70	18.15	0.010
C71	RO	0.38	0.48	39.15	0.010
C72	RO	0.40	0.40	130.37	0.010
C73	RO	0.50	0.38	46.29	0.010
C74	RO	0.51	0.20	27.29	0.010

*RC=Rectangular closed conduit, RO=rectangular open conduit, TR= trapezoidal conduit

5.9.1.4 Outfall nodes

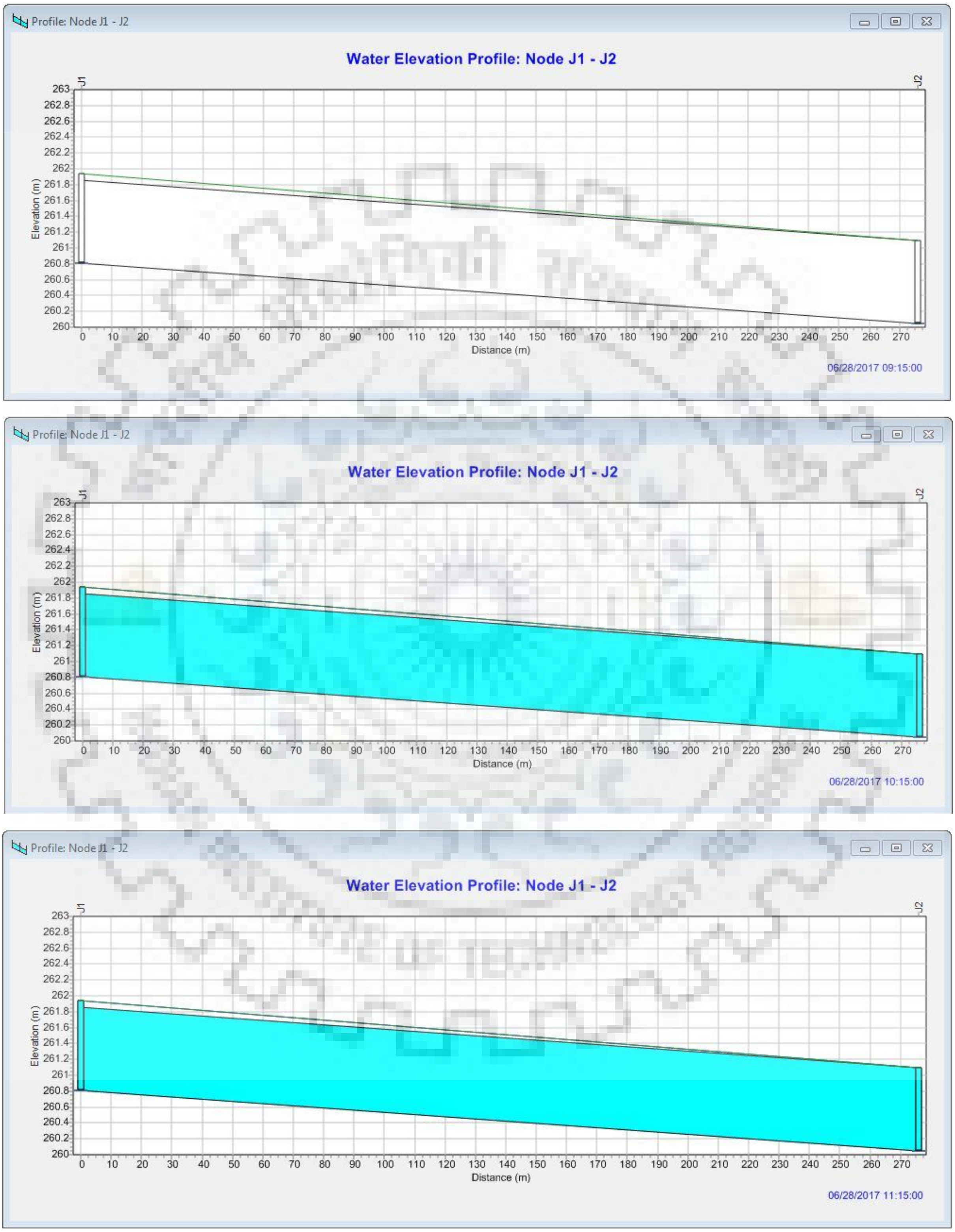
Outfalls are terminal nodes of the drainage system used to define final downstream boundaries. To an outfall node only a single link can be connected. The parameters required for outfalls are:

a. Invert elevation
b. Boundary condition type and stage description
c. Presence of a flap gate to prevent backflow through the outfall.

Table 5.14 Detail of outfalls used in SWMM model

Outfall	Outfall 1	Outfall 2	Outfall 3
Invert Elevation	259.962	258.827	257.347

To calculate the net flow at any junction, inflow from conduits and lateral inflow is added to get total inflow. As an example profile of water flows from J1 to J2 at 9:15 am, 10:15 am, 11:15 am, 12:15 pm and 12:45 pm is shown in Figure 5.18.



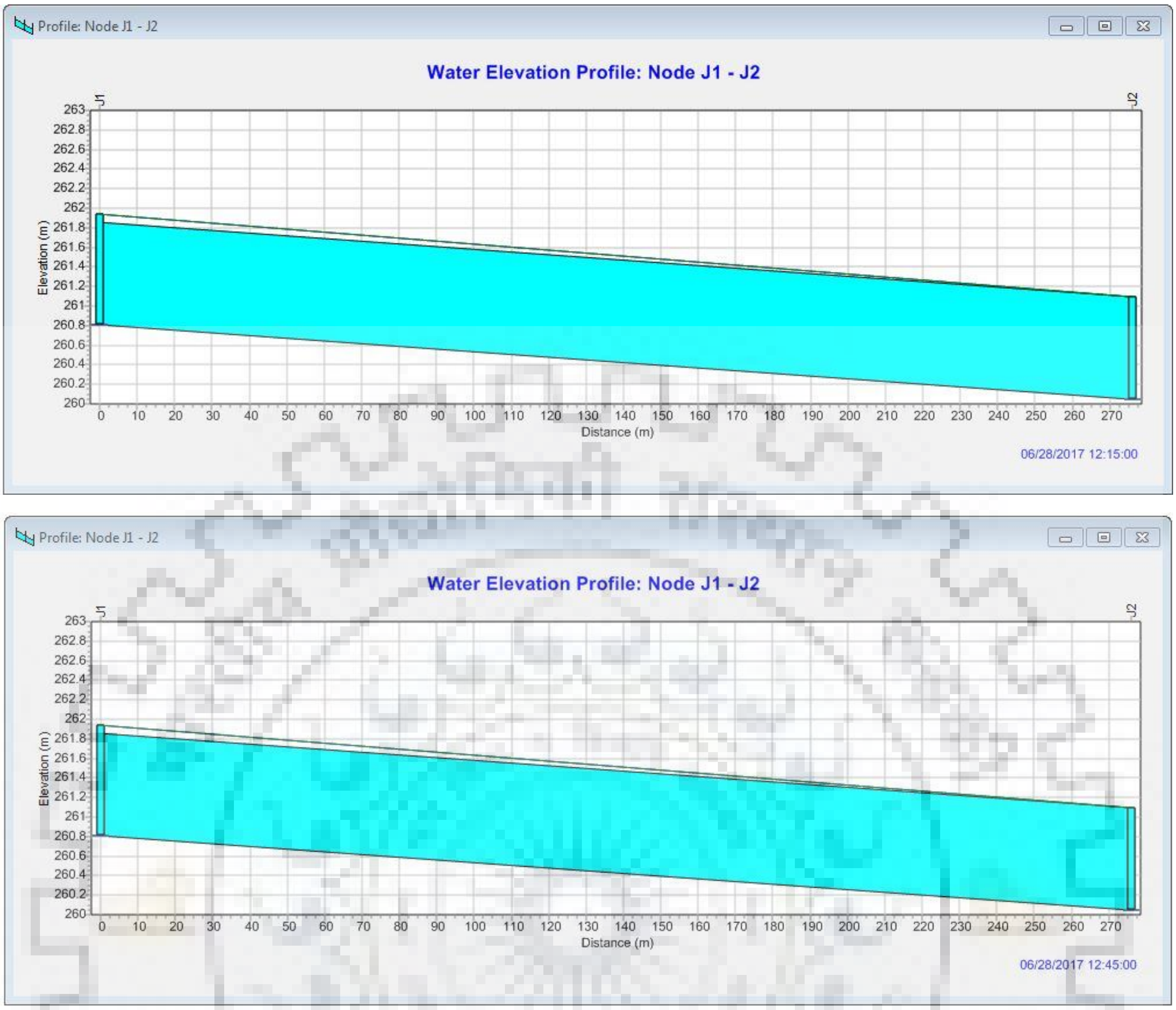


Figure 5.18 Inundation between node J1 and J2 over time

Similarly, the details of each subcatchment, nodes, and drains can be determined using Storm Water Management Model.

5.10 COMPARISON BETWEEN RESULTS OBTAINED USING RATIONAL FORMULA, MANNING'S EQUATION AND SWMM

The results obtained from the conventional method of rational formula at various points are compared to that obtained from Storm Water Management Model. Also, the existing drainage capacity calculated using Manning's formula is compared to the above two to determine the efficiency of the drain.

Table 5.15 Comparison between existing drainage capacity and calculated runoff using rational formula and Storm Water Management Model

Node	Existing drainage capacity Q(m^3/s)	Runoff generated (SWMM) Q(m^3/s)	Runoff generated (Rational formula) Q(m^3/s)	Inundation period (hrs)
J1	3.2	11.8	8.03	2:15
J2	8.8	15.06	11.65	2:15
J5	15.9	17.26	17.27	2:00

5.11 CALCULATION OF INTENSITY USING RATIONAL FORMULA FOR EXISTING DRAINAGE CAPACITY

The existing drainage capacity at J1, J2 and J3 are computed using manning's equation. The capacities are used to compute the intensity at all the three junctions using rational formula. Since the travel time at J1, J2 and J3 are known; we assume that travel time along with inundation at nodes is not more than 1 hr to stop flooding at nodes. The intensity obtained is shown in Table 5.16.

Table 5.16 Computation of intensity at nodes using rational formula

Watershed	WS1	WS2	WS3
tc (min)	30.12	35.66	42.58
Inundation (min)	29.88	24.34	17.42
C	0.33	0.38	0.37
A(km^2)	0.73	0.92	1.4
Q(m^3/s)	3.2	8.8	15.9
i(mm/hr)	47.82	90.62	110.50
depth (mm) at tc	47.82	90.62	110.50

Then, the annual maximum rainfall data for the past 116 years is used to compute the intensity at 1 hr and the graph is plotted as shown in Figure 5.19.

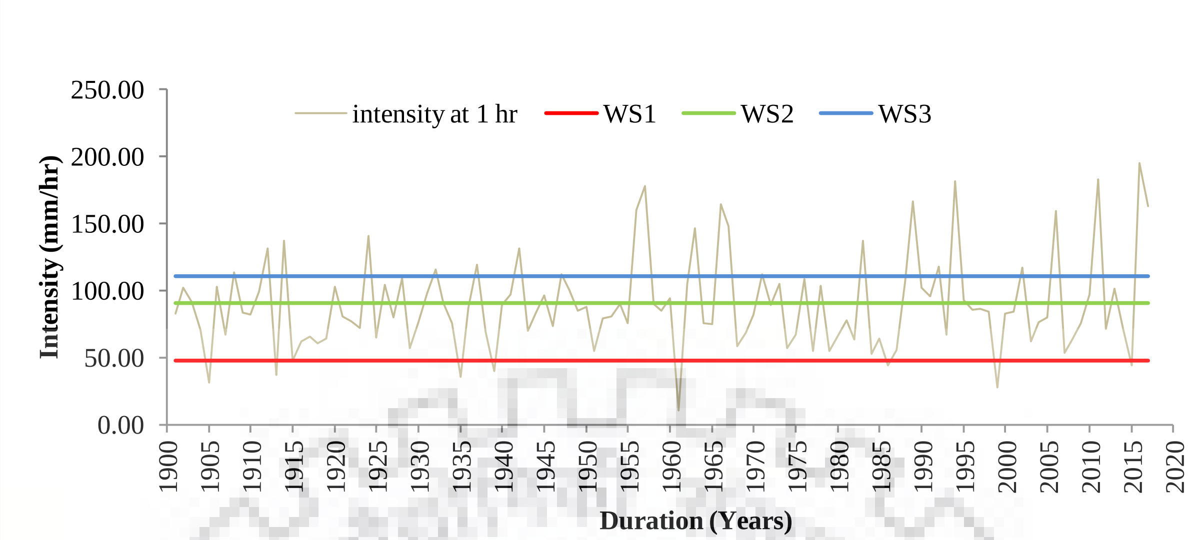


Figure 5.19 Exceedance of intensity above designed values

From the above figure it can be concluded that the number of times intensity exceeds the designed value for WS1, WS2, and WS3 is 108, 45, and 23 respectively.

The maximum intensity can be observed in the following years, for travel time of 30 minutes, the return period is computed using the equation:

$$I = \frac{KT^a}{(t+c)^b} \quad \ldots(5.7)$$

Table 5.17 Calculation of Return periods of maximum intensity rainfall

Year	Intensity(cm/hr)	Duration (0.5 hr)	Return Period
1957	17.81	0.5	141
1966	16.40	0.5	97
1983	13.70	0.5	43
1989	16.66	0.5	104
1994	18.15	0.5	153
2011	18.29	0.5	158
2016	19.54	0.5	214

From the above table it can be concluded that the capacity of drains should be increased.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

The study revolved around two aspects viz. (i) Frequency analysis of rainfall data of Roorkee and determination of the return period of June 28, 2017 event and development of IDF curves for Roorkee, and (ii) Evaluation of the adequacy of IIT Roorkee campus drainage system using Rational formula and SWMM model. The conclusions drawn from these two aspects are presented in the following sections.

6.1.1 Rainfall Analysis

The following conclusions can be drawn from the rainfall analysis.

a. From the hyetograph it can be seen that the storm lasted for 4 hrs, i.e. 9:00 am to 1:00 pm.
b. At 45 minutes from the start of rainfall, the storm reached the depth of 90 mm with the maximum intensity of 120 mm/hr.
c. The frequency analysis of past 116 year annual maximum rainfall data show that the rainfall data follows Generalized Logistic Distribution.
d. Using the equation of GLO, the return period of the storm for 24 hrs duration with total rainfall of 236.6 mm comes out to be 30 years.
e. The maximum return period observed at 3.00 hr (i.e. at 12 noon) was 176 years.
f. The storm event was unprecedented but the main reason of flooding in the campus has been the inadequate capacity of the drain and unplanned development around the main drain. The construction of continuing education guest house and Kasturba Bhawan worsened the situation. As a result old teacher’s hostel, new teacher’s hostel, continuing education guest house and Kasturba Bhawan are bond to face frequent flooding problems in future.
g. The covering of the main drain has resulted in the back flow of water from the drain to the surrounding area due to inadequate capacity of the drain and wrong positioning of the inlets.

6.1.2 Drainage Analysis for IIT Roorkee

Critical analysis of storm water drain of the campus at specific nodes has been done. The adequacy of the existing drainage system inside the campus is assessed. The analysis was done using rational formula and Storm Water Management Model, and from the study following conclusions can be drawn.

a. The drain is under designed at junction J1, J2, and overdesigned after J5 for the storm of return period of 176 years.
b. At J5 the natural flow of storm water is towards Gate no.5; overdesigned drain serves no purpose after this junction.
c. Drain is built to facilitate runoff after J5 (at Major Dhyan Chand stadium junction) but, inappropriate maintenance, and cleaning results in drain congestion (site visit analysis).
d. The drain also acts as an insufficient carrier of stormwater due to congestion and encroachment at most of the places. Improper solid waste management, garbage disposal in open drains leads to blockage of the drains, ultimately generates runoff.
e. Covering of drain with concrete slab resulted in reduction of the capacity of drain.

Based on the above conclusions some recommendations are given, which could be adopted in future to mitigate the urban flooding due to heavy rainstorm.

6.2 RECOMMENDATIONS FOR FUTURE WORK

Since the drain of IIT Roorkee campus is unable to handle the high intensity rainfall of more than 100 years return period, it is advisable to increase its capacity for at least a period of 200 years. Continuous haphazard urbanization in the catchment (in and outside the campus) shall increase the rate of runoff to a great extent in coming years. This event can be considered as an alarm to be prepared for such incidences in future. The recommendations for improving the drainage system of the campus are given below.

a. De-silting of the drains;
b. Increase the drainage capacity to successfully carry runoff generated through 200 years return period rainfall (since the recommended rainfall intensity is 195.41 mm/hr with return period of 214 years). The drainage capacity for 200 year return period should be increased in accordance to the runoff generated at various critical nodes. The detail of runoff generated at different points for 200 yrs. return period rainfall is

shown in Table 6.1. The intensity for 200 year return period has been taken from Intensity Duration Frequency curves of IIT Roorkee.

Table 6.1 Runoff generated at nodes for 200 yr. return period rainfall

	Node 1	**Node 2**	**Node 3**
Q (m^3/s)	12.85	17.05	23.16

c. Providing a drain along the natural flow of watershed may reduce the impact of flooding at nodes (Figure 6.1);

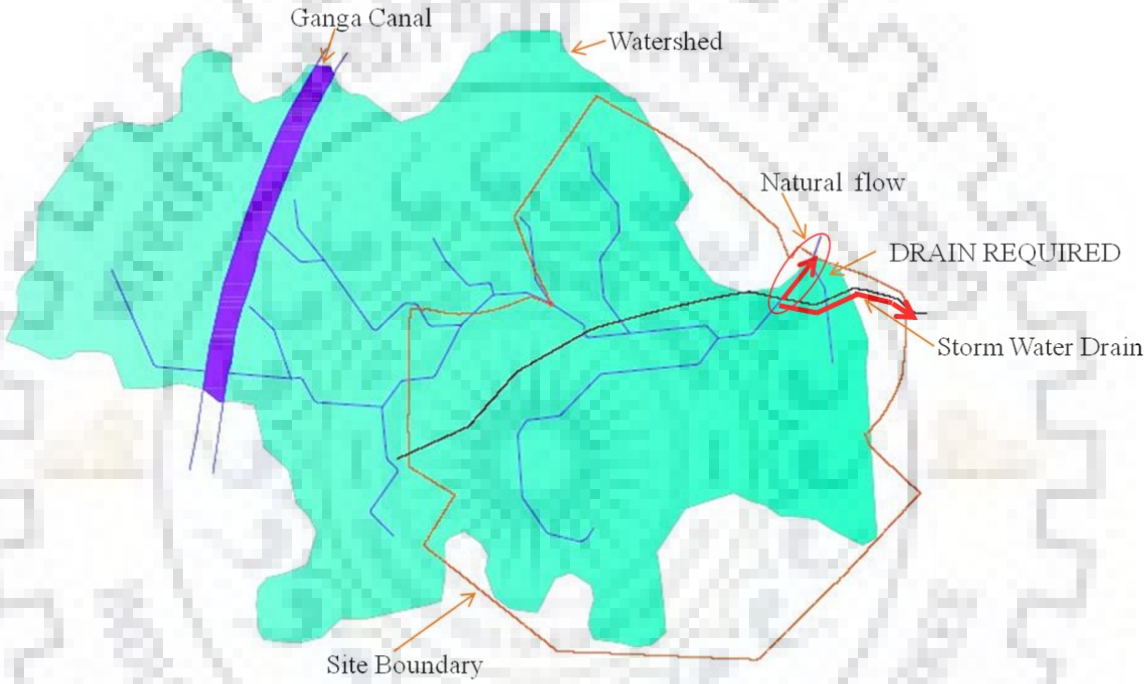


Figure 6.1 An alternative drain to carry storm water

d. Installation of one way non-returning covers for the inlets are also proposed to check the backflow of water from drain to the road; and

e. Using Rain Water Harvesting technique in individual pocket of the campus (sample case study given in Appendix D).

APPENDICES

Appendix A Annual maximum rainfall data for frequency analysis

Year	AMR(mm)	Year	AMR(mm)	Year	AMR(mm)	Year	AMR(mm)	Year	AMR(mm)
1901	119.9	**1926**	150.6	**1951**	79.5	**1976**	156.8	**2001**	121.6
1902	148.3	**1927**	115.6	**1952**	114.6	**1977**	80.0	**2002**	170
1903	132.6	**1928**	158.0	**1953**	117.3	**1978**	149.8	**2003**	90
1904	102.6	**1929**	82.3	**1954**	130.0	**1979**	79.4	**2004**	110.6
1905	45.0	**1930**	111.0	**1955**	109.5	**1980**	96.1	**2005**	115.8
1906	148.8	**1931**	141.7	**1956**	231.6	**1981**	112.5	**2006**	231
1907	96.8	**1932**	167.9	**1957**	258.1	**1982**	92.4	**2007**	77.5
1908	164.8	**1933**	131.1	**1958**	130.8	**1983**	198.6	**2008**	91.8
1909	120.7	**1934**	109.5	**1959**	123.4	**1984**	76.4	**2009**	109.1
1910	118.9	**1935**	51.6	**1960**	136.9	**1985**	93	**2010**	140.4
1911	143.5	**1936**	126.0	**1961**	15.2	**1986**	63.8	**2011**	265
1912	190.2	**1937**	173.2	**1962**	152.4	**1987**	80.5	**2012**	103
1913	53.6	**1938**	100.3	**1963**	212.0	**1988**	155	**2013**	146.8
1914	198.6	**1939**	57.2	**1964**	109.1	**1989**	241.5	**2014**	103
1915	69.1	**1940**	129.5	**1965**	108.4	**1990**	148	**2015**	63.6
1916	89.4	**1941**	141.0	**1966**	237.7	**1991**	138.2	**2016**	283.2
1917	95.3	**1942**	190.5	**1967**	214.6	**1992**	171		
1918	88.1	**1943**	101.6	**1968**	84.8	**1993**	97.5		
1919	92.7	**1944**	120.4	**1969**	99.1	**1994**	263		
1920	148.8	**1945**	139.2	**1970**	118.6	**1995**	134.6		
1921	116.6	**1946**	106.7	**1971**	162.6	**1996**	123.6		
1922	111.8	**1947**	162.3	**1972**	129.5	**1997**	125		
1923	104.1	**1948**	146.3	**1973**	152.6	**1998**	121.6		
1924	204.2	**1949**	123.4	**1974**	83.1	**1999**	40.1		
1925	94.0	**1950**	127.0	**1975**	96.8	**2000**	120		

Appendix B Computation of various parameters of frequency analysis

1. Computation of statistical parameters

	Original series	Log transformed series
Mean	128.891	4.782
Standard deviation	49.557	0.417
Cs	0.897	-1.108
C_k	4.196	7.731

2. Test for randomness and stationarity

Test	1% significance level	5% significance level	10% significance level
Turning point test	Random	Random	Random
Anderson correlogram test	Random	Random	Random
Kendall's rank correlation test	No trend	No trend	No trend

3. Check for goodness of fit tests

Distribution	K-S Test	Chi-square test	D-index test	
Normal distribution		Not fitting	1.53552	
Log normal distribution		Not fitting	0.58734	Minimum
PT III distribution		Fitting	0.64365	
LPT III distribution		Not fitting	1.702	
Gumbel distribution	Fitting	Fitting	0.59838	
Log Gumbel distribution		Not fitting	3.32805	

4. L-moment parameters estimation

Parameters		Values
β0	M100	128.890
β1	M110	77.894
β2	M120	57.164
β3	M130	45.732
λ1	β0	128.890
λ2	2β1-β0	26.898
λ3	6β2-6β1+β0	4.507
λ4	20β3-30β2+12β1-β0	5.574
LCv	λ2/λ1	0.208690
LCs	λ3/λ2	0.167564
LCk	λ4/λ2	0.207230

Appendix C Calculation of Intensity duration frequency curve

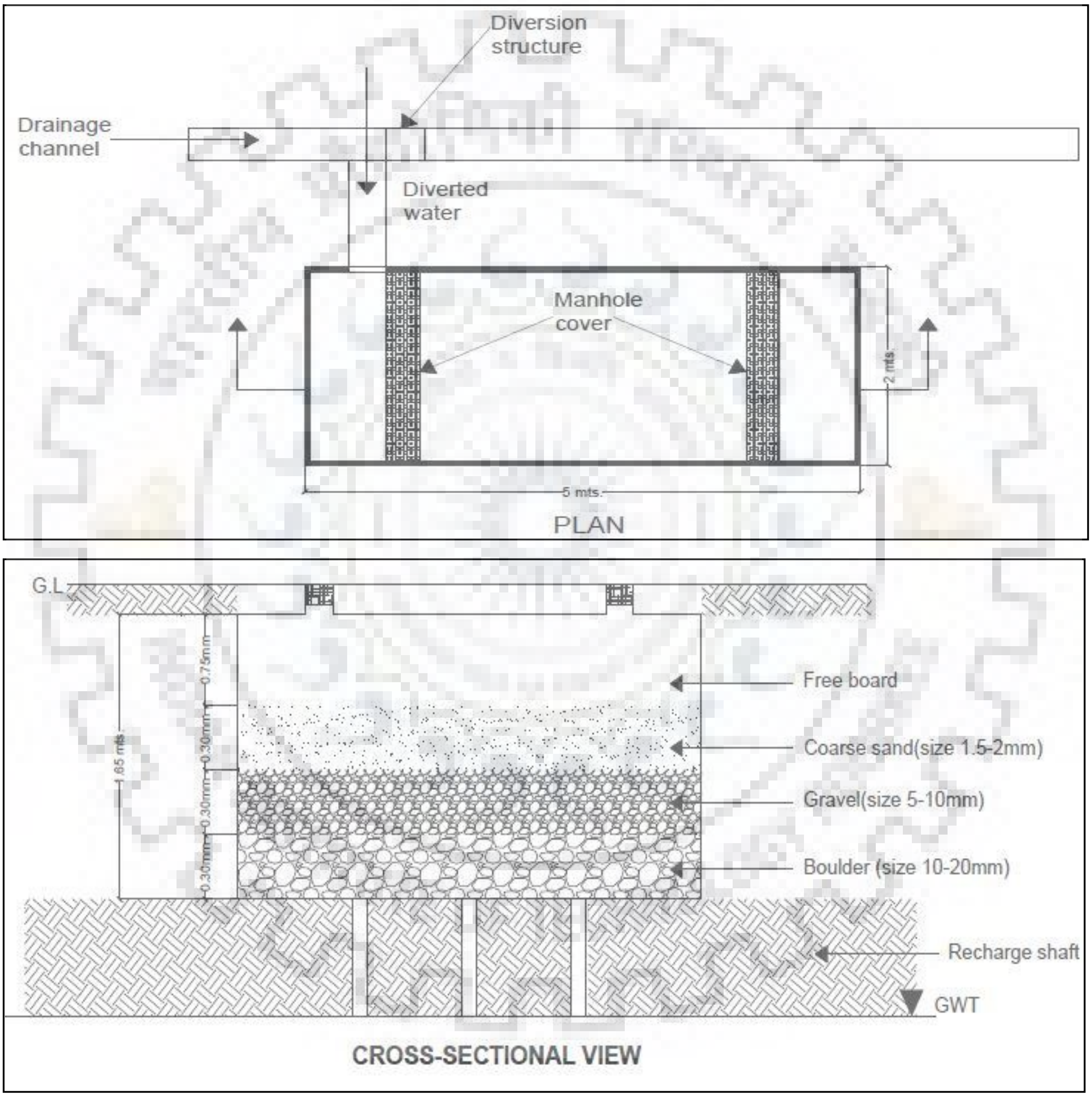
			5min.	10min.	15min.	30min.	1hr.	2hr.	6hr.	12hr.	24hr.
Year	**ARF(mm.)**	**Av. Intensity (cm/hr)**	**0.08hr.**	**0.17hr.**	**0.25hr.**	**0.50hr.**	**1hr.**	**2hrs.**	**6hrs.**	**12hrs.**	**24hrs.**
1901	119.9	0.5	11.53	10.71	9.99	8.33	6.24	4.16	1.78	0.96	0.50
1902	148.3	0.6	14.26	13.24	12.36	10.30	7.72	5.15	2.21	1.19	0.62
1903	132.6	0.6	12.75	11.84	11.05	9.21	6.91	4.60	1.97	1.06	0.55
1904	102.6	0.4	9.87	9.16	8.55	7.13	5.34	3.56	1.53	0.82	0.43
1905	45.0	0.2	4.33	4.02	3.75	3.13	2.34	1.56	0.67	0.36	0.19
1906	148.8	0.6	14.31	13.29	12.40	10.33	7.75	5.17	2.21	1.19	0.62
1907	96.8	0.4	9.31	8.64	8.07	6.72	5.04	3.36	1.44	0.78	0.40
1908	164.8	0.7	15.85	14.71	13.73	11.44	8.58	5.72	2.45	1.32	0.69
1909	120.7	0.5	11.61	10.78	10.06	8.38	6.29	4.19	1.80	0.97	0.50
1910	118.9	0.5	11.43	10.62	9.91	8.26	6.19	4.13	1.77	0.95	0.50
1911	143.5	0.6	13.80	12.81	11.96	9.97	7.47	4.98	2.14	1.15	0.60
1912	190.2	0.8	18.29	16.98	15.85	13.21	9.91	6.60	2.83	1.52	0.79
1913	53.6	0.2	5.15	4.79	4.47	3.72	2.79	1.86	0.80	0.43	0.22
1914	198.6	0.8	19.10	17.73	16.55	13.79	10.34	6.90	2.96	1.59	0.83
1915	69.1	0.3	6.64	6.17	5.76	4.80	3.60	2.40	1.03	0.55	0.29
1916	89.4	0.4	8.60	7.98	7.45	6.21	4.66	3.10	1.33	0.72	0.37
1917	95.3	0.4	9.16	8.51	7.94	6.62	4.96	3.31	1.42	0.76	0.40
1918	88.1	0.4	8.47	7.87	7.34	6.12	4.59	3.06	1.31	0.71	0.37
1919	92.7	0.4	8.91	8.28	7.73	6.44	4.83	3.22	1.38	0.74	0.39
1920	148.8	0.6	14.31	13.29	12.40	10.33	7.75	5.17	2.21	1.19	0.62
1921	116.6	0.5	11.21	10.41	9.72	8.10	6.07	4.05	1.74	0.93	0.49
1922	111.8	0.5	10.75	9.98	9.32	7.76	5.82	3.88	1.66	0.90	0.47
1923	104.1	0.4	10.01	9.29	8.68	7.23	5.42	3.61	1.55	0.83	0.43
1924	204.2	0.9	19.63	18.23	17.02	14.18	10.64	7.09	3.04	1.64	0.85
1925	94.0	0.4	9.04	8.39	7.83	6.53	4.90	3.26	1.40	0.75	0.39
1926	150.6	0.6	14.48	13.45	12.55	10.46	7.84	5.23	2.24	1.21	0.63
1927	115.6	0.5	11.12	10.32	9.63	8.03	6.02	4.01	1.72	0.93	0.48
1928	158.0	0.7	15.19	14.11	13.17	10.97	8.23	5.49	2.35	1.27	0.66
1929	82.3	0.3	7.91	7.35	6.86	5.72	4.29	2.86	1.22	0.66	0.34
1930	111.0	0.5	10.67	9.91	9.25	7.71	5.78	3.85	1.65	0.89	0.46
1931	141.7	0.6	13.63	12.65	11.81	9.84	7.38	4.92	2.11	1.14	0.59
1932	167.9	0.7	16.14	14.99	13.99	11.66	8.74	5.83	2.50	1.35	0.70
1933	131.1	0.5	12.61	11.71	10.93	9.10	6.83	4.55	1.95	1.05	0.55
1934	109.5	0.5	10.53	9.78	9.13	7.60	5.70	3.80	1.63	0.88	0.46
1935	51.6	0.2	4.96	4.61	4.30	3.58	2.69	1.79	0.77	0.41	0.22
1936	126.0	0.5	12.12	11.25	10.50	8.75	6.56	4.38	1.88	1.01	0.53
1937	173.2	0.7	16.65	15.46	14.43	12.03	9.02	6.01	2.58	1.39	0.72
1938	100.3	0.4	9.64	8.96	8.36	6.97	5.22	3.48	1.49	0.80	0.42
1939	57.2	0.2	5.50	5.11	4.77	3.97	2.98	1.99	0.85	0.46	0.24

			5min.	10min.	15min.	30min.	1hr.	2hr.	6hr.	12hr.	24hr.
Year	**ARF(mm.)**	**Av. Intensity (cm/hr)**	**0.08hr.**	**0.17hr.**	**0.25hr.**	**0.50hr.**	**1hr.**	**2hrs.**	**6hrs.**	**12hrs.**	**24hrs.**
1940	129.5	0.5	12.45	11.56	10.79	8.99	6.74	4.50	1.93	1.04	0.54
1941	141.0	0.6	13.56	12.59	11.75	9.79	7.34	4.90	2.10	1.13	0.59
1942	190.5	0.8	18.32	17.01	15.88	13.23	9.92	6.61	2.83	1.53	0.79
1943	101.6	0.4	9.77	9.07	8.47	7.06	5.29	3.53	1.51	0.81	0.42
1944	120.4	0.5	11.58	10.75	10.03	8.36	6.27	4.18	1.79	0.96	0.50
1945	139.2	0.6	13.38	12.43	11.60	9.67	7.25	4.83	2.07	1.12	0.58
1946	106.7	0.4	10.26	9.53	8.89	7.41	5.56	3.70	1.59	0.85	0.44
1947	162.3	0.7	15.61	14.49	13.53	11.27	8.45	5.64	2.42	1.30	0.68
1948	146.3	0.6	14.07	13.06	12.19	10.16	7.62	5.08	2.18	1.17	0.61
1949	123.4	0.5	11.87	11.02	10.28	8.57	6.43	4.28	1.84	0.99	0.51
1950	127.0	0.5	12.21	11.34	10.58	8.82	6.61	4.41	1.89	1.02	0.53
1951	79.5	0.3	7.64	7.10	6.63	5.52	4.14	2.76	1.18	0.64	0.33
1952	114.6	0.5	11.02	10.23	9.55	7.96	5.97	3.98	1.71	0.92	0.48
1953	117.3	0.5	11.28	10.47	9.78	8.15	6.11	4.07	1.75	0.94	0.49
1954	130.0	0.5	12.50	11.61	10.83	9.03	6.77	4.51	1.93	1.04	0.54
1955	109.5	0.5	10.53	9.78	9.13	7.60	5.70	3.80	1.63	0.88	0.46
1956	231.6	1.0	22.27	20.68	19.30	16.08	12.06	8.04	3.45	1.86	0.97
1957	258.1	1.1	24.82	23.04	21.51	17.92	13.44	8.96	3.84	2.07	1.08
1958	130.8	0.5	12.58	11.68	10.90	9.08	6.81	4.54	1.95	1.05	0.55
1959	123.4	0.5	11.87	11.02	10.28	8.57	6.43	4.28	1.84	0.99	0.51
1960	136.9	0.6	13.16	12.22	11.41	9.51	7.13	4.75	2.04	1.10	0.57
1961	15.2	0.1	1.46	1.36	1.27	1.06	0.79	0.53	0.23	0.12	0.06
1962	152.4	0.6	14.65	13.61	12.70	10.58	7.94	5.29	2.27	1.22	0.64
1963	212.0	0.9	20.38	18.93	17.67	14.72	11.04	7.36	3.15	1.70	0.88
1964	109.1	0.5	10.49	9.74	9.09	7.58	5.68	3.79	1.62	0.87	0.45
1965	108.4	0.5	10.42	9.68	9.03	7.53	5.65	3.76	1.61	0.87	0.45
1966	237.7	1.0	22.86	21.22	19.81	16.51	12.38	8.25	3.54	1.90	0.99
1967	214.6	0.9	20.63	19.16	17.88	14.90	11.18	7.45	3.19	1.72	0.89
1968	84.8	0.4	8.15	7.57	7.07	5.89	4.42	2.94	1.26	0.68	0.35
1969	99.1	0.4	9.53	8.85	8.26	6.88	5.16	3.44	1.47	0.79	0.41
1970	118.6	0.5	11.40	10.59	9.88	8.24	6.18	4.12	1.76	0.95	0.49
1971	162.6	0.7	15.63	14.52	13.55	11.29	8.47	5.65	2.42	1.30	0.68
1972	129.5	0.5	12.45	11.56	10.79	8.99	6.74	4.50	1.93	1.04	0.54
1973	152.6	0.6	14.67	13.63	12.72	10.60	7.95	5.30	2.27	1.22	0.64
1974	83.1	0.3	7.99	7.42	6.93	5.77	4.33	2.89	1.24	0.67	0.35
1975	96.8	0.4	9.31	8.64	8.07	6.72	5.04	3.36	1.44	0.78	0.40
1976	156.8	0.7	15.08	14.00	13.07	10.89	8.17	5.44	2.33	1.26	0.65
1977	80.0	0.3	7.69	7.14	6.67	5.56	4.17	2.78	1.19	0.64	0.33
1978	149.8	0.6	14.40	13.38	12.48	10.40	7.80	5.20	2.23	1.20	0.62
1979	79.4	0.3	7.63	7.09	6.62	5.51	4.14	2.76	1.18	0.64	0.33
1980	96.1	0.4	9.24	8.58	8.01	6.67	5.01	3.34	1.43	0.77	0.40

			5min.	10min.	15min.	30min.	1hr.	2hr.	6hr.	12hr.	24hr.
Year	**ARF(mm.)**	**Av. Intensity (cm/hr)**	**0.08hr.**	**0.17hr.**	**0.25hr.**	**0.50hr.**	**1hr.**	**2hrs.**	**6hrs.**	**12hrs.**	**24hrs.**
1981	112.5	0.5	10.82	10.04	9.38	7.81	5.86	3.91	1.67	0.90	0.47
1982	92.4	0.4	8.88	8.25	7.70	6.42	4.81	3.21	1.38	0.74	0.39
1983	198.6	0.8	19.10	17.73	16.55	13.79	10.34	6.90	2.96	1.59	0.83
1984	76.4	0.3	7.35	6.82	6.37	5.31	3.98	2.65	1.14	0.61	0.32
1985	93	0.4	8.94	8.30	7.75	6.46	4.84	3.23	1.38	0.75	0.39
1986	63.8	0.3	6.13	5.70	5.32	4.43	3.32	2.22	0.95	0.51	0.27
1987	80.5	0.3	7.74	7.19	6.71	5.59	4.19	2.80	1.20	0.65	0.34
1988	155	0.6	14.90	13.84	12.92	10.76	8.07	5.38	2.31	1.24	0.65
1989	241.5	1.0	23.22	21.56	20.13	16.77	12.58	8.39	3.59	1.94	1.01
1990	148	0.6	14.23	13.21	12.33	10.28	7.71	5.14	2.20	1.19	0.62
1991	138.2	0.6	13.29	12.34	11.52	9.60	7.20	4.80	2.06	1.11	0.58
1992	171	0.7	16.44	15.27	14.25	11.88	8.91	5.94	2.54	1.37	0.71
1993	97.5	0.4	9.38	8.71	8.13	6.77	5.08	3.39	1.45	0.78	0.41
1994	263	1.1	25.29	23.48	21.92	18.26	13.70	9.13	3.91	2.11	1.10
1995	134.6	0.6	12.94	12.02	11.22	9.35	7.01	4.67	2.00	1.08	0.56
1996	123.6	0.5	11.88	11.04	10.30	8.58	6.44	4.29	1.84	0.99	0.52
1997	125	0.5	12.02	11.16	10.42	8.68	6.51	4.34	1.86	1.00	0.52
1998	121.6	0.5	11.69	10.86	10.13	8.44	6.33	4.22	1.81	0.97	0.51
1999	40.1	0.2	3.86	3.58	3.34	2.78	2.09	1.39	0.60	0.32	0.17
2000	120	0.5	11.54	10.71	10.00	8.33	6.25	4.17	1.79	0.96	0.50
2001	121.6	0.5	11.69	10.86	10.13	8.44	6.33	4.22	1.81	0.97	0.51
2002	170	0.7	16.35	15.18	14.17	11.81	8.85	5.90	2.53	1.36	0.71
2003	90	0.4	8.65	8.04	7.50	6.25	4.69	3.13	1.34	0.72	0.38
2004	110.6	0.5	10.63	9.88	9.22	7.68	5.76	3.84	1.65	0.89	0.46
2005	115.8	0.5	11.13	10.34	9.65	8.04	6.03	4.02	1.72	0.93	0.48
2006	231	1.0	22.21	20.63	19.25	16.04	12.03	8.02	3.44	1.85	0.96
2007	77.5	0.3	7.45	6.92	6.46	5.38	4.04	2.69	1.15	0.62	0.32
2008	91.8	0.4	8.83	8.20	7.65	6.38	4.78	3.19	1.37	0.74	0.38
2009	109.1	0.5	10.49	9.74	9.09	7.58	5.68	3.79	1.62	0.87	0.45
2010	140.4	0.6	13.50	12.54	11.70	9.75	7.31	4.88	2.09	1.13	0.59
2011	265	1.1	25.48	23.66	22.08	18.40	13.80	9.20	3.94	2.12	1.10
2012	103	0.4	9.90	9.20	8.58	7.15	5.36	3.58	1.53	0.83	0.43
2013	146.8	0.6	14.12	13.11	12.23	10.19	7.65	5.10	2.18	1.18	0.61
2014	103	0.4	9.90	9.20	8.58	7.15	5.36	3.58	1.53	0.83	0.43
2015	63.6	1.2	27.23	25.29	23.60	19.67	14.75	9.83	4.21	2.27	1.18
2016	238.2	0.3	6.12	5.68	5.30	4.42	3.31	2.21	0.95	0.51	0.27
		Mean	**12.39**	**11.51**	**10.74**	**8.95**	**6.71**	**4.48**	**1.92**	**1.03**	**0.54**
		SD	**4.77**	**4.42**	**4.13**	**3.44**	**2.58**	**1.72**	**0.74**	**0.40**	**0.21**

Appendix D Case Study: NIH Roorkee (Rain Water Harvesting)

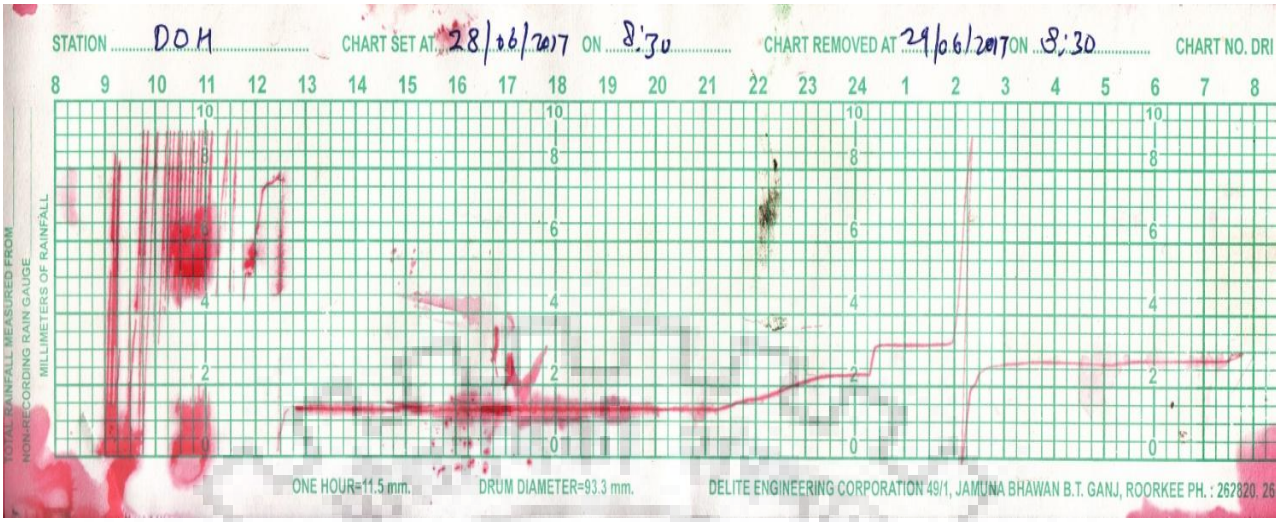
Rain Water Harvesting system was installed in the year 2012 at National Institute of Hydrology, Roorkee. It acts as an artificial ground water recharge for surface runoff of the NIH's campus. In the scheme, various recharge pits of 5m*2m*1.65 m are constructed and connected via covered channel. Detailed schematic diagram of recharge pits are shown figure below.



Recharge pit

Such small rainwater harvesting recharge pits will reduce the runoff generated during heavy storms. This facility should be compulsorily implemented in hostels, residential blocks, grounds etc. so that water can be recharged or reused for various purposes.

Appendix E Graph of June 28, 2017



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